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Six-Stage Flood Routing for Dams Having Gated Spillways

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Abstract

A six-stage operation policy for the routing of flood hydrographs with return periods from 1.01 years up to the Probable Maximum Flood (PMF) is proposed for any dam having a gated spillway. The gate opening rules are determined, based on the recent pool level. The magnitude of the incoming flood hydrograph does not have to be predicted beforehand, as the fixed rules of the six-stage operation policy provide optimum routing for all floods, which are classified into six different groups based on their return periods. 10-, 100-, 1,000-, 10,000-, 100,000-year floods, and PMF are the upper limits for the six groups. When the PMF is routed, the rising and falling limbs of the outflow hydrograph have the appearance of a six-step staircase with sudden jumps and sudden drops at definite times and smooth variations between steps. The computer program developed to perform calculations of the six-stage flood routing model is applied sequentially to Yedigöze, Çatalan, and Seyhan Dams, all on Seyhan River in Turkey.

Key Words: Flood Routing, Dams, Gated Spillways

Kapaklı Dolusavaklı Barajlar için Altı-Safhalı Taşkın Ötelenmesi

Özet

1,01 yıl tekerrürlüden Muhtemel Maksimum Taşkına (MMT) kadar olan bütün taşkınların hidrograflarının ötelenmesi için altı-safhalı bir işletme politikası önerilmektedir. Savak kapak açılma kuralları, baraj gölündeki su seviyesine bağlı olarak belirlenmektedir. Tekerrür peryotlarına göre altı farklı grupta kapsanmış bulunan bütün taşkınlar için optimum öteleme sağlayacak olan altı-safhalı kuralı, gelen taşkın hidrografının önceden tahmin edilmesini de gereksiz kılmaktadır. 10-, 100-, 1.000-, 10.000-, 100.000- yıl tekerrürlü taşkınlar, ve MMT, altı grubun üst sınırlarını teşkil etmektedir. MMT ötelendiğinde çıkan hidrografın alçalma ve yükselme dalları belirli zamanlarda ani iniş ve çıkışlar yapan, arada ise oldukça düzgün bir biçimde artan veya azalan altı basamaklı bir merdiven görünümünde olur. Altı-safhalı taşkın öteleme modelini hesaplayan bilgisayar programı, bir örnek çalışma olarak, Seyhan Nehri üzerinde seri halde bulunan Yedigöze, Çatalan, ve Seyhan Barajlarına sırayla uygulanmıştır.

Anahtar Sözcükler: Taşkın Ötelemesi, Barajlar, Kapaklı Dolusavaklar

Regulated Flood Routing

Effective attenuation of flood peaks with 100- to 1,000-year return periods has positive economic consequences. However, the safe passage of catastrophic floods of the order of the Probable Maximum Flood, PMF, without causing dam failure is a major concern.

Although telecommunications networks that instantly send and relay precipitation and runoff data between hydraulic structures in large basins have been put into service in recent years, prediction of both the magnitude and timing of intense floods, even a few days before their actual occurrence, is still unreliable. In Turkey, as in most developing countries, such networks have not yet been established. An attempt has been made by the Turkish Government in Seyhan Basin (Japan International Corp.Agency, 1994), but no network has been set up in that basin yet. A comprehensive Early Flood Warning System is not expected in Turkey in the near future.

Even in those basins equipped with an early flood warning network, the operation of gated spillways is based mainly on human experience and judgement. Linsley et al emphasize this fact:" The discharge from a storage reservoir is regulated by gates and valves operated on the basis of the judgement of the project engineer." (Linsley et al., 1992, page: 746). Sakakima et al make a similar comment: "... for the extremely big flood, a reservoir operator has to control the gates to protect the reservoir and the downstream reference point by relying on his judgement." (Sakakima et al., 1992).

For those reservoirs which serve both conservation and flood mitigation purposes, there is always active storage which is also kept ful during a flood season. Hence, the initial water surface elevation of the lake, Hb, is often already at a high leve upon the arrival of a flood wave. If the managers of the dam panic at the onset of the flood and open the gates too much, the peak of the outflow released will be greater than it would be through a tighter policy which would make full use of the flood retention storage and the spillway characteristics in proportion to the real strength of the incoming wave. Conversely, if the gate openings are kept too small at the rising limb of a serious flood, the peaks released later may have to be greater than they would under a better policy.

Variation of the water surface elevation, H, with respect to time indirectly reflects the magnitude of the inflow hydrograph. Therefore, the gate openings of the operable spillway can be determined as a function of the recent water surface elevation in the lake. It would be more practical if a fixed set of optimum operation rules could be developed, based only on the variation of the lake level.

Assuming that the actual magnitude of a flood cannot be predicted beforehand, the objective of this study is to propose a set of fixed spillway operation rules which will route floods of all magnitudes efficiently, as well as passing the PMF safely, with the purpose of eliminating human error during flood operations. The suggested scheme is a six-stage policy, treating floods in six different categories according to their return periods.

1. Six-Stage Operation Policy

Fairly small floods having return periods less than or equal to 10 years will be effectively routed within the 1^{st} stage. The operation policies for the 2^{nd}, 3^{rd}, 4^{th}, 5^{th}, and 6^{th} stages will route floods having return periods in the ranges: $10 < T \leq 100$ years, $100 < T \leq 1{,}000$ years, $1{,}000 < T \leq 10{,}000$ years, $10{,}000 < T \leq 100{,}000$ years, and 100,000 years $< T \leq$ PMF, respectively. As it may be seen in Table 2, these specific return periods subdivide the ultimate flood, the PMF, into well-dispersed fractions. The choice of the six stages is believed to be reasonable, as a greater number of stages would complicate the gate operations during the fairly short period of a particular flood.

