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14. ABSTRACT
In the planning and design of dams and embankments for large multiple-purpose projects, it is usually necessary to evaluate the effect of potentially disastrous extreme events to insure that the proposed developments do not produce an unnecessary increase in the disaster potential. One such case which is being encountered more frequently now involves the design of dams and embankments for projects located downstream from an existing dam. If the existing structure is relatively old or if there is reason to believe that the probability of failure is relatively great, it may be desirable to consider the effect of a potential failure during the design of the downstream structure.

In the design studies for a dam to be located downstream from an existing dam, the effects of failure of the upstream structure have been calculated. The flood wave which would result from failure of the upstream dam has been calculated and routed through the reservoir that would be formed by the downstream structure. The analysis utilized the unsteady flow equations to evaluate the effect of the flood wave under three different conditions of impoundment in the downstream reservoir at the time of failure. The structure that was assumed to fail was treated as a finite discontinuity in the water surface elevation at an internal point in the computation net rather than as a boundary condition. The design, verification and use of the digital simulation model for the analysis are presented.

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A METHOD FOR ANALYZING EFFECTS OF DAM FAILURES IN DESIGN STUDIES[1]

by William A. Thomas[2]

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In the planning and design of dams and embankments for large multiple-purpose projects, it is usually necessary to evaluate the effect of potentially disastrous extreme events to insure that the proposed developments do not produce an unnecessary increase in the disaster potential. One such case which is being encountered more frequently now involves the design of dams and embankments for projects located downstream from an existing dam. If the existing structure is relatively old or if there is reason to believe that the probability of failure is relatively great, it may be desirable to consider the effect of a potential failure during the design of the downstream structure.

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A METHOD FOR ANALYZING EFFECTS OF DAM FAILURES IN DESIGN STUDIES[1]

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In recent years, the failures of several small dams have prompted design, construction and regulatory agencies working in the field of water resources development to consider and analyze the effects of flood waves that could result from failure of existing or proposed dams. These flood waves, referred to herein as dam-break floods, are of particular importance because of the large momentum forces associated with them. Effects, such as the extent and duration of inundation in the flood plain below a breached dam, the rate of travel of the flood wave, the temporal and spatial variations in flood wave attenuation, the forces exerted by the flood wave on structures within the flood plain, and the environmental impact of the flood wave are examples of the types of effects that are important. In addition, each individual situation will introduce special considerations into the analysis.

The study which is discussed in this paper was conducted to determine how much force a dam-break flood wave would exert on a downstream dam. There are three major aspects to the problem: determining the discharge hydrograph when a dam is assumed to fail; routing that discharge downstream to the point of interest; and applying the resulting forces to the structure in question. The first two aspects are treated in this paper. The method that was utilized to calculate and route the dam-break

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flood waves in this study could also be used to analyze the dam-break flood wave in terms of the other effects mentioned above. However, special situations developed which required simplification of the problem in order to use this method, and these limitations are discussed also.

Interest in this analysis was stimulated by a difference in the design criteria used for two projects. The projects are owned by different agencies, and the criteria used to develop extreme hydrologic events to establish spillway requirements and the height of the dam for the proposed downstream structure produced a flood that would completely overtop the existing earthfill dam. Therefore, assumptions regarding failure had to be considered to adequately define loading conditions for the proposed dam. (At the present time, both the existing earth dam and its operating policy have been modified to reflect the same criteria being used for the proposed dam.)

Reservoir outlines of both the existing and proposed reservoirs are shown in figure 1.

Figure 2 shows a profile of the streambed from the site of the proposed Martins Fork Dam up Martins Fork of the Cumberland River to the confluence with Cranks Creek and thence up Cranks Creek. Pertinent elevations are shown for each dam.

Also, figure 2 illustrates a primary point of concern in the study. Cranks Creek Dam stands some 85 feet higher than the proposed Martins

Fork structure and is located only 3.9 miles upstream. How much of this energy head would be dissipated as the flood wave traveled to Martins Fork Dam? Saint Venant, Schoklitsch, Dressler and others have shown by theory and experiment that the energy producing a dam-break flood wave is the depth of water at the instant of failure, provided the downstream channel is dry. Their theoretical expression, in which energy losses are neglected, shows the peak discharge at a fully breached dam to be

$$Q_{max} = \frac{8}{27} W_d \sqrt{g}\, Y_o^{3/2} \tag{1}$$

where:

Q_{max} = peak discharge of the dam-break flood hydrograph

W_d = initial water surface width at the dam

Y_o = initial reservoir depth at the dam

g = acceleration of gravity

The value of peak discharge calculated with equation 1 agrees reasonably well with experimental data obtained from tests conducted by the U.S. Army Waterways Experiment Station (reference 6). Therefore, the fact that immediately after a failure the reservoir depth at the dam decreases rapidly to half its original value does not indicate a corresponding loss of total energy head at that location. Rather, total energy has been redistributed to include a large inertia component and a kinetic energy component, in addition to the pressure plus potential energy component; and the sum of these components yields the reservoir elevation just prior to failure. The important energy losses occur as the flood wave moves downstream and include friction losses, bend losses, expansion and contraction losses and others.

In addition to energy considerations, the volume of water which is available in Cranks Creek Reservoir to sustain the flood wave must be considered. Figure 3 shows the capacity of each reservoir as a function of its elevation. The increase in elevation of Martins Fork Reservoir due to storing the volume of water in the flood wave could possibly be greater than the energy consideration. This would depend on rate of energy dissipation and the rate of outflow from Martins Fork. It is also possible that the maximum force could result from some combination of the energy and volume considerations. Therefore, a time history of the flood wave motion is required to adequately analyze the problem.

Methods which are commonly used in flood routing, such as the modified Puls, Muskingum, Tatum, and straddle-stagger, permit direct consideration of volumes only. Indirectly, energy considerations are inferred by calibrating these methods to some experienced event. Such methods are inadequate for routing the dam-break flood--at least in the early stages, because energy plays such a dominant role in the movement. Therefore, one must resort to a solution of the basic equations of unsteady flow to consider both continuity and momentum.

