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Determination of Ungated Ogee Spillway Length so as to Pass Design Flood Safely

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ABSTRACT

Because the flowrate over an ungated ogee spillway depends on the net head of the water conveyed in the approach channel and because the energy losses depend on the flowrate, computation of the spillway discharge for a given gross head of water in the reservoir entering the approach channel necessitates a trial-and-error scheme. For given information of (a) the elevations of the lake water surface and of the spillway apex, (b) the energy loss coefficient at the entrance, the side wall inclination, the length, and the roughness coefficient of the trapezoidal approach channel, an iterative method for computing the discharge over an ungated ogee spillway is presented. Next, for given information of (a) the (volume) ↔ (water surface elevation) relationship of the reservoir, (b) the spillway apex elevation, (c) the maximum allowed lake water surface elevation, and (d) the design flood hydrograph, an iterative method for computing both the length of the ungated ogee spillway and the outflow hydrograph simultaneously such that the maximum water surface elevation reached during routing of the design flood hydrograph becomes equal to the maximum allowed elevation is presented. Matching of the maximum water surface elevation reached in the reservoir while routing of the design flood hydrograph to the pre-specified maximum lake elevation requires a trial-and-error scheme of reservoir routing computations over many different-length spillways. The iterative method presented in this study which is executed in a single run of the coded computer program is a short-cut alternative to the long approach. The developed method is applied to Catalan Dam, which is one of the large dams in Türkiye from reservoir capacity, flood attenuation, and hydroelectricity production aspects, as a case study. The length of an ungated spillway is computed by the method presented here as an alternative to the existing radial-gated spillway, and the reservoir routing computations are done with the design flood hydrograph given in its final project by both the ungated and the gated spillways, and the maximum lake water surface elevations and the outflow hydrographs are found to be fairly close to each other.

Keywords: Ogee profile ungated spillway, reservoir flood routing.

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1. INTRODUCTION

The book *Design of Small Dams* [1] is a commonly used classical reference for design of dams, and Yedigöze and Bayramhacılı Dams are just two of such cases [2, 3]. The method for computing the discharge passing over an ogee profile ungated (free flow) spillway for a given gross head over the spillway crest is described in detail with a couple of examples in that book. The spillway height is determined based on topographic conditions of the spillway site and on the experience of the designer. Conventionally, the design flood is the probable maximum flood for earth-fill dams. It may be a smaller magnitude flood like that having an average return period of 10,000 years [4]. The flood spillway will rout the design flood from the reservoir such that the maximum water surface elevation reached in the surcharge storage is below the dam crest elevation by a predetermined freeboard. The maximum water surface elevation in the lake is a critical value calculated by the economic cost incurred by inundation of the areas upstream of the dam. The dead storage is computed by the volume of sediment expected to settle behind the dam over a service life of about 100 years. The needed volume of the active storage above the dead storage is determined by a simulation operation model based on the projected yield from the dam, which can be as high as 90 % of the long-term average inflow, either in monthly or daily time steps over a critical dry period of 3 or 4 years.

