OPTIMUM RESERVOIR OPERATION DURING FLOODS

Leon Furstenburg

A project report submitted to the Faculty of Engineering, University of the Witwatersrand, Johannesburg, in partial fulfilment of the requirements for the degree of Master of Science in Engineering.

Johannesburg, 1985

ACKNOWLEDGEMENTS

The author is indebted to Binnie & Partners, Johannesburg, for providing computer facilities for the development of the RESIMO package.

DECLARATION

I declare that this project report is my own, unaided work. It is being submitted for the Degree of Master of Science in Engineering in the University of the Witwatersrand, Johannesburg. It has not been submitted before for any degree or examination in any other University.

28 day of Nov 1985.

ABSTRACT

This report presents a new reservoir simulation model (RESIMO) which is general and contains an optimization routine for the calculation of an operating rule, applicable to a specific flood, for manually controlled spillway gates. The operating rule is optimized in the sense that the peak outflow rate from the reservoir, during the flood, is minimized by using storage and pre-release volumes to their maximum extent. An extension of the storage-indication method, to accomodate controlled spillways when used in conjunction with uncontrolled spillways, is also presented. The application of the optimization routine as an aid to the reservoir manager during real time control of spillways is discussed. A limitation of the method is that it requires a full predicted inflow hydrograph which may not be available during real time operation. A method of allowing for this uncertainty during real time operation is discussed.

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1.0 INTRODUCTION

A reservoir site is a natural resource. As such it is important that it not be wasted by using it to less than its optimum potential. In this sense the word "optimum" may include the economic limit for a particular purpose or the inclusion of a variety of purposes to be served by a single reservoir.

For this reason reservoirs are seldom of single purpose and even where one purpose dominates the situation, lesser purposes must be considered. Multiple use is often made of the same storage space, or various head ranges in the same reservoir are often used for different purposes, such as flood control, power, irrigation, recreation, water supply, or navigation. Such combined operation requires very careful planning and control, as some of the uses are not compatible with others. For example, a power user, in order to be sure of the maximum amount of firm power, may wish to have the storage full when flood hazard is imminent and the need for available reservoir capacity greatest. Such combined operation is possible only with loss of some measure of benefit to one or all of the participants. Careful management is important in the operation of multiple-use reservoirs in order to maintain a balanced perspective in the matter of relative values.

As development takes place, be it urban, industrial or agricultural it invariably leads to an increased water demand and therefore naturally centres around water supplies. However advantageous a fixed-crest free overfall spillway may be from the operating standpoint the great value of property and improvements in many reservoir sites prohibits the backwater stages resulting above such a dam during floods. At the same time, the desirable storage or head requirements on projects concerned with the conservation of water or control of floods may often approach the limit that can be obtained at the feasible dam sites.

The solution to this problem lies in some form of moveable crest, by means of which increases in flood stage above the dam may be materially lessened within the limits of backwater effect, without seriously curtailing the low-water storage or head normally available.

However, moveable crests are not failsafe nor problem free. The most serious problem with any type of moveable crest is that it requires operation. Gate operation is by means of mechanical machinery which needs regular maintenance, inspection and test operation by suitably qualified persons. In addition this operation needs to be controlled and the method of control can be either manual or automatic. An automatic control system is invariably sophisticated and expensive, and requires regular maintenance. Manual control systems are usually simpler and cheaper but require input from a suitably trained and capable person.

With this in mind the Department of Water Affairs (14) made the following recommendation in 1983:

"12.1 General principles

- The cardinal principle to observe is to preferably have no gates on a dam at all, unless they are very clearly justified for reasons such as economics or hydrologic e.g. flood absorption or whatever other valid reaosn may exist.
- Control should preferably not be automatic unless very specific reasons exist therefore.
- The operation and maintenance of crest gates, especially when automated, create considerable difficulty with regard to manpower and training, so that due regard should be taken of these implications when such a decision for particular reasons is taken.

12.11 Operators

- Dams with flood gates should be designed so as to be safely operable by suitably trained capable operators. A suitable grading system and training programme for operators should be put into effect"

It is clear that the operator requires an operating rule to guide him into making the correct decision regarding the operation of the spillway during a flood. This operating rule should take due regard of the purpose of the reservoir but also of the size and shape of the inflow hydrograph and the status of the reservoir (water level, gate positions etc.), just before the flood. Due to the variability of these factors, and the fact that some of them may only be partly known or estimated during operations, it is very difficult and often impossible to determine one set of gate operating rules that will always apply. For this reason real time control of spillways is often exercised during floods. This type of control requires a means of quick and accurate floodrouting through the reservoir to test various modes of operation for a specific flood under specific conditions. Based on these results a decision can then be made on the operation of the spillway under those particular conditions.

In principle flood routing through a reservoir is very simple and numerous programs exist. Some of these programs are small enough to run on hand held programmable calculators. A complication is however added if the spillway contains manually controlled gates. Application of a program under these conditions usually requires the user to draw up a gate operation sequence for the entire period of the simulation. If the assumed operation sequence is inadequate the simulation has to be rerun with a revised sequence. This process must be repeated until a satisfactory sequence is found. This approach is both time consuming and inefficient.

For this reason various optimization techniques for reservoir operation have been developed. The majority of these techniques are stochastic dynamic programming models, as reviewed by Jery R. Stedinger et.al. (8). These algorithms are usually rerun every month to derive an optimal release for that month based on the conditional distribution of future inflows.