A number of solution techniques have been advanced over the past few years, but the method presented by Garrison, Granju and Price (reference 1) has been used in this study. It is, however, based on a solution of the gradually varied unsteady-flow equations through the use of an explicit finite difference scheme developed by Stoker. Terzidis and Strelkoff (reference 5), in studies involving a hydraulic bore, demonstrated that such a solution technique calculated the wave height correctly, but failed to maintain continuity in that an excess water volume was developed for the wave. The problem was corrected by accounting for energy losses resulting from flow conditions in the wave front. On the other hand, Martin and De Fazio (reference 3) studied cases of rapidly varied flow involving an undular type of flood wave movement rather than a bore type, and they found the solution developed by Stoker to be adequate for their design studies. Because of the depth of water downstream from Cranks Creek Dam, the flood movement in this study was expected to be more like the undular wave than a hydraulic bore. Therefore, Stoker's solution technique was considered satisfactory. However, continuity checks were made to insure a reasonable volume was being maintained during the routing.

Numerous assumptions were necessary in order to route the dam-break flood. Generally, they can be divided into two categories: (1) those basic to establishing the problem and method for solution, and

(2) those necessary in assigning values used in the actual calculations. In most cases, judgment was guided by the effort to produce the most critical situation for Martins Fork Dam. The assumptions are listed below:

a. The failure of Cranks Creek Dam is instantaneous and complete.

b. Pressure distribution is hydrostatic at each cross section.

c. Velocity distribution is uniform over that portion of the cross section conveying flow.

d. Steady flow n-values are applicable.

e. An existing railroad crossing just downstream from Cranks Creek Dam is washed out upon impact of the flood wave and the resulting energy loss is negligible.

f. Energy lost at the junction of Cranks Creek and Martins Fork mainstem is negligible.

g. The effect of transient waves on energy dissipation, due to sinuosity of the channel, can be ignored.

h. Changes in boundary geometry due to scour and fill can be neglected.

i. The model is verified when reservoir volumes and the spillway design flood outflow hydrograph matches data obtained from conventional routing techniques in earlier studies.

j. The overflow rating curve for Martins Fork Dam is not affected by tailwater.

Pertinent data for both projects are shown in table 1.

Table 1. Pertinent Data for Both Projects

Item	Cranks Creek Dam, Existing	Proposed Martins Fork Dam
Location by Stream Name	Cranks Creek	Martins Fork of the Cumberland River
Miles above mouth	2.4	15.6
Drainage Area (sq. mi.)	24.8	55.7
Type of Dam	Earth Fill	Concrete
Top of dam elevation (feet)	1443	1360
Streambed elevation (approx. feet)	1323	1263
Length of dam (feet)	640	504
Spillway		
Crest elevation (feet)	1430	1341
Crest length (feet)	200	200
Design flood, peak-discharge (cfs)	25100	57000
Design flood, peak-elevation (feet)	1441.5	1358.2
Reservoir Capacity		
Normal summer operating pool,		
elevation (feet)	1420	1310
capacity (acre-feet)	6400	6800
Capacity at spillway crest, elevation (acre-feet)	9000	21000
Top of dam elevation (acre-feet)	14000	33300
Distance between the two projects (miles)	3.9	

Reservoir Elevations at Failure

Three cases were selected for study. These differed only in elevation of the reservoirs as shown below:

Reservoir Conditions at Failure

Condition No.	Cranks Creek Res. Elev.	Martins Fork Res. Elev.	Net Head at Cranks Creek Dam	Remarks
I	1441.5	1358.3	83.2	SDF crest (1)
II	1419.5	1309.5	110.0	Typical summer pool
III	1430.0	1341.0	89.0	Spillway crest

(1) Spillway Design Flood (SDF)

Condition I was selected because it represented the largest total energy available at Cranks Creek Dam. However, it did not produce the largest head differential possible at Cranks Creek Dam, nor did it produce the most severe condition for impact when the flood wave reached Martins Fork Dam. Therefore, Conditions II and III were selected. Condition II corresponds to a typical summer operating policy for the reservoirs and represents maximum energy head for producing the flood wave at Cranks Creek Dam; whereas Condition III represents an extreme condition for developing impact loads at Martins Fork Dam.

The study was divided into several major steps, each of which is discussed in detail in the following paragraphs.

Establishing the Geometric Model

The proposed Martins Fork Dam formed the downstream boundary of the model, and the upstream end of Cranks Creek Reservoir formed the upstream boundary. Cross sections were located to define total volume in each reservoir, and, where they extended up tributary arms, artificial flow boundaries were imposed to separate that portion of the section which conveys flow from that portion which only stores water. From these sections geometric data were calculated for nodal points spaced 1/2 half mile apart, thus forming the basic geometric model for the unsteady flow computer program.

Determining Hydraulic Roughness

Since friction loss was such an important consideration in this study, the selection of realistic n-values for Manning's equation was of primary importance. Values of 0.05 for the channel and 0.10 for the overbanks had been used in other studies by engineers familiar with the streams in this area. These values were, however, based on natural conditions. Values of 0.03 and 0.01 had been used for reservoir conditions. Since the flow velocity associated with the dam-break flood was expected to be more nearly that for natural conditions than that which would be expected with the reservoir impounded, the natural conditions n-values were used. These were adjusted into composite values for the entire cross section and to account for the sinuosity of the valley, which resulted in a composite n-value of .07 for each cross section. In Cranks Creek this value was reduced to .05, since the flow conditions would tend toward preimpoundment conditions as the water drained out.

Verifying the Digital Model

Before routing the dam-break flood, the Spillway Design Flood for Martins Fork was routed through both reservoirs. The results compared favorably with those from earlier studies in which conventional routing techniques were used. Also, water surface profiles were calculated for the peak discharge of the spillway design flood. These profiles were compared with profiles calculated using a steady flow, backwater computer program. The comparisons from these tests were considered sufficient to verify the model for the proposed study.