There have been a few theoretical and experimental studies about the hydraulics of flow over free flow ogee spillways [e.g., 5, 6, 7, 8, 9, 10, 11, 12, 13]. Savage and Johnson [5] performed an experimental study with 10 different free flow conditions ranging from 7 % up to 120 % of the design head over an ogee-crested spillway model having a length of 1.83 m and a height of 0.80 m constructed with Plexiglas molded so as to conform to the exact shape of the designed ogee profile. And, they [5] concluded that there was reasonably good agreement among (1) the experimentally observed flowrates, (2) theoretically obtained flowrates from the FLOW-3D package program, and (3) the flowrates computed using the pertinent design charts from the book: *Design of Small Dams*. Chatilla and Tabbara [6] applied the ADINA-F software package which depicts the free surface flow over an ogee spillway by k-ε turbulent flow model to a few experimentally observed flows created in a long flume with glass side walls in their hydraulics laboratory and reported good agreement between the experimental and computational flowrates and the surface profiles. Alhashimi [7] performed experiments on the laboratory model of the ungated ogee spillway of Mandali Dam in Iraq and compared the experimental results with the theoretical values given by the FLUENT software package which modeled the spillway flow by k-ε turbulent flow, and found that the computational and experimental values differed about 3.7 % and 7.4 %. Goharrizi and Moghadam [8] analyzed the data obtained from experiments with various different heads and flowrates on a spillway model made of Plexiglas in the hydraulics laboratory. They also computed the relevant flow parameters by the ANSYS CFX software package applying the k-ε turbulent flow model and they concluded that both the experimental and theoretical values were close to each other and also the relevant charts in *Design of Small Dams* yielded close values to the experimentally observed ones. Salmasi and Abraham [9] performed experiments on a 22 cm high ogee spillway model in a 10 m long and 25 cm wide glass channel in the Hydraulics Laboratory of Tabriz University with various configurations of flowrates and heads, and they found that the main spillway discharge coefficient was a function of P/He (where, P is the spillway height and He is the net head over the spillway crest) similar to the main chart for this coefficient in *Design of Small Dams*, and their experimentally determined magnitudes for the discharge coefficient were close to those in the relevant chart in *Design of Small Dams*.

Kocaer and Yarar [10] performed experiments on an ogee spillway model in the hydraulics laboratory with various flowrates and heads and compared the magnitudes of the experimentally measured quantities with those given by two software packages of ANSYS-Fluent and OpenFOAM applying two different turbulence models of k-ε and k-ω, and concluded that these detailed finite-element models yielded close values to the experimentally observed ones. Fadafan and Kermani [11] applied the Moving Particle Semi-implicit (MPS) method, which is basically solution of partial differential equations of continuity and momentum with the help of some analytical arrangements, along with the software package FLOW-3D to the experimentally observed flows over a spillway model in the hydraulics laboratory, and they indicated that their MPS approach gave results close to both the experimental values and those given by FLOW-3D. Yildiz et al [12] performed experiments on a 7 cm high ogee spillway model in a laboratory flume of 7.5 cm width with various heads and flowrates, and they applied an adaptive neuro-fuzzy inference system (ANFIS) model along with the software package FLOW-3D to the flows over the spillway, and they concluded that both the ANFIS and FLOW-3D models yielded values close to the experimentally observed ones. Yildiz et al [12] also concluded that the relevant charts in *Design of Small Dams* gave values close to the experimentally observed values. Kumcu et al [13] after having performed detailed experimental studies on the 1/50 scale ogee spillway model of Kavşak Dam on Seyhan River in Türkiye in the Hydraulics Model Laboratory of the Research and Quality Control Department of the General Directorate of State Water Works concluded that for the free flow case the experimentally determined stage ↔ discharge relationship covering all magnitudes of flowrates up to the maximum design discharge was very close to the theoretically computed relationship using the relevant charts in *Design of Small Dams*. Although spillways other than ogee-crested ones such as labyrinth weirs are designed and constructed for some dams as mentioned by Daneshfaraz et al [14], still the ogee spillways are more common.

Quite a few experimental and theoretical studies related to ungated ogee spillway flows are summarized in the above paragraph. Some of those studies used one of popular finite-element software packages simulating the spillway flow by thousands of meshes. Naturally, they are respectable but mostly academic studies. Usage of the mentioned software packages requires special training and most of design engineers do not know how to use them. And yet, some of those studies indicated that the relevant design charts in the book: *Design of Small Dams* gave close values to the experimentally observed flows. It can be easily noticed that none of those studies are similar to the study summarized in this Technical Note. The objective of our study is computation of both the length of the ungated ogee spillway and the outflow hydrograph from that spillway simultaneously for given four peculiarities: (1) the incoming design flood hydrograph, (2) the spillway apex elevation, (3) the maximum allowed elevation of the surcharge storage, (4) the (valley volume)↔(water surface elevation) relationship of the reservoir. The developed method and the computer program executing it presents both the length of the designed spillway and the outflow hydrograph of the incoming design flood routed over the reservoir simultaneously such that the maximum water surface elevation reached equals the maximum allowed surcharge storage elevation.

