Vermont Flood Plain Management Services

Dam-Break Flood Analysis
Noyes Pond
Groton, Vermont

March 1994

US Army Corps of Engineers
New England Division

DAM-BREAK FLOOD ANALYSIS

NOYES POND
GROTON, VERMONT

Prepared for

The U.S. Army Corps of Engineers
New England Division

at the request of

State of Vermont
Department of Environmental Conservation
Dam Safety Program

Prepared by

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EXECUTIVE SUMMARY

The primary purpose of this study is to determine the downstream hazard potential of Noyes Pond Dam for the Dam Safety Program under the jurisdiction of the State of Vermont, Department of Environmental Conservation. The secondary purpose of the study is to provide introductory information for the dam owner and to develop an Emergency Action Plan (EAP) for impending dam failure.

Dam-break flood conditions were evaluated for both sunny-day and storm-day failures. Sunny-day failure was assumed to occur with minimal inflow to the reservoir. Storm-day failure was assumed to be caused by a significant inflow hydrograph. Five storms, the 100-year storm and four fractions (1, ¾, ½ and ¼) of the probable maximum precipitation (PMP), were analyzed. The hydrological model HEC-1 was employed to develop inflow hydrographs resulting from the five storms respectively, and to determine the corresponding outflow hydrographs. The inflow hydrograph which resulted in a maximum water depth of about 2 feet above the dam crest was considered to be the inflow condition for causing storm-day (overtopping) failure. This inflow was determined to be the ½ PMF.

The two dam-failure floods were analyzed using the National Weather Service DAMBRK Flood Forecasting Model. The analysis covered a reach of about 3.2 miles along the downstream channel. Peak stages and flows at various locations along the channel were determined. Maps of inundation caused by the floods are provided.

On the basis of the U.S. Army Corps of Engineers' guidelines for dam safety inspection, Noyes Pond Dam is classified as SMALL by its size. On the basis of its potential to cause downstream damage, in terms of either loss of life or economic loss, the dam is rated Class 2 or a SIGNIFICANT hazard category.

Four major components of the EAP are discussed: monitoring, evaluation, preventive action, and warning. The EAP also includes a current listing of officials to contact in the event of an impending dam failure.

A. DAM-BREAK FLOOD ANALYSIS

1. INTRODUCTION

a. Purpose

This report presents the findings of a dam-break flood analysis performed for Noyes Pond Dam located in Groton, Vermont. The dam is owned, operated and maintained by the Department of Environmental Conservation of the State of Vermont. The purpose of this investigation was to evaluate the effect of a hypothetical dam-break flood in the downstream valley and to determine hazard classification of the dam. The investigation was not performed because of any known likelihood of a breach of the dam.

The report provides a description of pertinent features of the watershed, reservoir, and dam. Procedure of the dam-break analysis, conditions for dam-break and resulting flooding effects in downstream areas are discussed in detail. Important results include: downstream hydrographs; peak flows, peak stages and their timings at all surveyed river cross-sections; and inundation maps for the river reach under study. The report also provides a current listing of local and state officials to contact in the event of a dam failure.

b. Authority

The U.S. Army Corps of Engineers, New England Division authorized Hydraulic and Water Resources Engineers, Inc. of Waltham, Massachusetts to conduct this dam-break flood study at the request of the Vermont Department of Environmental Conservation. The study was funded through the Corps of Engineers Section 206 Flood Plain Management Services (FPMS) Program.

c. Downstream Hazard Classification

Dams are classified according to their potential to cause loss of life and property damage in the area downstream of the dam if it were to fail. The hazard classification does not refer to the condition of the dam.

The classification system used in this study has been adopted by the U.S. Army Corps of Engineers and is used by the Vermont Department of Environmental Conservation to determine inspection frequency and spillway adequacy for dams under its jurisdiction. The categories and criteria for the hazard classification of dams, as reported in "Recommended Guidelines For Safety Inspection of Dams", Department of the Army, Sept. 1979 (Ref. 1), are listed in the following table.

DAM HAZARD CLASSIFICATION

Class	Potential Hazard Category	Loss of Life (Extent of Development)	Economic Loss (Extent of Development)
3	Low	None expected (No permanent structures for human habitation)	Minimal (Undeveloped to occasional structures or agriculture)
2	Significant	Few (No urban developments and no more than a small number of inhabitable structures)	Appreciable (Notable agriculture, industry or structures)
1	High	More than a few	Excessive (Extensive community, industry or agriculture)

2. PROJECT DESCRIPTION

a. General

Noyes Pond, formed by a curved earthen dam with a concrete core, is located in the headwaters of South Branch Wells River in the Town of Groton, Vermont (Fig. 1). The dam was constructed in 1933. A land marker at the site states that the purpose of constructing the dam was "to safeguard Groton's inhabitants and perpetuate the beauty of Seyon Lake," i.e., Noyes Pond. Total length of the dam is 453 feet. Average height is 17 feet from river bottom to top of the concrete core. The dam has an Ogee-type principal spillway and two overfall emergency spillways (Fig. 2).

Noyes Pond has a drainage area of 4 square miles. At normal pool elevation, the reservoir surface area is 39 acres. The principal spillway was reported to have a maximum discharge capacity of 1,800 cubic feet per second (cfs) as stated in the inventory prepared by the Corps of Engineers (Computed capacity for the spillway is 1,200 cfs based on a 3-foot head or difference between dam crest and spillway crest). The maximum capacities of the two emergency spillways (Aux. spillway #1 and #2) are estimated to be 330 cfs and 90 cfs, respectively, using the weir equation. South Branch Wells River merges with the north branch about 5 miles downstream from Noyes Pond to form Wells River which contributes to the Connecticut River. This study covers a 3.15 -mile reach of South Branch Wells River starting from the dam.

b. Community Description

The nearest community is West Groton which is located about 3.4 miles downstream from the dam. Starting from the confluence of South Branch Wells River and Heath Brook, there are houses scattered on the right bank of the river. The road leading to Noyes Pond runs along the river and merges with Highway 302 at West Groton.

c. Downstream Conditions

South Branch Wells River is a typical mountainous stream, narrow and steep. Channel bottom elevation is about 1767 feet above National Geographic Vertical Datum (NGVD, hereafter elevation is referred to NGVD) at the dam, and drops to 1324 feet at the wood bridge located 1.274 miles from the dam. Average channel slope is 340 feet/mile. At some locations, the slope is as high as 500 feet/mile. Downstream of the bridge, channel slope becomes milder, about 70 feet /mile over the 2-mile reach from the bridge to West Groton. Heath Brook merges with South Branch Wells River about 2.2 miles downstream from Noyes Pond. Between the wood bridge and the confluence of the river and Heath Brook, there exists an open area. The flood plains within this reach of the river are expected to provide some storage when overbank flow occurs.