Establishing Boundary Conditions and Initial Conditions

A discharge rating curve was used for the downstream boundary condition at Martins Fork Dam. It included flow over the top of the dam, as well as through the spillway.

The Spillway Design Flood discharge hydrograph was used at the upstream boundary in Condition I. In the other two cases, inflow was assumed to be zero.

Local area inflow entered the model at the confluence of Cranks Creek and Martins Fork mainstem. No other local inflow points were established.

For all three conditions of failure, Cranks Creek Dam was considered as a finite discontinuity in the initial water surface profile. Using this approach, as opposed to treating it as an end boundary, the increase in tailwater elevation accompanying the dam-break flood wave could be included in calculations for the discharge hydrograph at the dam axis.

Routing the Dam-Break Flood for Condition I

The results of routing the dam-break flood for Condition I are shown in figure 4 in the form of discharge and elevation hydrographs at Cranks Creek Dam axis and at Martins Fork Dam axis.

The impact of the flood wave on Martins Fork Dam resulted in a 28-foot increase in water surface elevation above the SDF peak, as the inertia and kinetic energy components were transformed back into a pressure energy. The peak discharge at Martins Fork was 190,000 cfs. Four minutes were required for the wave to travel from Cranks Creek Dam to Martins Fork Dam.

Two alternative methods for computing travel time, a wave celerity computation based on initial depth and a forward characteristic computation based on cerlerity plus flow velocity, produced travel times of 6 minutes and 5.5 minutes, respectively. The travel time calculated by the routing method used in the study appears to be reasonable, based upon comparison with these values.

The peak outflow from Cranks Creek Reservoir, curve one of figure 4, is 1,400,000 cfs. The value obtained by applying equation 1, 1,200,000 cfs, compares favorably with this peak outflow.

The peak discharge is important because it influences the rate of energy dissipation due to friction. This point was illustrated in the study by first including all storage volume on Martins Fork mainstem, upstream from Cranks Creek, then recalculating the routing with that volume excluded. The peak energy at Martins Fork Dam was 4 feet higher in the first case because the additional storage volume reduced the peak discharge and, consequently, the rate friction loss.

The results of a sensitivity study in which the composite n-values were varied is shown in figure 5. For Condition I failure, n-values of .05, .07 and .10 were assigned and the flood wave calculated and routed to Martins Fork Dam. The results show that total energy remaining in the flood wave when it reaches Martins Fork Dam is highly dependent upon the hydraulic roughness value. The determination of these values should, therefore, receive a great deal of consideration in planning a routing study.

Routing the Dam-Break Flood for Condition II

The flood that would result from a failure when both reservoirs are at normal, summer operating pools (Condition II) is represented in figure 6. Under this condition, the entire contents of Cranks Creek Reservoir would be stored in Martins Fork Reservoir and maximum pool elevation would be well below the spillway crest, even including the impact of the flood wave.

This condition was used to test for continuity by calculating the volume under the hydrograph at Cranks Creek Dam and comparing the result with the elevation-storage curve of figure 3. The comparison indicated that the volumes were within 15 percent of one another, with the routing method tending to produce an excess of water. This conformed with the experience reported by Terzidas and Strelkoff. The discrepancy in continuity was not considered to be unreasonable when evaluated with respect to the uncertainties associated with other aspects of the study.

Routing the Dam-Break Flood for Condition III

The results of routing Condition III are shown in figure 7. The full impact loading of the flood wave is reflected in the 24-foot increase in water surface elevation at Martins Fork Dam. The reservoir outflow which was zero initially reached a maximum of 100,000 cfs under this condition.

The results from the three conditions are shown in table 2.

Table 2. Comparison of Results

Condition	Peak Outflow in cfs		Total Energy Head at Martins Fork Dam	
	Cranks Creek	Martins Fork	Prior to Impact of Flood Wave	Peak
I	1,400,000	190,000	1358	1386
II	1,100,000	0	1310	1332
III	1,200,000	100,000	1341	1364

Several special situations which developed during the analysis are discussed in the following paragraphs.

The Junction Problem

In planning for the study, a technique was proposed whereby flows at the junction of Cranks Creek and Martins Fork mainstem would be determined by successive approximations using two routing models. Briefly, this technique was to be the following: The primary model would extend through Cranks Creek Reservoir to Martins Fork Dam. The secondary model would extend from the upstream end of the Martins Fork arm down to the confluence with Cranks Creek. The first routing would be made with the primary model, and would assume zero inflow from the Martins Fork mainstem. The second step would utlize elevations calculated for the junction in the first step and route down the Martins Fork arm of the reservoir to determine the discharge entering the junction. Step 1 would be repeated. This alternating between the primary and secondary routing models would continue until the same junction elevations and discharges resulted in two successive steps. Such "convergence" would indicate that a solution had been reached.

This technique proved to be unsuccessful. The changes in discharge and elevation were so rapid that convergence of results from the two models could not be achieved. Fortunately, the peak of the flood wave required only a minute to travel from the confluence to the dam. During this time, the wave would travel only a short distance up Martins Fork mainstem. Therefore, that wave travel distance could be neglected and the spillway design flood hydrograph could be entered as local inflow.

Critical Depth Controls

A sudden failure of a dam results in a negative wave which travels up the reservoir while the positive wave is being produced downstream from the dam. The geometry of Cranks Creek Reservoir caused a critical depth control to form near river mile 4 which restricted this negative wave from developing upstream from that point during Condition II. Fortunately, the amount of storage capacity upstream from mile 4 was small and could be neglected because the computer program did not perform satisfactorily for supercritical flow. Therefore, the upstream boundary was shifted down to mile 4 in the routing model and the calculations continued with no serious error resulting from this simplification.