2. METHOD

2.1. Review of Discharge from Ungated Ogee Spillway by *Design of Small Dams*

Figure 9-23 in the *Spillways* chapter of *Design of Small Dams* presents the discharge coefficient, C_0, versus the term P/H_0, where P is the height of spillway and H_0 is the design head (including the velocity head) over the spillway apex elevation [1]. Figures 9-24, 9-25, and 9-27 give the charts for correction coefficients accounting for the effects of a water head different from the design head, inclination of the upstream face of the spillway, and the apron effect caused by the supercritical flow at the downstream toe of the spillway, respectively [1]. In *Design of Small Dams* and in some other relevant publications [1, 15], the below equation is given for the discharge passing over an ungated (free flow) ogee profile spillway.

$$Q = C_{net} \cdot L_e \cdot H_e^{1.5} \tag{1}$$

Here, Q is the spillway discharge in ft^3/s, C_{net} is the net discharge coefficient, H_e is the total head (including the velocity head) at the downstream end of the approach channel with respect to the spillway apex elevation in ft, and L_e is the effective spillway length in ft, which equals the net spillway length excluding the widths of the piers if any minus a correction value due to the contraction effect by the piers and the approach abutments. Equation (1) is not dimensionally homogeneous, and C_{net} has the units of $ft^{0.5} \cdot s^{-1}$. Therefore, when equation (1) is used in metric system, C_{net} of the foot-pound-second system must be divided by 1.811309 which is equal to $(3.28084)^{0.5}$.

Mostly there is a fairly long chute channel having a rather steep slope conveying the spilled discharge down to the energy dissipating structure. Rarely for some small dams, there is not a chute channel and the spilled flow discharges directly into a stilling basin. For such small structures usually a drowned hydraulic jump occurs at the downstream face of the spillway and a correction factor depicted by Figure 9-28 of *Design of Small Dams* is needed. As seen in this figure however, even for cases of small submergence the effect on discharge is negligible. Hence, for spillways having steep chute channels, the net discharge coefficient, C_{net}, of equation (1) is computed by

$$C_{net} = C_0 \cdot C_{He/Ho} \cdot C_{incl} \cdot C_{aprn} \tag{2}$$

Here, C_0 is the major discharge coefficient for ogee spillways having vertical upstream faces and it depends on the ratio of (height of the ogee spillway)/(design head), P/H_0. The second coefficient, $C_{He/Ho}$, gives the correction factor for any head, H_e, other than the design head, H_0. The third coefficient, C_{incl}, gives the correction factor for the inclination of the upstream face of the spillway. The fourth coefficient, C_{aprn}, gives the correction factor for the apron effect of the supercritical flow at downstream toe of the spillway, which is determined as a function of the ratio: (flow depth + velocity head at downstream end)/H_e. The overall ranges of the three correction coefficients vary in the intervals of: $0.8 < C_{He/Ho} < 1.08$, $0.99 < C_{incl} < 1.04$, $0.77 < C_{aprn} < 1.00$ [1]. The major coefficient, C_0, and the correction factor, C_{incl}, have fixed values for a particular spillway structure for any head, H_e. The other two correction factors, $C_{He/Ho}$ and C_{aprn}, take on different magnitudes for every different head, H_e, and therefore, the net discharge coefficient, C_{net}, also assumes different magnitudes for different H_e's.

2.2. Spillway Discharge under a Given Head

If the head loss due to entrance into the approach channel and the friction loss along the channel were neglected, then H_e in equation (1) would be: H_e = (lake water surface elevation at that instant) – (spillway apex elevation). Accurately however, H_e should be computed by

$$H_e = WSE - SAE - (hl_{entrance} + hl_{friction}) \tag{3}$$

Here, WSE is the lake water surface elevation at that moment, SAE is the spillway apex elevation, $hl_{entrance}$ is the head loss incurred during entrance into the approach channel, and $hl_{friction}$ is the total friction loss from the beginning to the end of the approach channel. $hl_{entrance}$ is computed as a fraction of the velocity head in the channel by

$$hl_{entrance} = C_{entr}\cdot[Q^2 / (A_{apprch}{}^2\cdot 2g)] \tag{4}$$

where, C_{entr} is the entrance loss coefficient, for which a value in the range: 0.1 and 0.3 is recommended [1], A_{apprch} is the cross-sectional area of the approach channel while conveying the spilled discharge Q, and g is the acceleration of gravity. $hl_{friction}$ is computed with the help of the Manning equation by

$$hl_{friction} = L_{apprch}\cdot(Q\cdot n\cdot P_{apprch}{}^{2/3} / A_{apprch}{}^{5/3})^2 \tag{5}$$