3. DAM DESCRIPTION

a. Identification

Noyes Pond Dam is identified by the State of Vermont as 88-6. The national inventory prepared by the U.S. Army Corps of Engineers identifies this dam as VT00149.

b. Physical Characteristics

Type:	Earthfill
Length:	453 ft
Height:	17 ft
Top Width:	2.6 ft to 3.0 ft (crest of concrete core)
Side Slope:	Upstream face, 2.5:1 (Horizontal to Vertical) Downstream face, 1.75:1 (Horizontal to Vertical)

c. Spillways

Principal Spillway:

Type: Ogee-type concrete chute
Max. Hydraulic Capacity: 1,200 cfs (computed with a discharge coefficient of 3.9)

Emergency Spillways:

Auxiliary Spillway #1

Type: 7.7-foot wide weir
Max. Hydraulic Capacity: 330 cfs (without stop logs in place)

Auxiliary Spillway #2

Type: 4.2-foot wide weir
Max. Hydraulic Capacity: 90 cfs (without stop logs in place)

d. Impoundment Behind Dam

Surface Area:

At principal spillway crest	39 acres
At top of dam	44 acres

Height of Dam: 17 feet

Storage Volume (from Vermont State Dam Inventory):

At principal spillway crest	200 ac-feet

At top of dam	350 ac-feet

e. **Dam Site Elevations** (referred to NGVD)

Dam crest	1783.7 ft
Principal spillway crest	1780.7 ft
Emergency spillways-	
Aux. spillway #1	1778.8 ft
Aux. spillway #2	1780.5 ft
Streambed at downstream toe of dam	1766.7 ft

f. **Watershed Area**

Size: 4 square miles
Type: Primarily wooded land and undulating terrain

4. METHOD OF ANALYSIS

a. Introduction

Two types of dam failures were considered in this study: "sunny-day" failure and "storm-day" failure.

A sunny-day failure typically is a piping failure. Piping is internal erosion of the embankment through displacement of fines by seepage. The erosion creates voids in the embankment and, therefore, could lead to breach and eventually collapse of the dam. It was assumed in this study that sunny-day failure occurs with minimal inflow to the reservoir and normal condition downstream.

A storm-day failure is associated with significant inflow into the impoundment. As a result of inadequate spillway capacity and reservoir storage capacity, overtopping of the embankment occurs. As the embankment is eroded, breach and ensuing failure develops.

b. Hydrology

To accommodate the storm-day dam-break analysis, inflow hydrographs for the reservoir resulting from a 100-year storm and four fractions (1, ¾, ½, ¼) of probable maximum storm (PMS) were developed. Data necessary for generating the hydrographs include rainfall data and watershed characteristics.

The rainfall data for the 100-year storm were obtained from the National Weather Service's Rainfall Frequency Atlas of the United States Technical Paper 40 (Ref. 2) and HYDRO-35. To obtain a worst-case distribution, the rainfall data of 24-hour duration were critically arrayed such that the peak rainfall increment occurred at the 12th hour preceded by the second largest rainfall increment, followed by the third largest and so on.

The rainfall data, or probable maximum precipitation (PMP) data, for estimating the maximum probable storm (PMS) which yields the probable maximum flood (PMF) were obtained from Hydrometeorological Report No. 51 (HMR51) (Ref. 3). The 72-hour duration rainfall data from HMR51 were processed according to the guidelines provided in Hydrometeorlogical Report No. 52 (HMR52) to give an estimated 24-hour duration rainfall distribution of the PMS (Ref. 4). This 24-hour duration rainfall was comprised of the four greatest 6-hour incremental rainfalls from the 72-hour duration rainfall data. The resulting total rainfall of the PMS for a 24-hour duration was calculated to be 26.5 inches.

The watershed model, HEC-1 (Ref. 5), was used to generate the inflow hydrographs resulting from the 100-year storm and the various fractions of PMS. Rainfall loss was assumed to be uniform at the rate of 0.05 inches per hour. The SCS unit hydrograph method was utilized in computing the hydrographs. This method requires the input of lag time. Based on surface condition, land slope, channel slope and flow length of the watershed, lag time was calculated as 2.8 hours.

The rainfall data and watershed characteristics were prepared by the Corps of Engineers and furnished to HWRE. These data were then used by HWRE to develop inflow and outflow hydrographs in the subsequent analysis.

c. Spillway Hydraulic Capacity

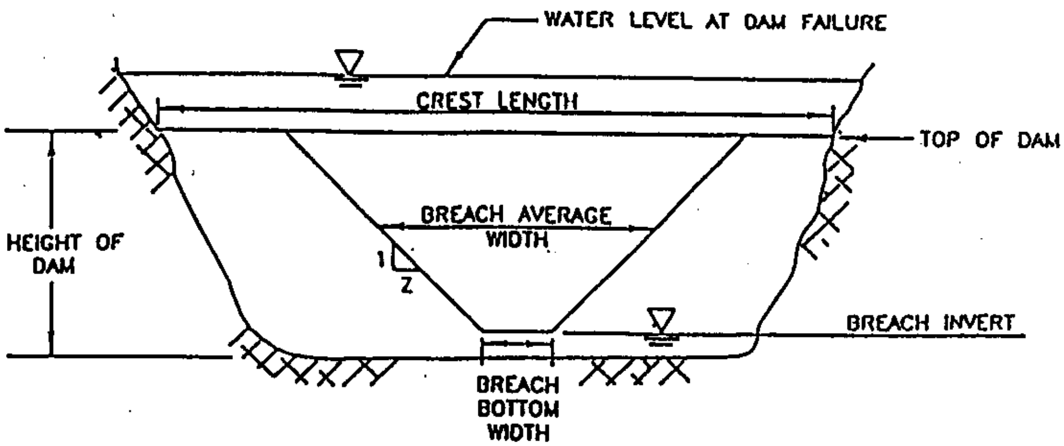
A composite rating curve was developed for the principal and emergency spillways. Weir equations were used to determine the flow rates. Flow which overtops the dam was also determined by the weir equation. The computations are included in Appendix 3.

d. Reservoir Routing

The inflow hydrographs were routed through the reservoir using the HEC-1 model to obtain outflow hydrographs. Information necessary for the reservoir routing includes elevation surface area relation of the reservoir, the composite spillway rating curve, dimensions and elevations of the dam. The Modified Puls Method is used in the HEC-1 model to solve the continuity equation. The purpose of reservoir routing was to determine the inflow hydrograph which could be used as inflow condition for the hypothetical overtopping failure of the dam.

c. Breach Discharge Hydrograph

The discharge hydrograph of a breach is a function of the inflow hydrograph, reservoir storage and breach parameters (Ref. 6). The sketch below illustrates the various dam breach parameters for a typical earth-fill dam. Total outflow from the reservoir is a combination of flows through the breach, spillways and over dam crest, if any. As the breach in the dam develops, so does the breach discharge.