Dry Channel at the Upstream Boundary

In Conditions I and III, there was sufficient initial depth so the negative wave could pass upstream from mile 4 without developing transitions between supercritical and subcritical flow. However, as strong negative waves would approach the upstream boundary, the water depth would temporarily go to zero which, again, caused the computer program to malfunction. Attempts to eliminate this problem by specifying a minimum stage at the upstream boundary caused other instabilities even though 1-second computation intervals were employed. Therefore, routings were terminated. In all cases, the peak flow had passed Martins Fork Dam before routings terminated.

Initial Depth in the Channel

In all three cases studied, there was a substantial depth of water downstream from Cranks Creek Dam. Otherwise, a hydraulic bore would have formed and a different formulation of the routing equations would have ben required. (This depth contributed to movement of the flood wave, resulting in larger wave velocities than would have been experienced in a dry channel.)

Any one of these special conditions could render a study impossible with this model if its influence predominated. It is believed that in this study they were treated in a conservative manner. A generally applicable solution technique which could be used regardless of whether the channel is wet or dry, junctions with tributaries are present or not, or the flow changes between subcritical and supercritical conditions is not presently available. Since 1968 The Hydrologic Engineering Center has been working, through contracted research, to obtain such a solution, and progress is being made; but the complete solution of the general case is still in the future.

SUMMARY AND CONCLUSIONS

The objective of this study was to develop a reasonable method for calculating energy remaining in the flood wave at Martins Fork Dam and for converting the inertia and kinetic energies into total energy from which pressure loadings could be determined. The body of theory appears to be reasonably well established, but methods for implementing this theory require numerical techniques which utilize the electronic computer, and a complete solution of the general case is not presently available. Some progress is being made toward such a solution; however, in the meantime, studies of dam-break floods will be required, and these studies will have to be limited in scope because of limitations in available methods. A great deal of engineering judgment will be required to simplify and approximate the actual problem so the existing methods can be employed.