Although these models are useful for determining long term reservoir operating rules they are generally unable to address a single isolated flood event. Orlovski et. al. (9) states that detailed analysis made in the field of management science has proved that decision makers very seldom consider long-term expected values of physical and/or economic indicators as representative measures of system performance. Indeed, very often a manager focuses his attention and effort on avoiding dramatic failures when the system is under stress. In other words, in most cases, decision makers are risk averse even if this entails a worse average performance of the system. Reservoir managers are no exception. For this reason he proposes a min - max approach to reservoir management which indicates an area of possible solutions that the reservoir manager can apply under given conditions. This method does however suffer from the disadvantage that it develops its feasible solutions from a selected 1 year-long daily inflow sequence and therefore still does not address the individual flood event.

Guidelines to the determination of a flood control procedure for a dam equipped with crest gates were presented by S.M. Nicol (7) in 1973. The method involves prerelease at a rate equal to bank full flow for the river reach downstream of the dam, until the flood enters the dam. Outflow is then increased to correspond to the inflow rate until the remainder of the flood peak can be absorbed in st[illegible]able in the dam. Outflow is then kept constant at this rate un[illegible] outflow rate exceeds the inflow rate.

This report presents a refinement of the above technique. The method involves the calculation of an outflow rate which once achieved, subject to the gate operation constraints, would provide enough storage in the dam to absorb the inflow peak without having to readjust the outflow rate. This ultimately leads to a peak outflow rate lower than that achieved by the method presented by Nicol.

In addition the new method is presented as a calculation option in a generalized reservoir floodrouting package. It can therefore be used to eliminate the iterative procedure described earlier to determine a satisfactory operating rule for a reservoir during a flood, and also presents an optimized outflow hydrograph in the sense that the outflow peak has been minimized, by optimal use of storage in the reservoir.

At present the Department of Water Affairs are exercising real time control on the Vaal Dam spillway in order to minimize flooding in the Vereeniging area and to maximize storage in the dam.

An extensive prewarning system exists in the Vaaldam catchment and an expected inflow hydrograph can be calculated for the dam. This predicted inflow hydrograph is then used as input into a reservoir routing program. A gate operation sequence is assumed and the program is rerun until a seemingly satisfactory operating rule for the expected flood is found. This exercise is repeated every six hours, with an updated inflow hydrograph, until the flood has passed.

Although this system works it has a number of shortcomings, the most serious being that although a workable solution is obtained, it takes time and in the end there is no way of measuring the adequacy of the solution i.e. the adequacy of the initial gate operation sequence, the refinements to the sequence, and the decision that a solution is workable all lie with the program user and depend solely on his experience. The second major shortcoming lies in the fact that the reservoir routing program used was written specifically for the Vaaldam and cannot, as it stands, be used for any other dam.

The new program is a generalized reservoir program that can be applied to any dam with any spillway configuration. Although the final decision regarding the operation of a manual spillway, using this program, would still be required from the user, the optimization routine can be used to calculate an initial gate operation sequence. Depending on the accuracy of the flood prediction method used at a particular dam, and the sensitivity of the dam and its environment to large floods, the user may adapt this "optimized" operation sequence as required. This constitutes a rationalization of the decision making process required to determine an adequate and workable gate operation sequence.

2.0 DESCRIPTION OF THEORY

This chapter describes the basic theory, equations and algorithms used in the floodrouting program. The first sub-section describes all the basic spillway equations used. The second and third sub-sections describe the two algorithms used by the program to calculate the volume balance during floodrouting operations, and the fourth sub-section describes the logic employed by the program to calculate the operation of a manually controlled spillway.

2.1 DESCRIPTION OF BASIC SPILLWAY EQUATIONS

2.1.1 Introduction

Numerous equations exist to describe flow through various types of spillways. The performance of a spillway does however depend on a number of factors which, among others, include the design flood, approach conditions and discharge conditions.

Design floods are generally determined by transporting great storms, which have been known to occur in the region, over the drainage area. This transportation is usually done mathematically and in its most simple form often involves the solution of a single equation (USBR(3), p.540). On the other extreme some very large computer based models (eg. ref. 15) that take many of the catchment features into account exist. Irrespective of the method the results must invariably be calibrated against measured storms and runoffs. In many areas of South Africa the rainfall-runoff records that are available, if any, are too short to be of any real value. The resulting hydrographs are therefore at the very best only good estimates of the design flood.

In determining the required discharge capacity for a new spillway consideration should be given to all contingencies. If the spillway contains gates the possibility that one or more gates may become inoperative cannot be excluded. Under some flood conditions operation errors may increase the discharge at the dam above the natural peak flow of the river. By storing too early on the rising side of the flood hydrograph, available storage space may be filled in advance of the peak inflow. Under those circumstances it may be necessary to release a discharge in excess of the natural peak inflow.

Emergency capacity, in excess of that required to discharge the spillway design flood, may be obtained by encroaching on the freeboard, providing emergency spillways, or providing a total freeboard which would permit a sizable flood to pass over the top of the gates, if they become inoperative, without overtopping the dam.

