COMPARISON OF HYDRAULIC AND HYDROLOGIC ROUTING METHODS FOR DAMS HAVING THIN AND LONG RESERVOIRS

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Bu çalışma, jürimiz tarafından İnşaat Mühendisliği Anabilim Dalı'nda YÜKSEK LİSANS TEZİ olarak kabul edilmiştir.

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ABSTRACT

The outflow hydrographs computed by both hydrologic routing (level-pool routing) and by hydraulic routing for three dams having considerably long lakes within narrow valleys in Turkey (Deriner, Berke, and Kayraktepe) were compared for three inflow hydrographs of different peaks with three unregulated ogee spillways of different capacities within considerably wide intervals and yet at reasonable proportions. Out of the total of 27 such combinations, it was observed that the difference between those outflow hydrographs was the greatest at the peak value. The difference grew larger as the spillway capacity became smaller relative to the magnitude of the inflow hydrograph, and it changed between 2 to 10% of the peak of the outflow by hydrologic routing, for these three thin-and-long reservoirs. The peak of the outflow by hydraulic routing turned out to be smaller than that by hydrologic routing for all the 27 different routing cases, as expected. The difference being at most about 10% even for such long lakes was considered to indicate that the level-pool routing for reservoirs of short valleys should yield outflow hydrographs reasonably close to the real ones.

For thin-and-long reservoirs, a simple idea is suggested to modify the outflow hydrograph computed by the hydrologic routing without necessitating any excessive data.

ÖZ

Bu çalışmada, Türkiye'de dar vadilere uygulanmış ince ve uzun göle sahip üç barajda (Deriner, Berke, Kayraktepe), üç farklı pike sahip giren akım hidrografı ve herbiri için üç farklı dolusavak genişliği alınarak hidrolojik ve hidrolik metodlarla taşkın ötelemesi yapılmış ve çıkan akım hidrografları karşılaştırılmıştır. Toplam 27 kombinasyondan her iki metod sonucunda çıkan akımlar arasındaki farkın pik değerlerde en fazla olduğu gözlenmiştir. Sözü edilen farkın, giren akımın büyüklüğüne bağlı olarak dolusavak genişliği küçüldükçe arttığı ve ince-uzun göle sahip reservuarlar için hidrolojik öteleme sonucundaki çıkan akım pikinin %2'si ile %10'u arasında değiştiği saptanmıştır. Tahmin edildiği gibi hidrolik ötelemedeki çıkan akım piki hidrolojik ötelemedeki çıkan akım pikinden daha küçük olmuştur. İnce uzun göle sahip reservuarlarda bile bu fark en fazla %10 oranına ulaştığına göre ince-uzun olmayan göllerde hidrolojik öteleme ile daha gerçekçi öteleme olan hidrolik öteleme sonuçları birbirine oldukça yakın olsa gerekir.

Sonuç olarak ince ve uzun reservuarlar için çok fazla data yükü gerektirmeyen hidrolojik öteleme yöntemindeki çıkan akım pikini, hidrolik ötelemedeki çıkan akım pikine yaklaştıran bir modifikasyon geliştirilmiştir.

1. INTRODUCTION

Flood routing is the process which shows how a flood wave may be reduced in magnitude and lengthened in time by the use of storage in a river reach or through a reservoir in a valley. Routing procedure involves determination of the outflow hydrograph at a downstream point of river or reservoir by using given inflow hydrograph at an upstream point.

Recently, storage reservoirs, relief channels or storage basins have been built for flood routing. For the design of these structures, it is very important to estimate their effects on the flood waves.

Two main methods have been developed for routing floods in reservoirs and rivers: Hydrologic and hydraulic routing [1,3]. In general, hydrologic methods are simpler and based on solution of the continuity equation only, together with the spillway rating and reservoir storage-elevation curves. It is assumed that the water surface in the reservoir is horizontal at all times. Because the velocities in large reservoirs even during a flood event is negligibly small, the wall shear forces and forces due to changes in momentum are mostly of negligible magnitudes, also. Therefore, the hydrologic routing may be satisfactory for reservoirs where absolute precision is not a must.

In hydraulic routing methods hydraulic characteristics of channel, and dynamic effects are taken into account. Two differential equations which are continuity and momentum govern these methods. Contrary to the simplicity of hydrologic routing, hydraulic routing is more complicated, and it generally requires the use of a high speed computer. Although a package program like DUFLOW is used for the complicated and heavy computational load of hydraulic routing, preparation of input data takes a long time due to the necessity of the topographic study.

The hydrologic routing may not produce correct outflow hydrographs for thin-and-long reservoirs, simply because the long lake of such a reservoir may act like a deep river reach and the dynamic effects of the flood wave may not be negligible.

There are yet many reservoirs in the world, existing or planned, which have thin-and-long lakes located in rather narrow and deep valleys. In this study, three thin-and-long reservoirs: Deriner, Berke, Kayraktepe were chosen from different parts of Turkey.

In Chapter 2, lumped and distributed system routings are presented with their theoretical backgrounds. Chapter 3 contains application of routing along a considerably long and thin lake by hydrologic and hydraulic methods and comparison of the results. In Chapter 4, a practical modification for method of hydrologic routing to improve the outflow hydrograph obtained by it is suggested.

2. REVIEW OF LITERATURE ABOUT THE PROBLEM OF ROUTING IN RIVERS AND RESERVOIRS

2.1 Introduction

Flow routing is a procedure to determine the time and magnitude of flow at a point downstream from known or assumed hydrograph at one or more points upstream. If the flow is a flood, the procedure is specifically known as flood routing. There are lumped and distributed routing systems in the literature [1,3]. The flow is calculated as a function of time alone at a particular location called as a lumped flow. In a distributed system routing, the flow is calculated as a function of space and time throughout the system. Routing by lumped system methods is called hydrologic routing, and routing by distributed systems methods is referred to as hydraulic routing, also.

In this chapter, four particular methods are explained. First, hydrologic river routing by the Classical Muskingum method. Second, hydrologic routing methods for reservoirs by the different types of methods which are based on the continuity equation are reviewed. Finally, hydraulic routing methods in rivers and reservoirs by distributed flow routing by using the partial differential equations are summarized [1-8].

2.2 Lumped System Routing (Hydrologic Routing)

The flow is calculated as a function of time alone at a particular location in a lumped system model. Governing equation is the continuity equation:

$$\frac{dS}{dt} = I(t) - Q(t) \tag{2.1}$$

where I(t) is the input, Q(t) is the output, S(t) is the storage.

2.2.1 Classical Muskingum Method

The Muskingum method is a commonly used hydrologic routing method for handling a variable discharge-storage relationship. This method models the storage volume of flooding in a river channel by a combination of wedge and prism storages. During rising stages a considerable volume of wedge storage may exist. During the recession, outflow exceeds inflow, resulting in a negative wedge [3-6]. In addition, there is a prism of storage which is formed by a volume of constant cross section along the length of prismatic channel (Fig.(2.1)).

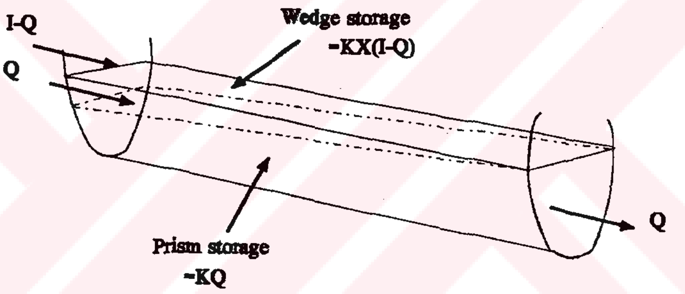

Figure 2.1 Prism and wedge storages in a channel reach

The volume of prism storage is equal to KQ where K is a proportionality coefficient, and the volume of wedge storage is equal to KX(I-Q) where X is a weighing factor having the range $0 \leq x \leq 0.5$. The total storage is therefore the sum of two components which can be arranged as follows:

$$S = K[XI + (1-X)Q] \tag{2.2}$$

Eq.(2.2) is written for two successive time intervals j and j+1, respectively. The change in storage over time interval Δt is

$$S_{J+1} - S_J = K[[XI_{J+1} + (1-X)Q_{J+1}] - [XI_J + (1-X)Q_J]] \tag{2.3}$$

The change in storage can also be expressed, using the continuity equation, as

$$S_{j+1} - S_j = \frac{I_j + I_{j+1}}{2}\Delta t - \frac{Q_j - Q_{j+1}}{2}\Delta t \tag{2.4}$$

Combining the Eqs. (2.3) and (2.4) gives the simple form of the equations.

$$Q_{j+1} = C_1 I_{j+1} + C_2 I_j + C_3 Q_j \tag{2.5}$$

which is the routing equation for the Muskingum method where,

$$C_1 = \frac{\Delta t - 2KX}{2K(1-X) + \Delta t} \tag{2.6}$$

$$C_2 = \frac{\Delta t + 2KX}{2K(1-X) + \Delta t} \tag{2.7}$$

$$C_3 = \frac{2K(1-X) - \Delta t}{2K(1-X) + \Delta t} \tag{2.8}$$

$$C_1 + C_2 + C_3 = 1 \tag{2.9}$$

If the observed inflow and outflow hydrographs are available for a river reach, the values of K and X can be determined. Assuming various values of X and using known values of the inflow and outflow, successive values of the numerator and denominator of the following expression for K, derived from Eqs.(2.3) and (2.4) can be computed.

$$K = \frac{0.5\,\Delta t\ [(I_{j+1} + I_j) - (Q_{j+1} + Q_j)]}{X(I_{j+1} - I_j) + (1 - X)(Q_{j+1} - Q_j)} \tag{2.10}$$

The value of X that produces a loop closest to a single line is taken to be the correct value for the reach and K according to Eq.(2.10) is equal to slope of the line. Since K is the time required for incremental flood wave to traverse the reach.

2.2.2 Hydrologic Routing Methods for Reservoirs (Level Pool Routing)

Hydrologic routing is a procedure for calculating the outflow hydrograph from a reservoir with a horizontal water surface, given its inflow hydrograph and storage-outflow characteristics. A fundamental characteristic of a reservoir is that its storage is closely related to its outflow rate as it is shown in Fig.(2.2).

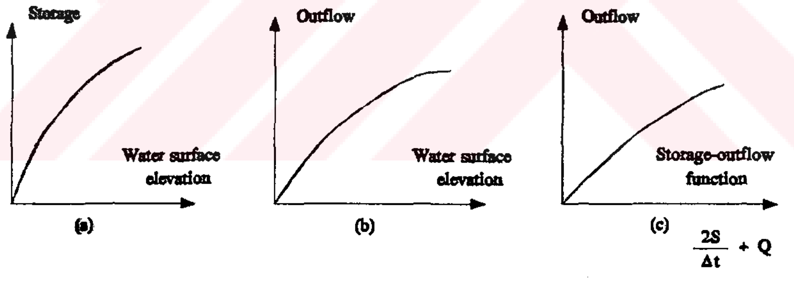

Figure 2.2 Storage-outflow function for hydrologic routing on the basis of S-H and H-Q curves.

In the hydrologic routing, many methods have been proposed. All of them are only based on the continuity equation. It is assumed that, the water surface in the reservoir is horizontal at all times. It means water surface level rises or falls horizontally. The time horizon is broken into intervals of duration Δt, indexed by j, and the continuity equation (Eq.(2.1)) integrated over each time interval (Fig.(2.3)). For the j^{th} time interval:

$$\int_{S_j}^{S_{j+1}} dS = \int_{J\Delta t}^{(j+1)\Delta t} I(t)\,dt - \int_{J\Delta t}^{(j+1)\Delta t} Q(t)\,dt \tag{2.11}$$

If the variation of inflow and outflow over the time interval j is approximately linear, the change in storage over the interval can be found by rewriting Eq.(2.11) as

$$S_{j+1} - S_j = \frac{I_j + I_{j+1}}{2}\Delta t - \frac{Q_j + Q_{j+1}}{2}\Delta t \tag{2.12}$$

The values of I_j, I_{j+1}, Q_j, S_j are known but Q_{j+1},S_{j+1} are unknowns. Two unknowns are isolated by multiplying the each side by $2/\Delta t$ and Eq.(2.12) is rearranged as:

$$\left(\frac{2S_{j+1}}{\Delta t} + Q_{j+1}\right) = (I_j + I_{j+1}) + \left(\frac{2S_j}{\Delta t} - Q_j\right) \tag{2.13}$$

In order to calculate the outflow, Q_{j+1} from Eq.(2.13) a storage-outflow function relating $2S/\Delta t+Q$ to Q is needed. This is obtained by using elevation versus storage and elevation versus outflow relationships (Fig.(2.2a), (2.2b)).

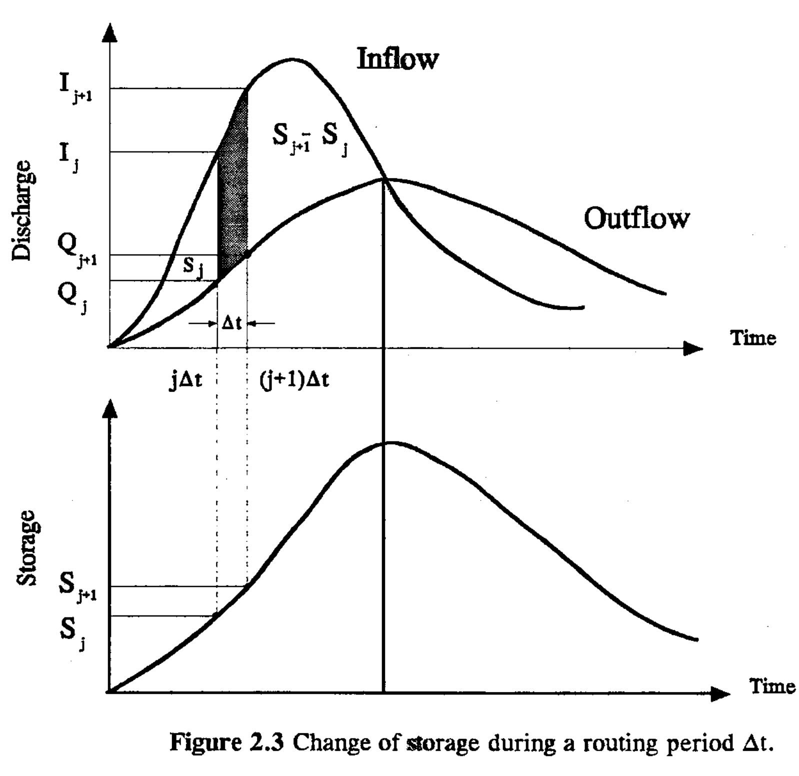

Figure 2.3 Change of storage during a routing period Δt.

The relationship between elevation and storage can be derived by planimetering topographic maps or from field surveys. The temporary storage S for h is approximately determined by using Eq.(2.14). In which A_1, A_2 are the surface area of

$$S=\frac{h}{2}(A_1+A_2) \tag{2.14}$$

the reservoir when h=0 and h=h respectively. The elevation versus discharge relation is derived from hydraulic relating head and discharge for various types of spillways and outlet works, e.g. most frequently used one is the uncontrolled overflow ogee-crest spillway including discharge formula of $Q=CLH^{3/2}$ [8]. The value of Δt is taken as the time interval of the inflow hydrograph. It should be chosen such that it will be neither too long nor too short. If it is too long and exceeds the travel time through reservoir, then peak discharge could pass through the reservoir between time intervals and could therefore not be computed. If Δt is too short, then it takes a longer to perform flood routing. Δt must be one third or one half of the travel time through reservoir. For a given value of water surface elevation, the values of storage S and discharge Q are determined (parts (a) and (b) of Fig.(2.2)), then the value of $2S/\Delta t+Q$ is calculated and plotted (part c of Fig.(2.2)).

2.3 Distributed System Routing (Hydraulic Routing)

Hydraulic Routing methods are used where hydrologic method may not suffice in stream channels of a watershed, where the flowrate, velocity, and depth vary in space throughout the watershed. The advantage of distributed flow routing is the computation of the flowrate and water level simultaneously instead of separately. This type of method is based on partial differential equations which are known as the Saint-Venant equations, first developed by Barre de Saint-Venant in 1871, describing one dimensional unsteady open channel flow, which is applicable in this case[1,3].

2.3.1 Saint-Venant Equations

The following assumptions are necessary for derivation of the Saint-Venant equations:

1. The flow is one dimensional; depth and velocity vary only in the longitudinal direction of the channel.

2. Flow is assumed to vary gradually along the channel so that hydrostatic pressure prevail and vertical acceleration can be neglected.

3. The longitudinal axis of the channel is approximated as a straight line.

4. The bottom slope of the channel is small and the channel bed is fixed; that is effects of scour and deposition are negligible.

5. Resistance coefficients for steady uniform turbulent flow are applicable so that relationship such as Manning's equations can be used to describe resistance effects.

6. The fluid is incompressible and of constant density throughout the flow.

Continuity Equation :

$$\frac{d}{dt}\iiint_{c.v} \rho \, d\forall + \iint_{c.s} \rho \, V \cdot dA = 0 \tag{2.15}$$

where, ρ is the fluid density, first term is the rate of change of mass stored within the control volume. The second term is the net mass outflow across the control surface. After performing some differential operations, conservation form of the continuity equation can be written as

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q \tag{2.16}$$

Momentum Equation :

Newton's second law is written in the form of Reynold's transport theorem as in Eq.(2.17). This states that the sum of the forces applied is equal to the rate of change of momentum stored within the control volume plus the net outflow of momentum across the control surface. This equation in the form of $\Sigma F=0$ was applied to steady

$$\Sigma F = \frac{d}{dt}\iiint_{c.v} V\rho d\forall + \iint_{c.s} V\rho V\cdot dA \tag{2.17}$$

uniform flow in an open channel. In this study unsteady nonuniform flow is considered.