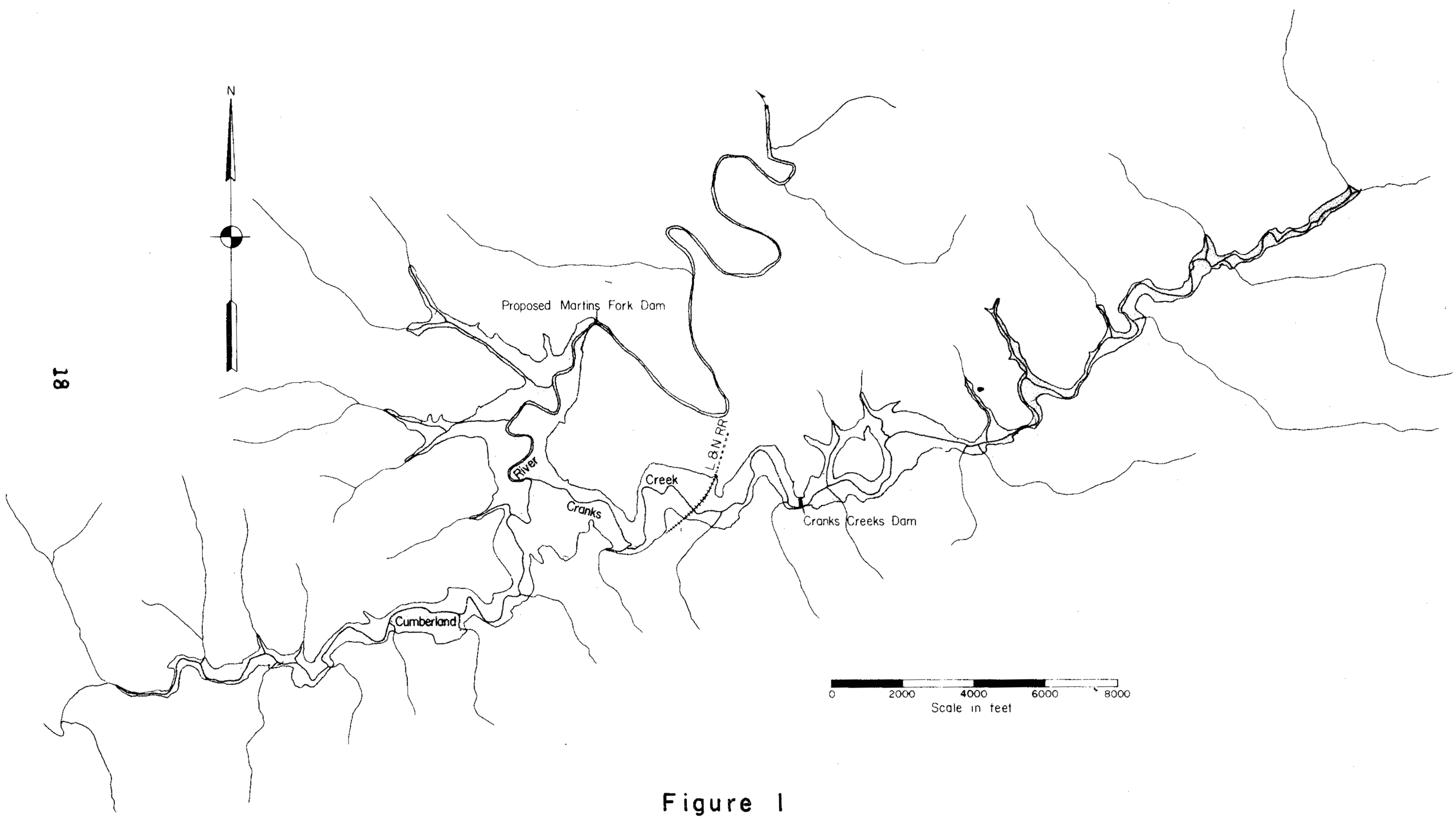

Figure 1

MARTINS FORK AND CRANKS CREEK RESERVOIRS

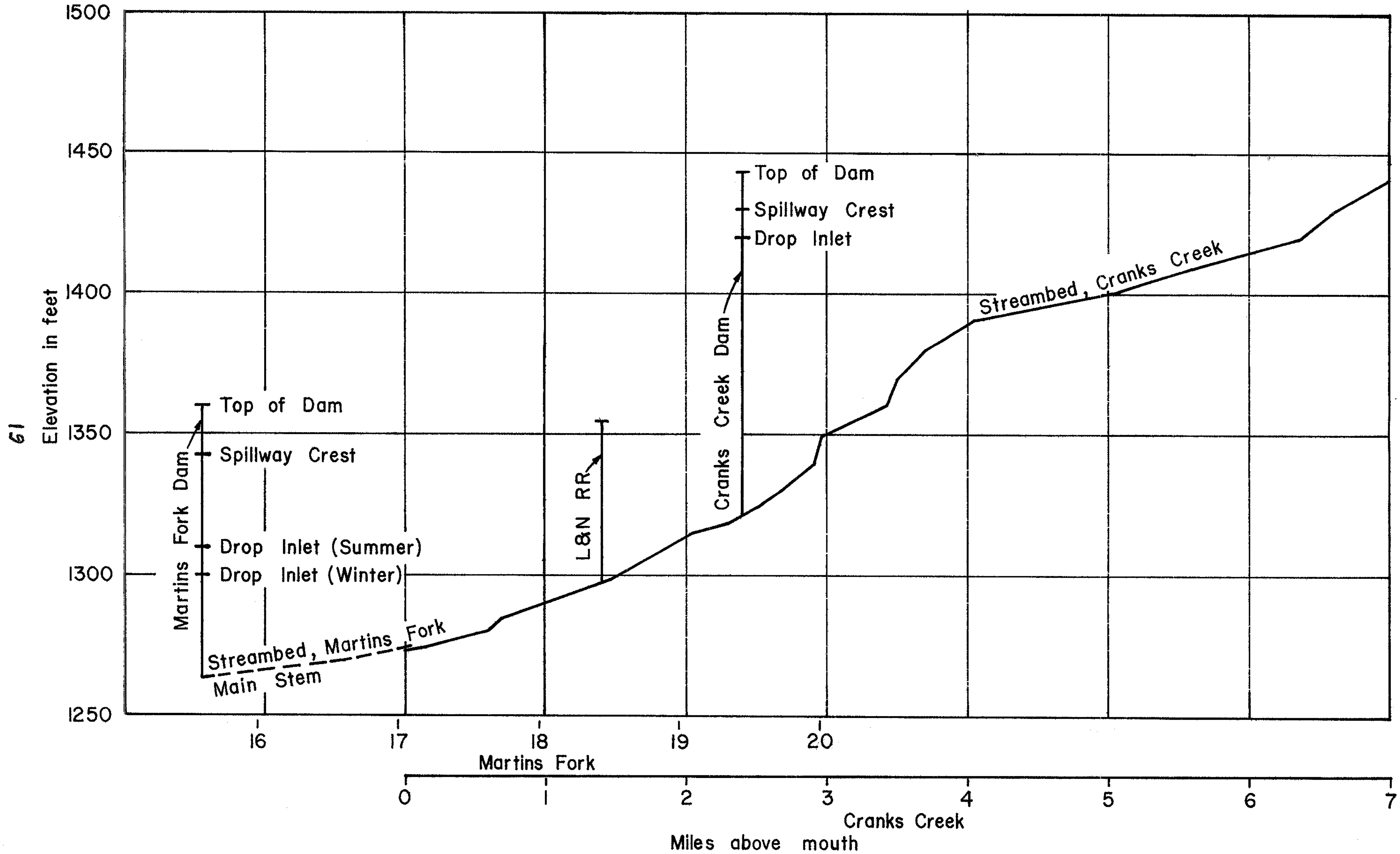

Figure 2. STREAMBED PROFILES