The maximum water surface elevation occurring when a flood with a 10-year return period is routed will make the ceiling elevation of the 1st stage, called the 1st critical level, Hcr1. When a flood of greater magnitude occurs, it will cause the lake surface elevation, H, to rise above Hcr1 with the operation policy of the 1st stage. This will be a clearcut indication that the flood being routed is greater than the 10-year flood, and the spillway gate opening will be enlarged a little more than in the 1st stage, resulting in the release of greater outflow discharges. The maximum H when the 100-year flood is routed will be the upper level of the 2nd stage, or the 2nd critical level, Hcr_2. When any flood having a return period in the range 10-year $< T \leq$ 100-year is routed, the maximum H will remain within the 2nd stage. For a 100-year flood, for example, the initial part of it will be routed by the operation policy of the 1st stage: $Hb < H \leq Hcr_1$. During routing, once H reaches Hcr_1, the spillway operation policy of the 2nd stage will be implemented. Similarly, the initial part of the rising limb of the 1,000-year flood will be routed with the operation policy of the 1st stage; later, when $H > Hcr_1$, routing will continue according to the operation policy of the 2nd stage; and finally, the lake level will rise over Hcr_2, and from then on routing will continue according to the operation policy of the 3rd stage. In short, the outflow from any flood having a return period greater than 10 years will have a hydrograph increasing smoothly within each stage, but rising abruptly from one stage to the subsequent stage. Figure 1 schematically depicts these stages, the critical levels, and behaviour of the water surface elevation during the routing of any flood.

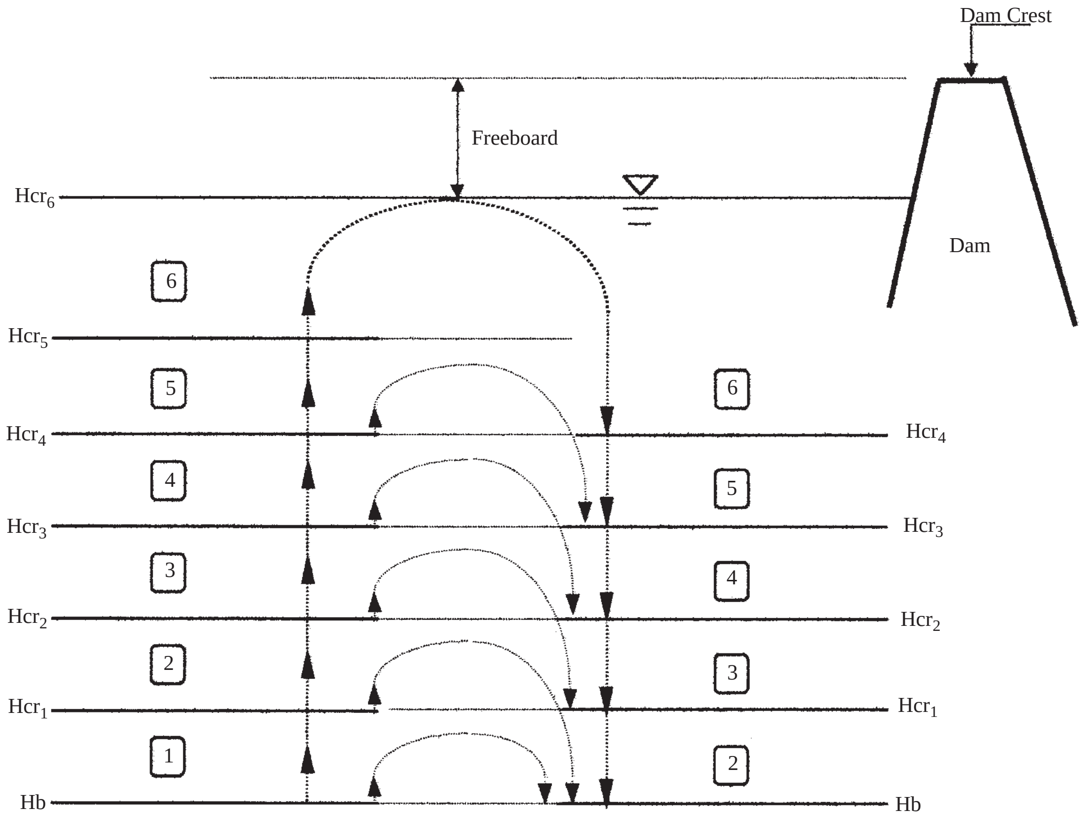

Figure 1. Schematic description of six-stage routing

When the PMF strikes, the managers of a dam may not realize its real size, and they may assume it is a flood with a 10-year return period. Hence, initially they will apply the operation policy of the 1^{st} stage. Because of its intense magnitude, however, the PMF will cause the lake level to rise above Hcr1 in a few hours. Next, the managers will not panic but will implement the operation policy of the 2^{nd} stage, and so on, until the lake surface rises above the uppermost critical level, Hcr_5, and enters the 6^{th} stage. At that moment, not too much water will have been released from the reservoir, because of the encroached gate opening policy up to the 6^{th} stage. Gradual opening of the gates however, will cause quite a fast increase in H. Therefore, when the spillway starts to operate at full capacity in the 6^{th} stage, the lake surface is at a high elevation, huge quantities of water will be released and there will be no risk of overtopping the dam.