Forces :

There are five forces acting on the control volume.

$$\Sigma F = F_g + F_f + F_e + F_w + F_p \tag{2.18}$$

where,

F_g = gravity force along the channel due to weight of water in the c.v.

F_f = friction force along the bottom and sides of the c.v.

F_e = contraction/expansion force along the bottom and sides of the c.v.

F_w = wind shear force on the water surface.

F_p = unbalanced pressure force.

After also performing some differential operations, Eq.(2.17) takes the following conservation form.

$$\frac{1}{A}\frac{\partial Q}{\partial t} + \frac{1}{A}\frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + g\frac{\partial y}{\partial x} - g(S_0 - S_f) = 0 \tag{2.19}$$

In which, first term is the local acceleration term, second term is the convective acceleration term, third term is the pressure force term, fourth term is the difference between gravity force and friction force.

Eqs.(2.16) and (2.17) are known as the Saint-Venant equations for one dimensional unsteady flow in an open channel [1,3].

Wave Motion :

There are three types of wave models in the distributed system routing. The simplest distributed model is the kinematic wave model, which neglects the local acceleration, convective acceleration, and pressure terms in the momentum equation. The diffusion wave model neglects the local and convective acceleration terms but incorporates the pressure term. The dynamic wave model considers all the acceleration and pressure terms in the momentum equation.

Kinematic waves govern flow when inertial and pressure forces are not important. Dynamic waves govern flow when these forces are important, such as in the movement of a large flood wave in a wide river. Fig.(2.4) illustrates the difference between kinematics and dynamic wave motion within a differential from the viewpoint of a stationary observer on the river bank.

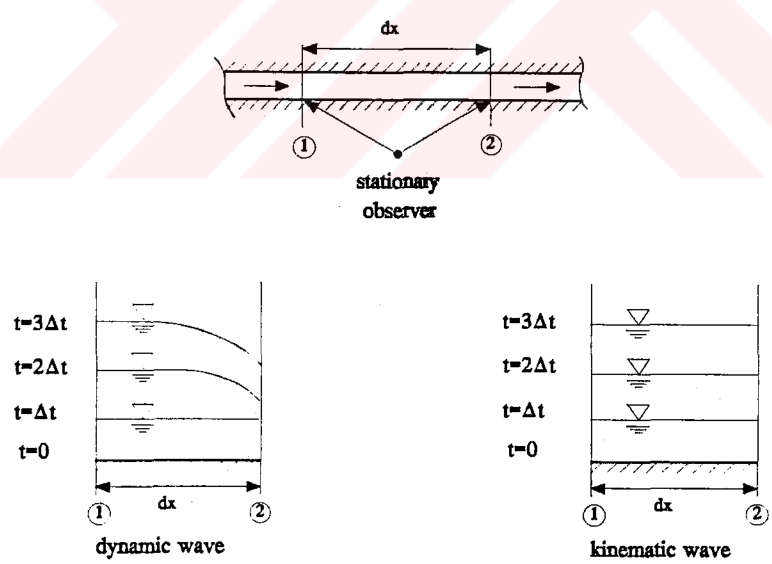

Figure 2.4 Kinematic and dynamic waves in a short reach of channel as seen by stationary observer.

2.3.2 Methods for Solving Saint-Venant Equations

Methods for solving partial differential equations may be classified as direct numerical methods and characteristic methods. In direct numerical methods, finite-difference equations are formulated from the original partial differential equations for continuity and momentum. Solutions for the flow rate and water surface elevations are then obtained for incremental times and distances along the stream or river. In characteristic methods, the partial differential equations are first transformed to a characteristic form, and the characteristic equations are solved analytically.

In numerical methods for solving partial differential equations, the calculations are performed on a grid placed over x-t plane. The x-t grid is a network of points defined by taking distance increments of length Δx and time increments of duration Δt. As shown in Fig. (2.5) the distance points are denoted by index i and the time points by index j. A time line is a line parallel to the x-axis through all the distance points at a given value of time.

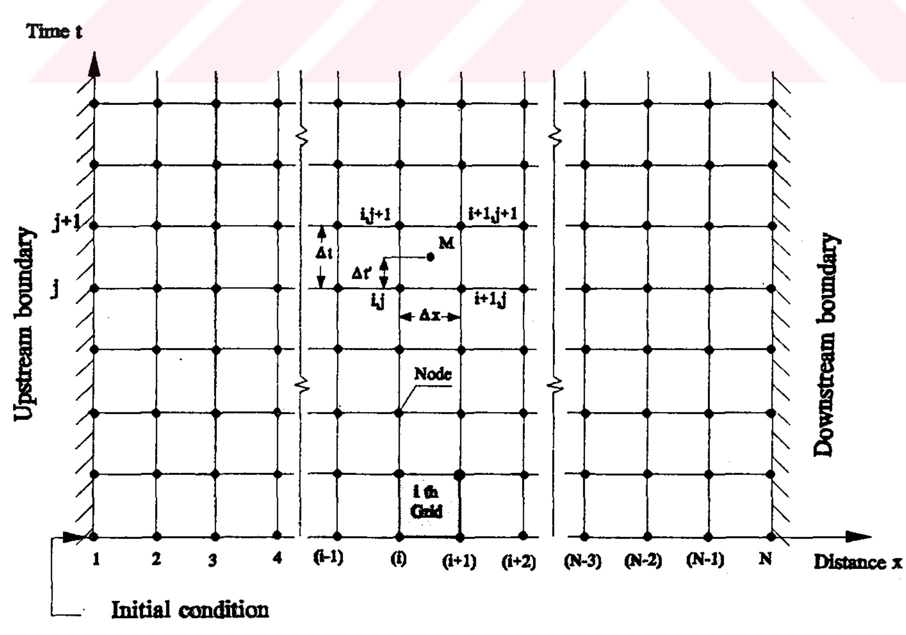

Figure 2.5 The finite-difference forms of the Saint-Venant equations on the x-t plane

The Saint-Venant equations are solved from one time line to the next simultaneously for all points along the time line.

The implicit finite-difference scheme uses a weighted four point method between adjacent time lines at a point M, as shown in Fig.(2.5). The conservation form of the Saint-Venant equations are used.

Continuity Equation :

$$\frac{\partial Q}{\partial x}+\frac{\partial A}{\partial t}-q=0 \tag{2.20}$$

Momentum Equation :

$$\frac{\partial Q}{\partial t}+\frac{\partial(\beta Q^2/A)}{\partial x}+gA\left(\frac{\partial h}{\partial x}+S_f+S_e\right)-\beta q v_x+W_f B=0 \tag{2.21}$$

where:

x = longitudinal distance along the channel or river

t = time

A = cross sectional area of flow

q = lateral inflow per unit length along the channel

h = water surface elevation

ν_x = velocity of lateral flow in the direction of channel flow

S_f = friction slope

S_e = eddy loss slope

B = width of the channel at the water surface

W_f = wind shear force

β = momentum correction factor

g = acceleration due to gravity

The partial dervatives in Eqs.(2.20) and (2.21) are computed numerically making use of below approximations:

$$\frac{\partial Q}{\partial t} \approx \frac{Q_i^{j+1} + Q_{i+1}^{j+1} - Q_i^j - Q_{i+1}^j}{2\Delta t} \quad \textbf{(2.22)}$$

$$\frac{\partial Q}{\partial x} \approx \theta \frac{Q_{i+1}^{j+1} - Q_i^{j+1}}{\Delta x} + (1-\theta) \frac{Q_{i+1}^j - Q_i^j}{\Delta x} \quad \textbf{(2.23)}$$

$$\frac{\partial h}{\partial x} \approx \theta \frac{h_{i+1}^{j+1} - h_i^{j+1}}{\Delta x_i} + (1-\theta) \frac{h_{i+1}^j - h_i^j}{\Delta x_i} \quad \textbf{(2.24)}$$

θ is a weighing factor to define the spatial derivative. If θ is taken 0.5 this means point M is located at the center of the grid in the Fig.(2.5).

$$\frac{\partial Q}{\partial x} \approx \frac{Q_{i+1}^{j+1} - Q_i^{j+1} + Q_{i+1}^j - Q_i^j}{2\Delta x_i} \quad \textbf{(2.25)}$$

$$\frac{\partial h}{\partial x} \approx \frac{h_{i+1}^{j+1} - h_i^{j+1} + h_{i+1}^j - h_i^j}{2\Delta x_i} \quad \textbf{(2.26)}$$

It is assumed that, $\beta = 1$ and S_e, W_f and q are neglected.

$$\frac{\partial (Q^2/A)}{\partial x} \cong \frac{(Q^2/A)_{i+1}^{j+1} - (Q^2/A)_i^{j+1} + (Q^2/A)_{i+1}^j - (Q^2/A)_i^j}{2\Delta x_i} \quad \textbf{(2.27)}$$

$$\frac{\partial A}{\partial t} = \frac{A_i^{j+1} + A_{i+1}^{j+1} - A_i^j - A_{i+1}^j}{2\Delta t} \qquad (2.28)$$

$$A = \bar{A} = \frac{A_i^j + A_{i+1}^j + A_{i+1}^{j+1} + A_i^{j+1}}{4} \qquad (2.29)$$

Substituting Eqs.(2.25), (2.28), into (2.20), finite difference form of the continuity equation becomes:

$$\frac{Q_{i+1}^{j+1} - Q_{i+1}^j - Q_i^j}{2\Delta x} + \frac{A_i^{j+1} - A_{i+1}^{j+1} - A_i^j - A_{i+1}^j}{2\Delta t} = 0 \qquad (2.30)$$

Similarly, the finite difference form of the momentum equation is:

$$\frac{Q_i^{j+1} - Q_i^j + Q_{i+1}^{j+1} - Q_{i+1}^j}{2\Delta t} + \frac{(Q^2/A)_{i+1}^j - (Q^2/A)_i^j + (Q^2/A)_{i+1}^{j+1} - (Q^2/A)_i^{j+1}}{2\Delta x_i}$$

$$+\ g\,(\frac{A_i^j + A_{i+1}^j + A_{i+1}^{j+1} + A_i^{j+1}}{4})[(\frac{h_{i+1}^j - h_i^j + h_{i+1}^{j+1} - h_i^{j+1}}{2\Delta x_i}) + S_f] = 0 \qquad (2.31)$$

Finite difference form of the continuity equation (2.30) is multiplied by $2\Delta x_i$ for simplification and Eq.(2.32) is obtained.

$$Q_{i+1}^{j+1} - Q_i^{j+1} + Q_{i+1}^j - Q_i^j + \frac{\Delta x_i}{\Delta t}(A_i^{j+1} + A_{i+1}^{j+1} + A_i^j - A_{i+1}^j) = 0 \qquad (2.32)$$

Similarly, finite difference form of the momentum equation (2.31) is multiplied by Δx_i, which, results in the equation given below:

$$\frac{\Delta x_i}{\Delta t}(Q_i^{j+1} - Q_i^j + Q_{i+1}^{j+1} - Q_{i+1}^j) + ((Q^2/A)_{i+1}^j - (Q^2/A)_i^j + (Q^2/A)_{i+1}^{j+1} - (Q^2/A)_i^{j+1})$$

$$+ g\bar{A}_i^j\,(h_{i+1}^j - h_i^j + h_{i+1}^{j+1} - h_i^{j+1} + \bar{S}_{f_i}^{\,j}) = 0 \qquad \textbf{(2.33)}$$

The continuity and momentum equations are nonlinear and solved at each one of the N-1 grids shown in Fig.(2.5). Q, h are unknown at each one of the N grids, so there are 2N unknowns in all. The two additional equations required to complete the solution are supplied by the upstream and downstream boundary conditions. Upstream boundary is the inflow hydrograph while downstream boundary is the stage and discharge hydrograph.

The system of nonlinear equations can be expressed in functional form in terms of the unknowns h and Q at time level j+1, as follows :

$UB\ (h_1, Q_1) = 0$	upstream boundary condition
$C_1\ (h_1, Q_1, h_2, Q_2) = 0$	continuity for grid 1
$M_1\ (h_1, Q_1, h_2, Q_2) = 0$	momentum for grid 1
.	
.	
.	
$C_i\ (h_i, Q_i, h_{i+1}, Q_{i+1}) = 0$	continuity for grid i
$M_i\ (h_i, Q_i, h_{i+1}, Q_{i+1}) = 0$	momentum for grid i
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$C_{N-1}(h_{N-1}, Q_{N-1}, h_N, Q_N) = 0$	continuity for grid N-1
$M_{N-1}(h_{N-1}, Q_{N-1}, h_N, Q_N) = 0$	momentum for grid N-1

$DB\ (h_N, Q_N) = 0$ downstream boundary condition

Newton-Raphson method is used to solve the system of 2N nonlinear equations $x^k=(Q_1^k, h_1^k, Q_2^k, h_2^k,\ldots,Q_N^k, h_N^k)$ is the vector of unknown quantities for iteration k. First values of Q and h are known from initial conditions. The partial derivative terms are solved to define the Jacobian coefficient matrix by using the values for x^k. Then, residuals of above system are computed one by one for iteration k. After that, a system of linear equations is solved for dh_i and dQ_i using Gaussian elimination. Finally, the values of h_i^{k+1} and Q_i^{k+1} are determined by using Eqs.(2.34), (2.35). The process is repeated until $(x_{k+1}- x_k)$ is smaller than some specified tolerance [1,3,5].

$$h_i^{k+1} = h_i^k + dh_i \tag{2.34}$$

$$Q_i^{k+1} = Q_i^k + dQ_i \tag{2.35}$$

3. ROUTING THROUGH RESERVOIRS WITH A CONSIDERABLY LONG AND THIN LAKE

3.1 Introduction

For large reservoirs, application of hydraulic routing is not necessary due to its excessive computational load. Since the velocities even during a flood event are negligibly small, the wall shear forces and the forces due to changes in momentum are of negligible magnitudes also.

On the other hand, there are many reservoirs in the world, existing or planned, which have thin-and-long lakes due to the natural shapes of deep and narrow valleys. Hydrologic routing for thin-and-long reservoirs may not yield correct outflow hydrographs, because the long-and-thin lake of such reservoirs may act like a deep river reach, and the dynamic effects of the flowing flood wave are not taken into account by the simple algorithm of the hydrologic routing.

Therefore, in order to assess quantitatively the error of the classical hydrologic routing in the computed outflow hydrograph, the hydraulic routing was performed on three reservoirs in Turkey, which have very long and narrow lakes positioned in deep and narrow valleys. For this purpose, the verified package program, DUFLOW, was used. Next, the hydrologic routing computations were also applied on the same three reservoirs with the help of another sound computer program developed at the Civil Engineering Department of the Çukurova University [2].

In this chapter, acquisition of data, information about the package program DUFLOW [7], application of hydrologic and hydraulic routing methods are explained. Then, comparison of the results by two routing systems are exhibited in tabular and graphical forms.

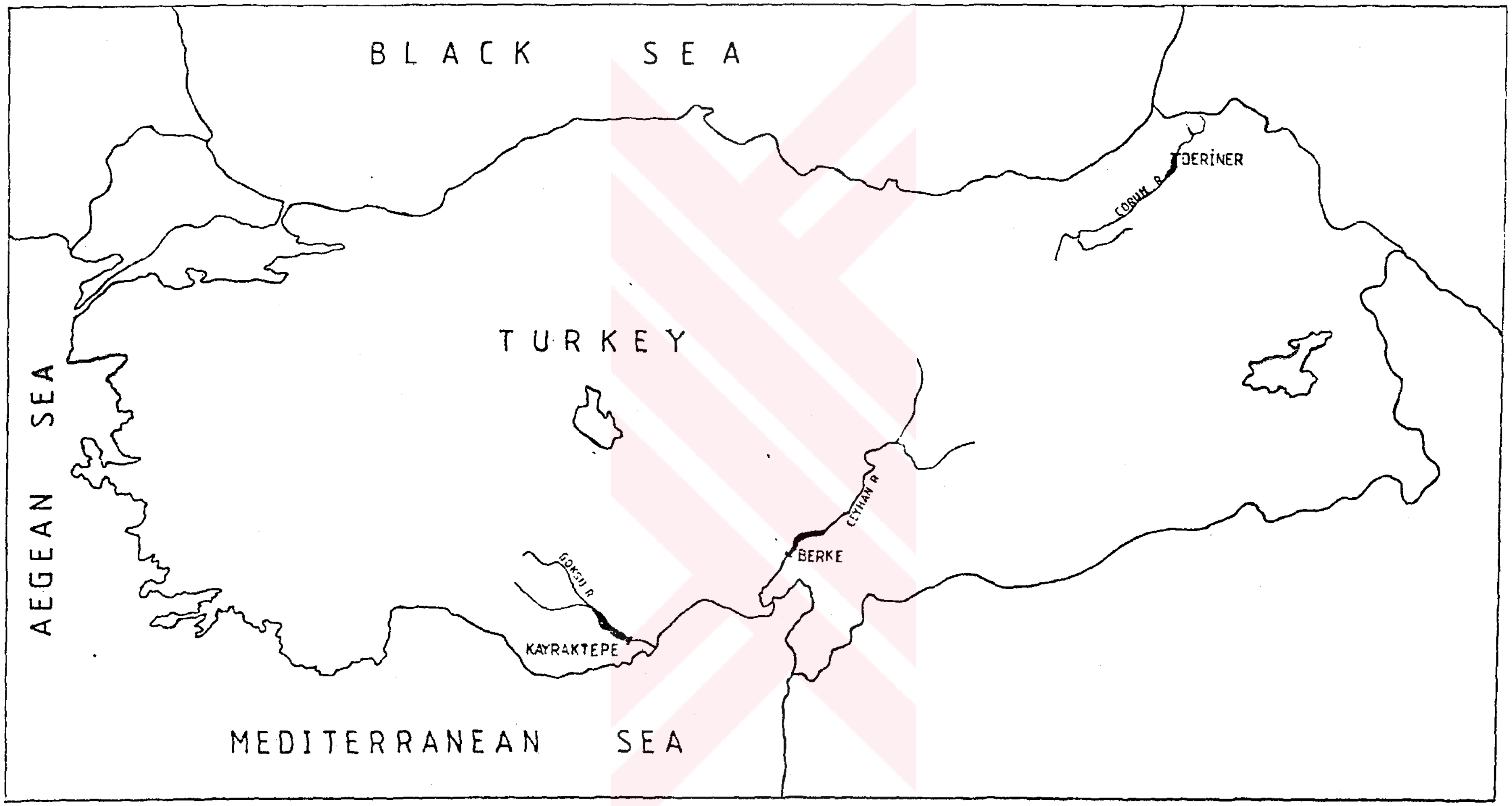


Figure 3.1 Positions of the three thin-and-long reservoirs chosen.