DEFINITION SKETCH OF BREACH PARAMETERS

f. **Assumed Breach Parameters**

Two of the parameters for a dam-break flood study are the average breach width b and breach time t (time from the beginning to full formation of breach). Fread (Ref. 6) has developed two equations to estimate these two parameters. For the current dam, the equations yield b = 117 feet, t= 0.88 hours for overtopping failure, and b = 83 feet, t = 0.88 hours for piping failure (Appendix 3). Experience shows that $b \approx 3\ h_d$, where h_d = dam height. Noyes Pond Dam has a height of 17 feet. Therefore, an average of b = 50 feet seems reasonable for both failure scenarios. A breach time of 0.5 hours was selected. The breach discharges computed by DAMBRK with b = 50 feet and t = 0.5 hours agree fairly well with that given by another equation developed by Fread for checking the parameters (Appendix 3).

It was assumed in this analysis that the breach for overtopping failure is of trapezoidal shape with side slope equal to 1H:1V. Therefore the breach bottom width is approximately 30 feet. The shape of the breach for piping failure was assumed to be rectangular because piping failure usually develops from within the embankment. These and other parameters necessary for the dam-break flood studies are listed below:

Assumed Sunny-Day (Piping) Failure Condition:

i) Initial pool level: 1780.7 feet NGVD

ii) Dam failure level (water surface that triggers beginning of breach): 1780.7 feet NGVD

iii) Breach invert elevation: 1766.7 feet NGVD

iv) Breach bottom width: 50 feet with side slopes 1 V : 0 H

v) Time to complete formation of breach: 0.5 hours

vi) Downstream reach roughness (Manning's n values):

0.050 to 0.100 for channel
0.040 to 0.100 for overbank

vii) Embankment dimensions:

Height of dam = 17 feet
Crest length = 390 feet

Assumed Storm-Day (Overtopping) Failure Condition:

i) Initial pool level: 1780.7 feet NGVD

ii) Dam failure level (water surface that triggers beginning of breach): 1785.4 feet NGVD or 1.7 feet above dam crest

iii) Breach invert elevation: 1766.7 feet NGVD

iv) Breach bottom width: 30 feet with side slope 1 V : 1 H

v) Time to complete formation of breach: 0.5 hour

vi) Downstream reach roughness (Manning's n values):

0.050 to 0.100 for channel
0.040 to 0.100 for overbank

vii) Embankment dimensions:

Height of dam = 17 feet
Crest length = 390 feet

g. Downstream Channel Routing

Downstream channel routing was performed using the flood forecasting model, DAMBRK. A downstream channel routing analysis allows the breach discharge hydrograph to be characterized at points of interest along the stream. The breach discharge is attenuated and stored through a downstream channel and flood plain in a manner similar to that by which an inflow hydrograph is routed through a reservoir. The degree of attenuation of this breach discharge hydrograph is a function of downstream valley storage capacity and valley roughness characteristics.

(1) Method

The dynamic wave method of channel routing is used in DAMBRK to route the flood wave downstream. This is a hydraulic routing method that solves the complete equations for unsteady flow. Output from the computer code includes flood discharge, stage, and their timings at various locations along the channel.

(2) Downstream Cross-Sections

Cross-sections of the river reach for this study were obtained from field survey by HWRE. USGS topographic maps were used to supplement HWRE's survey data. Manning's "n" values for the channel and overbanks were determined based on the size of channel bed material and vegetation condition (Ref. 7). These values are listed in the previous section.

(3) Downstream Flow Structures

The wood bridge located 1.27 miles downstream from Noyes Pond is the only structure across South Branch Wells River within the study reach. The bridge was entered into the program as a restricted cross-section. At the confluence of South Branch Wells River and Heath Brook, there is a bridge across the brook. This bridge, though not involved in the computations, may not be accessible during a dam-break flood in South Branch Wells River.

In the routing procedure, the wood bridge is assumed not to fail. However, because of the increased flood stage and velocity associated with a dam-break, failure of the structure is possible. This study does not attempt to predict if any downstream structure will fail during failure of Noyes Pond Dam.

(4) Antecedent Channel Flow

Initially, a flow of 10 cfs (minimum required by DAMBRK) was tested. The flow was then increased until the program converged. It was found that, for both sunny-day and storm-day failure scenarios, an antecedent flow of 50 cfs was required and, therefore, used in the simulations. Since the magnitude of this flow rate is less than one percent of the flood peak flow, any effect on the routing results due to the antecedent flow would be negligible.

h. Lateral Flow

Flow from Heath Brook was treated as lateral flow into South Branch Wells River. For sunny-day scenario, a constant lateral flow of 10 cfs was assumed. For storm-day scenario, a hydrograph with the same occurrence frequency as that (which was later determined to be 1/2 PMF) of the inflow hydrograph for the reservoir was used. The derivation of this hydrograph is included in Appendix 2.

i. Calibration

Before the simulation of a dam-break flood, the program (DAMBRK) should be calibrated. For a gaged stream, a rating curve is the ideal data for the calibration. This type of information is, however, not available for the river reach under study. The calibration for this study was done by routing the 1/2 PMF through the river and comparing the range of inundation with elevation contours in the USGS topographic map. Necessary adjustments in roughness, location and geometry of cross-sections were made until reasonable agreement was reached.

j. Project Mapping

The project mapping was developed by enlarging the USGS 1:24,000 Metric Quadrangle (7.5 x 15 minute) of Groton, Vermont. Locations of structures within the inundation limits were verified through field survey and site reconnaissance.

k. Vertical Control

Vertical Control for this investigation was established from a level run starting at a chiseled square located on top of the south concrete wingwall of auxiliary spillway #1 at Noyes Pond as shown on a State of Vermont, Agency of Natural Resources, Department of Environmental Conservation plan sheet 1 of 2 dated May 1989. The level run ended on a standard USGS disc stamped "BC 20 1935 1192" located in a granite boulder 90 feet south of the road junction in West Groton.

5. RESULTS OF ANALYSIS

a. Inflow Hydrograph

The results of reservoir routing using HEC-1 are summarized in Table 1. The complete computer output is included in Appendices 4 and 5. Flow hydrographs for the 100-year storm and the four fractions of PMF are shown in Fig. 3. It is seen that the inflow hydrograph resulting from the 100-year storm peaks at 16 hours after beginning of the storm with a peak discharge of 1,750 cfs. This 100-year discharge appears high but it is adequate for a dam-break flood analysis. The PMF inflow hydrograph peaks also at 16 hours but has a peak discharge of 9,790 cfs. Since the reservoir storage is very small, the outflow hydrographs are almost identical to the inflow hydrographs. In general, the difference between peak inflow and peak outflow is less than 5%.