1.1. Algorithm for Determining Critical Levels and Gate Opening Rules

The initial constraints are reservoir surface elevation at the beginning, Hb, and the upper-bound surface elevation during routing of the PMF, Hcr_6, which equals dam crest elevation minus a freeboard (Figure 1). The volume of the natural valley between elevations Hcr6 and Hb is available for flood retention. The six stages and their critical levels will be placed within this total empty storage capacity, and the corresponding spillway gate openings will be determined accordingly.

The magnitudes of the decision floods, which are 10-year, 100-year, ..., 100,000-year floods, are always well-distributed portions of the PMF. Hence, the empty volume of the valley allotted to each consecutive decision flood and the PMF should be in proportion to the total volume of the flood. The initial estimate of Hcr_1 (Hcr_1-trial) is based on this criterion. Namely, the ratio of the valley storage between Hcr_1-trial and Hb to the total empty valley storage is proportional to the ratio of the volume of the 10-year flood to that of the PMF. Routing of the 10-year flood is performed with closed gates, and it is checked that the maximum water surface elevation, Hmax, with zero spillway outflow and known turbine discharge remains below Hcr_1-trial. Next, routing of the 10-year flood is repeated with the smallest spillway gate opening if Hmax turns out to be greater than Hcr_1-trial with the first option of closed gates. Once Hmax $\leq$ Hcr_1-trial, then Hcr_1 = Hmax; and, Hcr_2-trial is calculated using the proportionate part of the empty valley between Hcr_6 and Hcr_1. Next, routing of the 100-year flood is repeated as many times as required until Hmax $\leq$ Hcr_2-trial. The spillway gate opening of the 2^{nd} stage is increased one step further during each such repetition. Hcr_3, Hcr_4, and Hcr_5, and the corresponding gate opening values are determined in a similar manner using the 1,000-, 10,000-, and 100,000-year floods, individually, in this order. The gate opening of the first trial routing of the PMF is taken as being one step further than the gate opening of the previous decision flood, the 100,000-year flood. Routing of the PMF is repeated until Hmax $\leq$ Hcr_6. If Hmax $>$ Hcr_6, even with the gates fully open, then routing cycles drop one step below, to the routing of the 100,000-year flood. The gate opening of the 100,000-year flood is increased yet one step further, then calculations start for the PMF once again. In a case in which the constraint: Hmax $\leq$ Hcr_6 cannot be satisfied, then the large loop of routing calculations drops down to the 10,000-year flood. This large loop can go as low as the 10-year flood in order to satisfy the crucial ultimate constraint of Hmax $\leq$ Hcr_6.

By giving more weight to floods with higher probabilities of occurrence, the optimization of critical levels and gate openings of the stages summarized above is based on the principle that floods with shorter return periods should be routed more efficiently than those with longer return periods.

1.2. Algorithm for Routing in the Falling Limb

In the proposed six-stage routing procedure, the same operation rules as those for a rising limb are applied during a falling limb, with one distinct modification. In a falling limb, the operation policy of the jth stage is kept as it is both in the jth and in the (j-1)th stages down to the (j-2)th critical level. In other words, as the jth operation policy is executed in the jth stage during a rising limb, the jth operation policy is still applied in the jth stage just after turning around a peak, if that happens within the jth stage; the jth operation policy is still applied one stage further below in the (j-1)th stage, down to Hcr_{j-2}, below which, in the (j-2)th stage, the (j-1)th operation policy is implemented, and so on.

This one-stage-lagging operation policy during a falling limb is useful in preventing unwanted oscillations between two stages, which is a possibility if the (j-1)th operation policy is applied strictly as the lake level drops below the (j-1)th critical level. If the gate openings of the (j-1)th stage are too narrow, causing the outflow to become less than the inflow during a falling limb, the DS will become positive immediately after encroaching the gates and this will push the lake level back up over Hcr_{j-1} , into the j stage, which wil cause the gates to be opened wider, the policy of the jth stage. Once opened wider, the spillway will pass an outflow greater than the inflow again, which will make H drop below Hcr_{j-1} into the (j-1)th stage once again. Therefore, although theoretically possible and numerically solvable, such a vicious cycle of oscillations between the jth and the (j-1)th stages for a few time steps would be very impractical in a real-life situation, causing the gates to be lowered and lifted every one hour or so. Continuation of the jth policy in the (j-1)th stage prevents that and also helps empty the reservoir at a slightly faster rate. This one-stage-lagging operation during the falling limb is also shown in Figure 1.

1.3. Algorithm for Routing in Each Stage

In calculating the outflow hydrograph from a reservoir, the difference between the more detailed hydraulic routing and the hydrologic routing is negligible for most practical purposes (e.g. Hager et el., 1984; Haktanır and Özmen, 1997). Level-pool routing is executed numerically by an algorithm different from any used to date, like the pulse or Runge-Kutta methods (Chow et al.,1988, Ch.8). Computations are performed tepwise at small time increments, Dt, (e.g. 0.2, 0.5, 1, or 2 hour) on the basis of the continuity equation in conjunction with the storage versus head relationship of the reservoir and the discharge versus head characteristics of the spillway for the chosen option of gate opening.