3.2 Acquisition of Pertinent Data for Typical Reservoirs in Anatolia

The first site chosen (Deriner), presently under planning stage, is on Çoruh River at the Northeastern part of Turkey. This is a river with a steep slope conveying wild waters in a narrow and deep valley. The second one (Berke) is a dam presently under construction on Ceyhan River in Southern Anatolia. The third one (Kayraktepe) is also under construction on Göksu River again in the South (Fig.(3.1)). The geometrical properties such as reservoir lengths, valley widths, channel slopes and actual cross-sections of the three reservoirs were carefully extracted from 1/25000 and 1/5000 scale topographic maps. The storage versus elevation relationship of each of the three valleys chosen was computed stepwise in classical way by using cross section profiles. Brief geometrical data of these reservoirs are given in Table (3.1).

Table 3.1 Summary of some peculiarities of the three thin-and-long reservoirs chosen

Name of reservoir	Storage capac. at initial water surface elev. (Mill.m^3)	Reservoir length at initial water surface elev. (m)	Aver. valley width at init. w. s. elev. (m)	Aver.channel bottom slope of the valley
Deriner	15.8	8,880	150	0.00302
Berke	260	22,000	250	0.00695
Kayraktepe	748	48,475	680	0.00146

As inflow hydrographs, the curve of the Gamma distribution was made use of in a fashion similar to those of Croley [4] and Perumal [6]. The shape parameter was taken as 3.0 for all the hydrographs. Coincidentally, Perumal [6] used 3.02 for the shape parameter of the inflow in his study.

For each reservoir three different peaks for these inflow hydrographs were decided upon, and three different spillway lengths were taken, which altogether resulted in 9 different inflow hydrograph-spillway combinations. The peaks and spillway lengths chosen were proportionate with the storage capacity of the reservoir above the spillway crest elevation. H-Q relationship is determined by using Eq.(3.1) which is the unregulated ogee-crested spillway formula [8].

$$Q = C\ L\ H^{3/2} \tag{3.1}$$

where C is the discharge coefficient, L is the effective length of the weir crest, and H is the measured head above the crest excluding the velocity head. The tabular forms of Q-H and S-H relationships are illustrated in section (3.5).

3.3 Information About the Package Program DUFLOW

3.3.1 History and Purpose of the Model

The introduction and wide-spread utilization of personal computers stimulated the demand for easy to use computer models.

The Department of Sanitary Engineering and Water Management, Faculty of Civil Engineering, Delft, University of Technology and the International Institute for Hydraulic and Environmental Engineering, Delft, in Netherlands, stimulated various organizations to contribute to this model development. As a result of this comprehensive endeavour, the package program called DUFLOW was developed for unsteady flow computations in networks of open water courses. The DUFLOW system consists of three modules which are controlled by MASTER MENU. The interactive input module and output module are written in GW-Basic, the noninteractive computational module is written in Fortran-77 [7].

The DUFLOW program package is designed for various categories of users. The model can be used by water managers and designers and has proved to be a very useful tool in educations. Since it runs on IBM-compatible PC computers it can be operated in almost every scientific or engineering environment. In engineering education, the major advantage is the short learning time due to its menu structure and screen-oriented input and output. Allowing up to 250 elementary sections, (river, canal, control structure) DUFLOW can tackle most of the real-life problems.

3.3.2 Options and Elements of DUFLOW

In the DUFLOW system a large range of applications, such as propagation of tidal waves in estuaries, flood waves in rivers, operation of irrigation and drainage systems, etc. can be conducted. Basically free flow in open channel systems is simulated, where control structures like weirs, pumps,culverts and siphons can be included. In case of considering a flood wave in a river, the discharge imposed at the upstream boundary of a river strech is transmitted through a sequence of river sections which may be separated by weirs. Weirs with movable gates can be modelled for overflow and underflow conditions between two nodes i and j.

Boundary conditions can be specified as water levels and discharges, either constant or in the form of a time series or Fourier series. The inflow hydrograph at the first node and the rating curves in tabular form at the last node are used.

A bird's eye view of the network provided by the program allows a quick check of the network including its orientation and the connections between the nodes. Section numbers are located between subsequent nodes and simple shaped cross-sections can be specified with a depth against corresponding width. Initial values of H and Q are required to start the computations. These initial values must be provided by the user, they may be historical measurements, obtained from former computations or just a first reasonable guess. Two types of resistance formulae can be used, Manning and De Chezy. Additionally wind stresses and rainfall conditions can be specified.

3.3.3 Physical and Mathematical Background of DUFLOW

The DUFLOW package is based on the one-dimensional partial differential equations that describe non-stationary flow in open channel as explained in the previous chapter. The continuity and momentum Eqs.(2.16) and (2.17) were also used in DUFLOW package. These equations are discretized in space and time using the finite difference method (Fig.(2.5)). Defining a section Δx_i from node x_i to node x_{i+1} and a time interval Δt from time t_j to t_{j+1}, in a similar way explained in chapter 2, the transformed partial differential equations are written as a system of algebraic equations by replacing the derivatives by finite difference expressions. In short, DUFLOW performs the numerical procedure outlined in the previous chapter.

3.4 Application of the Hydraulic Routing by DUFLOW

The network of three reservoirs was specified by the number of nodes, branches and structures. The geometrical peculiarities of the three thin-and-long reservoirs were illustrated in section 3.2 (Table 3.1). The network of Deriner reservoir was modelled by splitting the length into 15 sections. The model consists of 17 nodes, a structure which is an unregulated ogee spillway at the end node, two boundary conditions which were inflow hydrograph at most the upstream node and rating curve at the most downstream node. Similarly, the models of the other two reservoirs, Berke and Kayraktepe, had 13 sections, 14 nodes and also a spillway and two boundary conditions. Figs.(3.2), (3.3), (3.4) indicate the networks and longitudinal profiles of three models.

The cross-section profiles of the considered intermediate reaches were specified as input data, by entering valley widths against depths at begin and end nodes of each sections. Cross sectional profiles at the most upstream and the most downstream nodes of three reservoir are illustrated in Figs.(3.5), (3.6), (3.7).