As seen in Table 1, all the inflow hydrographs result in flow overtopping the dam. Water depth above the dam crest varies from 0.2 feet for the 100-year storm to 3.3 feet for the full PMF. The 1/2 PMF yields a water stage 1.7 feet above the dam crest. According to experience and recommendation in the scope of work for this study, overtopping failure should be considered to occur when an inflow hydrograph results in a water stage no more than a few feet above dam crest. The 1/2 PMF inflow hydrograph is therefore selected to be the inflow condition for storm-day failure analysis.

b. Reservoir Storage Capacity

The maximum storage capacity of the reservoir, i.e., storage at dam crest, is approximately 300 acre-feet (calculated by HEC-1). The calculated storage is less than the original design storage (350 acre-feet) primarily because the method in reservoir routing treats the reservoir storage as an inverse cone. However, it should be pointed out that continuous deposition in the reservoir over the years since construction of the dam is expected to have reduced the storage. The calculated storage is probably more realistic. As the 1/2 PMF outflow reaches its peak stage of 1785.4 feet, the volume of water stored in the reservoir is calculated to be 390 acre-feet.

c. Spillway Hydraulic Capacity

The combined maximum hydraulic capacity for the principal and emergency spillways (top of stop log at the same elevation as the principal spillway crest) is approximately 1,400 cfs. It is obvious that Noyes Pond does not have adequate storage and spillway capacities to route and pass any of the floods treated in this study although the computed 100-year flood overtops the dam by only 0.2 feet..

d. Breach Discharge Hydrograph

Tables 2 and 3 summarize the peak discharges and stages at critical stations along the downstream channel due to sunny-day and storm-day failures, respectively. The discharge and stage hydrographs at these stations are shown in Figs. 4 and 5. The complete computer output for each of the two scenarios is included in Appendices 6 and 7 respectively. Sunny-day failure is assumed to start at 0.0 hour. A peak flow of 5,870 cfs is produced at 0.50 hours due to failure of the dam. At the wood bridge, peak flow occurs at 0.63 hours and is reduced to 5,530 cfs. The reduction is small because the valley has virtually no storage upstream of the bridge. At the confluence with Heath Brook, peak flow is reduced to 4,450 cfs. The flood is attenuated as it passes the flood plains in the reach between the wood bridge and the confluence with Heath Brook. In the downstream reach, peak flow remains nearly the same, 4,360 cfs at West Groton.

The storm-day dam failure results in a peak flow of 10,790 cfs at the dam site 16.08 hours after the beginning of the storm, or, at the time when breach starts. Peak flow at the bridge is 10,370 and it occurs at 16.60 hours. The flood is reduced to 8,990 cfs at the section just before the confluence with Heath Brook. The reduction is attributed to the storage over the flood plains. Immediately downstream of the confluence, peak flow is increased to 14,800 cfs due to the flood contributed by the Heath Brook. The flow is slightly reduced to 14,700 cfs at West Groton.

e. Downstream Channel Routing

One of the major parameters which define the severity of a flood is the flood stage. The peak flood stages resulting from the two hypothetical dam-break floods at the surveyed cross-sections along the river under study are depicted in Figs. 6 and 7 , respectively. Flow conditions at critical locations are described below (all elevations are referred to NGVD).

(1) Sunny-Day Results

At the wood bridge (1.27 miles), peak stage is 1334.7 feet. Water surface is 1.7 feet above top of the bridge (elevation: 1333.0 feet). At the confluence with Heath Brook, peak stage is 1268.7 feet. The road on the right bank is inundated by about 1.5 feet (road surface elevation: 1267.1 feet). The side bridge across Heath Brook has a top elevation of 1271.1 feet and, therefore, is not inundated. However, the bridge is not accessible due to inundation of the road downstream.

At the location of 2.83 miles, peak stage is 1229.5 feet. The flow is confined in the channel. The road on the right bank is accessible (road surface elevation: 1234.7 feet). At the downstream end of the study reach, or, West Groton, peak stage is 1203.5 feet. No overbank flow occurs. Road surface (elevation: 1207.1 feet) is 3.6 feet above water surface.

(2) Storm-Day Results

The storm-day failure results in a peak stage of 1337.0 feet at the wood bridge, overtopping the bridge by 4 feet. The road leading to the bridge is inundated. At the confluence with Heath Brook, peak stage reaches 1274.1 feet, overtopping the road by 7 feet and the upstream side bridge by 3 feet.

At the location of 2.83 miles, peak stage is 1233.5 feet, below the road on the right bank. It is noted that the road at this location is not inundated by either the sunny-day failure flood or the storm-day failure flood because the road is much higher above the river bed at this location than the upstream section (2.23 miles). At West Groton, peak stage reaches 1209.1 feet, inundating the road by 2 feet. The inundation occurs primarily because the channel narrows toward this location.

f. Inundation Mapping

The limits of inundation caused by the two hypothetical dam failure floods are defined by the boundaries of flow at the peak stages along the downstream channel. The flood resulting from the sunny-day failure is depicted in Fig. 8 and predicted to inundate the houses on the right bank of the river between the 2.23-mile and 2.83-mile sections. At the downstream end, the flood is confined in the channel. The houses in West Groton are not expected to be affected by the flood.

Fig. 9 shows the inundation limit of the storm-day failure flood. This flood is predicted to overtop the bridge across Heath Brook, inundate the road, houses and barns on the right bank of South Branch Wells River from its confluence with Heath Brook to the section at 2.83 miles. A portion of the road through West Groton, located beyond the limit of the river reach under study, is expected to be overtopped by the flood.

g. Size Classification

Noyes Pond Dam is 17 feet high and its design maximum storage is 350 acre-feet. According to Article 2.1.1 of the "Recommended Guidelines for Safety Inspection of Dams" (Ref 1), the dam is classified as small in size.

h. Hazard Classification

As shown in the inundation maps, the floods due to a sunny-day failure and a storm-day failure would inundate several houses on the right bank of the river. The houses in West Groton might also be inundated by the storm-day failure flood. On the basis of its potential to cause downstream damage, in terms of either loss of life or economic loss, Noyes Pond Dam is rated Class 2 or a significant hazard category.

TABLE 1

RESERVOIR ROUTING SUMMARY
(HEC-1 Model Results)

Flood Frequency	Peak Inflow (cfs)	Peak Outflow (cfs)	Peak Stage (ft. NGVD)	Water Depth above Dam Crest (ft)	Flow Condition
100-year	1750	1640	1783.9	0.2	overtopped
1/4 PMF	2430	2360	1784.4	0.7	overtopped
1/2 PMF	4965	4730	1785.4	1.7	overtopped
3/4 PMF	7510	7100	1786.3	2.6	overtopped
1 PMF	9790	9470	1787.0	3.3	overtopped

TABLE 2

DOWNSTREAM CHANNEL ROUTING RESULTS FOR SUNNY-DAY FAILURE

Downstream Location	Peak Discharge (cfs)	Peak Stage (ft NGVD)	Depth Above Streambed (ft)	Time to Peak Stage After Breach (hours)
Noyes Pond Dam (at 0.0 mi.)	5870	1780.7	14.0	0.00
Wood Bridge (at 1.27 mi.)	5530	1334.7	10.2	0.63
Heath Brook (at 2.23 mi.)	4450	1268.7	8.7	1.13
Road (at 2.83 mi.)	4380	1229.5	8.4	1.30
West Groton (at 3.15 mi.)	4360	1203.5	8.4	1.35

TABLE 3

DOWNSTREAM CHANNEL ROUTING RESULTS FOR STORM-DAY FAILURE

Downstream Location	Peak Discharge (cfs)	Peak Stage (ft NGVD)	Depth Above Streambed (ft)	Time to Peak Stage After Start of Storm (hours)	Time to Peak Stage After Start of Breach** (hours)
Noyes Pond Dam (at 0.0 mi.)	10790	1785.4	18.7	16.08	0.00
Wood Bridge (at 1.27 mi.)	10370	1337.0	12.5	16.60	0.52
Heath Brook (at 2.23 mi.)	14810*	1274.1	14.1	16.97	0.89
Road (at 2.83 mi.)	14700	1233.5	12.4	17.08	1.00
West Groton (at 3.15 mi.)	14690	1209.1	14.0	17.13	1.05

* Including lateral inflow.