Assuming the stepwise calculations have already proceeded for some time, the outflow at the end of the ith time step is calculated through a trial-and-error procedure. The initial estimate of the reservoir water surface elevation at the end of the ith time step is made simply by:

$$H_{i_1} = H_{i-1} \quad (1)$$

where, H_{i-1} is the actual water surface elevation at the end of the i-1st (previous) time step. The initial estimate of the outflow at the end of the ith step, is made by extrapolating the second degree polynomial passing through the previous three points of the outflow hydrograph, which is:

$$O_{i_1} = O_{1-3} - 3.O_{1-2} + 3.O_{i-1} \quad (2)$$

This is a reasonable assumption because the outflow hydrograph can be approximated by a parabola in a short period, and O_{i_1} given by Eq.2 should be close to the actual value. Next, the storage increment (or decrement) is calculated by

$$DS_i = [(I_{i-1}+I_i)/2-(O_{i-1}+O_{i_1})/2]Dt.0.0036 \quad (3)$$

where, DS_i is the storage increment or decrement over the ith time step, and Ii-1 and Ii are inflows at the beginning and at the end of the ith time step (all I's and O's are in m^3/s, Dt is in hours, and DS_i is in Million m^3). Now, the storage in Million m^3 at the end of the ith Dt is calculated by

$$S_i = S_{i-1} + DS_i \quad (4)$$

The water surface elevation in m versus S_i, H_{i2}, is calculated by interpolation with a third-degree polynomial passing through four points enclosing S_i at equal distances from the initially given table of storage versus elevation of the reservoir. If then is the correct value of the outflow hydrograph at the end of the ith time step. Otherwise, the next estimate for O_i is made by a third-degree polynomial interpolation again from the initially set discharge - head relationship of the spillway versus H_{i2}; namely, is calculated by

$$O_{i2} = QSP(H_{i2}) \quad (5)$$

where QSP(...) symbolizes the discharge head relationship of the spillway. Next, is calculated using Eq.3 again, with replacing and iterations stop when is within 6 significant digits. This numerical algorithm, which computes the values of outflow, lake surface elevation, and storage simultaneously, requires two iterations for most of the steps, and only rarely requires three or at most four iterations.

1.4. Algorithm for Passage from One Stage to the Next

During calculations, if the water surface elevation, H_i, happens to be close to any one of the four critical elevations, Hcr_j, within 1 cm, passage from the (j-1)th to the jth stage takes place at the end of the ith time step. If $H_i < Hcr_j$, then routing computations proceed normally as explained above. If $H_i > Hcr_j$ as $H_{i-1} < Hcr_j$, however, this means that the lake surface elevation will reach the jth critical level after the (i-1)th time step in a period less than Dt.

Fig.2 depicts O vs time, I vs time, and H vs time for that time step in which passage from the (j-1)th to the jth stage takes place. Symbolizing the storage corresponding to Hcr_j by Sint, which is easily interpolated versus Hcr_j, the increment of storage from time t_{i-1} to time $(t_{i-1}+Dt1)$, (from H_{i-1} to Hcr_j), is given by:

$$DSint = Sint - S_{i-1} \quad (6)$$

By continuity, DSint is also equal to:

$$DSint = (l\bar{i}nt - O\bar{i}nt).Dt1.0.0036 \quad (7)$$

where $l\bar{i}nt$ and $O\bar{i}nt$ are average flow rates of the inflow and outflow hydrographs from time t_{i-1} to $(t_{i-1} + Dt1)$. Assuming the rate of increase (or decrease) of the inflow hydrograph is approximately linear, which is a reasonable assumption if Dt is small, $l\bar{i}nt$ is given by:

$$l\bar{i}nt = (l_{i-1} + lint)/2 \quad (8)$$

It can be shown after algebraic manipulation that $l\bar{i}nt$ can be expressed in terms of l_{i-1} and l_i as

$$l\bar{i}nt = l_{i-1} + (Dt1/Dt).(l_i - l_{i-1})/2 \quad (9)$$

Similarly, $O\bar{i}nt$ is given by:

$$O\bar{i}nt = (O_{i-1} + Oint)/2 \quad (10)$$

where $O\bar{i}nt$ is calculated by the precise interpolation versus Hcr_j using Eq.5, discharge - head relationships of the spillway. Combining Eqs.6, 7, 9, and 10, the following equation can be written:

$$\{[0.5.(l_i - l_{i-1})/Dt].0.0036\}.(Dt1)^2 + \{[l_{i-1} - 0.5.(O_{i-1} + Oint)].0.0036\} .Dt1 - (Sint - S_{i-1}) = 0 \quad (11)$$

where, Sint and S_{i-1} are in Million m^3, all I's and O's are in m^3/s, and Dt1 and Dt are in hours. Eq.11 is a parabolic equation whose reasonable root will yield the time period Dt1. After computation of Dt1, the program reports Iint, Oint, Hcrj, and Sint values also at the time of $(t_{i-1} + Dt1)$.

The other part of this particular time step, Dt, is calculated by

$$Dt2 = Dt - Dt1 \quad (12)$$

and net outflow, water surface elevation, and storage at the end of the ith time step are calculated as explained below.