Here, L_{apprch} is the length of the approach channel, P_{apprch} and A_{apprch} are the wetted perimeter and the flow area in the channel, and n is the Manning roughness coefficient of the channel. Equation (5) in this form is valid in metric system of units, and n should be replaced by (1.49)·n if it is to be used in foot-pound-second system.

If the approach channel is not short and if its abutments have dull shapes, the head losses, $hl_{entrance}$ and $hl_{friction}$, will assume non-negligible magnitudes. Usually, their sum is of the order of a couple of decimeters at most. In the realistic case of accounting for the approach channel head losses, because Q depends on H_e by equation (1), while H_e is related to Q according to equations (3), (4), and (5), computation of the spillway discharge at any lake water surface elevation necessitates a trial-and-error procedure. The iterative numerical algorithm as a practical alternative put forth in this study is summarized in the following.

(i) Assume ($hl_{entrance} + hl_{friction}$) = 0 and compute an initial estimate for H_e denoting it by H_{e1}.

(ii) Inserting H_{e1} for H_e, compute an initial estimate for the spillway discharge, Q, by equations (3), (2), and (1), executing them in this order.

(iii) With Q computed at previous step, compute $hl_{entrance}$ and $hl_{friction}$ by equations (4) and (5).

(iv) Inserting the magnitudes of $hl_{entrance}$ and $hl_{friction}$ in equation (3) compute the improved magnitude for H_e denoting it by H_{e2}.

(v) Compute the absolute relative difference between H_{e2} and H_{e1} by:

$$ARD = \left| (H_{e2} - H_{e1}) / H_{e2} \right| .$$

(vi) If ARD $\leq 1\cdot 10^{-6}$, then $H_e = H_{e2}$ and the spillway discharge at that lake water surface elevation equals the magnitude of Q computed at step **(ii)**, and the iterations stop here.

(vii) If RD > $1\cdot10^{-6}$, then make the assignment: $H_{e1} = H_{e2}$ and go to step **(ii)** above and repeat the iterations.

This algorithm converges in two or three iterations and Q under that WSE is hence computed.

2.3. Spillway Length For Given Design Flood Hydrograph and Spillway Design Head

Determination of the length of the free flow spillway such that when the design inflow hydrograph is routed over the reservoir, the maximum water surface elevation reached in the reservoir be close to a pre-specified maximum allowed elevation requires a trial-and-error approach involving a few reservoir routing computations with many different-length spillways [e.g., 1, 15]. In this study, we propose a numerical approach which eliminates this long method of trials. Our method simultaneously determines (a) the length of that free flow spillway such that the maximum water surface elevation reached in the reservoir is equal to the pre-specified maximum allowed elevation and (b) the outflow hydrograph resulting from routing of the incoming design flood hydrograph. In the following, the method devised in this study is explained.

The dead storage and the active storage capacity of the reservoir of a dam are computed by the known methods. The apex elevation of the ungated flood spillway equals the elevation of the top of full active storage. The maximum allowed water surface elevation to occur during passage of the design flood from the reservoir is determined beforehand independently from reservoir routing computations based on comprehensive economic analyses to maximize the financial difference of benefits on account of water releases from the active storage and costs due to the inundated upstream areas. And, the spillway design head, H_0, is computed by

$$H_0 = \mathrm{MWSE} - \mathrm{SAE} \qquad (6)$$

Here, MWSE is the maximum allowed lake water surface elevation, SAE is the spillway apex elevation. Next, the length of the spillway whose design head is H_0 is computed by an iterative procedure. The initial estimate of the spillway length is made by equation (1) assuming the maximum discharge of the outflow hydrograph, which is the same as the maximum spillway discharge, equals 70 % of the peak of the incoming design flood hydrograph. The spillway length which makes the spillway discharge equal to the peak of the outflow hydrograph under the design head of H_0 is computed by a recursive algorithm. At the last one of the iterations, the outflow hydrograph is also determined along with the correct spillway length.