** Dam begins to break at 16.08 hrs when the reservoir water surface reaches 1785.4 feet NGVD.

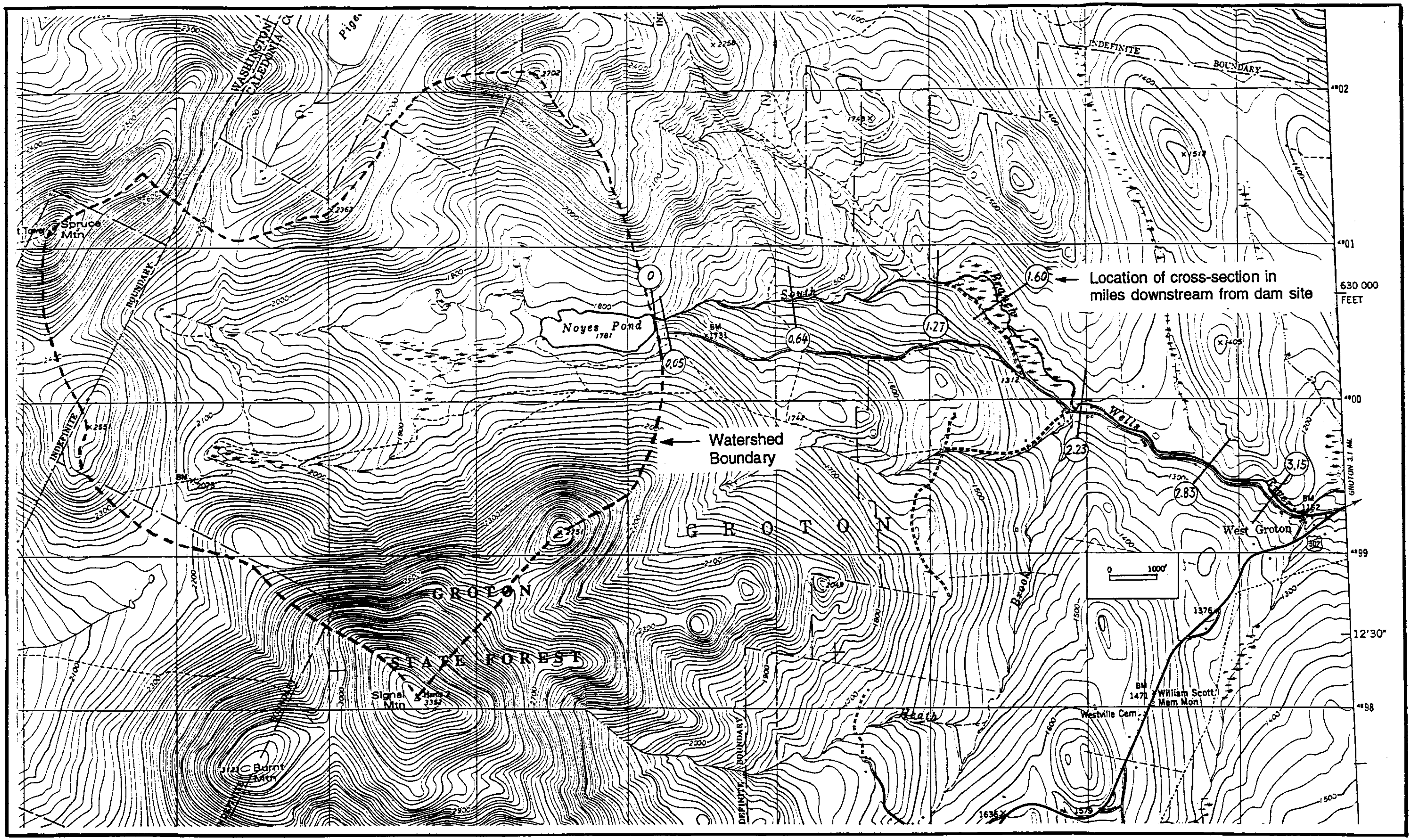


Figure 1. Index Map