Symbolizing the net outflow about one minute after $(t_{i-1} + Dt1)$ by Oint*, namely, the net outflow at the beginning of the jth stage immediately after the spillway gates are opened, as dictated by the policy of the jth stage, the algorithm explained in the previous section is applied to the time step of Dt2, from $(t_{i-1} + Dt1)$ to $(t_{i-1} + Dt1 + Dt2)$ $(= t_i)$, with Iint replacing I_{i-1}, $O^\star_{int}$ replacing O_{i-1}, Sint replacing S_{i-1}, Hcr_j replacing H_{i-1}, and Dt2 replacing Dt in Eqs.1, 3, 4, and 5. The first estimate of O_i at the end of time step Dt2 is made simply by:

$$O_{i_1} = Oint^\star \quad (13)$$

instead of Eq.2.

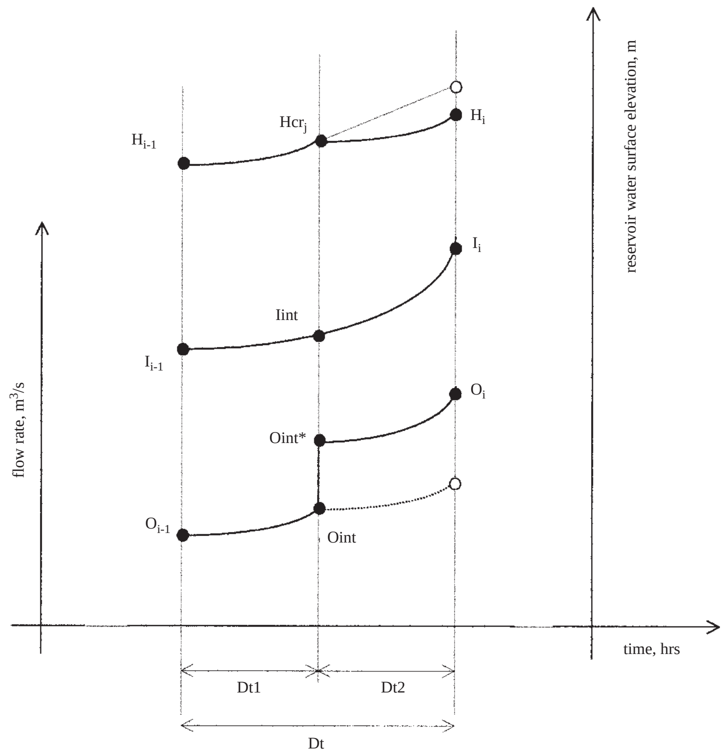

Figure 2. Inflow, outflow, and reservoir water surface elevation versus time during passage from one stage to the next stage

So far, passage from the (j-1)th to the jth stage during a rising limb of the outflow hydrograph has been explained. During a falling limb, Eq.11 is also applicable as it is, which can be verified by deriving the same equation with the help of a figure similar to Fig.2, in a falling limb.

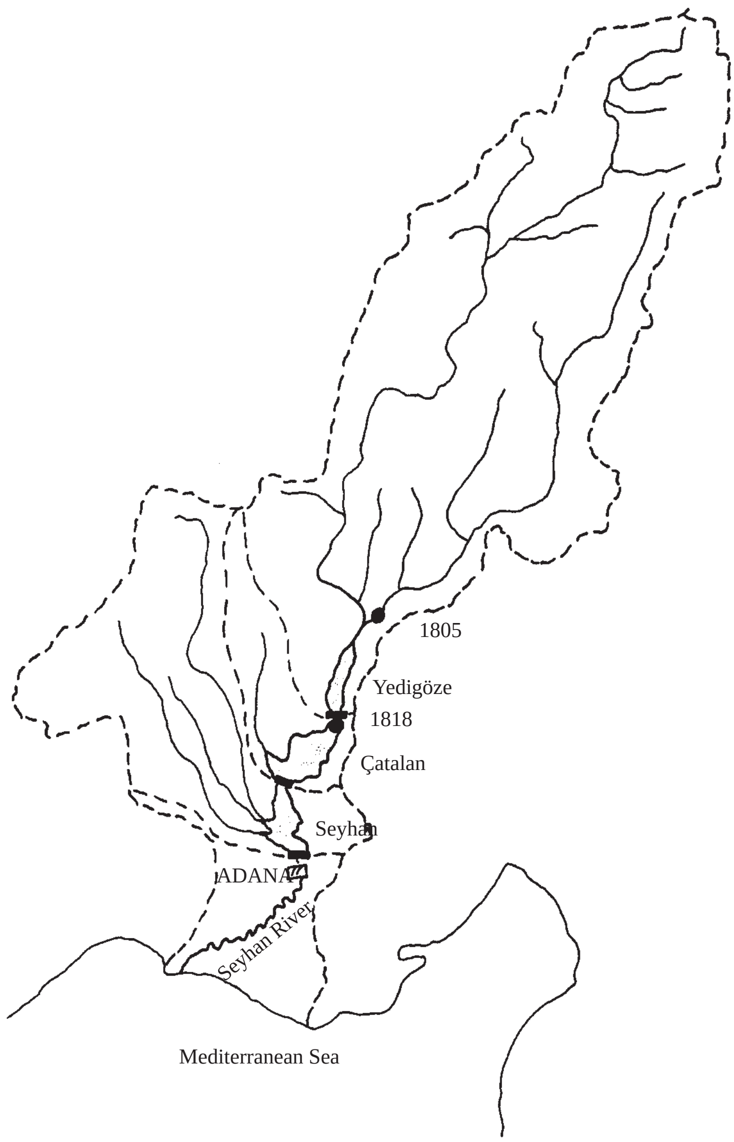

Figure 3. Locations of the dams in Seyhan River Basin

2. Application of the six-stage operation in three dams in Seyhan Basin

2.1. Data for the dams and their reservoirs

Seyhan Basin, with a total area of about 20,000 km^2, is one of the major basins in Anatolia. Seyhan River discharges unpolluted waters of about 6.5 Billion m^3 per year into the Mediterranean Sea (DSİ, 1988; Verbund-Plan, Romconsult, Temelsu, 1980a).