The shape of the downstream face of an ogee spillway with a design head of H_0 closely resembles the lower surface of the nappe of the freely shooting flow over a sharp-crested weir [1, 15]. Hence, the water flowing over the ogee profile spillway is not carried by its downstream part and the efficiency of spillage becomes almost as high as that of the free flow over the sharp-edge spillway of the same height. If it were not a free jet but rather an open channel flow carried by the downstream part of the spillway, the discharge in that case would be smaller than the free jet due to wall friction losses. Because it is almost a freely shooting nappe the hydrostatic pressure of the spilled water on the bottom of the downstream face of the spillway is close to zero. For a flowrate smaller than the design discharge, the nappe profile of the spilling water is shorter than that of the design discharge, and the downstream face of the spillway begins acting like a channel causing a reduction in the discharge coefficient C_0 hence in C_{net}. In this case, the pressure on the downstream face is positive, and the spilling water does not have a tendency to separate from the crest curve, which is an advantage actually along with the disadvantage of reduction in C_{net}. According to Figure 9-24 of *Design of Small Dams*, the highest drop in C_{net} is $C_{He/Ho}$ = 0.80. In other words, alleviation of the possibility of cavitation damage versus a reduction in discharge efficiency of at most 20 % actually is an advantage rather than a disadvantage. Again, according to the same figure, for heads greater than the design head (H_e>H_0) the discharge efficiency increases as much as 8 %. This is because the spilling jet shoots farther away under a head greater than the design head and the suction induced by the negative pressure incites a higher flow. This slight increase in discharge efficiency may seem to be advantageous; but, along with it cavitation damage to the concrete material of the spillway may take place. Therefore, in this study, the maximum discharge over the spillway is not allowed to exceed the design discharge, and the head forming at the maximum spillway discharge during routing of the design flood hydrograph is taken equal to the spillway design head, H_0.

The volume of the valley upstream from the dam between the maximum allowed water surface elevation and the top of full active pool is known as the surcharge storage. The ratio of the surcharge storage to the total volume of the design flood hydrograph is directly effective on abating the peak of the outflow hydrograph. The surcharge storage for dams whose service objectives do not include flood damage reduction is usually small, and the peak of the outflow hydrograph becomes close to the peak of the inflow hydrograph of the design flood. In this study, the peak of the outflow hydrograph is made equal to the discharge of the spillway occurring under the design head, and this condition is satisfied by an iterative method, which is summarized in the following step by step.

(1) At the arrival of the design flood to the reservoir, the lake water surface elevation equals the spillway apex elevation (WSE = SAE). First, the incremental time step, Δt, is chosen. For dams having large reservoirs, Δt may be taken as 1 hour, and small or large for any dams, it could be chosen as small as 0.2 hr or even 0.1 hr. The ordinates of the design flood hydrograph at Δt time steps throughout its time base should be computed and stored in a file beforehand. And, the length (L_{apprch}) and the Manning roughness coefficient (n) of the approach channel are determined considering the area upstream from the spillway and the channel material.

(2) The spillway design head, H_0, is determined by equation (6).

(3) The spillway height with respect to the bottom of the approach channel can be taken as

$$P \approx (0.5) \cdot H_0 \qquad (7)$$

This yields a reasonable value for P (spillway height) because, as seen in Figure 9-23 of *Design of Small Dams*, the major spillway discharge coefficient, C_0, rapidly drops down to very low values for spillways whose heights are smaller than half of the design head. Yet, equation (7) is not a strict rule, and another value for P dictated by the topography of the area where the spillway is situated may be taken.