At the beginning of the routing, each reservoir was assumed to be full up to the

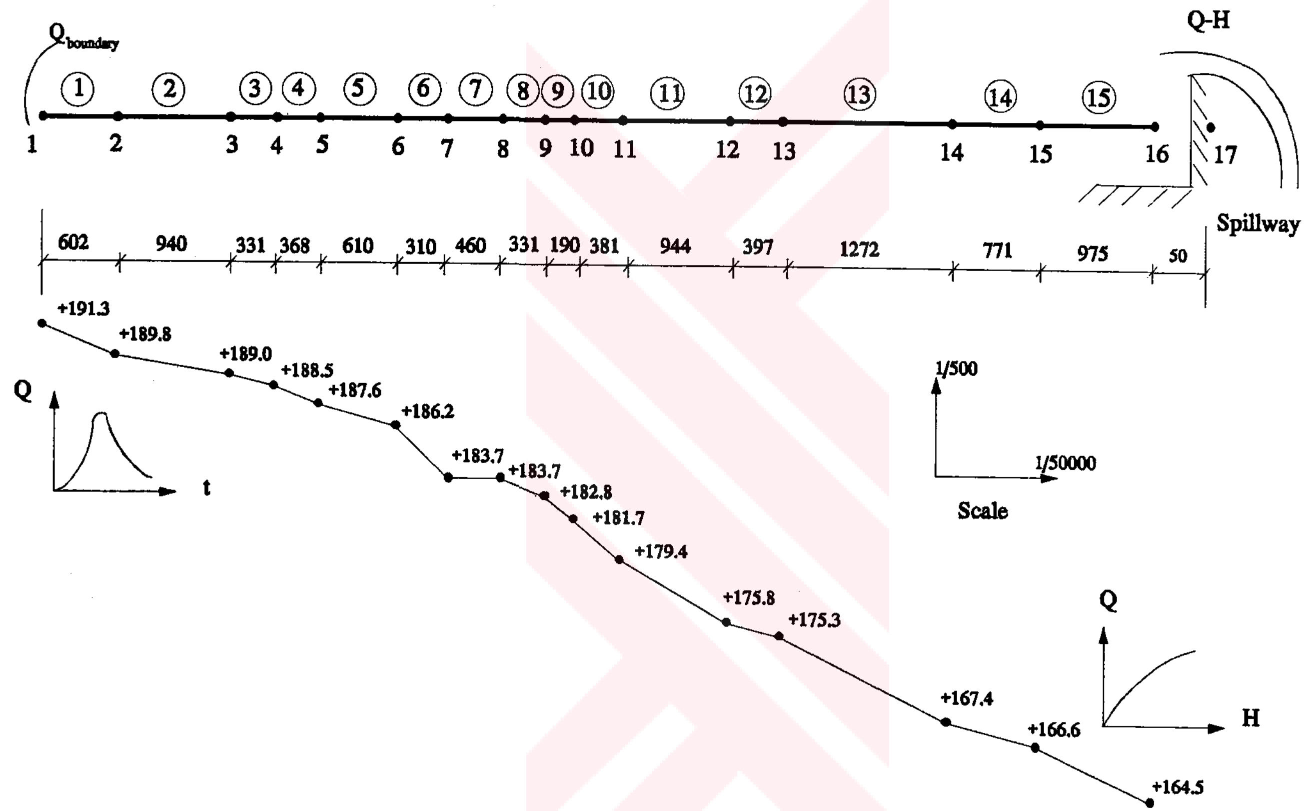


Figure 3.2 Network and longitudinal profile of Deriner Reservoir

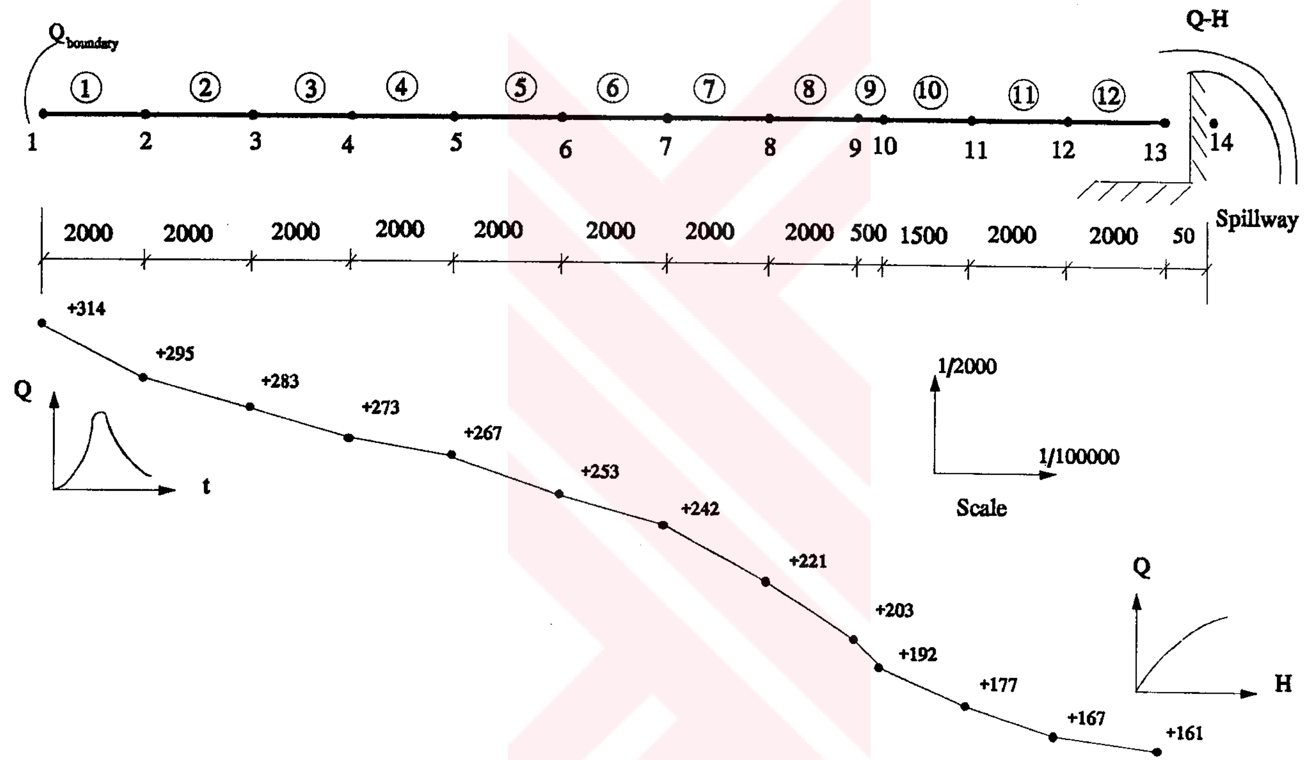


Figure 3.3 Network and longitudinal profile of Berke Reservoir

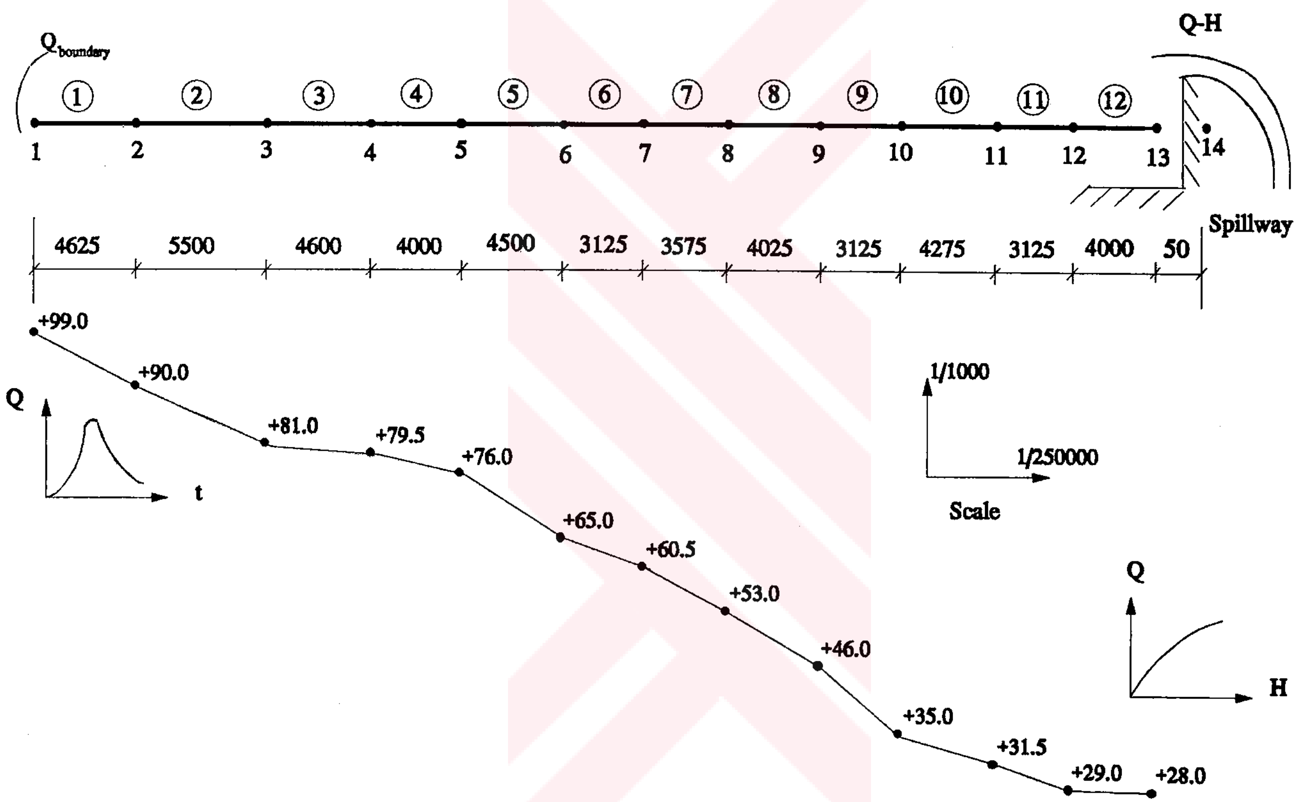


Figure 3.4 Network and longitudinal profile of Kayraktepe Reservoir

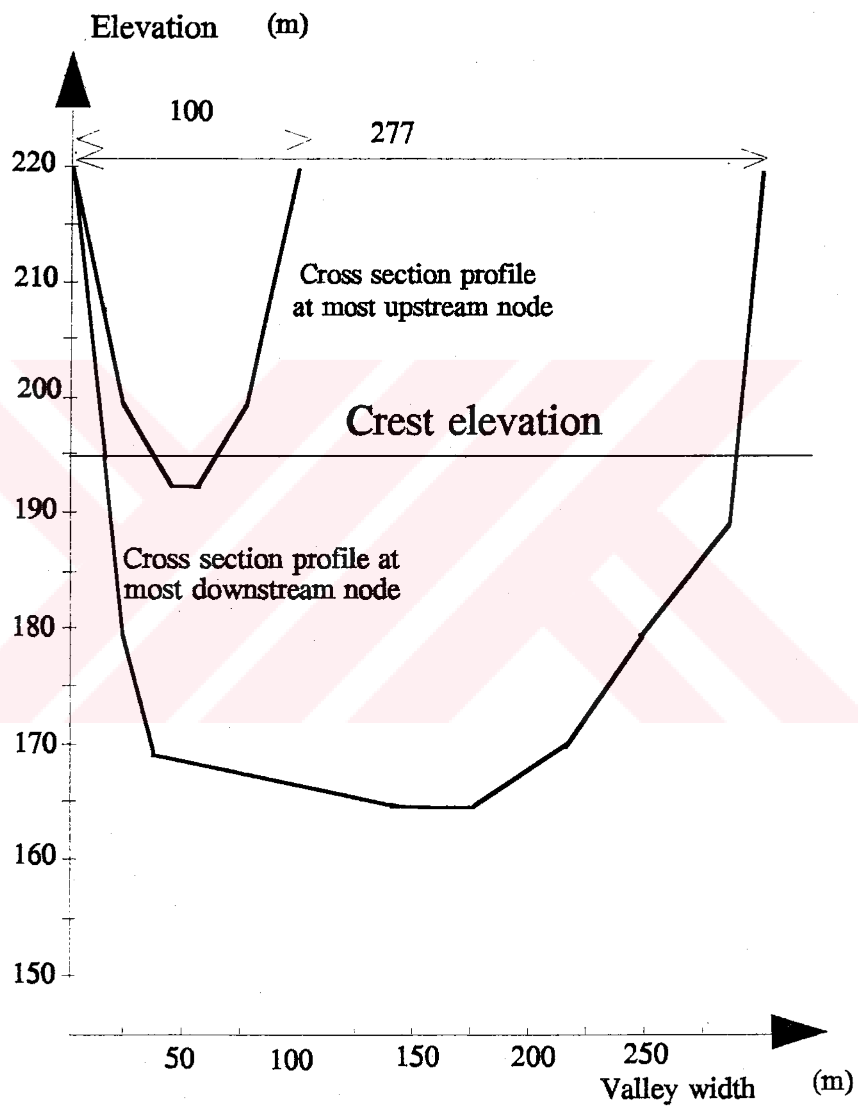


Figure 3.5 Cross section profiles of Deriner reservoir at most upstream and most downstream nodes

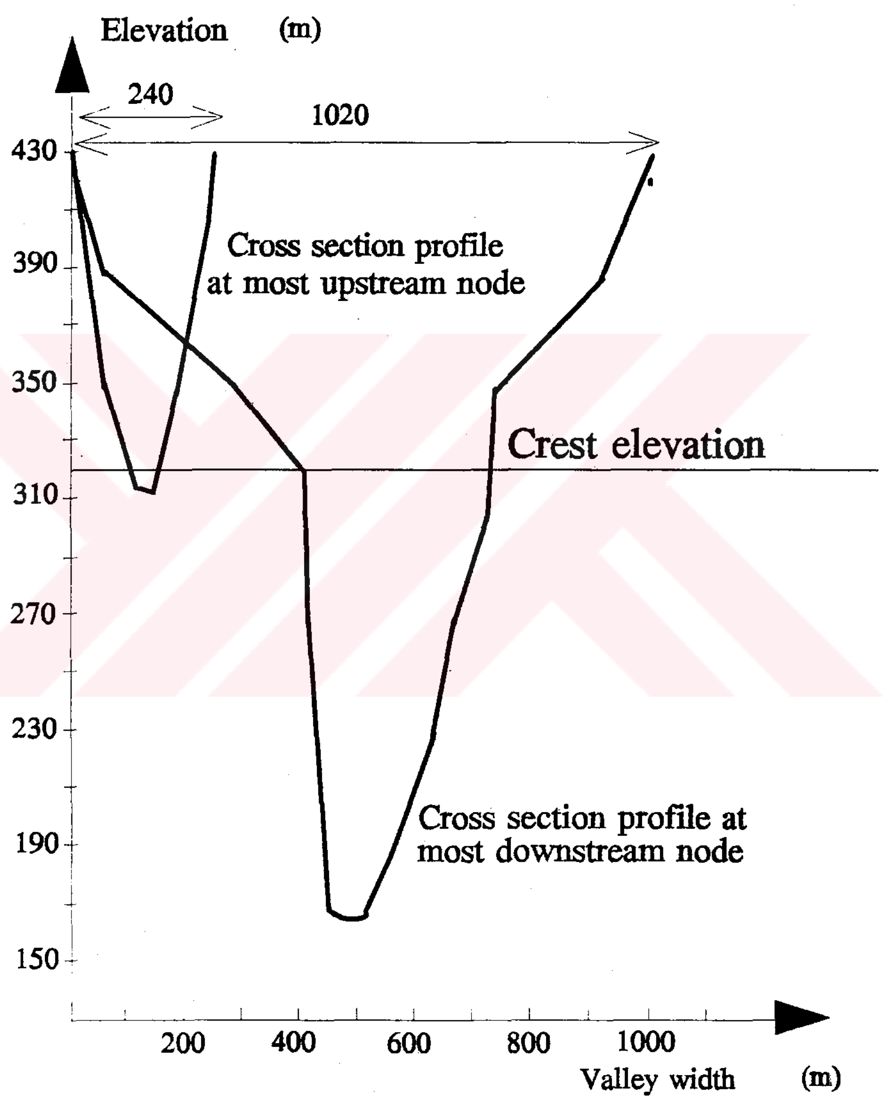


Figure 3.6 Cross section profiles of Berke reservoir at most upstream and most downstream nodes

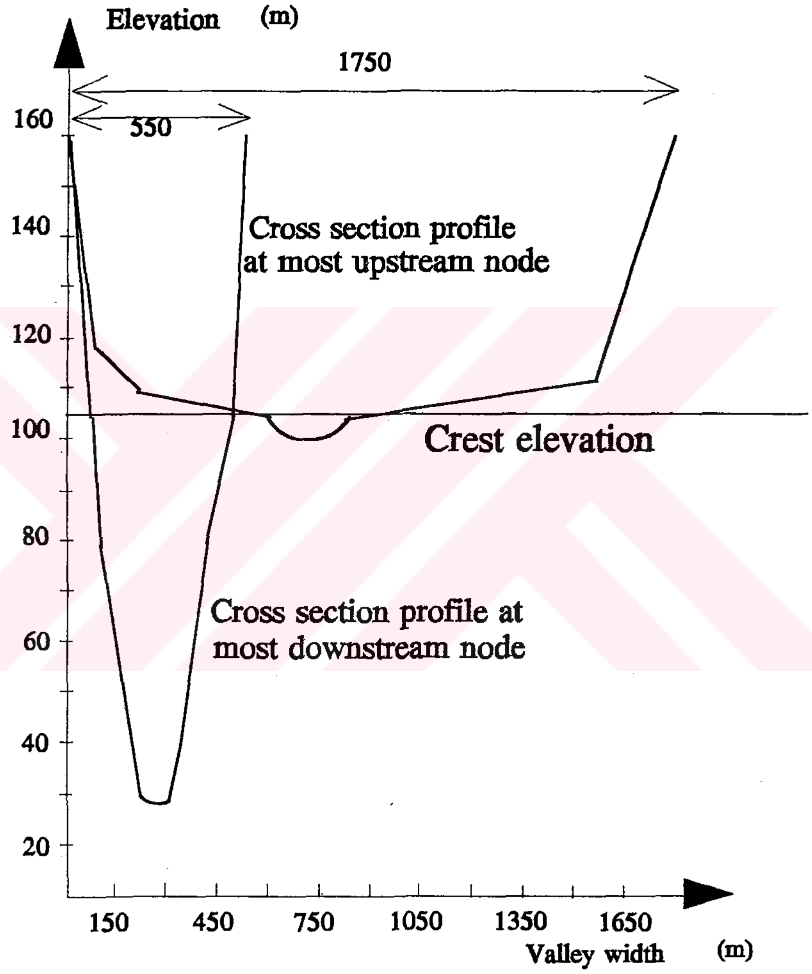


Figure 3.7 Cross section profiles of Kayraktepe reservoir at most upstream and most downstream nodes

spillway crest elevation and the initial value of outflow was taken to be zero. Each inflow hydrograph started with a zero discharge, also. The Chezy friction factors were taken as 45 for Deriner and Berke reservoirs, and 33 for Kayraktepe reservoir.

The package DUFLOW was run for all the 27 combinations of the three reservoirs with a computational time step of 12 minutes. As results, hydrographs at all the intermediate slices of each reach and water surface elevations at the all the nodes were obtained as a function of time.

3.5 Application of the Hydrologic Routing by Level Pool Method

The level pool routing for all these 27 cases were performed by another computer program which correctly computes the outflow hydrographs by an efficient numerical algorithm [2]. This program executing hydrologic routing required the relationships of discharge-elevation of the spillway and storage-elevation of the reservoir as input data. Geometrical and hydraulic peculiarities of the intermediate sections of the valley were not needed, so the level pool routing method was simple. The determination of the Q-H and S-H as explained in section (3.2) are given in tabular in Tables (3.2) and (3.3).

Table 3.2 Storage-elevation values of three reservoirs

DERİNER		BERKE		KAYRAKTEPE	
H(m)	S(10^6m^3)	H(m)	S(10^6m^3)	H(m)	S(10^6m^3)
165	0.000	260	59.536	40	13.460
170	0.433	270	76.916	50	39.583
175	1.400	280	100.224	60	76.416
180	3.470	290	130.491	70	133.988
185	6.177	300	167.180	80	210.490
190	10.134	310	209.908	90	357.320
195	15.770	320	260.276	100	604.563
200	22.965	330	316.918	110	910.253

Table 3.3 Spillway discharge and water surface elevation values of three reservoirs

DERİNER				BERKE				KAYRAKTEPE			
H (m)	Q(m^3/sec)			H (m)	Q(m^3/sec)			H (m)	Q(m^3/sec)		
	L=30	L=20	L=10		L=50	L=40	L=30		L=100	L=80	L=60
195	0	0	0	320	0	0	0	105	0	0	0
196	62	41	20.5	321	103	82	62	106	205	164	123
197	174	116	58	322	290	232	174	107	580	464	348
198	320	213	107	323	533	426	320	108	1065	852	639
199	492	328	164	324	820	656	492	109	1640	1312	984
200	688	458	229	325	1146	917	688	110	2292	1834	1375
201	904	603	301	326	1506	1205	904	111	3013	2410	1808
202	1139	759	380	327	1898	1519	1139	112	3797	3037	2278
203	1392	928	464	328	2319	1855	1392	113	4639	3711	2783
204	1661	1107	553	329	2768	2214	1661	114	5535	4428	3321
205	1945	1297	648	330	3241	2593	1945	115	6483	5186	3890

3.6 Comparison of the Results Obtained by the Two Routing Methods

The same values of three storage-elevation and discharge-elevation relationships were used in both methods. The results of twenty-seven combinations are indicated in Table(3.4).

Table 3.4 The outflow peaks by the two routing methods together with inflow peaks and relative differences between peaks of hydrologic & hydraulic routing (%)

RESV.	Spillway length (m)	Peak of inflow (m^3/s)	Peak of Outflow by hydrologic routing (m^3/s)	Peak of Outflow by DUFLOW (m3/s)	Rel.diff.B/W peaks of hydrologic & hydrau. rout. (%)
DERİNER	30	1000	947	933	1.50
	20	1000	903	886	1.92
	10	1000	This	case is	excluded
	30	500	466	451	3.33
	20	500	440	423	4.02
	*10	500	375	354	5.93
	30	200	179	170	5.30
	20	200	166	156	6.41
	10	200	137	124	10.48
BERKE	50	3000	2779	2702	2.85
	40	3000	2703	2609	3.60
	30	3000	2581	2455	5.13
	50	2000	1819	1761	3.30
	40	2000	1762	1693	4.08
	30	2000	1671	1588	5.23
	50	1000	870	834	4.32
	40	1000	835	795	5.03
	*30	1000	782	736	6.25
KAYRAKTEPE	100	6000	4435	4140	7.13
	80	6000	4124	3809	8.27
	60	6000	3697	3368	9.77
	100	4000	2831	2634	7.50
	80	4000	2619	2410	8.67
	*60	4000	2334	2121	10.00
	100	2000	1285	1183	8.62
	80	2000	1178	1073	9.78
	60	2000	1037	931	11.38

As it is seen from the Table(3.4), peaks of the outflow hydrographs by hydrologic routing were all greater than those by hydraulic routing. The difference grew larger as the spillway capacity became smaller relative to the magnitude of the inflow hydrograph and it changed between approximately 2 to 10% of the peak of the outflow by hydrologic routing.

Out of these 27 cases, three of them which yielded greater differences were selected (marked with * in Table(3.4)) in order to elaborate the overall differences in graphical form. In Figs.(3.8), (3.9), (3.10), the outflow hydrographs computed by DUFLOW and by hydrologic routing are given together with the particular inflow hydrographs for three cases chosen.

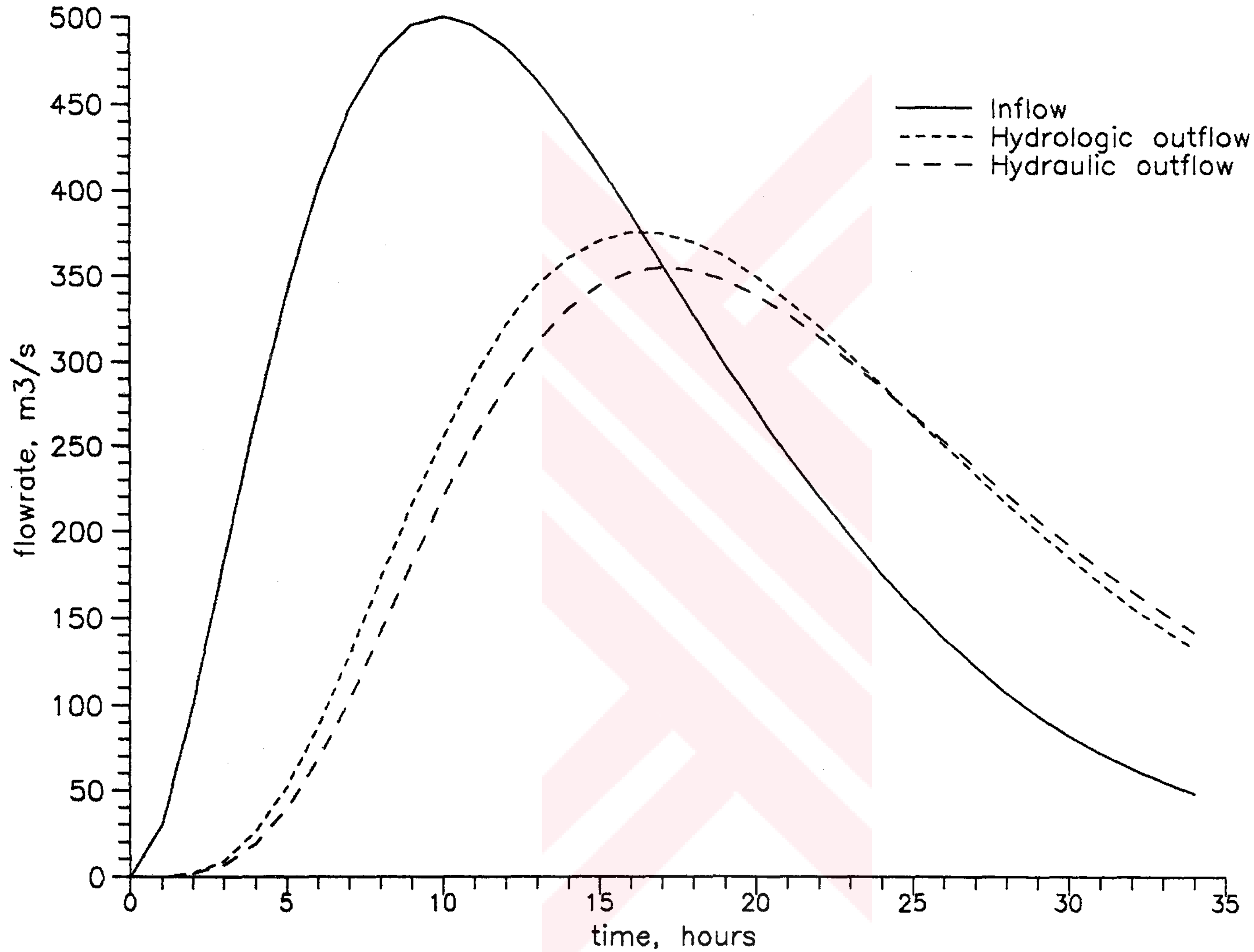


Figure 3.8. Hydrographs of inflow, hydrologic outflow, and hydraulic outflow at Deriner R. with inflow peak of 500 m3/s and spillway length of 10 m