Approach conditions, denoting the hydraulic properties of the water passages leading to the point of control in the spillway, are another important consideration. Spillways of large capacity can produce discharges with great momentum in the approach channel. Undesirable conditions in the approach channel can reduce the spillway capacity, produce troublesome disturbances, contribute to the possibility of cavitation, prevent the satisfactory passage of floating debris through the spillway, and even produce erosion and undermining of the upstream portions of the spillway structure itself. It is important therefore to evaluate the approach conditions very carefully.

Discharge conditions, denoting the hydraulic properties of the water passages leading away from the point of control in the spillway, are equally important. Flow released through the control structure is usually conveyed to the streambed below the dam in a discharge channel or waterway, the exception being the discharge from an arch dam crest which falls freely to the river bed.

As the spillway flow falls from pool level to downstream river level, either freely or through a chute or channel, the static head is converted to kinetic energy. This energy manifests itself in the form of high velocities which if impeded result in large pressures or forces. Means of safely returning the flow to the river without serious scour or erosion of the toe of the dam, damage to adjacent structures, or damage of the downstream portions of the spillway structure itself, must therefore be provided.

After a spillway control of certain dimensions has been selected, the maximum spillway discharge and the maximum reservoir water level can be determined by flood routing. The other components of the spillway can then be proportioned to conform to the required capacity and to the specific site conditions, and a complete layout of the spillway can be established. Cost estimates of the spillway and dam can then be made. Estimates of various combinations of spillway capacity and dam height for an assumed spillway type, and of alternative types of spillways, will provide a basis for selection of the economical spillway type and the optimum relation of spillway capacity to a height of dam.

2.1.2 Box Culvert Spillway

The culvert spillway ordinarily consists of a simple culvert conduit placed through the dam or along the abutment, generally on a uniform grade with the entrance placed vertically or inclined. The culvert cross-section may be round if it is constructed of fabricated or precast pipe, or it may be square, rectangular or of some other shape if cast in-situ. The culvert can discharge freely, or it can empty into an open channel so that the outflowing jet is supported along the channel floor.

The factors which combine to determine the nature of flow in a culvert spillway include such variables as the slope, size, shape, length and roughness of the conduit barrel, and the inlet and outlet geometry.

A culvert will flow full when the outlet is submerged or when the outlet is not submerged but the headwater is high and the barrel is long.

According to laboratory investigations, Ven Te Chow (4), p.495 the entrance of an ordinary culvert will not be submerged if the headwater is less than a certain critical value which varies between 1.2 and 1.5 times the height of the culvert, depending on the entrance geometry, barrel characteristics and approach conditions.

Laboratory investigations also indicate that a culvert, usually with a square edge at the top of the entrance, will not flow full even if the entrance is below headwater level when the outlet is not submerged. Under these conditions, the flow entering the culvert will contract to a depth less than the height of the culvert barrel in a manner very similar to the contraction of flow in the form of a jet under a sluice gate. This high velocity jet will continue through the barrel length, but its velocity will be reduced slowly as head is gradually lost by friction. If the culvert is not sufficiently long to allow the expanding depth of flow below the contraction to rise and fill the barrel, the culvert will never run full. Such a culvert is considered hydraulically short. Otherwise the culvert is hydraulically long.

For practical purposes culvert flow may be classified into six basic types as shown in Figure 1. Each type of flow may be identified according to the following schematic:

Description	Flow type
1. Outlet submerged	(1)
2. Outlet unsubmerged	
2.1 Headwater greater than the critical value	
i. Culvert hydraulically long	(2)
ii. Culvert hydraulically short	(3)
2.2 Headwater less than the critical value	
i. Tailwater higher than critical depth	(4)
ii. Tailwater lower than critical depth	
- Slope subcritical	(5)
- Slope supercritical	(6)

The first two types of flow are pipe flow and the other types are open channel flow. For type 3 flow, the culvert acts like an orifice and the control is therefore at the inlet. For type 4, 5 and 6 flows, the entrance is not sealed by water and it acts like a weir. Type 4 flow is subcritical throughout the barrel length. Type 5 flow is subcritical and the control section is at the culvert outlet. Type 6 flow is supercritical and control is at the inlet.

Flow through a culvert spillway may therefore occur as one of the types of flow from the above list and a number of checks are required to determine which flow type applies. In "Design of Small Dams" (3) pages 430 and further this procedure is described. However, most culverts used as spillways run through the dam wall and are therefore usually short in length and steep in slope and seldom run full. In other words, culvert spillways are usually inlet controlled.

The discharge for an inlet controlled box culvert may be computed by the following equations :-