At present, there are two dams in series on Seyhan River: Çatalan and Seyhan; and, a third one, Yedigöze Dam, is under construction. Brief information about these dams and their reservoirs is given in Table 1, and their locations in Seyhan Basin are shown in Figure 3. The dam furthest upstream, Yedigöze, has a small reservoir with the single purpose of hydroelectricity and so it has no flood retention storage. The central has the largest storage capacity, and it serves both flood mitigation and hydroelectric generation purposes. The dam furthest downstream is Seyhan Dam, which went into operation in 1956. The construction of Çatalan Dam was recently completed in 1997, and Yedigöze Dam is still being built.

Table 1. Some charactestics of the three dams and their reservoirs considered in the study (DSİ, 1988; Verbund-Plan, Romconsult, Temelsu, 1980a)

	YEDİGÖZE	ÇATALAN	SEYHAN
Date of beginning and completion of construction	1997-	1982-1997	193-1956
Embankment type	rockfill	earthfill	earthfill
Purpose*	E	E+F	E+I+F
Drainage area, km^2	13830	15387	19000
Total storage capacity, $10^6 m^3$	662	2126	883
Dam crest elevation, m	240.0	130.0	72.7
Dam height, m	105.0	70.0	50.7
Top elvt. of active storage, m	233.9	118.6	63.5
Spillway crest elevation, m	220.0	110.0	61.0
Emergency spillway crest elvt., m	-	-	67.5
Max. spillway discharge, m^3/s	8760	10055	2500
Max. emerg. spillway disch., m^3/s	-	-	2950
Max. turbine outflow, m^3/s	2×92.5=185	3×120=360	3×77=231

* E: Energy production, F: Flood mitigation, I: Irrigation

2.2. Calculation of the PMF and 10-year, ..., 100,000-year Floods

The 24-hour Probable Maximum Precipitation, PMP-24, was calculated using the Hershfield method as explained in Report No.1 by the World Meteorological Organization (WMO, 1973, Ch.4). The envelope curve for Anatolia developed by the General Directorate of State Hydraulic Works of Turkey (Özdemir, 1978) was used in conjunction with pertinent recorded data (DSİ, 1990) from all precipitation gauges in and around Seyhan Basin with records going back more than 20 years. Using the classical Thiesen polygons method, the areal averages of the PMP-24 were calculated for the three subbasins of the three dams. The time distribution of the 24-hour PMP for each subbasin was calculated with the help of a chart prepared by the Turkish State Hydraulic Works (Özdemir, 1978). The areal average infiltration indices previously determined for these basins from recorded hyetograph-hydrograph data in Seyhan Basin were also obtained (Haktanır and Sezen, 1990; Türksoy, 1979; Verbund-Plan, Romconsult, Temelsu, 1980b). Next, synthetic unit hydrographs were calculated using a procedure suitable for the Turkish basins (Haktanır and Sezen, 1990), and the Probable Maximum Flood, PMF, for the three subbasins was calculated by the unit hydrograph theory.

The streamgauging station 1818-Üçtepe is situated almost on the axis of Yedigöze Dam, and it has record dating back 26 years. About 30 km upstream, again on Seyhan River, the 1805-Gökdere station has records for the last 54 years (EİEİ, 1955-92). In one study related to flood frequency analyses on unregulated streams throughout Turkey, the two particular probability models: 1) 3-parameter log-normal distribution with the method of maximum likelihood, LN3-ML, and 2) 3-parameter log-normal distribution with the method of zero skewness, LN3-CSx=0 (proposed in Haktanır, 1992), were found to predict the high return period peaks better than many other distributions (Haktanır, 1997).

Although not needed for routing calculations, the PMF at the 1805-Gökdere section was also calculated by the same method as that explained in the previous paragraph. The application of these two models to the recorded annual flood peak series of stations 1818 and 1805 indicated that the peak with a 1,000,000-year return period was very close to the peak of the PMF calculated for the same station. The peak of the PMF turned out to be 9000 m^3/s and 5800 m^3/s for 1818 and 1805, respectively; whereas the

1,000,000-year return-period peaks by the LN3-ML and LN3-CSx=0 models were 10,100 m^3/s, 10,200 m^3/s, and 5600 m^3/s, 5900 m^3/s, for 1818 and 1805, respectively. Because stations 1818 and 1805 are both on Seyhan River and close to the study area, they were assumed to represent the main channel characteristics of Seyhan River. Since the 1,000,000-year peaks by the popular distributions were very close to the peaks of the PMFs for the same sections, it was assumed that the PMF on any section of Seyhan River had a return period of 1,000,000 years.

In Table 2, the averaged ratios of the 10-, ..., 100,000-year return-period peaks by both the LN3-ML and LN3-CSx=0 models are given. Assuming that the total volumes of the flood hydrographs had the same probability distributions as their peaks, all flood hydrographs with return periods of 10-, ..., 100,000-years were calculated by multiplying the ordinates of the PMF by the ratios given in Table 2.