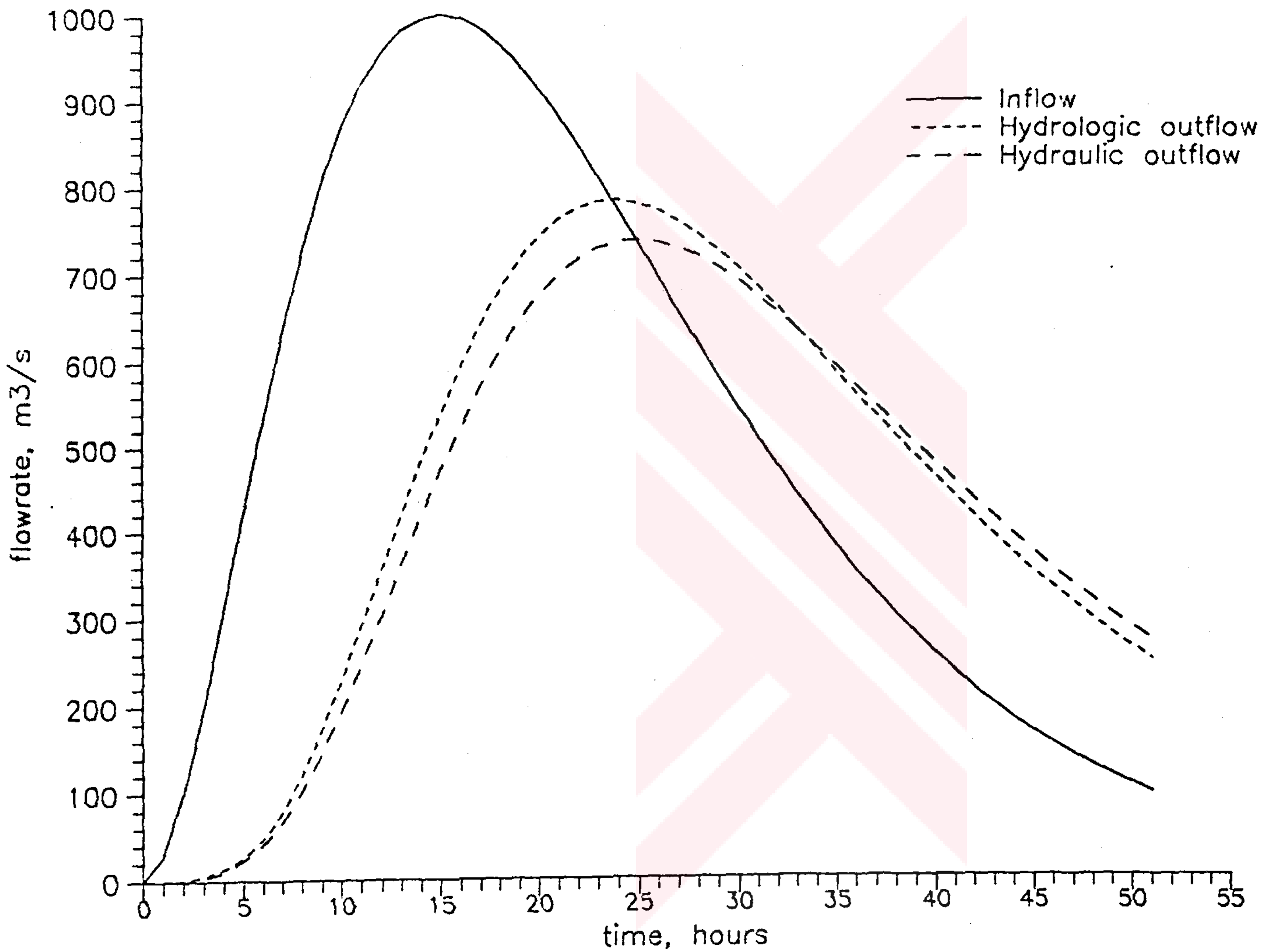


Figure 3.9. Hydrographs of inflow, hydrologic outflow, and hydraulic outflow at Berke Res. with inflow peak of 1000 m3/s and spillway length of 30 m

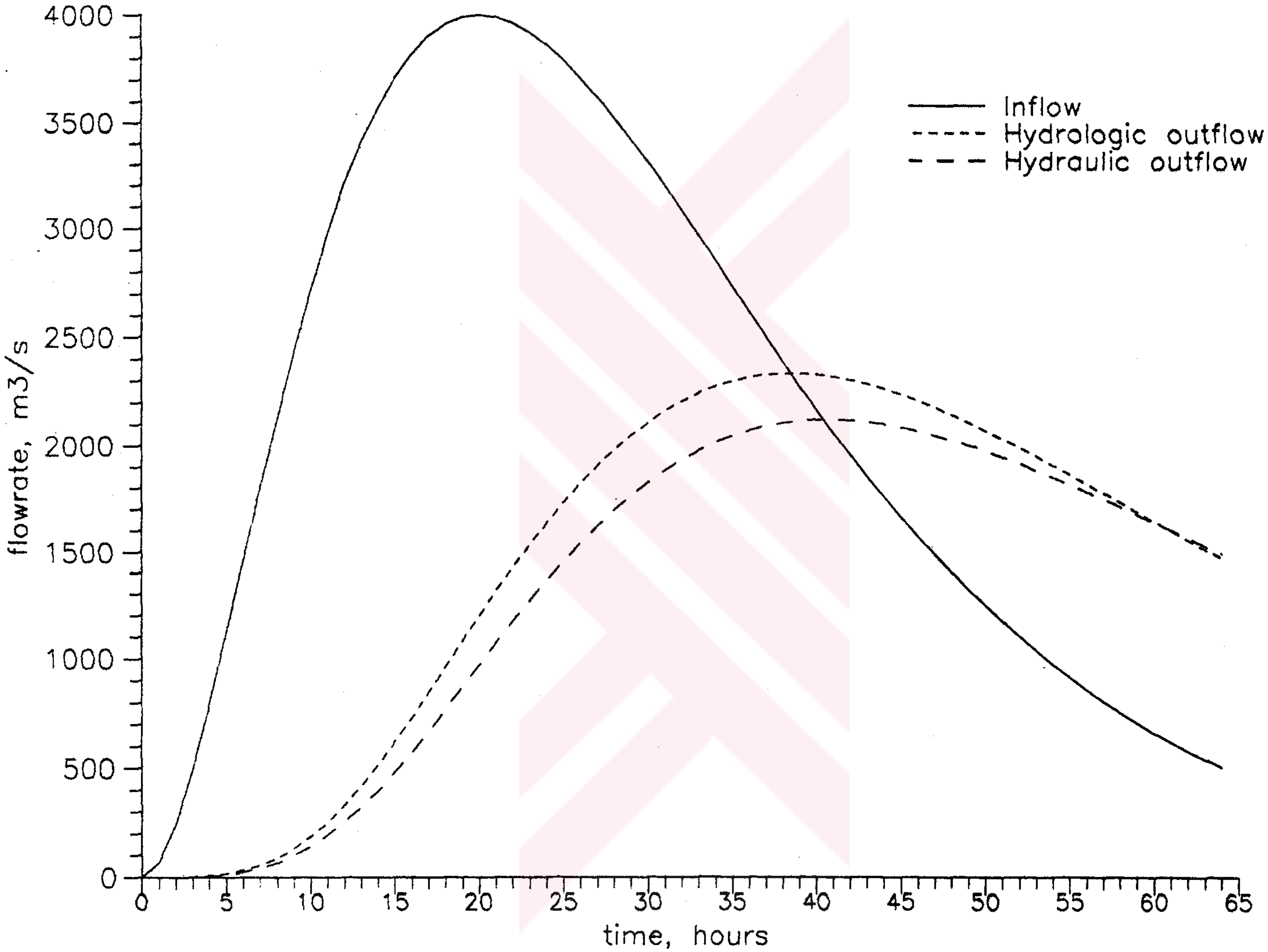


Figure **3.10.** Hydrographs of inflow, hydrologic outflow, and hydraulic outflow at Kayrcktepe R. with inflow peak of 4000 m3/s and spillway length of 60 m

4. ATTEMPTS TO DETERMINE THE DIFFERENCE BETWEEN OUTFLOW HYDROGRAPHS OF THE HYDROLOGIC AND HYDRAULIC ROUTINGS

4.1. Introduction

As already explained in Chapter 3, considering a thin-and-long reservoir as a reach of a natural river, the governing equations of the unsteady open-channel flow have been solved with the help of the verified package program: DUFLOW. The hydraulic routing method of DUFLOW considers the thin-and-long reservoir as a long reach of a natural river by taking into account the geometric and hydraulic properties of so many cross-sections of the valley of the lake. It solves simultaneously the continuity and momentum equations of unsteady flow in differential forms by a finite-difference based numerical scheme, which results in solving a large system of as many simultaneous nonlinear equations at each time step as double the number of all the cross-sections chosen [7]. Although, such a large and formidable system of simultaneous nonlinear equations is solved differently at each time step, these different systems always lend themselves to convergent solutions because firstly good initial estimates for the two unknowns, water surface and flowrate at each computational grid, can be made close to the actual values, and secondly the Newton-Raphson method is convergent for this system due to the analytic nature of the resulting difference equations [1,3]. Still, the amount of computations involved for even one time step, Δt, is enormous and manually impossible to overcome.

Hence, there is a huge difference in 1. input data requirements, 2.computational load, and computer memory necessities between the classical hydrologic routing (level-pool routing) and the more precise hydraulic routing methods. Because of the level-pool assumption behind the simple algorithm of continuity-based method of hydrologic routing, it is obvious that the peak of the outflow hydrograph computed by the hydrologic routing method will be greater than that by hydraulic routing, especially for a thin-and-long reservoir. The reason for this fact lies behind the reality that the flood wave must travel from the uppermost entering section down to the section at the beginning of the spillway approach channel of the reservoir; and hence, the stored water within the lake over the time step Δt, will not spread throughout the lake instantly forming a level pool surface. Because of this delay, the water surface elevation at the beginning of the spillway approach channel will be slightly less than that of level-pool assumption. The shorter the reservoir length, the more realistic the level-pool assumption should be.

4.2. Inclined Water Surface Assumption

The results of DUFLOW have indicated that the water surface profile in the valley of a thin-and-long reservoir is not horizontal, but is inclined downwards in the upstream-downstream direction (Figs.(4.1), (4.2), (4.3)). It is clearly verified that the water surface slope is almost horizontal at the beginning of a flood hydrograph entering the thin-and-long reservoir, and the inclination becomes steeper towards the peak time of the outflow hydrograph. The average slope of the water surface is always the steepest at the time of the peak outflow. At the recession limb of the outflow hydrograph the average water surface slope drops again gradually back to an almost horizontal value (Figs.(4.1), (4.2), (4.3)).

In order to compute the numerical value of the slope of the water surface assumed to be linearly declined in the upstream-downstream direction, the flowrates of the outflow hydrograph by DUFLOW were firstly accepted to be the real ones. Next, the elevations at the beginning of the spillway approach channel were computed from the discharge-elevation (rating) curve of the spillway, symbolized here by H_1. The valley of the thin-and-long reservoir was assumed to be a rectangle in the plan view, so as to preserve the storage-elevation relationship of the valley, still.

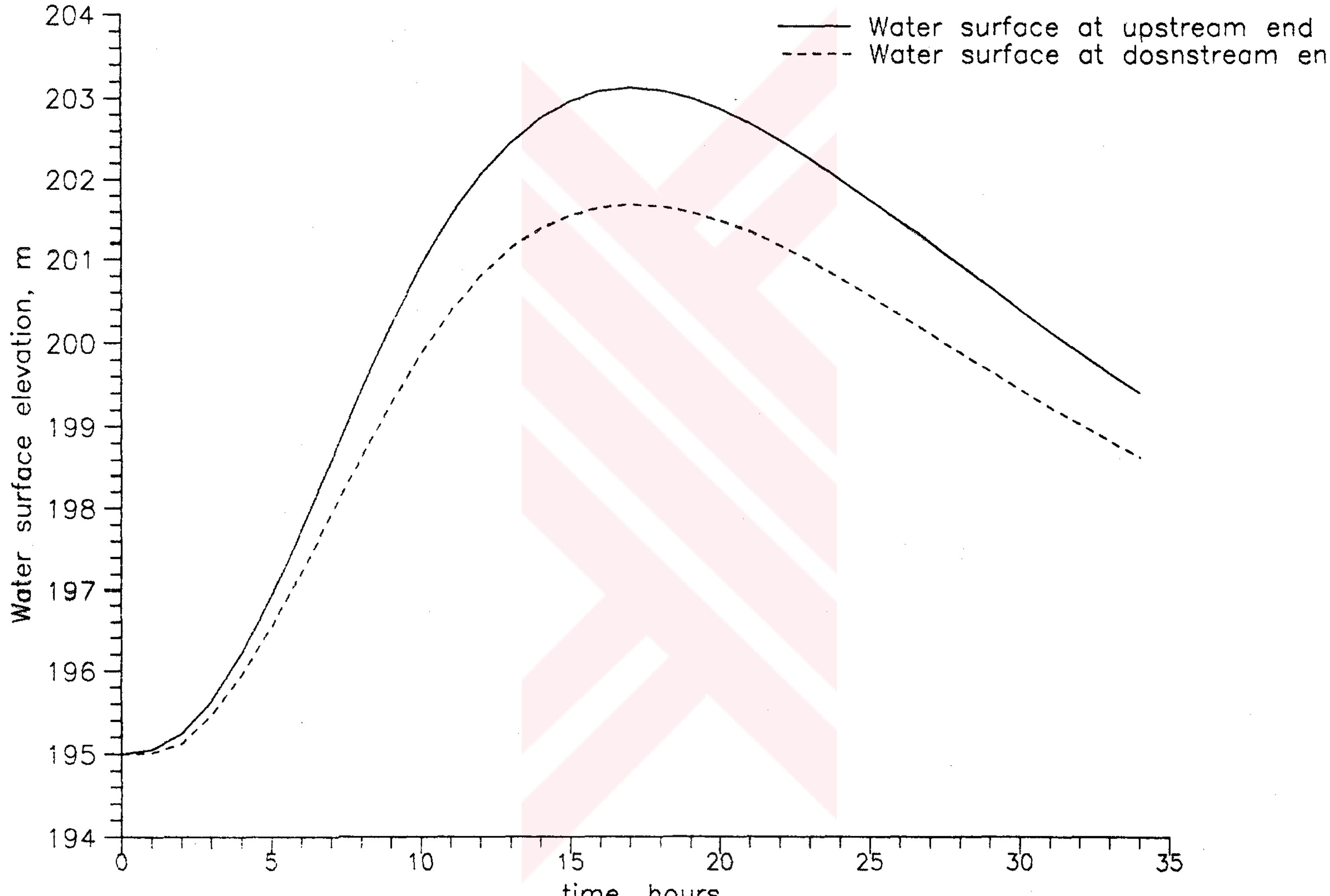


Figure 4.1. Water surface elevations versus time at both upstream and downstream ends of Deriner R. with Ip = 500 m3/s and spillway length of 10 m