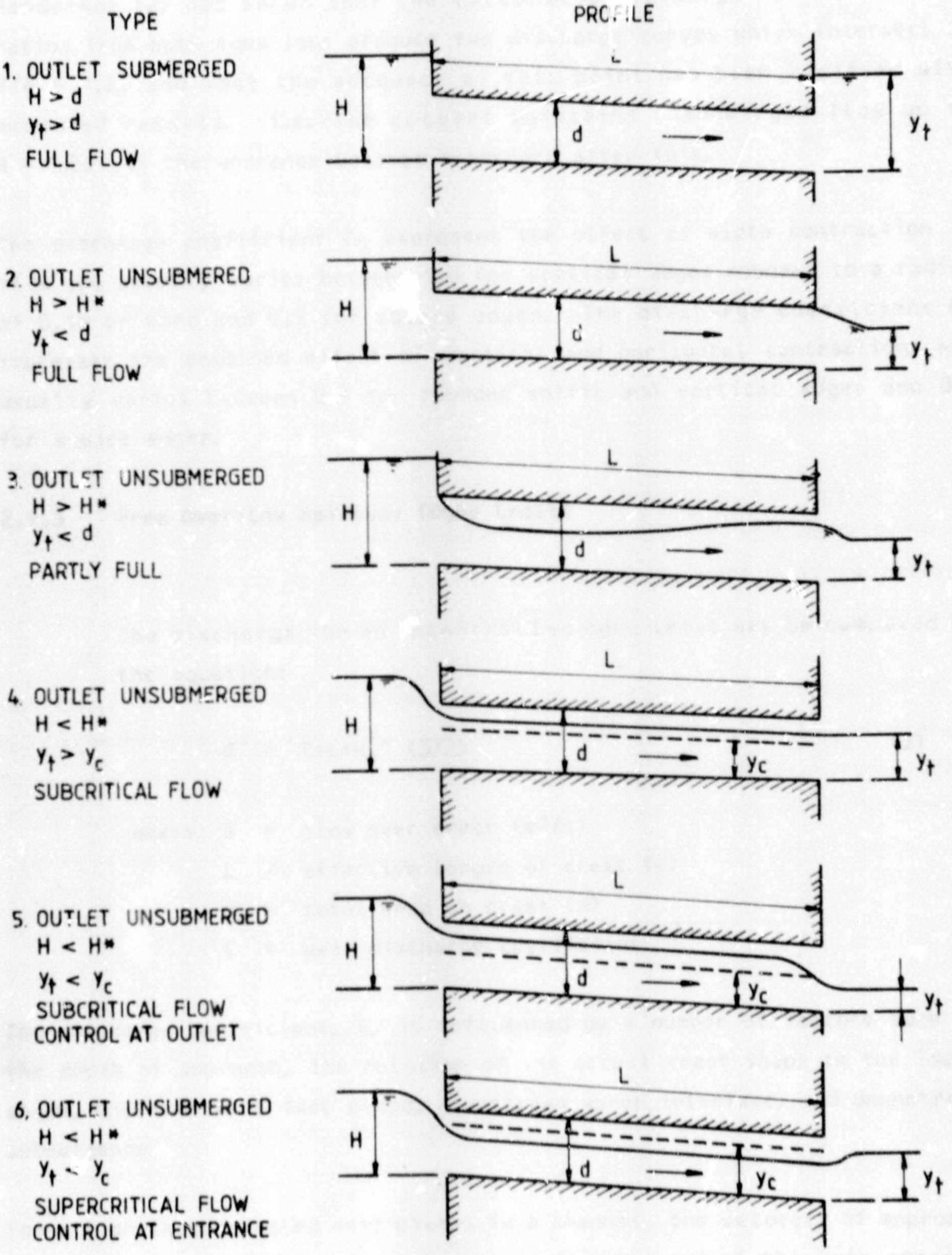
$$Q = 2/3*C_B*B*H*SQR(2/3*G*H) \tag{1}$$

when H/D<=1,2 - unsubmerged flow

and $$Q = C_h*B*D*SQR(2*G*(H-C_h*D)) \tag{2}$$

when H/D>1,2 - submerged flow

where Q = flow from one culvert (m^3/s)
B = width of culvert (m)
D = height of culvert (m)
G = acceleration due to gravity (usually 9,81 m/s^2)
C_B = unsubmerged discharge coefficient
C_h = submerged discharge coefficient



TYPES OF CULVERT FLOW

FIGURE 1

Henderson (2) has shown that the calculated discharge for various H/D ratios from both equations produce two discharge curves which intersect at H/D = 1.2, and that the adequacy of this point has been verified with measured results. i.e. The culvert maintains unsubmerged flow up to H = 1.2D and the entrance becomes submerged after this.

The discharge coefficient C_B expresses the effect of width contraction in flow and usually varies between 1,0 for vertical edges rounded to a radius of 0,1B or more and 0,9 for square edges. The discharge coefficient C_h expresses the combined effect of vertical and horizontal contractions and usually varies between 0,8 for rounded soffit and vertical edges and 0,6 for square edges.

2.1.3 Free Overflow Spillway (Ogee Crest)

The discharge for an uncontrolled ogee crest may be computed by the equation:

$$Q = C*L*H_e\hat{}\ (3/2) \qquad (3)$$

where Q = flow over crest (m^3/s)
L = effective length of crest (m)
H_e = total head on crest (m)
C = weir discharge coefficient

The discharge coefficient, C, is influenced by a number of factors such as the depth of approach, the relation of the actual crest shape to the ideal shape, upstream wall face slope, downstream apron interface, and downstream submergence.

For a high sharp-crested weir placed in a channel, the velocity of approach is small and the underside of the nappe flowing out of the weir attains maximum vertical contraction. As the approach depth decreases, the velocity of approach increases and the vertical contraction diminishes. If the sharp-crested weir coefficients are related to the head measured from the point of maximum contraction (Ho) instead of to the head above the sharp crest, coefficients applicable to ogee crests shaped to profiles of under nappes for various approach velocities can be established. The relationship of the ogee crest coefficient, C, to various values of the dam wall height P, is shown in Figure 2 (Figure 2 is a metricated version of Figure 249 from "Design of small dams". (3)). These coefficients are valid only when the ogee is formed to the ideal nappe shape.