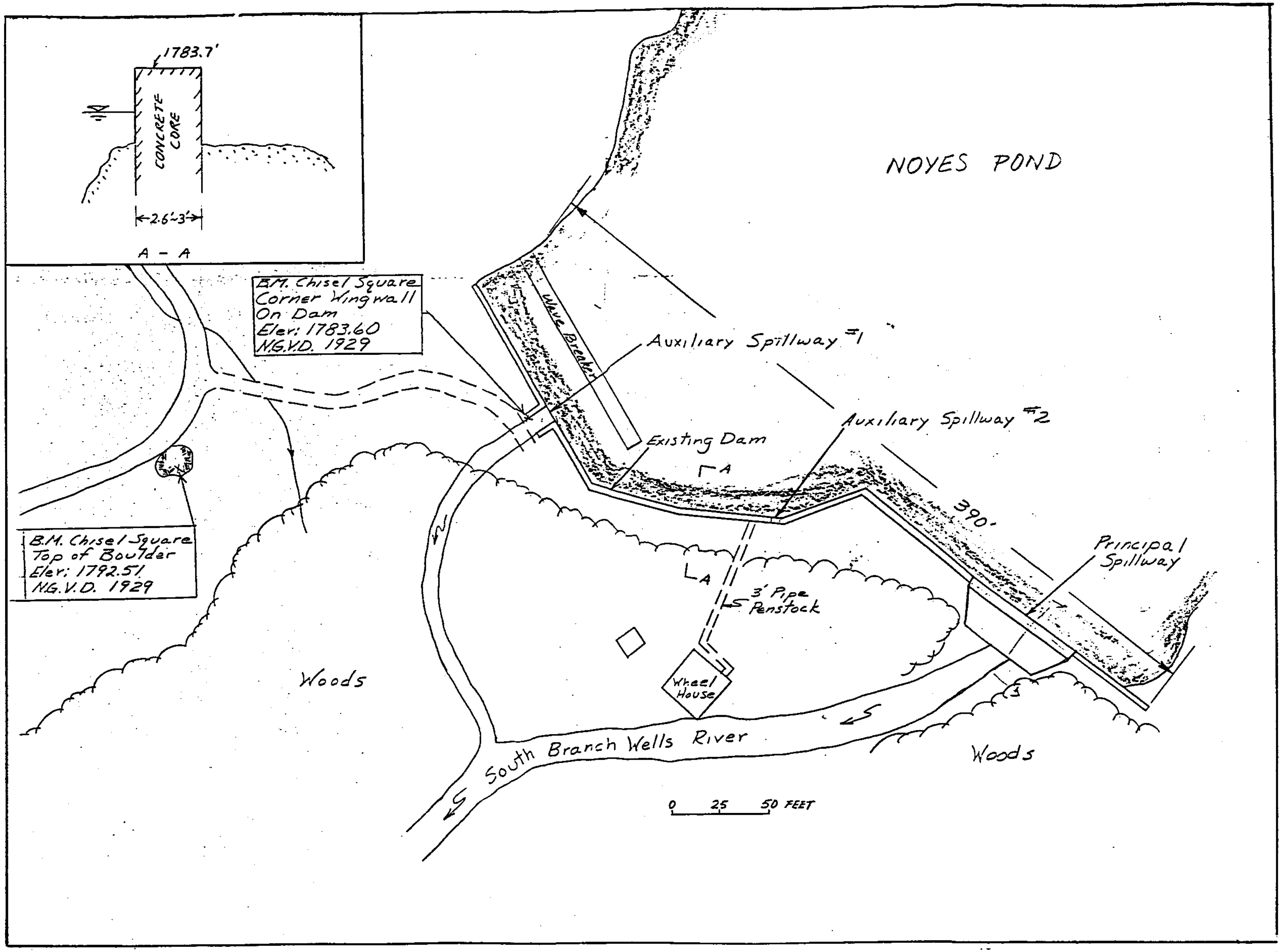


Figure 2. Plans of Noyes Pond and Dam Appurtenances.

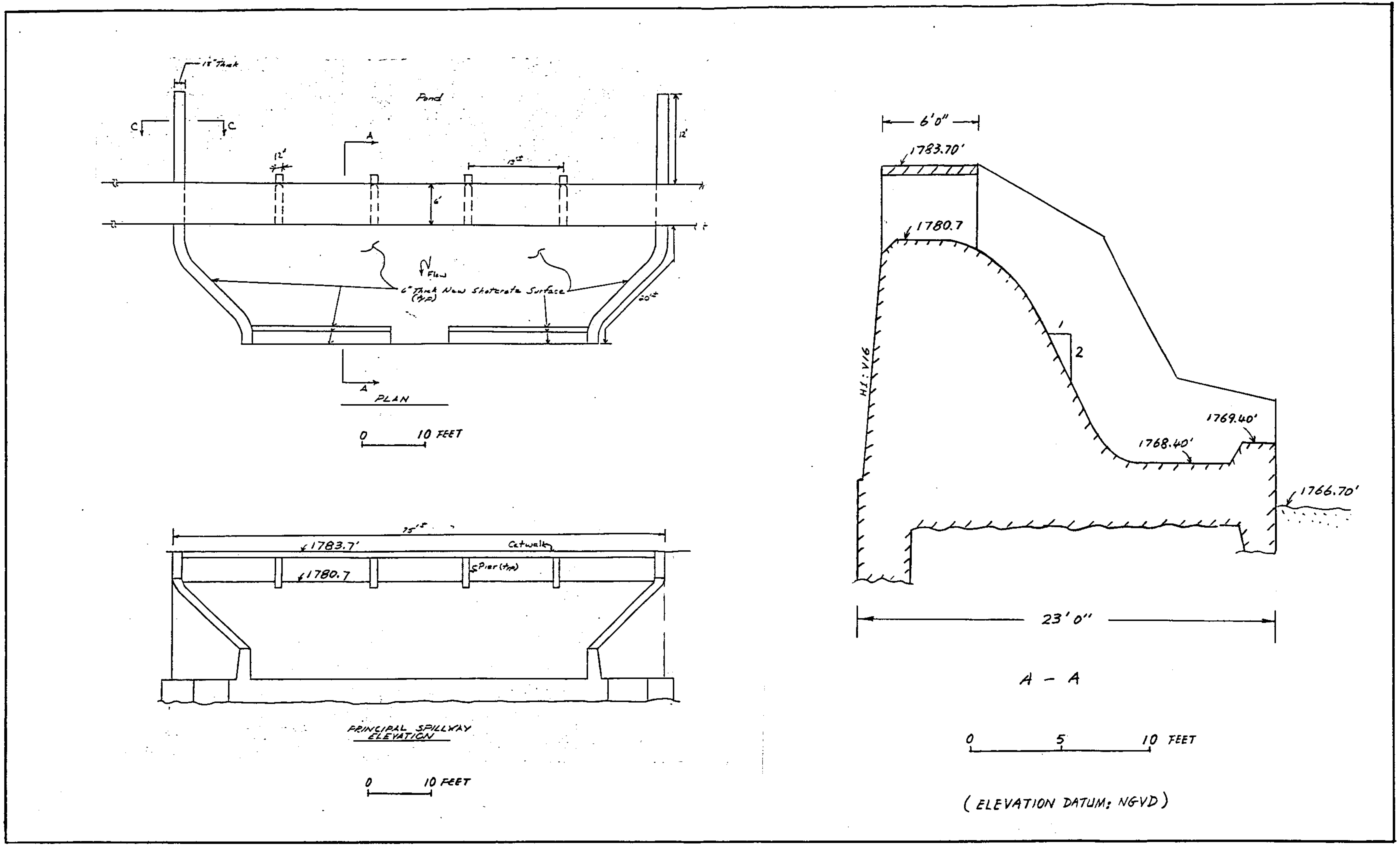


Figure 2. (continued)