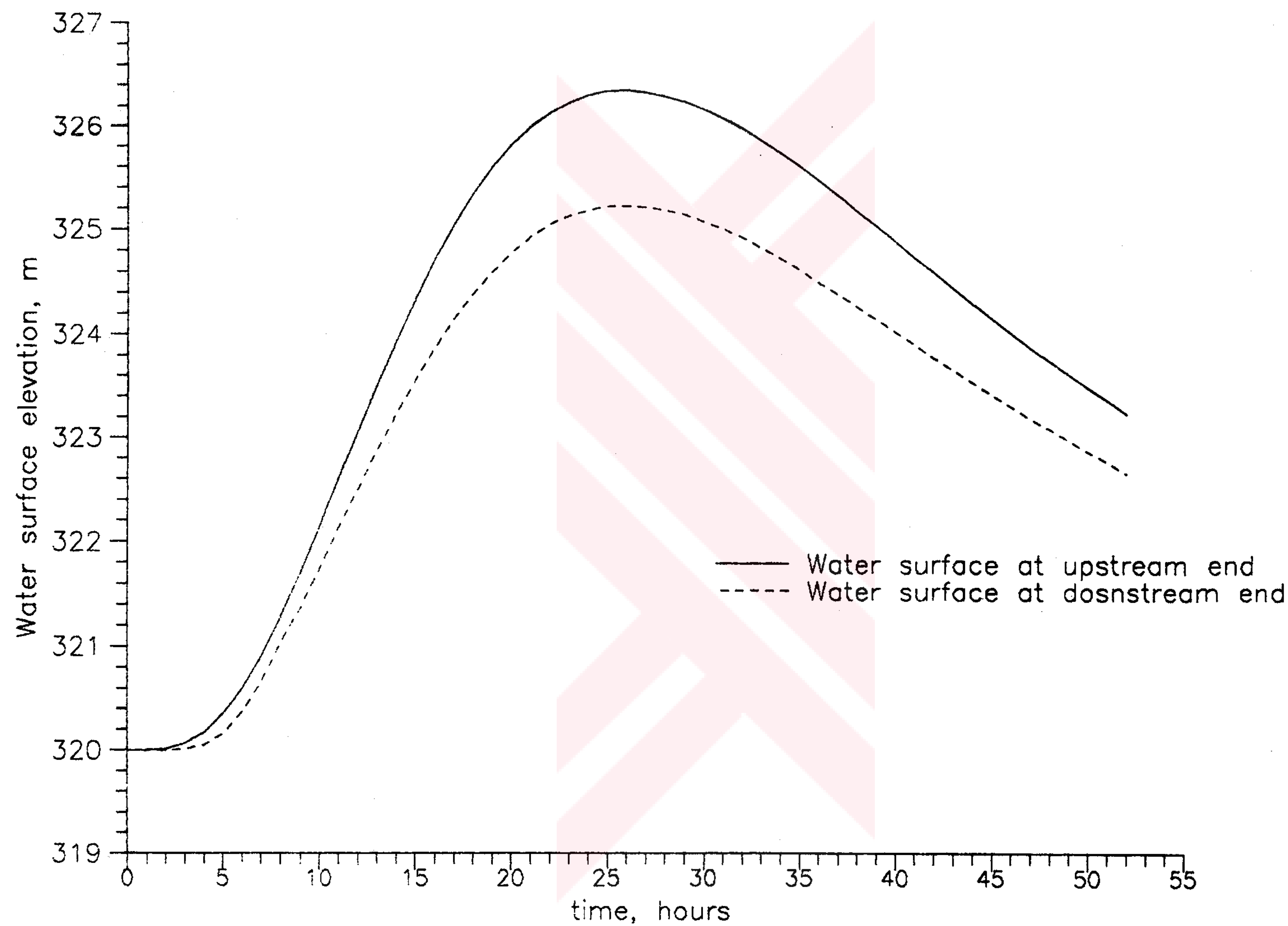


Figure 4.2. Water surface elevations versus time at both upstream and downstream ends of Berke R. with Ip = 1000 m3/s and spillway length of 30 m

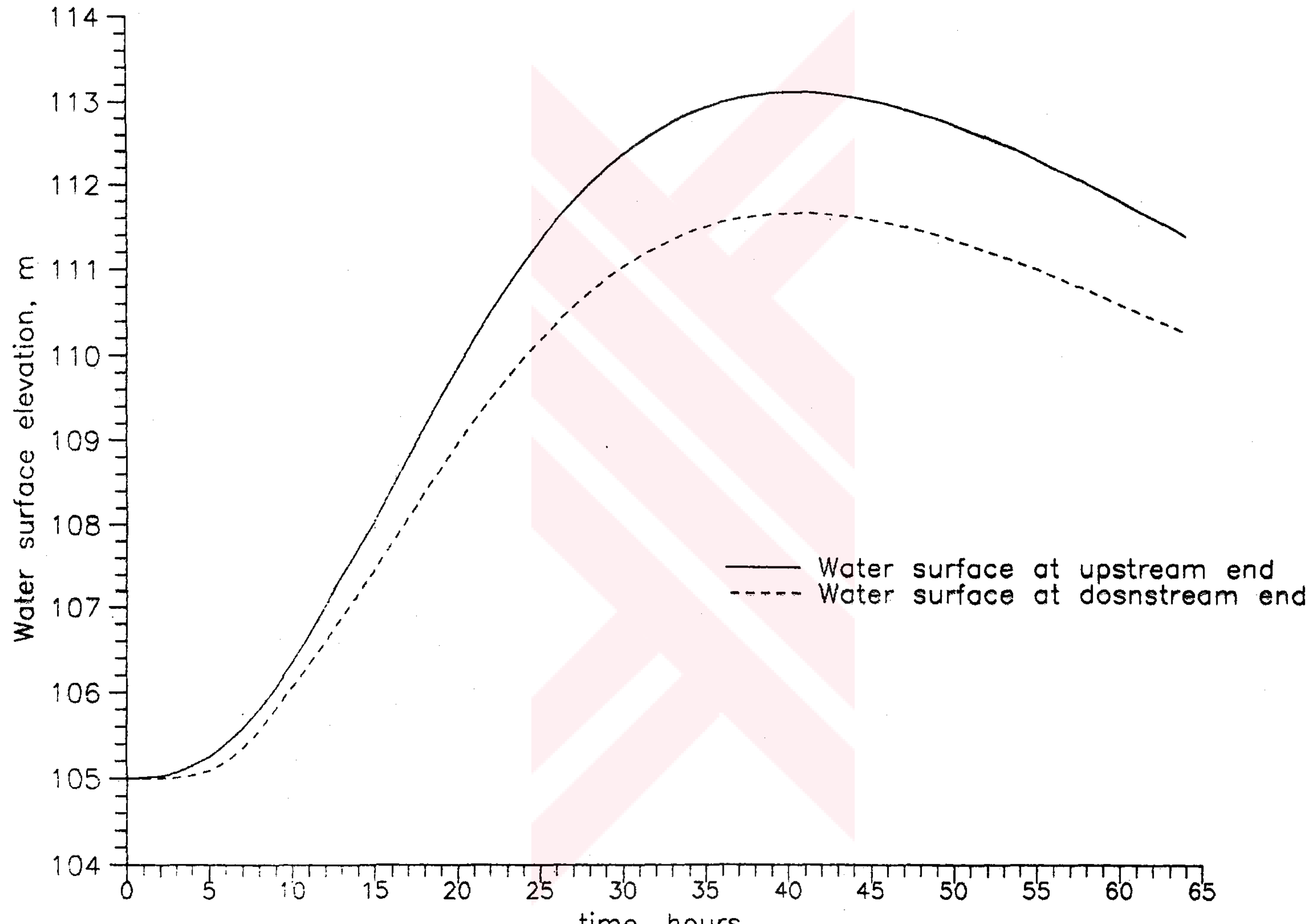


Figure 4.3. Water surface elevations versus time at both upstream and downstream ends of Kayraktepe R. with Ip = 4000 m3/s and spillway length of 60 m

Then, the water surface elevation at the uppermost cross-section of the reservoir, symbolized here by H_2, was computed so that the volume within the paralleloidal prismatic slice formed through the time step Δt, was equal to the water stored in (or released from) the reservoir computed by the simple and general continuity equation with known inflow and outflow values at the beginning and end of that particular time step, Δt (Fig.(4.4)).

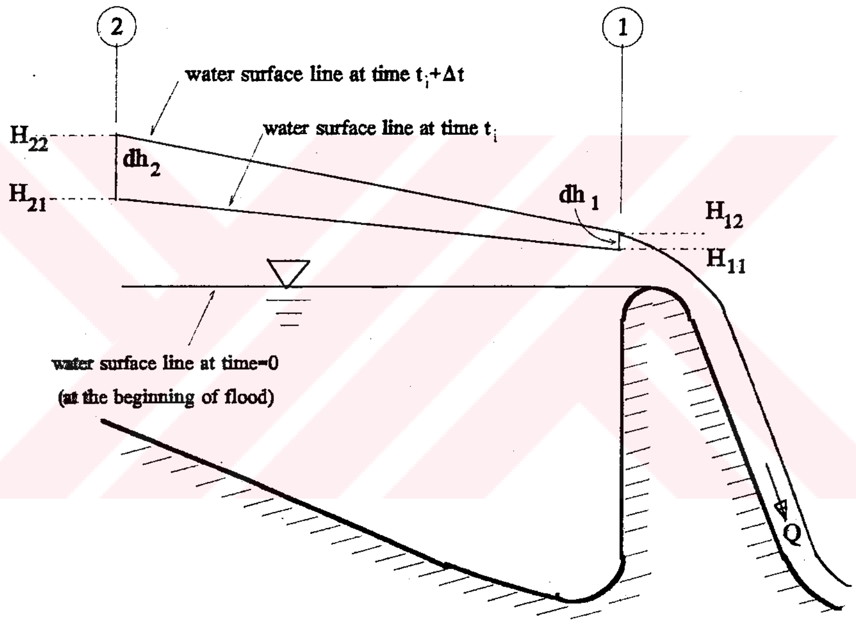


Figure 4.4 Water surface elevations computed by DUFLOW at the upstream and downstream ends for all of the reservoirs considered

Finally, average slope of the water surface over the time step Δt was computed by

$$S_{ws} \approx (H_2 - H_1)/L \tag{4.1}$$

Such computed water surface slopes were plotted versus time for all the 27 routing combinations from all the three reservoirs taken. Here, only three of these relationships are given in Figs.(4.5), (4.6), and (4.7).

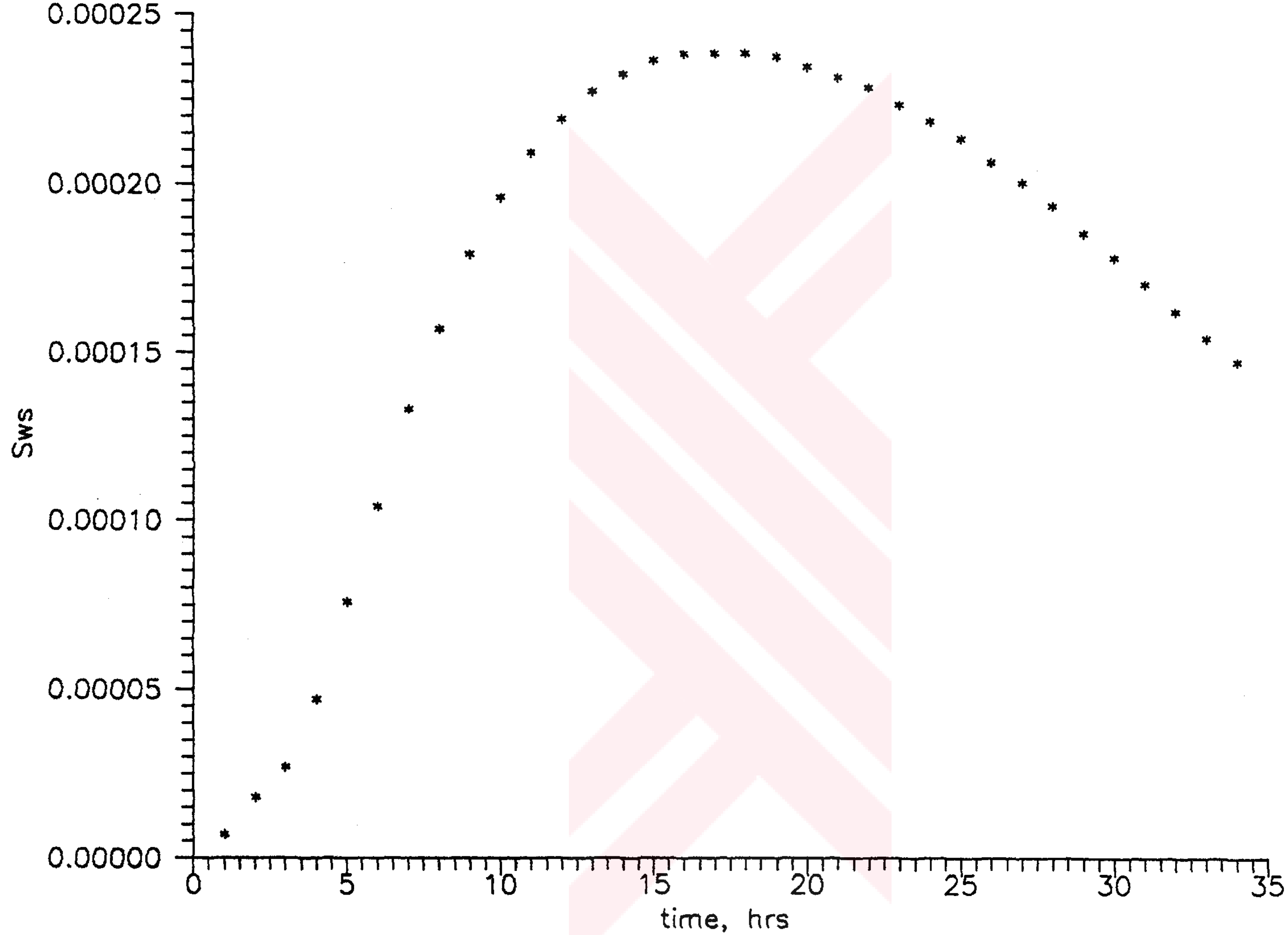


Figure 4.5. Sws–time relationship for Deriner R. with Ip = 500 m3/s & SL = 10 m

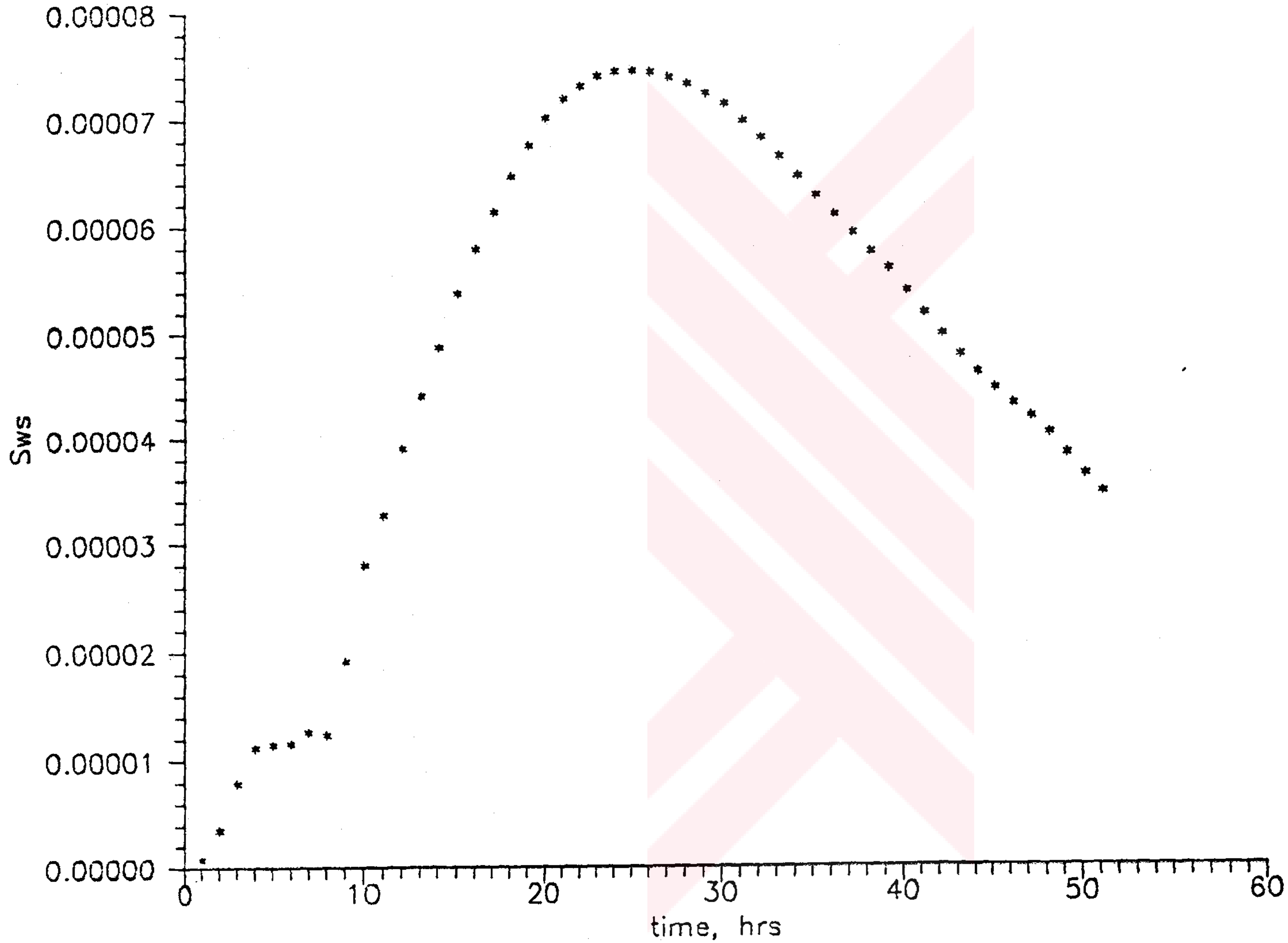


Figure 4.6. S_{ws}–time relationship for Berke R. with Ip = 1000 m3/s & SL = 30 m