When the ogee crest is formed to a shape differing from the ideal shape or when the crest has been shaped for a head larger or smaller than the one under consideration, the coefficient of discharge will differ from that shown in Figure 2. A widened shape will result in positive pressures along the crest contact surface and thus reduce the discharge. A narrower crest will result in negative pressures along the contact surface and thus increase the discharge. Figure 3 shows the variation of the coefficient as related to values of He/Ho, where He is the actual head being considered. (Figure 3 has been drawn from Figure 250 from "Design of small dams"(3)).

For small ratios of approach depth to head on the crest, sloping the face of the overflow results in an increase in the coefficient of discharge. For large ratios the effect is a decrease in the coefficient. Figure 251, from "Design of small dams" (3) shows the ratio of the coefficient for an overflow ogee crest with a sloping face to the coefficient for a crest with a vertical upstream face as obtained from Figure 2 and adjusted by Figure 3 if appropriate, as related to values of P/Ho.

The effect of downstream apron interference and downstream submergence shall not be discussed here as they do not usually influence flow from ogee crests used as reservoir spillways. For a discussion of these effects the user is referred to paragraph 197 (d) on page 376 of "Design of Small Dams" (3).

The total head on the crest, He, does not include allowances for approach channel friction losses or other losses due to curvature of the upstream channel, entrance loss into the inlet section, and inlet transition losses.

Where the design of the approach channel results in appreciable losses, they must be added to He to determine reservoir elevations corresponding to the discharges given by the above equation.

Where crest piers and abutments are shaped to cause side contractions of the overflow, the effective length, L, will be less than the net length of the crest. The effect of the end contractions may be taken into account by reducing the net crest length using the following equation:

$$L = L^{1} - 2*(N*K1+K2)*H_e \tag{4}$$

where L = effective length of crest (m)

L^1 = net length of the crest (m)

N = number of piers (if any)

K1 = pier contraction coefficient

K2 = abutment contraction coefficient

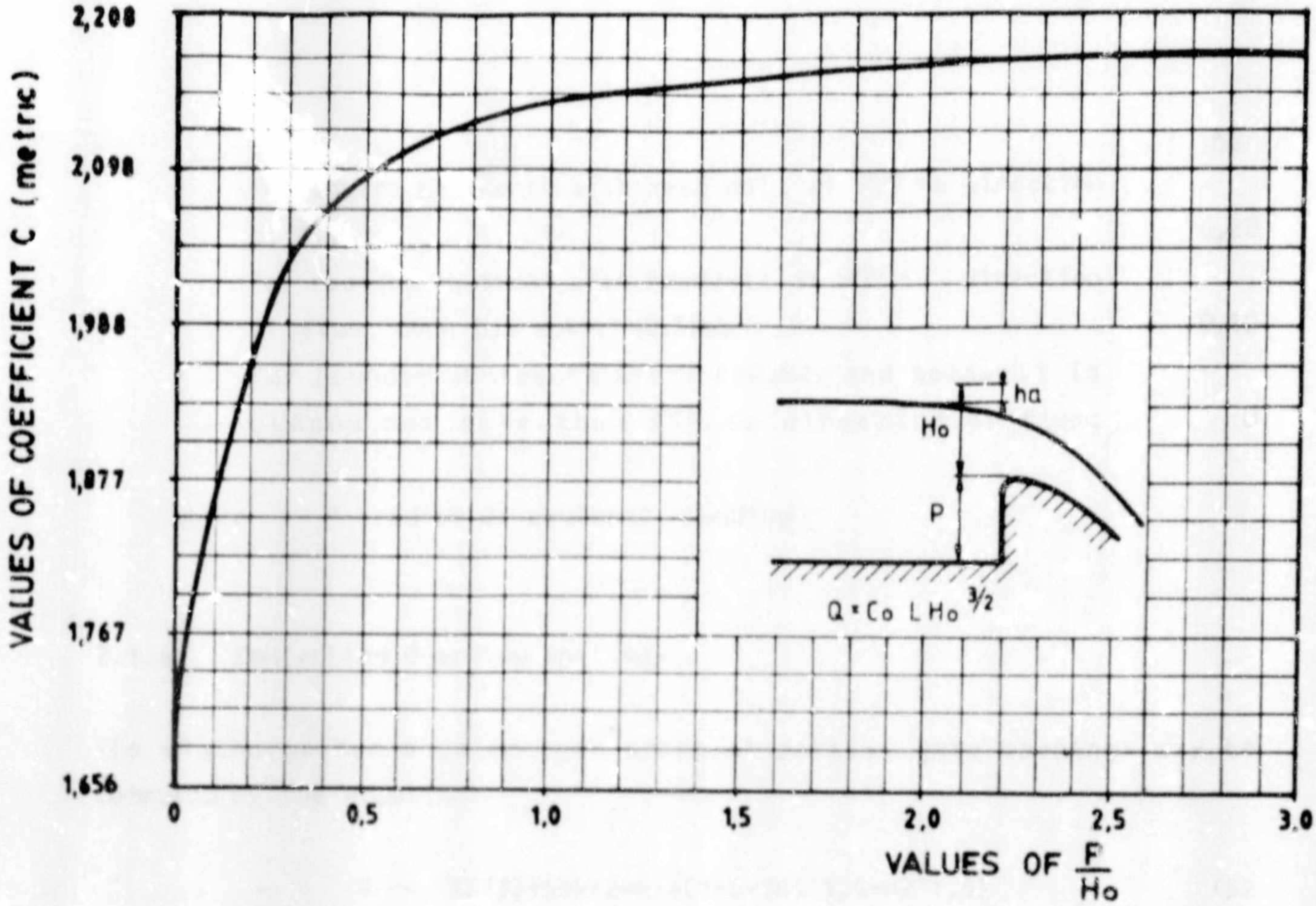
H_e = total head on crest (m)

In "Design of Small Dams" (3) the following values for the coefficients K1 & K2 are suggested.