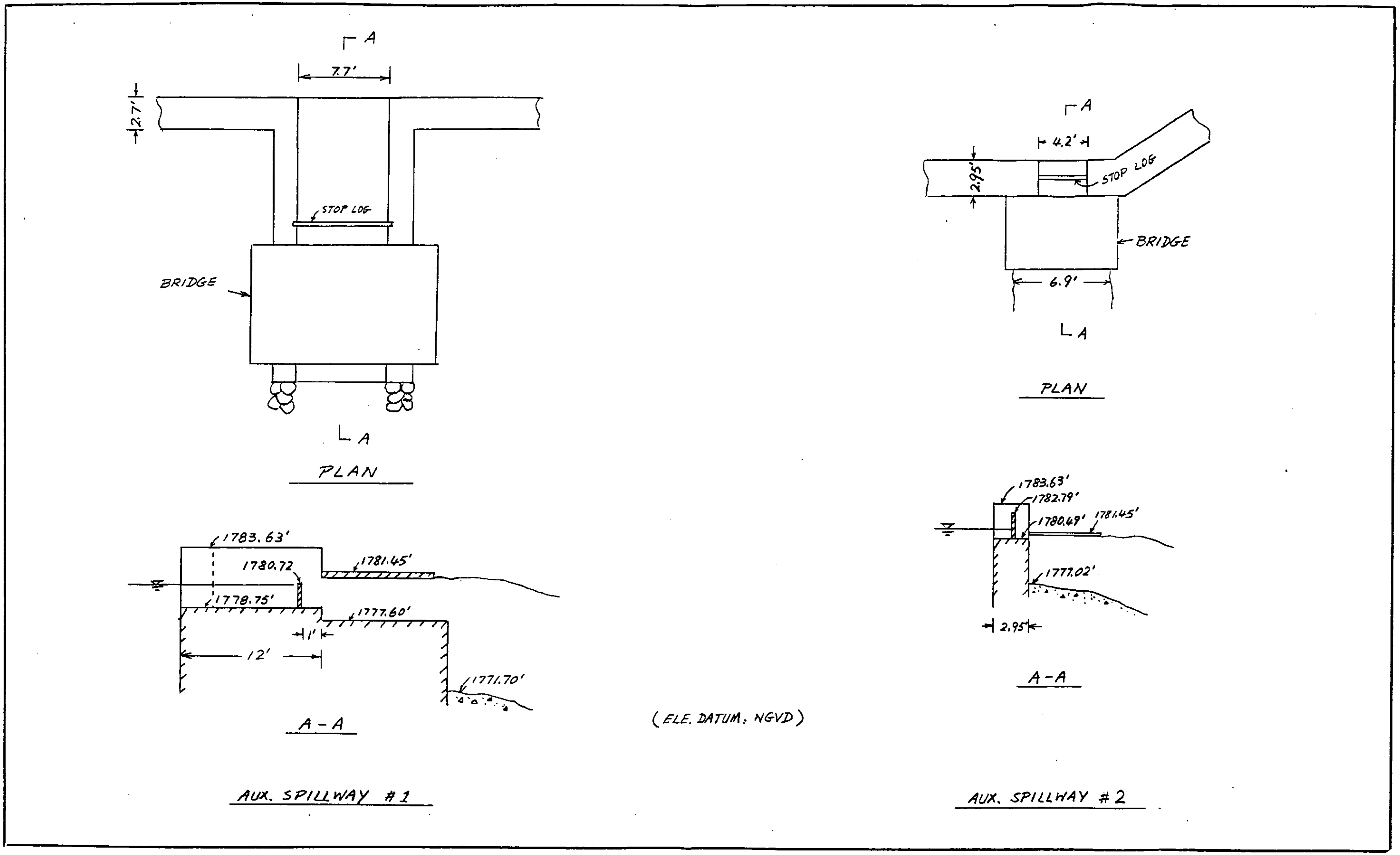


Figure 2. (continued)

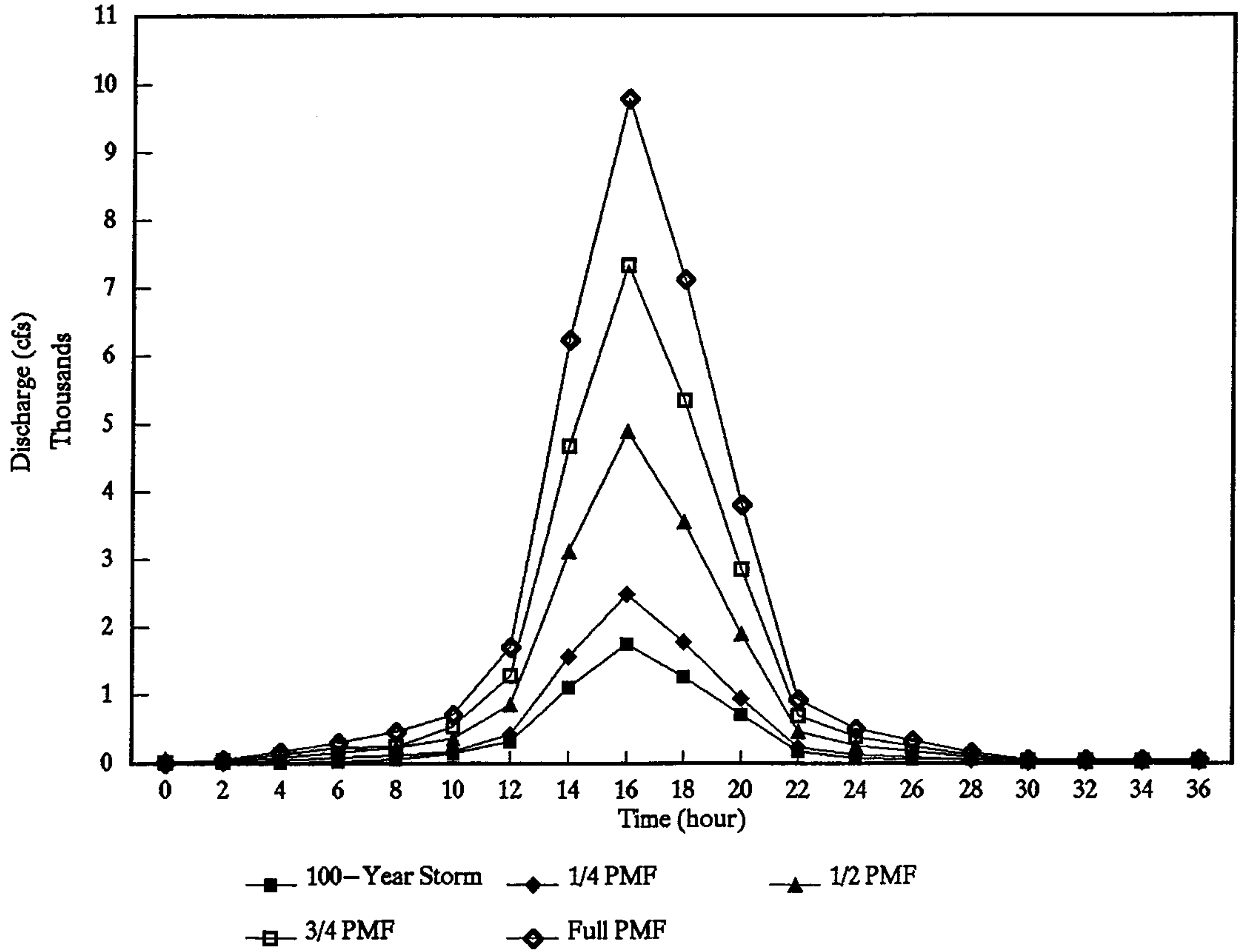


Figure 3. Inflow Hydrographs.