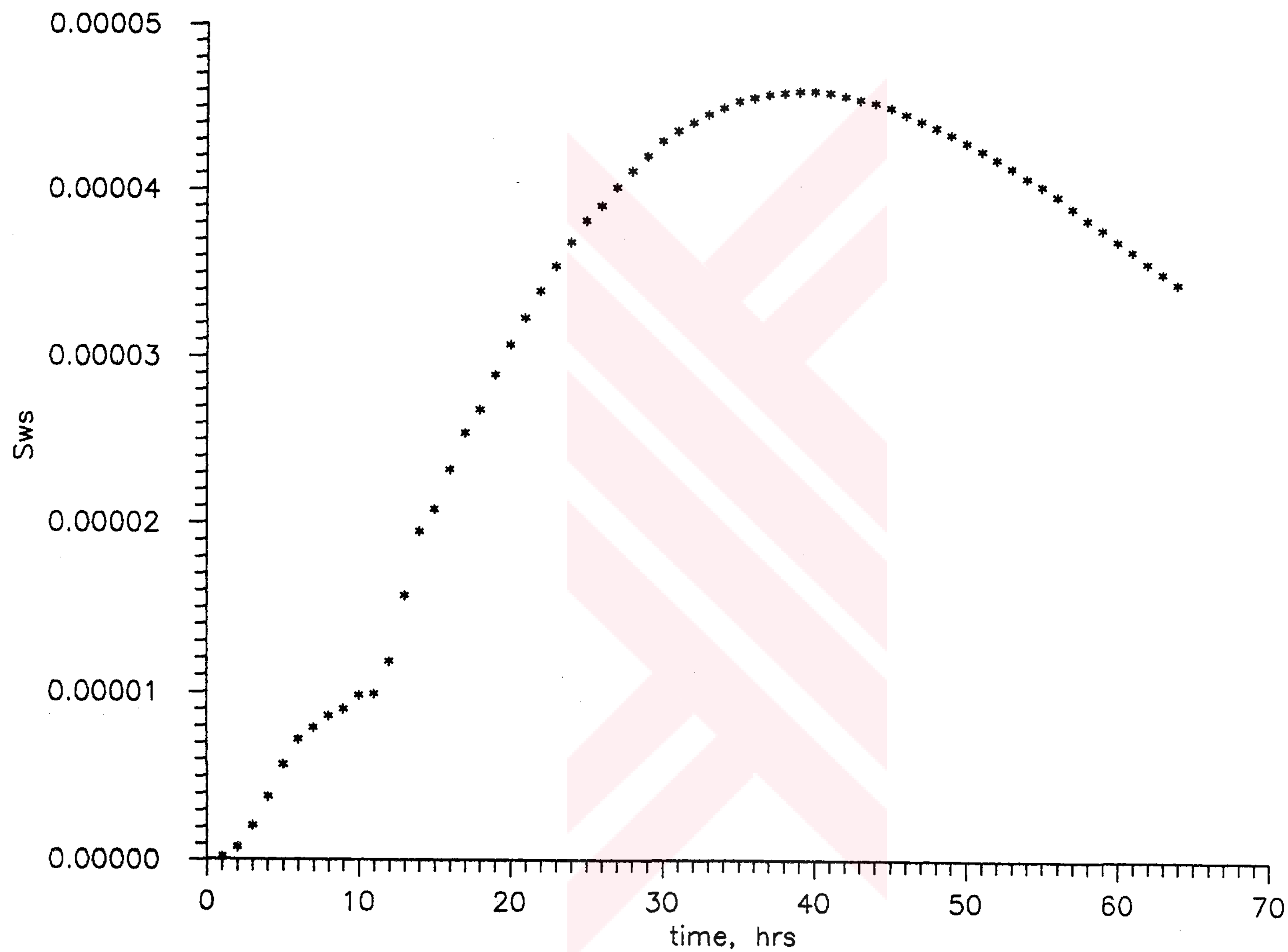


Figure 4.7. Sws–time relationship for Kayraktepe R. with Ip = 4000 m3/s & SL = 60 m

All the others looked similar, namely S_{ws} started from zero, increased to a maximum value exactly at time-to-peak of the outflow hydrograph, and dropped back again to values close to zero.

The results of the DUFLOW applied on 9 different inflow hydrograph and spillway dimension combinations for each one of the three thin-and-long reservoirs chosen in Turkey, have also indicated that the difference in peaks of the hydrologic and hydraulic routing outflows is not too great. It changed between 2 to 10 percent for the total of 27 various inflow-hydrograph, spillway-capacity combinations; the difference getting bigger as the relative difference between the inflow hydrograph and the design capacity of the spillway grows bigger.

Thus, aside from the main objective of assessing the difference between the peaks of the outflow hydrographs by the hydrologic and hydraulic routings quantitatively in the manner explained before, a practical modification of the simple method of hydrologic routing so as to yield an outflow hydrograph close to that by the hydraulic routing was also attempted in this study. The difference between the hydrologic and hydraulic routing was not too great to cause alarm in the first place even for those three considerably long reservoirs. It meant that for normal reservoirs with not too long lakes, then, the margin of error involved with the simplistic hydrologic routing would be still smaller. Therefore, the hydrologic routing for reservoirs based on the level-pool assumption should not be notably off from the reality, and therefore should be taken seriously.

Although the water surface slope between the beginning and end cross-sections of a thin-and-long reservoir is not necessarily constant, the results of the DUFLOW on those 27 different combinations have revealed that the water surface is declined in the upstream-downstream direction. Moving on this observation, the hydrologic routing has been modified in this study, making the assumption that initially horizontal water surface slope increases gradually up to the time of peak of outflow, and then drops back again close to horizontal towards the end of the recession limb of the outflow hydrograph. The assumption of linearly declined water surface line in the thin-and-long reservoir may not be strictly correct, but it will help compute an outflow hydrograph by hydrologic routing very close to that of hydraulic routing.

Once this assumption of linearly declined water surface is made, the volume of water, ΔS, stored during any time step, Δt, can be defined as below, which is explained in Fig.(4.4).

$$\Delta S = \frac{(dh_1 + dh_2)}{2} B_a L \tag{4.2}$$

where, B_a is the average valley width at that particular water surface elevation of the thin-and-long reservoir, L is the total length of the reservoir at that elevation, and dh_1 and dh_2 are the differences in water surface elevations at the downmost and uppermost cross-sections of the reservoir at the beginning and end of the time step, Δt. dh_1 and dh_2 are defined as:

$$dh_1 = H_{12} - H_{11} \tag{4.3a}$$

$$dh_2 = H_{22} - H_{21} \tag{4.3b}$$

where, H_{12} and H_{11} are water surface elevations at the beginning of the approach channel of the spillway, at the beginning and at the end of the time step, Δt; and H_{22} and H_{21} are the same values at the uppermost cross-section, where the incoming flood wave firstly enters the reservoir, respectively. H_{11} and H_{21} are known from the computations one step before.

From the principle of continuity, ΔS of Eq.(4.2) is also defined as:

$$\Delta S = \left(\frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2}\right)\Delta t \tag{4.4}$$

In Eqs.(4.2) and(4.4), there are three unknowns: O_2, H_{12}, and H_{22}. Therefore, three independent equations are needed. The two such equations are Eqs.(4.2) and(4.4). In this study, the third one is obtained with the help of regression analysis as explained below.

As seen in Figs.(4.5), (4.6) and (4.7), the water surface slope of the linearly declined water surface reaches a maximum value, and then drops back to almost horizontal values. Therefore, firstly, it was hoped to relate this maximum value of the water surface slope to some relevant peculiarities defined by some parameters such as: 1. a parameter reflecting the slenderness (shape of the valley) of the thin-and-long reservoir, 2. a factor reflecting the spillway capacity relative to the magnitude of the incoming wave, 3. average slope of the channel bottom of the valley of the reservoir, etc. It was decided that the average width of the valley assumed to be a rectangle in plan view, defined as the average of the valley width for the water surface elevation at the beginning of the flood and that at the maximum water surface elevation during routing, might be a parameter reflecting the slenderness of the valley of the reservoir. A dimensionless number obtained by dividing this average valley width during routing by the total length of the reservoir would be a better parameter reflecting the slenderness or shape of the thin-and-long reservoir (symbolized by B_a/L). Next, the dimensionless ratio: $[(I_p-O_{hp})/I_p]$ where, I_p is the peak of the inflow hydrograph and O_{hp} is the peak of the outflow hydrograph by hydrologic routing. $[(I_p-O_{hp})/I_p]$, was taken as an important independent variable comprising such peculiarities as magnitude (peak and volume) of the incoming flood wave, magnitude of the lake volume available for routing, comparative capacity of the spillway, and the routing characteristic of the long valley of the reservoir, all inherently in a lumped manner.

Contradictory to the expectations, meaningful correlations among the maximum value of water surface slope as the dependent variable versus quite a few parameters like the ones mentioned above could not be obtained. Then, another simple regression was tried. This time, the difference in peaks of the outflow hydrographs by hydrologic and hydraulic routings was aimed to relate by regression to the parameters explained above. The data obtained for the 27 combinations obtained for the three reservoirs are given in Table(4.1). The chosen dependent variable was $(O_{hp}-O_{dp})/O_{hp}$ where, O_{hp} denotes peak of outflow by hydrologic routing and O_{dp} denotes the peak of outflow computed by the package program: DUFLOW.

As a result of various regression trials among these variables including S_o (channel bottom slope), the following simple one was found to be the most significant:

$$\frac{O_{hp}-O_{dp}}{O_{hp}} = 0.0169 + 0.186\,\frac{I_p-O_{hp}}{I_p} \qquad (4.5)$$

The summary of this regression together with its plot is given in Appendix. Inclusion of either (B_a/L) or S_o did not improve the relationship any more. O_{hp}, the peak of the outflow hydrograph by hydrologic routing was easily computed beforehand. Therefore, the difference between O_{hp} and O_{dp} could be computed by

$$O_{hp}-O_{dp} = O_{hp}\,\frac{O_{hp}-O_{dp}}{O_{hp}} \qquad (4.6)$$

where, the ratio $[(O_{hp}-O_{dp})/O_{hp}]$ would be determined from Eq.(4.5). Next, it was assumed that the difference between outflow hydrographs by the hydrologic and hydraulic routings would increase from zero up to the value given by Eq.(4.6) and would drop back to zero towards the end of the recession limb linearly. Therefore, it was not too difficult to compute the magnitudes of the modified outflow hydrograph, afterwards.

4.3 A Practical Method for the Modified Hydrologic Routing

Initially, the classical hydrologic routing computations are performed and all the flowrates of the outflow hydrograph by the hydrologic routing are obtained. Next, up to the time-of-peak, the following steps are performed:

$$O_{mi} = O_i - \frac{i}{p_i}(O_{hp} - O_{dp}) \tag{4.7}$$

where, p_i is the number of steps to peak of outflow hydrograph, i is the integer number of the i^{th} step, O_{mi} is the flowrate of the modified outflow at the i^{th} step, and O_i is the flowrate of the outflow by level-pool routing. Then, ΔS_i is computed from Eq.(4.4) by:

$$\Delta S_i = \left(\frac{I_{i-1} + I_i}{2} - \frac{O_{m_{i-1}} + O_{m_i}}{2}\right)\Delta t \tag{4.8}$$

Also, with O_{mi}, computed with the help of Eq.(4.7), H_{12} is determined by a 3^{rd} degree interpolation polynomial from the Q-H relationship of the spillway of the reservoir, which is another essential data used in normal hydrologic routing, and which is usually given in a tabular form containing two columns of numbers. After determination of H_{12} in this manner, dh_1 is easily computed from Eq.(4.3a). H_{11} and H_{21} are known values from the previous step of computations. Next, dh_2 is computed from Eq.(4.2), and finally H_{22} is determined from Eq.(4.3b). The values of B_a and L in Eq.(4.2) are determined by interpolation again from the initially computed tables of these quantities versus elevation; and, as elevation, the average water surface elevation of the previous time step is used.

Hence, the objective to obtain a precision by the hydrologic routing close to that of the hydraulic routing without going through the agony of preparing so much input data and finding a sound computer package and learning how to use it to perform the complicated unsteady flow computations was achieved to some extent in a practical

The outflow hydrographs computed by the modified hydrologic routing method, together with those by hydrologic and DUFLOW routings are shown in Figs.(4.8),(4.9) and (4.10) for those three particular cases out of the 27 combinations.

Table 4.1 Summary of the results of and some parameters for 26 different routings on the three thin-and-long reservoirs

RESV.	Spillway length (m)	Peak of inflow (m^3/s)	Peak of Outflow by hydrologic routing (m^3/s)	Peak of Outflow by DUFLOW (m^3/s)	$\frac{(O_{hp}-O_{dp})}{O_{hp}}$	$\frac{(I_p-O_{hp})}{I_p}$	B / L	$S_{ws\text{-}max}$
DERİNER	30	1000	947	933	0.0148	0.053	0.0189	0.000223
	20	1000	903	886	0.0188	0.097	0.0197	0.000239
	10	1000	This case is	excluded	because	the	spillway is	too small.
	30	500	466	451	0.0322	0.068	0.0178	0.000169
	20	500	440	423	0.0386	0.120	0.0184	0.000198
	10	500	375	354	0.0560	0.250	0.0192	0.000238
	30	200	179	170	0.0503	0.105	0.0170	0.000098
	20	200	166	156	0.0602	0.170	0.0172	0.000118
	10	200	137	124	0.0949	0.315	0.0177	0.000154
BERKE	50	3000	2779	2702	0.0277	0.074	0.0117	0.000132
	40	3000	2703	2609	0.0348	0.099	0.0118	0.000151
	30	3000	2581	2455	0.0488	0.140	0.0119	0.000184
	50	2000	1819	1761	0.0319	0.0905	0.0115	0.000097
	40	2000	1762	1693	0.0392	0.119	0.0116	0.000110
	30	2000	1671	1588	0.0497	0.1645	0.0117	0.000130
	50	1000	870	834	0.0414	0.130	0.0113	0.000057
	40	1000	835	795	0.0479	0.165	0.01135	0.000064
	30	1000	782	736	0.0588	0.218	0.0114	0.000075
KAYRAKTEPE	100	6000	4435	4140	0.0665	0.261	0.0145	0.000055
	80	6000	4124	3809	0.0764	0.313	0.0147	0.000060
	60	6000	3697	3368	0.0890	0.384	0.0149	0.000066
	100	4000	2831	2634	0.0696	0.292	0.0141	0.000038
	80	4000	2619	2410	0.0798	0.345	0.0142	0.000042
	60	4000	2334	2121	0.0913	0.417	0.0144	0.000046
	100	2000	1285	1183	0.0794	0.357	0.0137	0.000021
	80	2000	1178	1073	0.0891	0.411	0.0137	0.000023
	60	2000	1037	931	0.1022	0.481	0.0138	0.000025

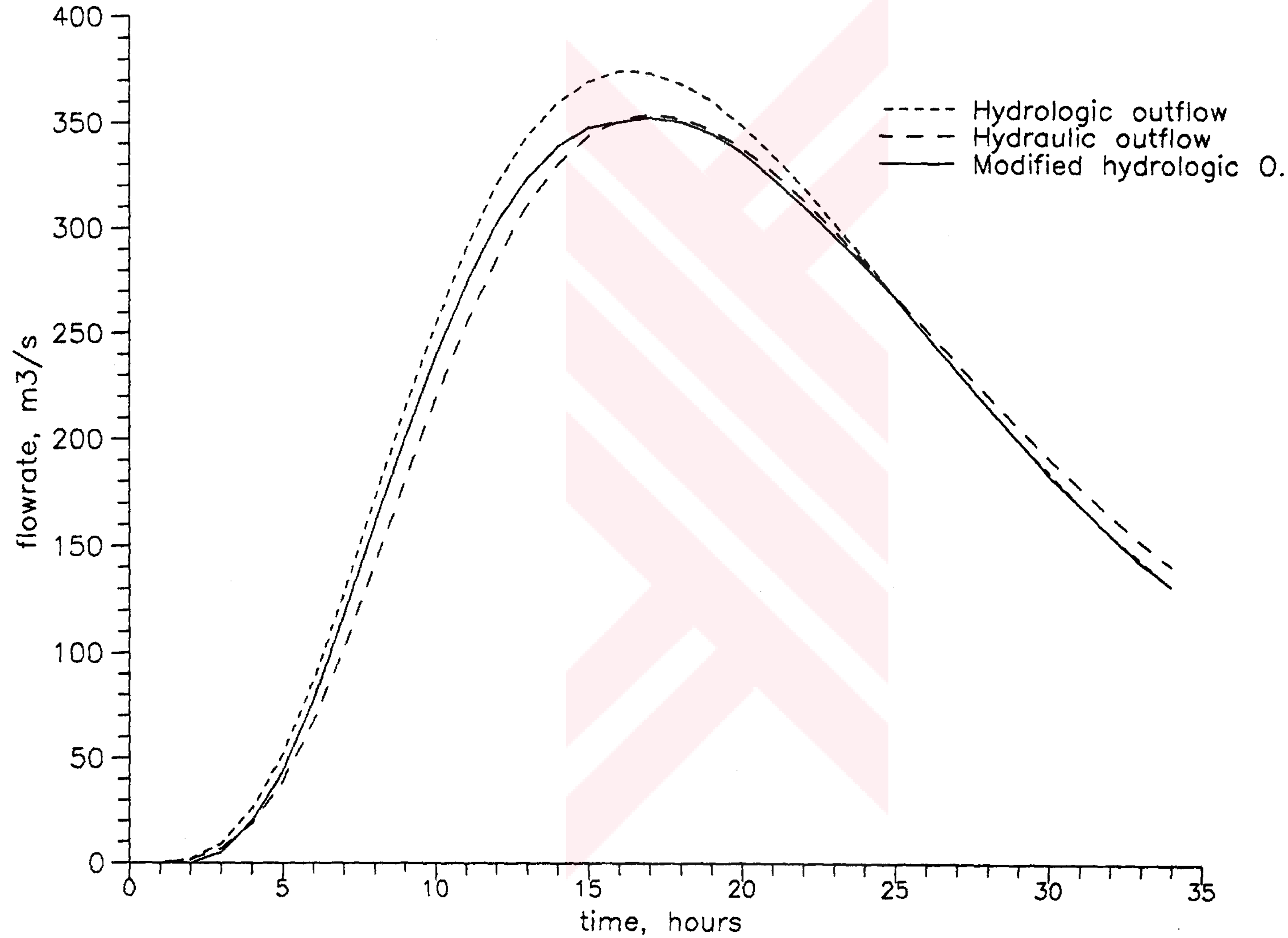


Figure 4.8. Hydrographs of hydrologic outflow, hydraulic outflow, and modified hydrologic outflow at Deriner R. with Ip = 500 m3/s and SL = 10 m