Table 2. Ratios of various return-period peaks to the 1,000,000-year peak by the LN3-ML and LN3-CSx=0 models

QT/QPMF	Average value
Q10/QPMF	0.19
Q20/QPMF	0.22
Q50/QPMF	0.27
Q100/QPMF	0.31
Q200/QPMF	0.35
Q500/QPMF	0.41
Q1,000/QPMF	0.45
Q2,000/QPMF	0.49
Q5,000/QPMF	0.56
Q10,000/QPMF	0.61
Q100,000/QPMF	0.79

2.3. Application of the Six-Stage Operation Model

The developed computer program first reads all possible spillway gate opening rules, the inflow hydrograph of the PMF, ratios of the 10-year, ..., 100,000-year floods to the PMF, and the storage - surface elevation, S-H, relation of the reservoir. After executing all the algorithms summarized above, it determines the critical levels and gate opening rules of the six stages, and finally it reports the inflows, outflows, and water surfaces for all decision floods.

Yedigöze and Çatalan Dams are at the upstream ends of the Çatalan and Seyhan reservoirs, respectively; this means that the spillway outflow from an upstream dam discharges directly into the reservoir of the downstream dam with no river in between. Therefore, the inflow hydrographs into Çatalan and Seyhan Reservoirs were calculated by adding the outflow hydrograph from the upstream dam to the natural hydrograph of the intermediate subbasin between the two dams.

The six-stage operation scheme was applied to each of these three dams in sequence so as to keep the flooding downstream of the last dam, Seyhan, to a minimum.

The spillway rule curves and reservoir S-H relations of the dams used in the study were taken from the following sources: (DSİ, 1981a, 1981b, 1988; Japan International Corporation Agency, 1994; Verbund-Plan, Romconsult, Temelsu 1980a, 1980b. The top of active pool elevations in the original projects of the three dams are: 233.9 m, 118.6 m, and 63.5 m, respectively for Yedigöze, Çatalan, and Seyhan reservoirs (DSi, 1988; Verbund-Plan, Romconsult, Temelsu, 1980a). The six-stage operation policies determined for this case by applying the program to these three dams in sequence going downstream are given in Table 3. In Figures 4-6, the routings of the PMFs from Yedigöze, Çatalan, and Seyhan Dams respectively are shown for this case.

To show that the fixed rules of the proposed six-stage routing adjust automatically to any multi-peaked hydrographs in the most effective way, the routing of a triple-peaked hydrograph formed sequentially joining PMF, 10,000-year, and 100,000-year floods from Çatalan Dam is given in Figure 7.

2.4. Results and Discussions

With the aim of minimizing human error in the operation of spillway gates during the passage of any flood from a gated dam, a fixed six-stage gate operation model was developed. The coded computer program was applied to three dams located in series on Seyhan River, and the results show that this model provides a plausible set of rules for any dam having a gated spillway.

Table 3. Summary of six-stage routing operations through the three reservoirs for initial reservoir elevations of 233.9 m, 118.6 m, and 63.5 m

Flood Return Period (years)		Reservoirs		
		Yedigöze	**Çatalan**	**Seyhan**
	Dam Crest Elvtn.(m)	240	130	72.70
	Beginning Lake Elv (m)	233.9	118.6	63.5
	Critical Levels (m)	233.9 236.0 236.6 237.2 237.5 238.2	118.6 121.5 122.6 123.7 124.8 126.3	63.5 66.4 69.0 69.6 70.1 70.6
	Gate Openings (m)	1 3 5 8 11 F.	0 1 2 3 4 5	0 0 1 2 4 6
10	Peak Inflow (m^3/s)	1696	1565	1719
	Peak Outflow (m^3/s)	962	360	231
	Max. Lake Elvtn. (m)	235.88	121.32	66.35
100	Peak Inflow (m^3/s)	2806	3133	2608
	Peak Outflow (m^3/s)	2456	1085	941
	Max. Lake Elvtn. (m)	236.54	122.47	68.84
1,000	Peak Inflow (m^3/s)	4051	4850	3606
	Peak Outflow (m^3/s)	3702	1785	1925
	Max. Lake Elvtn. (m)	237.16	123.56	69.44
10,000	Peak Inflow (m^3/s)	5485	6964	4819
	Peak Outflow (m^3/s)	5278	2476	2803
	Max. Lake Elvtn. (m)	237.41	124.57	69.98
100,000	Peak Inflow (m^3/s)	7136	9144	6631
	Peak Outflow (m^3/s)	6785	3337	3851
	Max. Lake Elvtn. (m)	237.92	126.12	70.46
PMF	Peak Inflow (m^3/s)	9022	12147	8559
	Peak Outflow (m^3/s)	9328	4115	5199
	Max. Lake Elvtn. (m)	238.20	128.05	71.20