(4) The angle of inclination of the upstream face of the spillway is determined. Recently, an inclination of 45° ~ 33° with the vertical is commonplace [e.g. 2, 3].

(5) The elevation difference between the upstream and downstream toes of the spillway is determined, basically by topographic factors. This value is usually of the order of 1 m ~ 3 m.

(6) The initial estimate for the peak of the outflow hydrograph, Qp_{outf}, is made as:

$$Qp_{outf} = (0.7)\cdot Qp_{inf}. \tag{8}$$

Here, Qp_{inf} is the peak of the design flood hydrograph. And, the initial estimate of the design spillway discharge, Q_{design}, is made as:

$$Q_{design} = Qp_{outf1}. \tag{9}$$

(7) Assigning $H_e = H_0$, the coefficients: C_0, C_{incl}, and C_{aprn} are taken from Figures 9-23, 9-25, and 9-27 of *Design of Small Dams* [1], and C_{net} is computed by equation (2).

(8) The effective spillway length is computed by equation (10) below, which is another form of equation (1)

$$L_e = Q_{design} / (C_{net}\cdot H_0^{1.5}) \tag{10}$$

(9) The net spillway length, L, is computed by adding the contraction effects of the piers and of the approach abutments to L_e.

(10) The width of the approach channel is equal to the gross spillway length (b = L).

(11) In routing computations, the spilled discharge at the end of each Δt time step is computed by an iterative numerical method. The algorithm used here and in some other studies [e.g. 16] is briefly summarized in the following. The initial estimate for the spillway discharge at the end of any Δt time step is made by extrapolation using the second-degree polynomial passing through discharges at the ends of the three preceding Δt's. Beginning with this initial estimate, by two or three iterations, the lake water surface elevation and the spillway discharge at the end of that Δt is computed to six significant digits. At each iteration, the discharge passing over the ogee spillway is computed by the recursive method summarized in subsection: 2.2. Spillway Discharge under a Given Head.

(12) When the routing computations at Δt time steps all over the time base of the design flood hydrograph are completed, the peak of the outflow hydrograph, Qp_{outf}, is determined by finding its maximum ordinate. If the absolute relative difference between Qp_{outf} and the previous design spillway discharge, Q_{design}, is not smaller than $1\cdot 10^{-6}$, then the new spillway design discharge is taken as $Q_{design} = Qp_{outf}$, the computations are sent back to the **8**th step above, and the iterations between the **8**th and **12**th steps are repeated until convergence to six significant digits is achieved. Ultimately, the length of the ogee spillway computed at the **9**th step is the searched length of the spillway such that (a) its design discharge is equal to the peak of the outflow hydrograph routed by it, (b) the maximum lake water surface elevation is equal to the pre-specified value, MWSE, and (c) the outflow hydrograph computed at the **11**th step of the last iteration is the final outflow hydrograph. A computer program is coded performing this iterative procedure. The program can be rerun with a spillway height different from the one at the **3**rd step.

3. AN EXAMPLE: SPILLWAYS OF CATALAN DAM ON SEYHAN RIVER

3.1. Length of Ungated Spillway

Catalan Dam exists on a cross-section of Seyhan River in the Mediterranean Region of Anatolia. This dam serves for both energy production and flood abatement. The capacities of its active storage and surcharge storage are 990 hm^3 and 590 hm^3. Its flood spillway is a radial-gated ogee spillway having a net length of 66 meters with six gates [17]. The data of Catalan Dam about (a) its flood spillway, (b) the probable maximum flood (PMF), which has a peak flowrate of 10,055 m^3/s, (c) the maximum allowed water surface elevation of the surcharge storage, which is 126.5 m, (d) the top elevation of the full active storage, which is 118.6 m, and (e) the (reservoir volume, hm^3)↔(water surface elevation, m) relationship are taken from its final project documents (Sheets HD-001, HD-002, HD-003 in [17]). The numerical details of the routing of the PMF through the flood spillway are not given in the final project, except for a table of 5-step operation of releases from the existing radial-gated ogee spillway (HD-003 in [17]). The operation steps given in that table are rewritten here in Table 1.