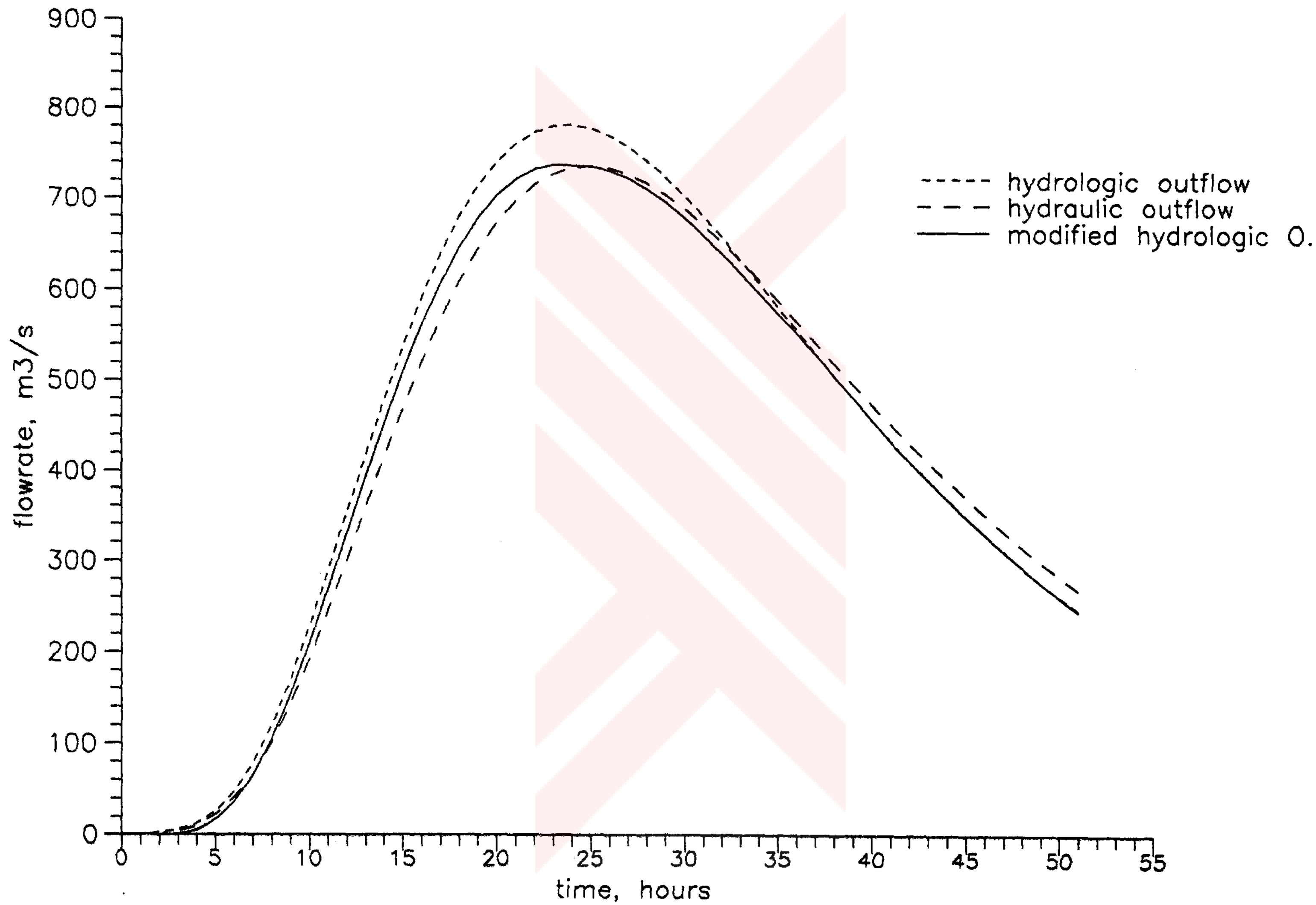


Figure 4.9. Hydrographs of hydrologic outflow, hydraulic outflow, and modified hydrologic outflow at Berke Res. with Ip = 1000 m3/s and SL = 30 m

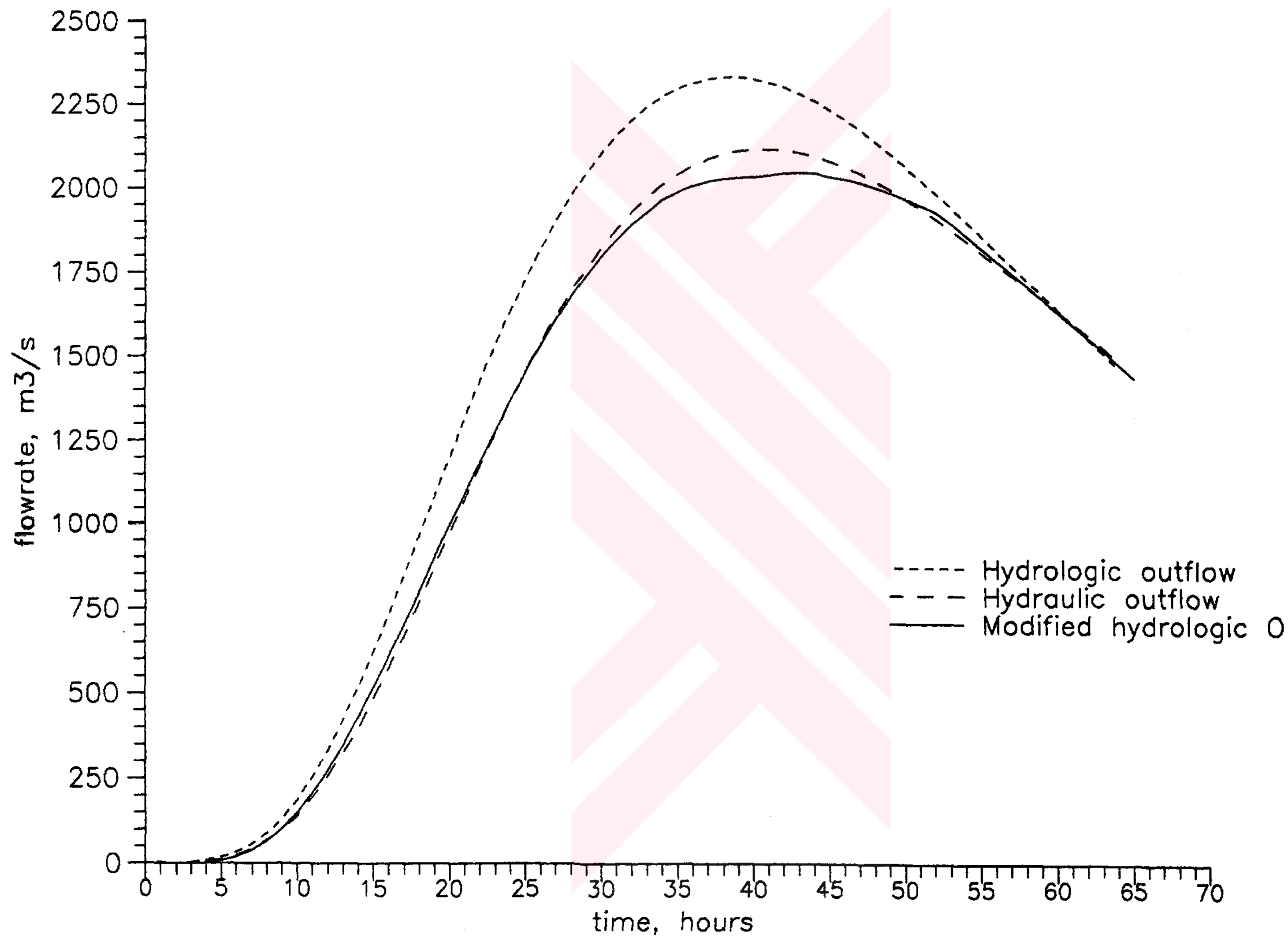


Figure 4.10. Hydrographs of hydrologic outflow, hydraulic outflow, and modified hydrologic outflow at Kayraktepe R. with Ip = 4000 m3/s and SL = 60 m

5. CONCLUSIONS

1. The outflow hydrographs were computed by both hydrologic and hydraulic routing for three dams having considerably long lakes within narrow valleys in Turkey. The computation of both methods were executed for 27 combinations which were three inflow hydrographs of different peaks with three unregulated ogee spillways of different capacities. The peak of outflow by hydrologic routing was greater than that by hydraulic routing in all cases, as expected. As it is seen on Table(3.4), this difference changes between 2 to 10% and grows larger as the spillway capacity becomes smaller relative to the magnitude of the inflow hydrograph.

2. Eq.(4.5) obtained as a result of 27 combinations on three reservoirs in Turkey cannot be accepted as a generalization. However, it indicates that regressions of that type or similar ones could be derived as a result of more detailed analyses to be applied not on 3 but maybe on 30 actual thin-and-long reservoirs in Turkey and elsewhere in the world.

The modified hydrologic routing suggested in this study does not require any excessive data. The only extra information is the total length of the reservoir or its values versus elevation in tabular form. Then the relationship of the average valley width, B_a, versus elevation can be computed easily from the storage-elevation relationship, which is essential data for the normal hydrologic routing. The modified hydrologic routing suggested herein does not require an excessive load of computations, either. It simply suggests to lower at reasonable values the flowrates computed by the classical hydrologic routing. However, the goodness of the simple modified hydrologic routing depends to a great extent on the soundness of the empirical model predicting the difference between the peak of the hydrologic routing and the peak of the modified hydrologic routing. The one suggested here Eq.(4.5) cannot be a general equation valid for all the thin reservoirs. It should be regarded simply as a suggestion for better regression models to be derived from a more comprehensive statistical analysis including many thin-and-long reservoirs all over the world.

Yet, one may not even bother about the difference between the outflow hydrographs by the hydrologic and more realistic hydraulic routings, because firstly this difference is not too great, and secondly it is on the safe side.

APPENDIX

The Regression Analysis

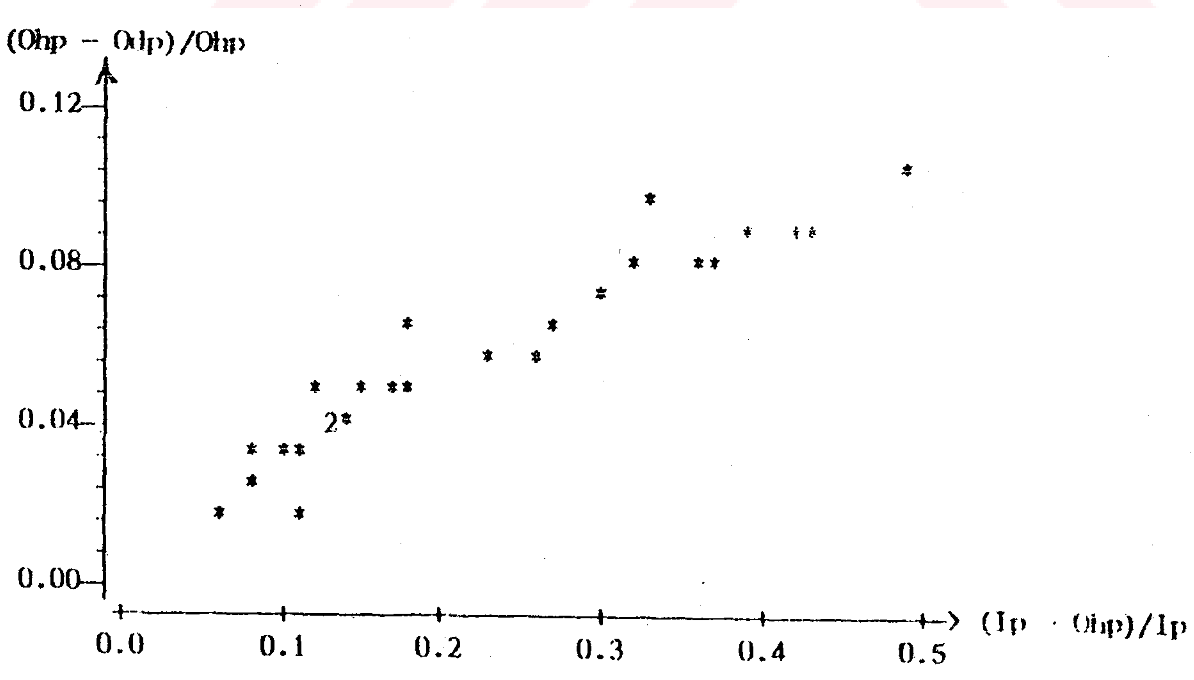
In the following is the result of the regression for the relative difference:

$$\frac{\text{Peak of O. by hydrologic routing - Peak of O. by DUFLOW}}{\text{Peak of O. by hydrologic routing}}$$

versus the relative difference:

$$\frac{\text{Peak of Inflow - Peak of O. by hydrologic routing}}{\text{Peak of Inflow}}$$

applied on the 26 permutations of different inflow hydrographs and different spillways of 3 different thin-and-long reservoirs in Turkey: Deriner, Berke, and Kayraktepe.



THE REGRESSION EQUATION IS

$$\frac{(O_{hp} - O_{dp})}{O_{hp}} = 0.0169 + 0.186 \frac{(I_p - O_{hp})}{I_p} \tag{A.1}$$

COLUMN	COEFFICIENT	ST. DEV. OF COEF.	T-RATIO = COEF/S.D.
	0.016885	0.002885	5.85
$RDI_p\&O_p$	0.186250	0.011530	16.16

S = 0.007338

R-SQUARED = 91.6 PERCENT
R-SQUARED = 91.2 PERCENT, ADJUSTED FOR D.F.

ANALYSIS OF VARIANCE

DUE TO	DF	SS	MS=SS/DF
REGRESSION	1	0.014059	0.014059
RESIDUAL	24	0.001292	0.000054
TOTAL	25	0.015351	

SUMMARY

The outflow hydrographs computed by both hydrologic routing (level-pool routing) and by hydraulic routing for three dams having considerably long lakes within narrow valleys in Turkey (Deriner, Berke, and Kayraktepe) were compared for three inflow hydrographs of different peaks with three unregulated ogee spillways of different capacities within considerably wide intervals and yet at reasonable proportions. Out of the total of 27 such combinations, it was observed that the difference between those outflow hydrographs was the greatest at the peak value. The difference grew larger as the spillway capacity became smaller relative to the magnitude of the inflow hydrograph, and it changed between 2 to 10% of the peak of the outflow by hydrologic routing, for these three thin-and-long reservoirs. The peak of the outflow by hydraulic routing turned out to be smaller than that by hydrologic routing for all the 27 different routing cases, as expected. The difference being at most about 10% even for such long lakes was considered to indicate that the level-pool routing for reservoirs of short valleys should yield outflow hydrographs reasonably close to the real ones.

A simple model was suggested to modify the outflow hydrograph computed by the hydrologic routing for long and thin reservoirs. The modified model does not require any excessive data. The only extra information is the total length of the reservoir or its values versus elevation in tabular form. The modified hydrologic routing suggested herein does not require an excessive load of computations. It simply suggests to lower at reasonable values the flowrates computed by the classical hydrologic routing.

ÖZET

Bu çalışmada, Türkiye'de dar vadilere uygulanmış ince ve uzun göle sahip üç barajda (Deriner, Berke, Kayraktepe), üç farklı pike sahip giren akım hidrografı ve herbiri için üç farklı dolusavak genişliği alınarak hidrolojik ve hidrolik metodlarla taşkın ötelemesi yapılmış ve çıkan akım hidrografları karşılaştırılmıştır. Toplam 27 kombinasyondan her iki metod sonucunda çıkan akımlar arasındaki farkın pik değerlerde en fazla olduğu gözlenmiştir. Sözü edilen farkın, giren akımın büyüklüğüne bağlı olarak dolusavak genişliği küçüldükçe arttığı ve ince-uzun göle sahip reservuarlar için hidrolojik öteleme sonucundaki çıkan akım pikinin %2'si ile %10'u arasında değiştiği saptanmıştır. Tahmin edildiği gibi hidrolik ötelemedeki çıkan akım piki hidrolojik ötelemedeki çıkan akım pikinden daha küçük olmuştur. İnce uzun göle sahip reservuarlarda bile bu fark en fazla %10 oranına ulaştığına göre ince-uzun olmayan göllerde hidrolojik öteleme ile daha gerçekçi öteleme olan hidrolik öteleme sonuçları birbirine oldukça yakın olması gerekir.

Bu çalışma sonucunda ince uzun göle sahip barajlar için, hidrolojik öteleme yöntemi ile bulunmuş olan çıkan akım hidrografını hidrolik öteleme sonucundaki çıkan akım hidrografına yaklaştıran basit bir model geliştirilmiştir. Bu modifikasyon modeli çok fazla data yükü getirmemektedir. Hidrolojik öteleme yönteminde gerekli olan dataya ek olarak toplam reservuar uzunluğu veya reservuar uzunluğu ve ona karşılık gelen kot ilişkisi gerekmektedir. Ayrıca, sözü edilen model daha gerçekçi öteleme olan hidrolik ötelemenin hesap yükünü de ortadan kaldırmıştır.

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