For conditions of design head (Ho) average pier contraction coefficients (K1) may be assumed as follows:

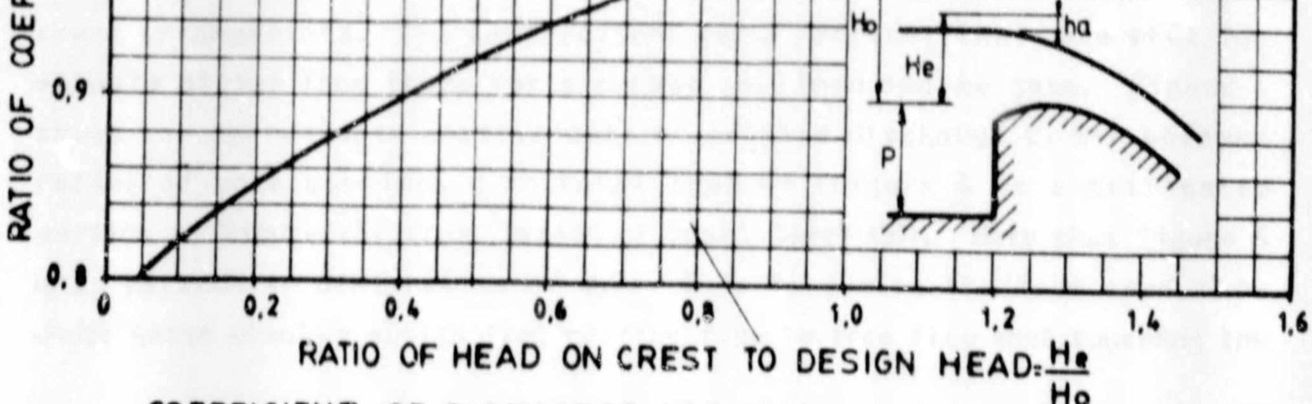
	K1
- For square - nosed piers with corners rounded on a radius equal to about 0,1 of the pier thickness:	0,02
- For round - nosed piers:	0,01
- For pointed - nose piers:	0

For conditions of design head (Ho) average abutment contraction coefficients (K2) may be assumed as follows:



DISCHARGE COEFFICIENTS FOR VERTICAL - FACED OGEE CREST

FIGURE 2



COEFFICIENT OF DISCHARGE FOR OTHER THAN THE DESIGN HEAD

FIGURE 3

		K2
-	For square abutments with head wall at 90° to direction of flow:	0,20
-	For rounded abutment with headwall at 90° to direction of flow, when 0,5 Ho>=r>=0,15Ho:	0,10
-	For rounded abutments where r>0,5Ho and headwall is placed not more than 45° to direction of flow:	0