Table 1 - Discharges to be released during routing of the PMF from Catalan Dam given in Sheet HD-003 in [17]

Range of water surface elevation (WSE) (m)	Spillway discharge to be released (m^3/s)
118.60 < WSE ≤ 125.15	1200
125.15 < WSE ≤ 125.30	2500
125.30 < WSE ≤ 125.45	4000
125.45 < WSE ≤ 125.60	6000
125.60 < WSE ≤ 126.50	6500

Next, using the developed computer program, the length of the alternative ungated ogee spillway is computed. The flowrates of the PMF are taken from its final project at 4 hour steps, and the intermediate discharges at time steps of 0.2 hour are computed by third-degree polynomials fitted to every four sequential values surrounding the interpolated value and typed in the input data file. Although the initial estimate for the design discharge of the spillway is equal to (0.7)×10,055, which is 7038 m^3/s, the final spillway design discharge turns out to be: Q_{design} = 6629 m^3/s, and the length of the ungated ogee spillway is computed to be 141 m. The water surface elevation at the arrival of the design flood hydrograph, which is the PMF for this dam, is 118.6 m and the spillway design head is 8.0 m. The maximum water surface elevation reached during routing of the PMF from the reservoir with the design spillway of length of 141 m turns out to be 126.46 m.

3.2. Outflow Hydrographs from Ungated Spillway and From Radial-Gated Spillway

Along with the design of the ungated ogee spillway alternative to the existing radial-gated ogee spillway for Catalan Dam, the 15-stage operation model put forth by Haktanir et al [16]

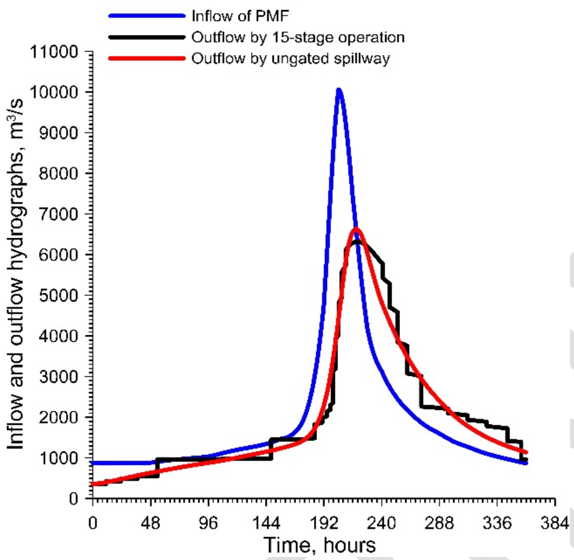

Figure 1 - Hydrographs of the incoming PMF and of the outflows (1) by the ungated ogee spillway and (2) by the existing radial-gated ogee spillway by the 15-stage operation model for Catalan Dam

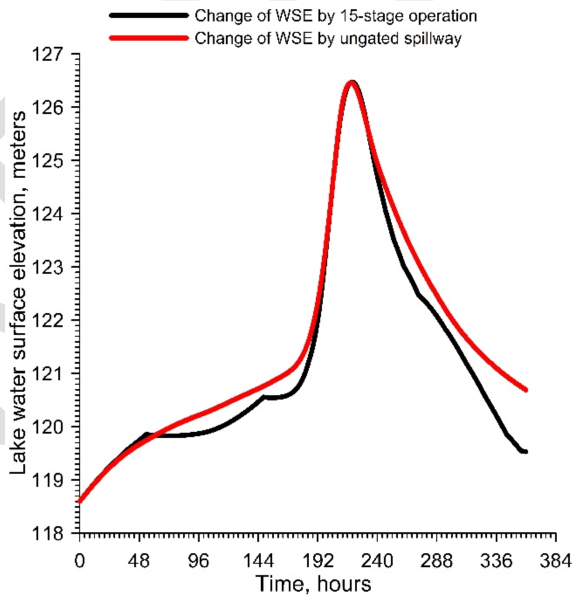

Figure 2 - Variations of the lake water surface elevations during routing of the PMF (1) by the ungated ogee spillway and (2) by the existing radial-gated ogee spillway by the 15-stage operation model for Catalan Dam