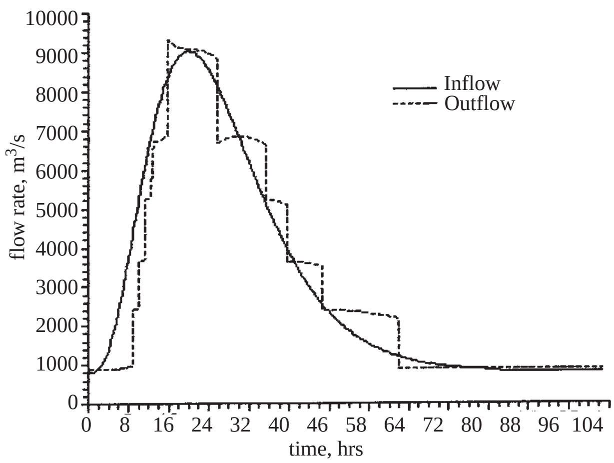

Figure 4. Routing of PMF by six-stage operation at Yedigöze Dam

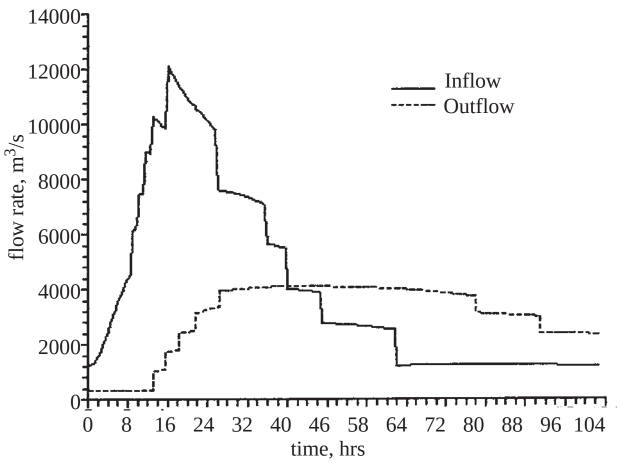

Figure 5. Routing of PMF by six-stage operation at Catalan Dam

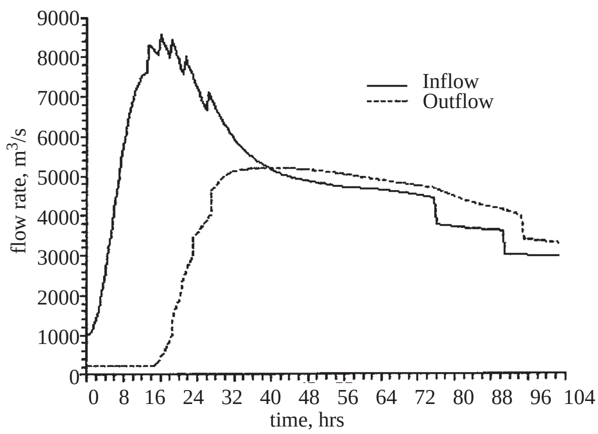

Figure 6. Routing of PMF by six-stage operation at Seyhan Dam

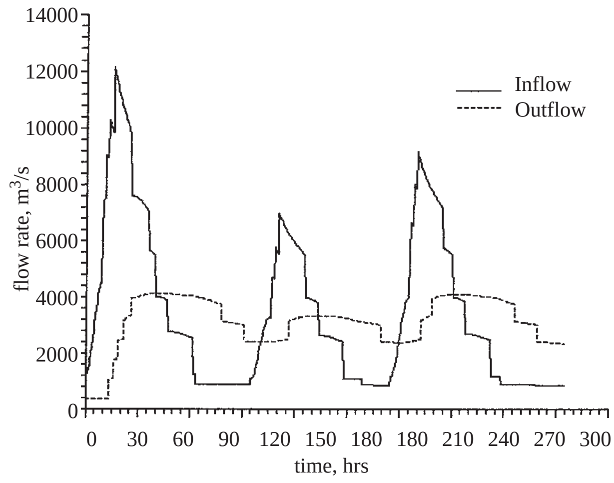

Figure 7a. Routing of PMF +10,000-year flood + 100,000-year flood by six-stage operation of Catalan Dam

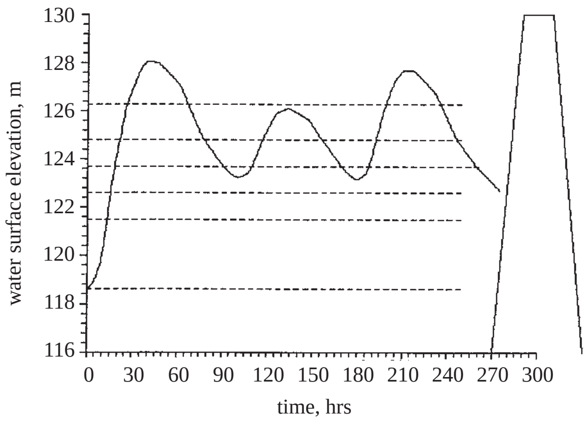

Figure 7b. Variation of water surface elevation during routing of PMF + 10,000-year + 100,000-year floods at Catalan Dam

An interesting finding peculiar to the application of this study was that the existing spillway of Çatalan Dam seems to be overdesigned. It can safely pass even the PMF with a gate opening of only 6 m. The gate opening is 13 m when fully open. Considering additional floods from three creeks directly discharging into the reservoir of Seyhan Dam, downstream of Çatalan Dam the spillway gates of Çatalan Dam must in no case be opened too wide. Because the active storage of Çatalan Dam is quite large and the elevation difference between the active pool surface and spillway crest is great, opening Çatalan spillway gates too much would cause very high flow rates from Çatalan Dam, discharging directly into Seyhan Reservoir. These excessive flows would cause volumes of water too great to be safely handled by Seyhan Dam to fill up the reservoir of Seyhan Dam, ultimately leading to overtopping of the embankment; This could result in catastrophic damage in the city of Adana just 10 km downstream.

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