where r = radius of abutment rounding

2.1.4 Controlled Overflow Spillway

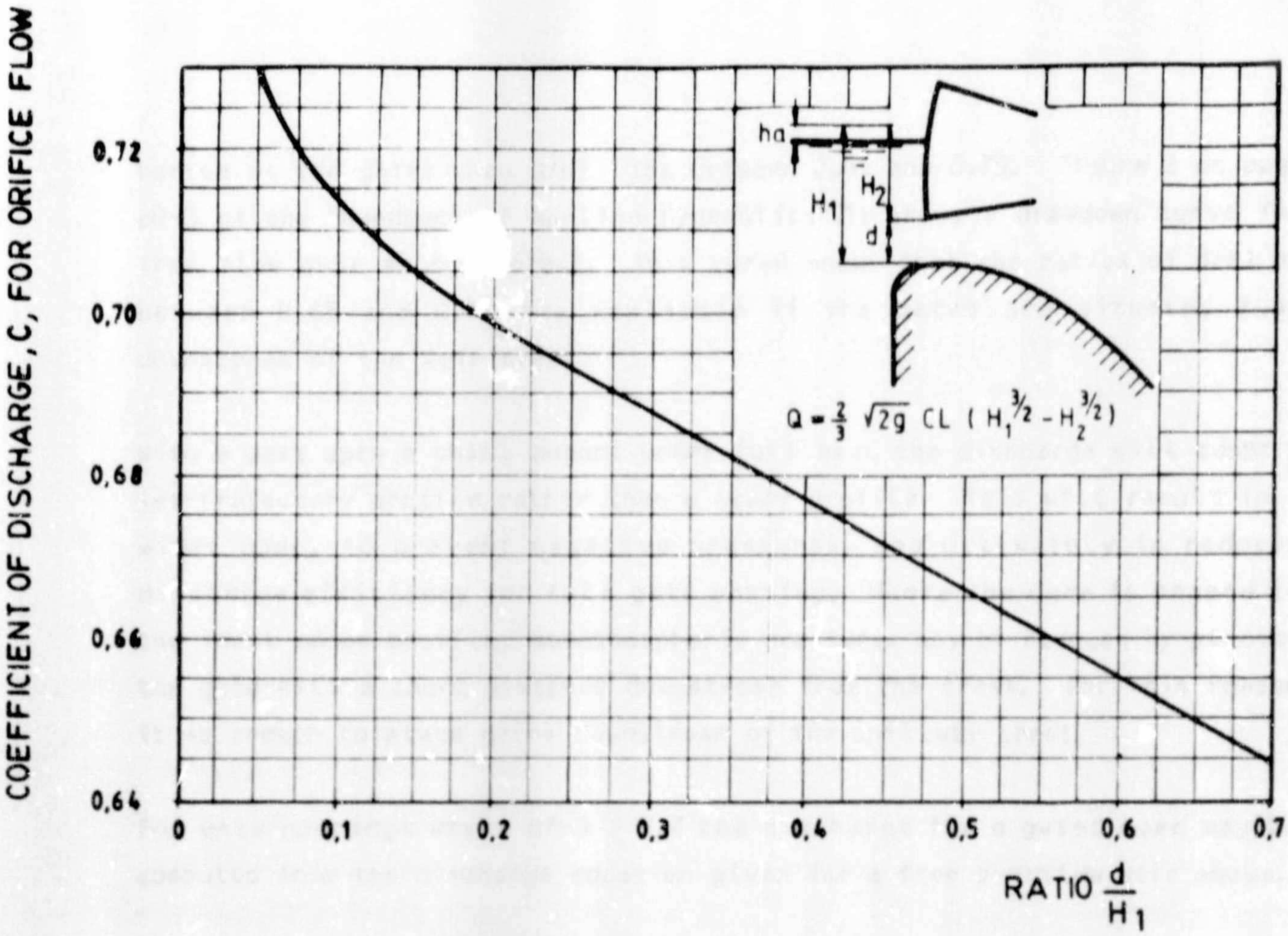
The discharge for a gated ogee crest at partial gate openings may be computed by the equation:

$$Q = (2/3)*SQR(2*G)*C1*L*(H1^{1,5}-H2^{1,5}) \quad (5)$$

where Q = flow from one gate (m^3/s)
G = acceleration due to gravity (usually 9,81 m/s^2)
L = gate width (m)
H1 = total head (including velocity head of approach) to bottom of orifice (m)
H2 = total head to top of orifice (m)
C_1 = discharge coefficient for orifice flow

also let d = height of gate opening (m)

The orifice discharge coefficient C1 will differ with different gate and crest arrangements. The contractions for a vertical leaf gate will for example differ from those for a curved inclined radial gate. Figure 4 shows the approximate coefficients of orifice discharge C1 for various ratios of gate openings d to total head H1 (Figure 4 is a metricated version of Figure 257 from "Design of Small Dams" (3)). Note that Figure 4 only extends to d/H1 ratios of 0,7. This is due to the fact that flow under gates usually switch from orifice flow to free flow (not touching the



COEFFICIENT OF DISCHARGE FOR FLOW UNDER GATES

FIGURE 4

bottom of the gate) when d/H1 lies between 0,65 and 0,75. Figure 2 on page 20-3 of the "Handbook of Applied Hydraulics"(1) shows a drawdown curve for free flow over an ogee crest. This curve shows that the ratios of d/H1 of between 0,65 and 0,75 are realistic if the gates are situated just downstream of the weir crest.

With a gate open a small amount under full head the discharge will adopt a jet-trajectory profile rather than a nappe profile. This will result in a wider ogee, to prevent negative pressures, and ultimately in reduced discharge efficiency for full gate opening. Where the ogee is shaped to the ideal nappe profile, subatmospheric pressures may be reduced by placing the gate sill a short distance downstream from the crest. For this reason it is common to place gates downstream of the spillway crest.

For gate openings where $d/H1 \geq 0,7$ the discharge for a gated ogee may be computed from the discharge equation given for a free overflow weir above.

2.1.5 Controlled High Pressure Bottom Outlet

The discharge for a controlled high pressure bottom outlet may be computed by the equation for orifice flow:

$$Q = C*A*SQR(2*G*H) \tag{6}$$

where Q = flow from one outlet (m^3/s)

A = area of the opening

G = gravitational acceleration (usually 9,81 m/s^2)

H = total head to the centreline of the orifice (m)

C = orifice discharge coefficient

An orifice is a geometric opening in a thin walled tank or dam wall. The usual shape is circular with either a sharp edge or a square upstream edge and downstream bevel. Due to the vertical component of the flow near the wall, the streamlines continue to converge for a short distance downstream of the orifice. They become parallel at a point of minimum cross-section called the vena contracta. The pressure here is atmospheric and, since the streamlines are close together, the velocity V_c is approximately constant over the section. This velocity may be determined by applying Bernoulli's equation to the free surface and the vena contracta to yield:

$$h = V_c{}^2/2g \tag{7}$$

In the case of a real fluid there is a small energy loss due to viscous effects and this necessitates the introduction of a coefficient of velocity. The discharge is then equal to $a_c V_c$ where a_c is the sectional area of the jet at the vena contracta. It is more convenient, however, to express the discharge in terms of the orifice area A which is constant and easily measured. This necessitates the introduction of a coefficient of contraction. Combining the coefficient of velocity and the coefficient of contraction we get the orifice discharge coefficient C.