has been applied to Catalan Dam for routing of its PMF, also. The hydrographs of the PMF and of the outflows by the alternative ungated spillway and by the existing radial-gated spillway operated by the 15-stage operation model for Catalan Dam are given in Figure 1. The variations of the lake water surface elevation by routing of the PMF both by the ungated spillway and by the 15-stage operation model over the time base of the PMF are given in Figure 2. Coincidentally, routings of the PMF both by the ungated spillway and by the existing radial-gated spillway by the 15-stage operation model turn out to be fairly close to each other. The peaks of the outflow hydrographs by the ungated spillway and by the existing radial-gated spillway operated by the 15-stage operation model are 6629 m^3/s and 6315 m^3/s, and the maximum lake water surface elevations are 126.46 m and 126.48 m, respectively. In sheet HD-003 in its final project [17], these values are given as 6500 m^3/s and 126.44 m. We have not been able to superpose the outflow hydrograph to form by the operation policy mentioned in the final project of Catalan Dam (Table 1) in Figure 1 because the relevant quantitative data are not given in the final project sheets [17].

4. CONCLUSION AND DISCUSSION

Using the charts for the spillway discharge coefficients in the *Spillways* chapter of the book: *Design of Small Dams* [1], a new iterative method is developed to determine the length of the ungated ogee spillway such that the maximum lake elevation reached during routing of the design flood hydrograph becomes equal to the pre-specified maximum allowed water surface elevation. This method simplifies the conventional trial-and-error procedure for computation of the spillway length because it yields both the spillway length and the outflow hydrograph in a single run of the coded computer program.

As for the limitations of the proposed methods, the correction factor for the negative effect of a possible submergence due to a drowned hydraulic jump for those cases where a chute channel downstream of the spillway does not exist and the hydraulic jump of the spilled water forms right at the toe of the spillway is missing for the spillway discharge of an ogee spillway of given length. Symbolizing this coefficient by $C_{submrgnc}$, actually $C_{submrgnc}$ should be the fifth coefficient next to C_{aprn} in equation (2). Figure 9-28 of the book: *Design of Small Dams* [1] gives the chart for $C_{submrgnc}$. Lack of this factor from equation (2) should not be a serious drawback of the proposed method because most of the dams in Türkiye have long chute channels between the toe of the spillway and the energy dissipating structure and hence $C_{submrgnc}$ = 1.0 for most dams, anyway. Yet, for slight submergences $C_{submrgnc}$ = 1.0 also. In short, missing of $C_{submrgnc}$ in equation (2) should not be a worrisome issue. However, it should be kept in mind that for small dams having no chute channels the first method presented here may yield slightly greater magnitudes for ogee spillway discharge.

It is a known fact that because of the existence of the gates, the apex elevation of a gated spillway is much lower than the top of the full active storage (maximum operation elevation for water supply purposes), and hence when the gates are opened and begun to be operated the net head over the spillway crest is already very high and in parallel with that the discharge of the spilled water also becomes a high value. Therefore, gated spillways discharge high magnitudes of flowrates because of high net heads, and consequently they need smaller lengths for higher discharges. The apex elevation of a free flow (ungated) spillway however must necessarily be equal to the top of the full active storage. The developed method and the computer program executing it can be used to compare the financial costs of the ungated spillway and of the radial-gated spillway. For dams whose surcharge storages are fairly large, the length of the ungated spillway may not be too long. Hence, the total cost of the ungated spillway may become smaller than that of the shorter radial-gated spillway, because the sum of the costs of the concrete spillway unit, the steel gates, the motors, the hoisting mechanisms, the trunnion pins, and the anchorages and the support piers of the trunnions may outweigh the total cost of the ungated spillway. Besides, the operation and maintenance cost of a radial-gated spillway will also be much greater than that of an ungated spillway. A third advantage of the ungated spillway is that it does not require finding an optimum operation rule of gate openings during floods of any magnitudes.

The developed approach is applied to Catalan Dam on Seyhan River in Türkiye which yields accurate and reasonable results.

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