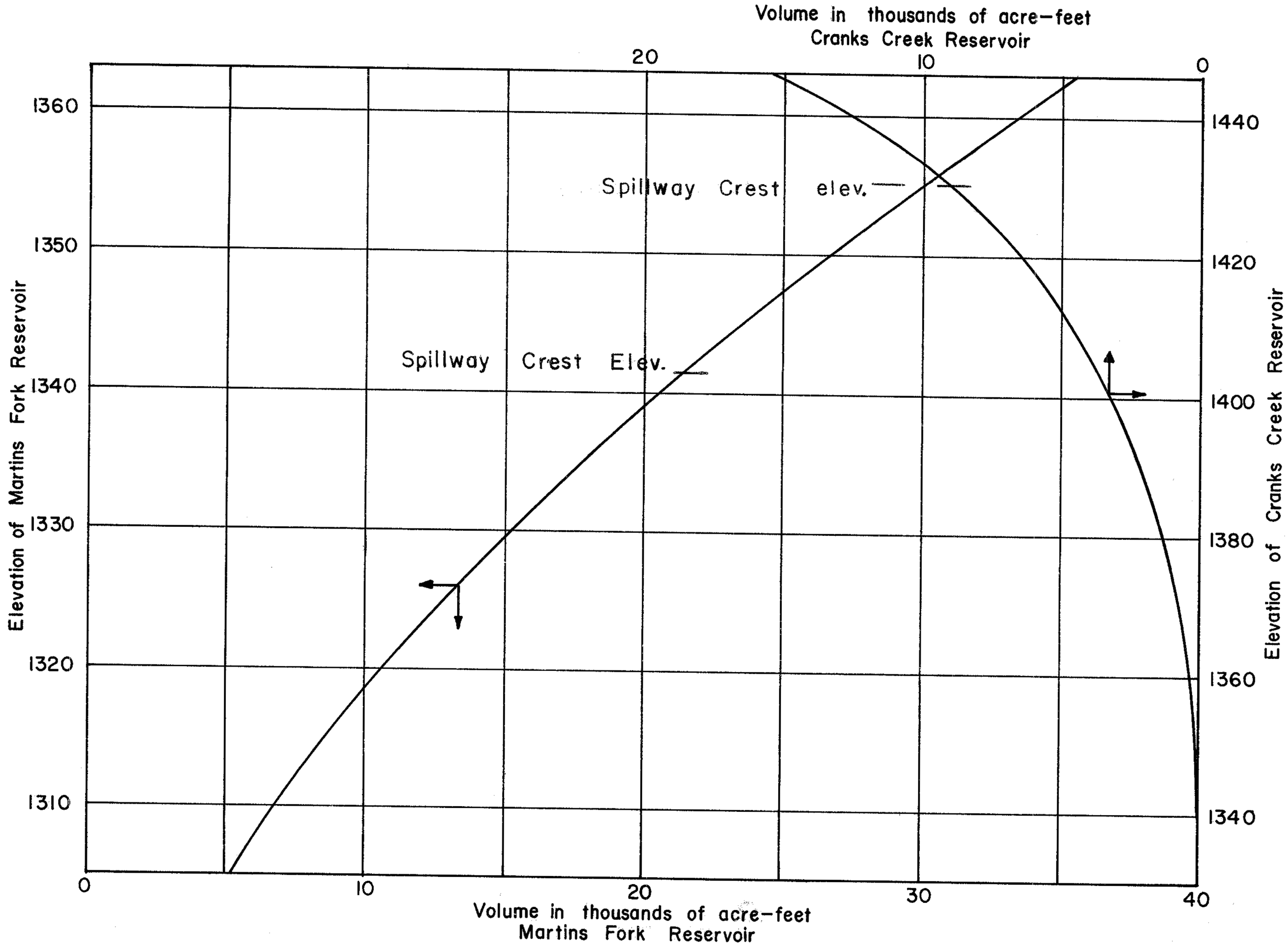

Figure 3. STORAGE IN EACH RESERVOIR

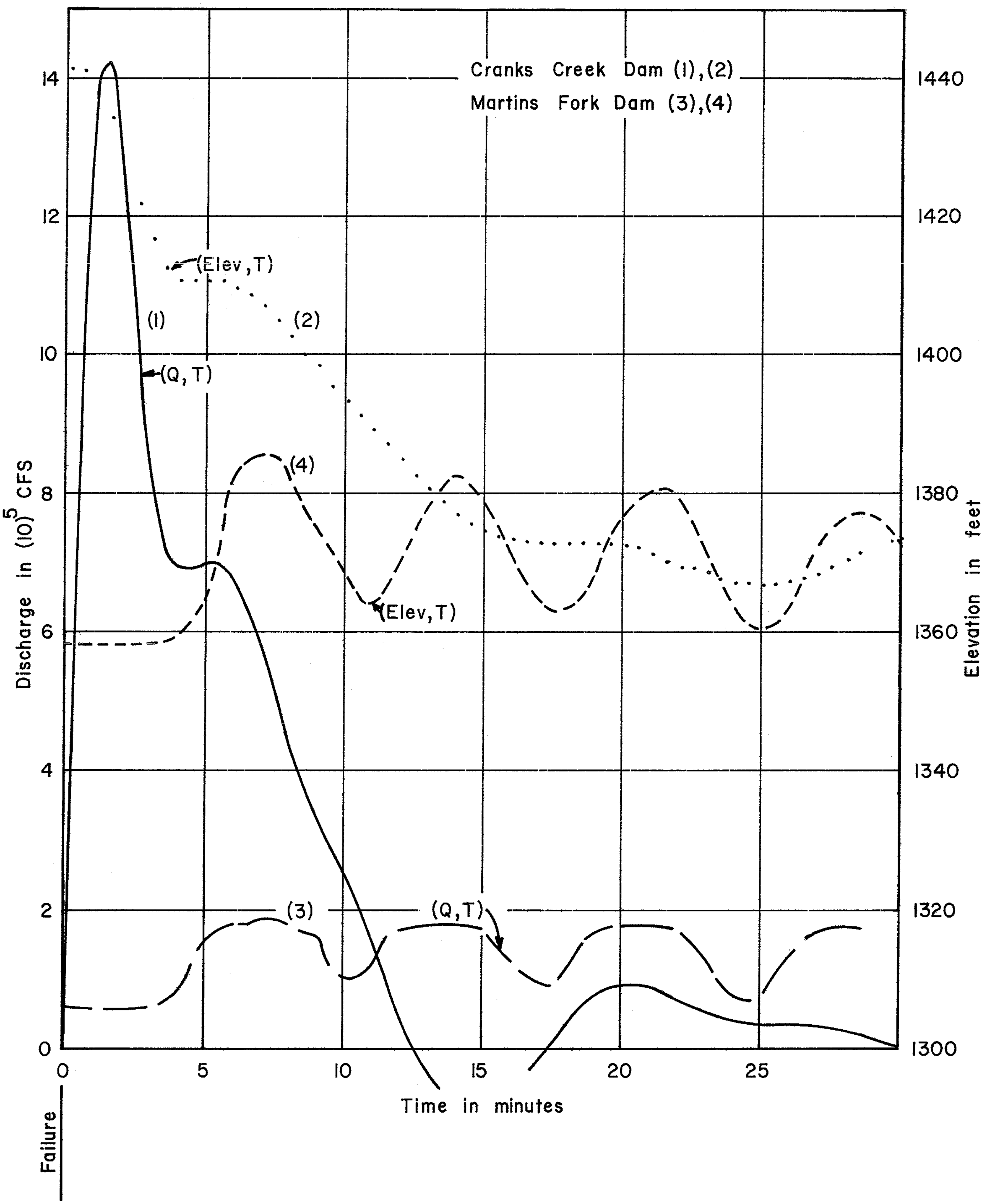

Figure 4. RESULTS FROM FAILURE AT PEAK OF S. D. F. (CONDITION I)