Various forms of mouthpiece may be fitted which will alter the pattern of the streamlines and therefore the value of the coefficient C. It is therefore suggested that the coefficient C be determined experimentally for a particular spillway before this equation is used.

2.1.6 Fuseplug Spillway

A fuseplug spillway is a spillway which has been plugged to a predetermined level using an erodable material. Flow through the spillway is therefore zero until water reaches the top of the plug. Flow over the plug erodes it away to expose the spillway at a lower level. Once the spillway has been exposed it operates as an uncontrolled spillway. The discharge for an operating fuseplug spillway is calculated from an equation relevant to the spillway shape used.

The flow rate required over the top of the plug to start the washout depends on a number of factors including the type of material used to construct the plug, the type and amount of vegetation that has been allowed to grow on the plug surface, the age of the plug and the antecedent moisture condition of the material in the plug. These factors will invariably also influence the rate of washout. A given plug may therefore wash out at a different rate under different conditions.

Due to this great variance potential it is extremely difficult to model a fuseplug washout with any degree of certainity. The RESIMO package provides a fuseplug option which assumes instantaneous washout. Although this can never happen in practice the assumption yields acceptable results for a number of reasons which include:

- A fuseplug is an emergency spillway which will only operate under extreme conditions. Generally the flow volumes under these conditions are large in relation to the volumetric inaccuracy in the assumption i.e. the inaccuracy inherent in the assumption is a very small percentage of the overall calculation.

- During a simulation, spillway performance is averaged over a time step. If the modelling time step is longer than the washout time (which is likely) the assumption of instantaneous washout yields a final calculated result of a washout equal to the modelling time step.

2.1.7 User Defined Spillway

A user defined spillway can describe any conceivable spillway configuration. No equation is used for the calculation of discharge. A table, relating discharge to total head, is drawn up either from calculations or more usually from model or prototype tests. This table is then used to read off discharge for a particular head during simulation calculations. Intermediate values are usually interpolated linearly. As spillway discharge curves are almost invariably non-linear, the user should ensure that the discharge-head points are chosen in such a way that linear interpolation would yield acceptable results.

2.2 DESCRIPTION OF STORAGE INDICATION METHOD OF FLOOD ROUTING

If the continuity of flow through a river or reservoir reach is considered, it is clear that what enters the reach must either emerge or temporarily move into storage.

$$I = Q + \frac{ds}{dt} \tag{8}$$

where I = inflow to the reach

Q = discharge from the reach

$\frac{ds}{dt}$ = rate of change in reach storage with respect to time

This equation is approximated, for a time interval Δt, by:

$$\tfrac{1}{2}(I1 + I2)\Delta t = (S2 - S1) + \tfrac{1}{2}(Q1 + Q2)\Delta t \tag{9}$$

where I1 and I2 = inflow at beginning and end of period

S1 and S2 = storage at beginning and end of period

Q1 and Q2 = outflow at beginning and end of period

Δt = routing period

The routing period, Δt, must be chosen sufficiently short, so that the assumption implicit in the equation, i.e. that the inflow and outflow hydrographs consist of a series of straight lines, does not depart too far from actuality. In particular, if Δt is too long it is possible to miss the peak of the inflow hydrograph, so that the period should be kept shorter than the travel time of the flood wave crest through the reach. On the other hand, the shorter the routing period the greater the amount of computation required.

We can rewrite equation (9) to the form:

$$S1 + \frac{\Delta t}{2}(I1 + I2) = S2 + \frac{\Delta t}{2}(Q1 + Q2)$$

therefore
$$S1 + \frac{\Delta t}{2}(I1 + I2) - \frac{\Delta t Q1}{2} = S2 + \frac{\Delta t Q2}{2}$$

$$(S1 + \frac{Q1\Delta t}{2}) - Q1\Delta t + \frac{\Delta t}{2}(I1 + I2) = (S2 + \frac{\Delta t Q2}{2}) \tag{10}$$

All terms on the left hand side of equation (10) are known and a value for $S2 + \frac{\Delta t Q2}{2}$ can be obtained directly. The corresponding value of Q2 is then obtained from the relationship connecting storage and discharge. This method was first developed by L.G. Puls of the U.S. Army Corps of Engineers (see Wilson (6)).

The simplest case is that of a reservoir receiving inflow at one end and discharging through a spillway at the other. In such a reservoir it is assumed that there is no wedge storage i.e. that the water surface is level, and that the discharge is a function of the surface elevation, in other words that the spillway arrangements are either free over or through flow or gated with fixed gate openings. Reservoirs with gates or sluices may also be treated as simple reservoirs if the gates or sluices are opened to defined openings at specified surface-water levels, so that an elevation - discharge curve may be drawn. (e.g. Automatic float controlled gates) The other required data are the elevation - storage curve of the reservoir and the inflow hydrograph.

If a spillway consists only of manually controlled components an iterative search method of flood routing (described in chapter 2.3) is more appropriate. However, if the spillway contains a manual component in parallel with one or more non-manual components the storage indication method, modified as described below, would require less computational effort than an iterative search technique.