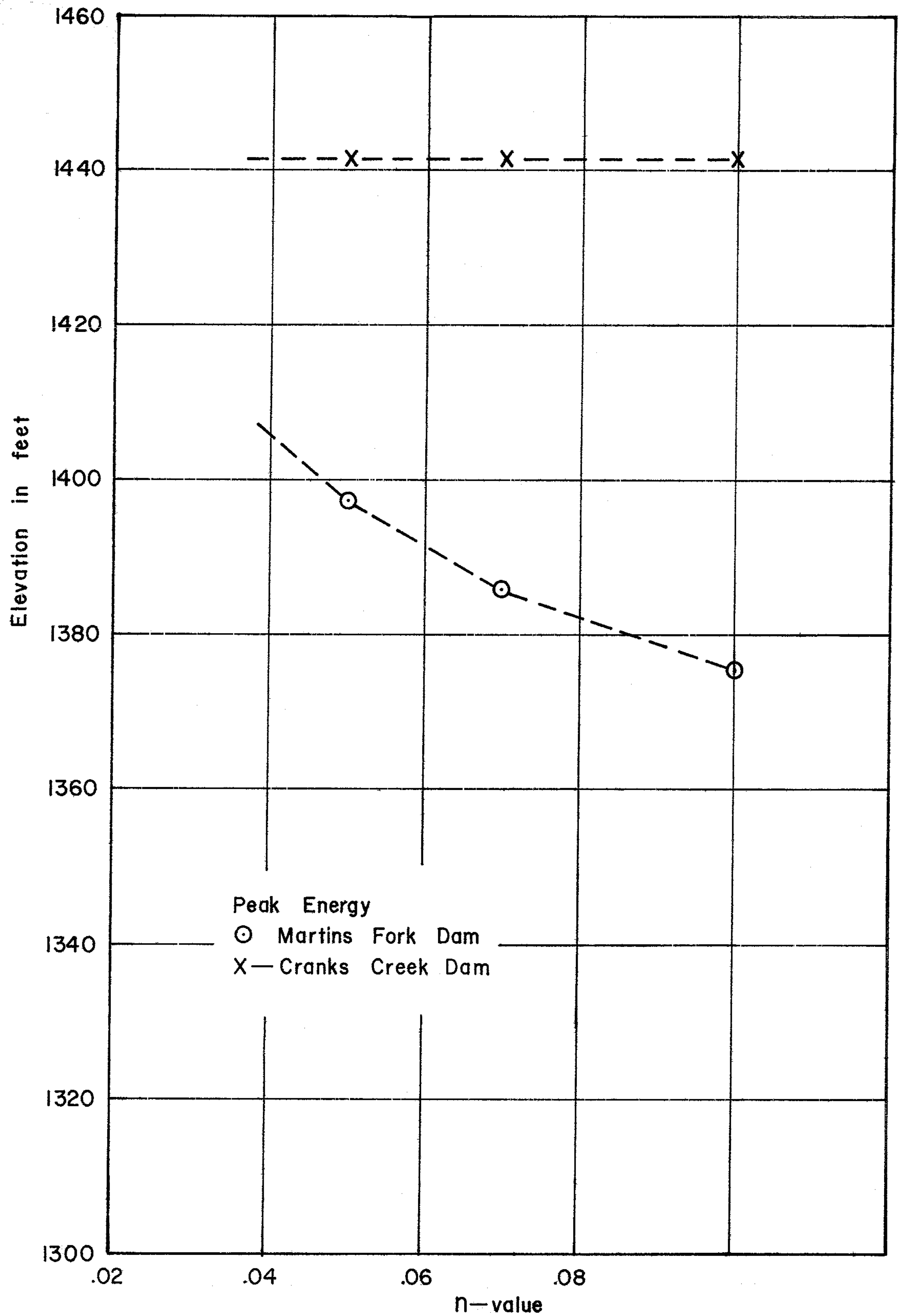

Figure 5. THE INFLUENCE OF HYDRAULIC ROUGHNESS ON PEAK ENERGY AT MARTINS FORK DAM (CONDITION I)

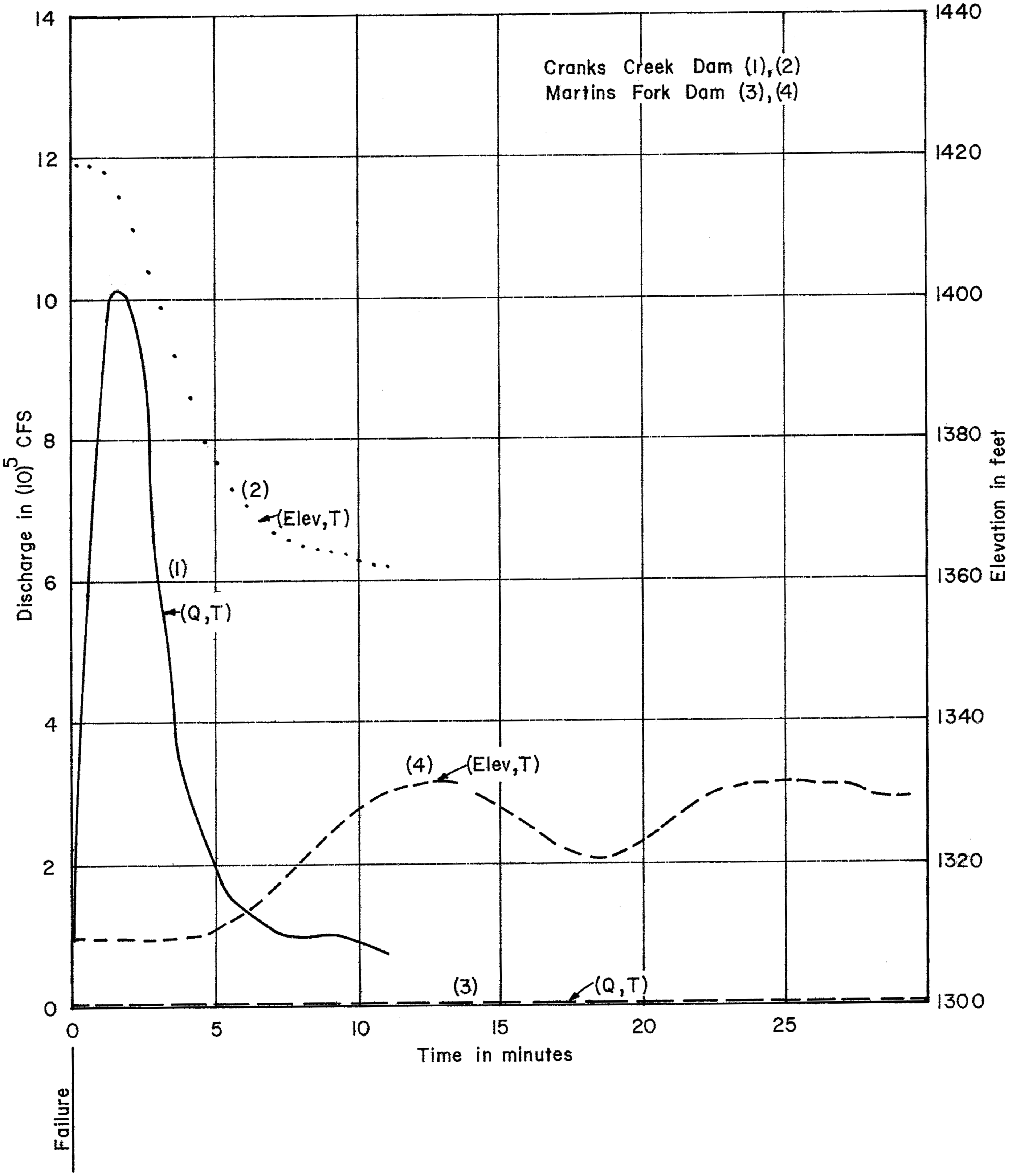

Figure 6. FAILURE WITH BOTH RESERVOIRS AT SUMMER POOL ELEVATION (CONDITION II)

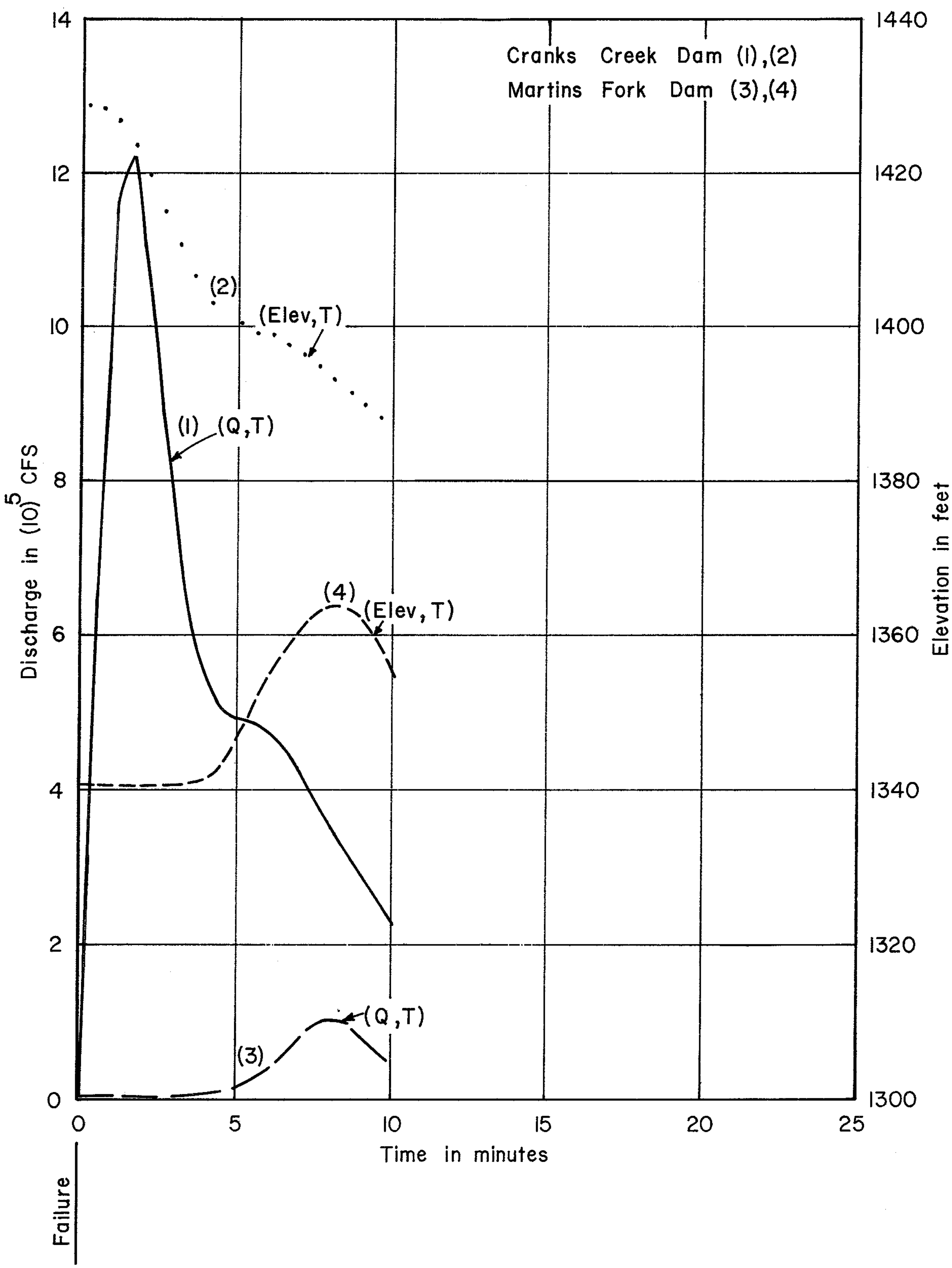

Figure 7. FAILURE WITH BOTH RESERVOIRS AT SPILLWAY CREST ELEVATIONS (CONDITION III)

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