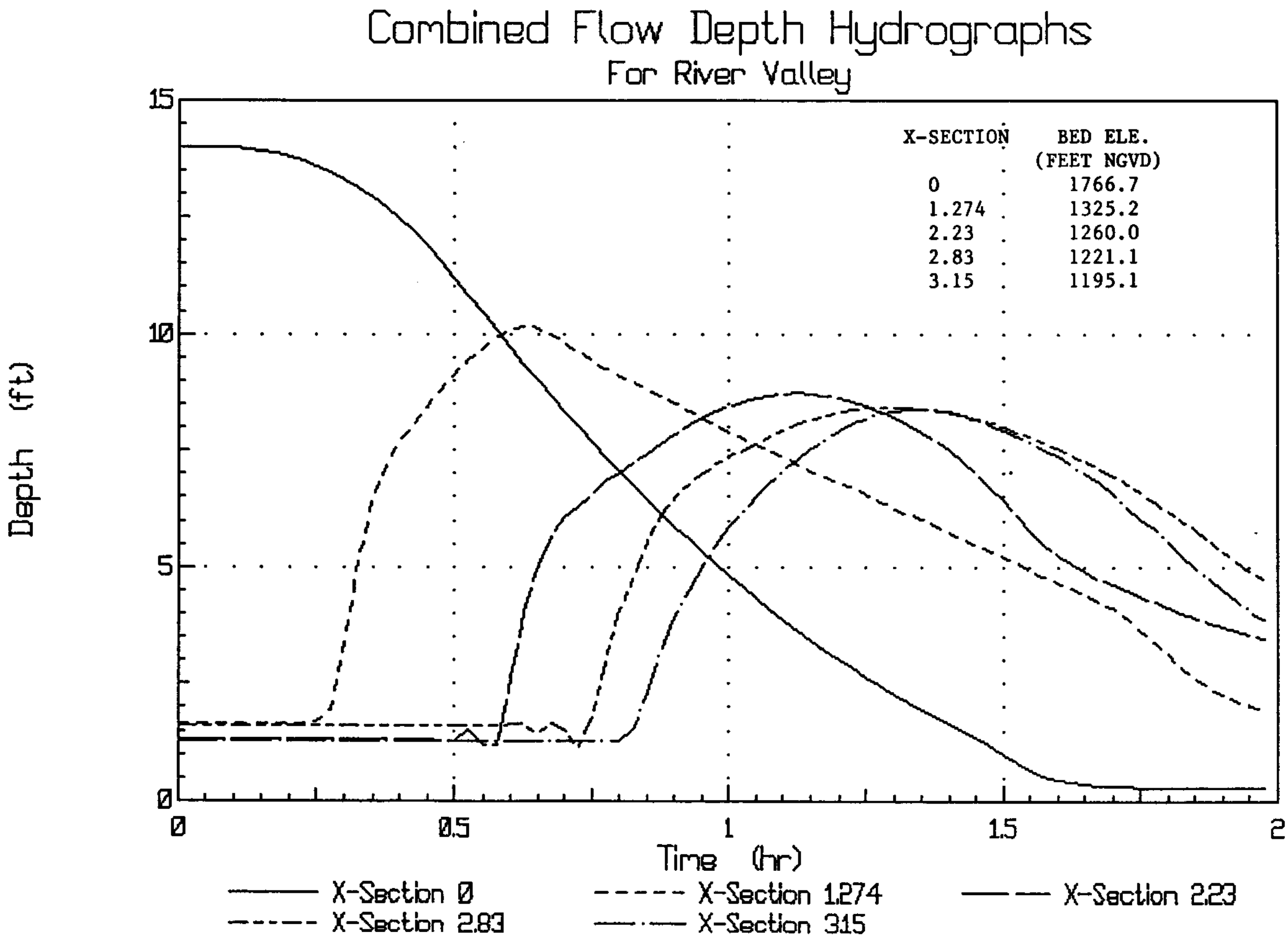


Figure 4. Stage and Flow Hydrographs Resulting from Sunny-Day Dam-Break Flood.

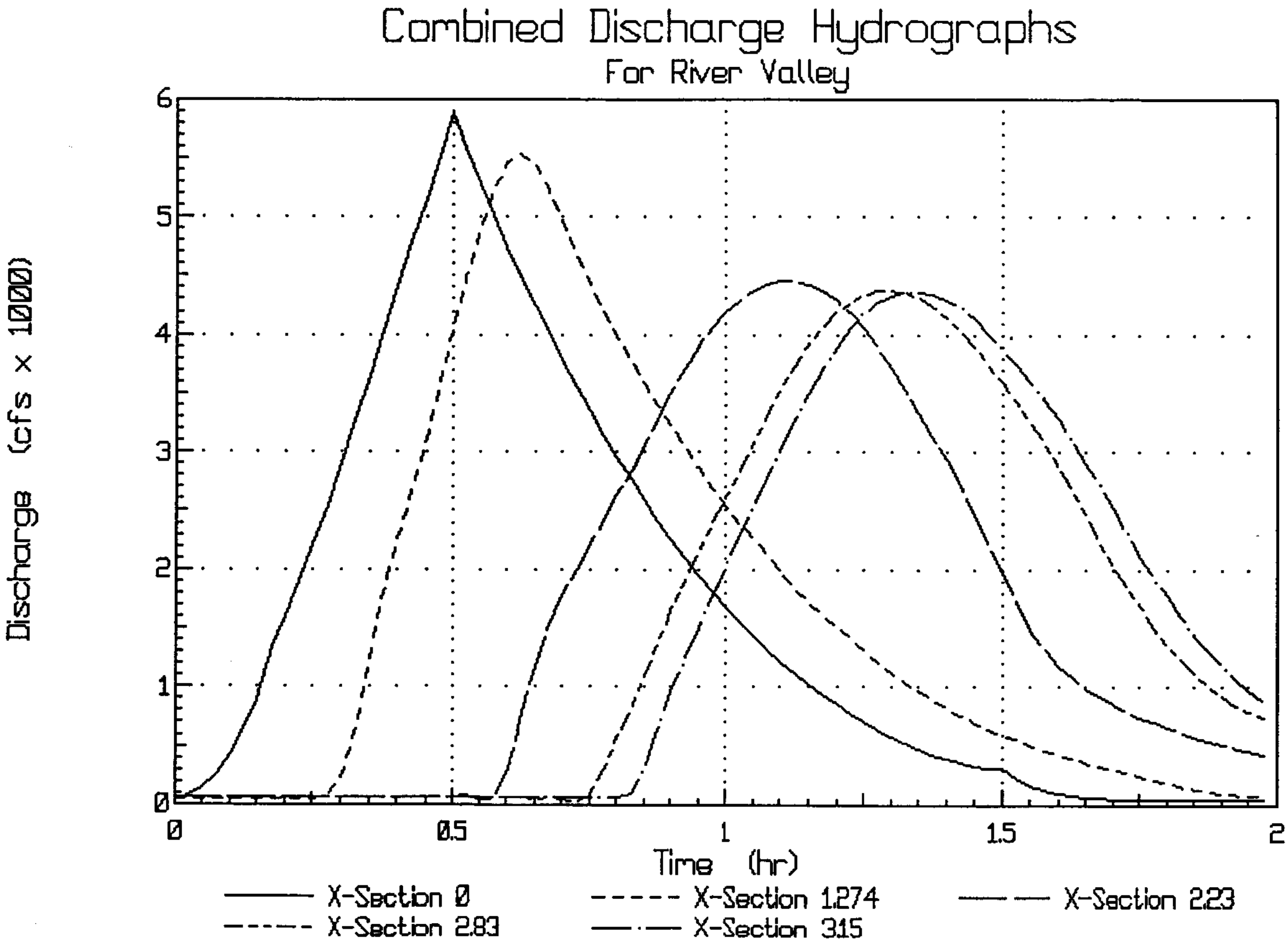


Figure 4. (continued)

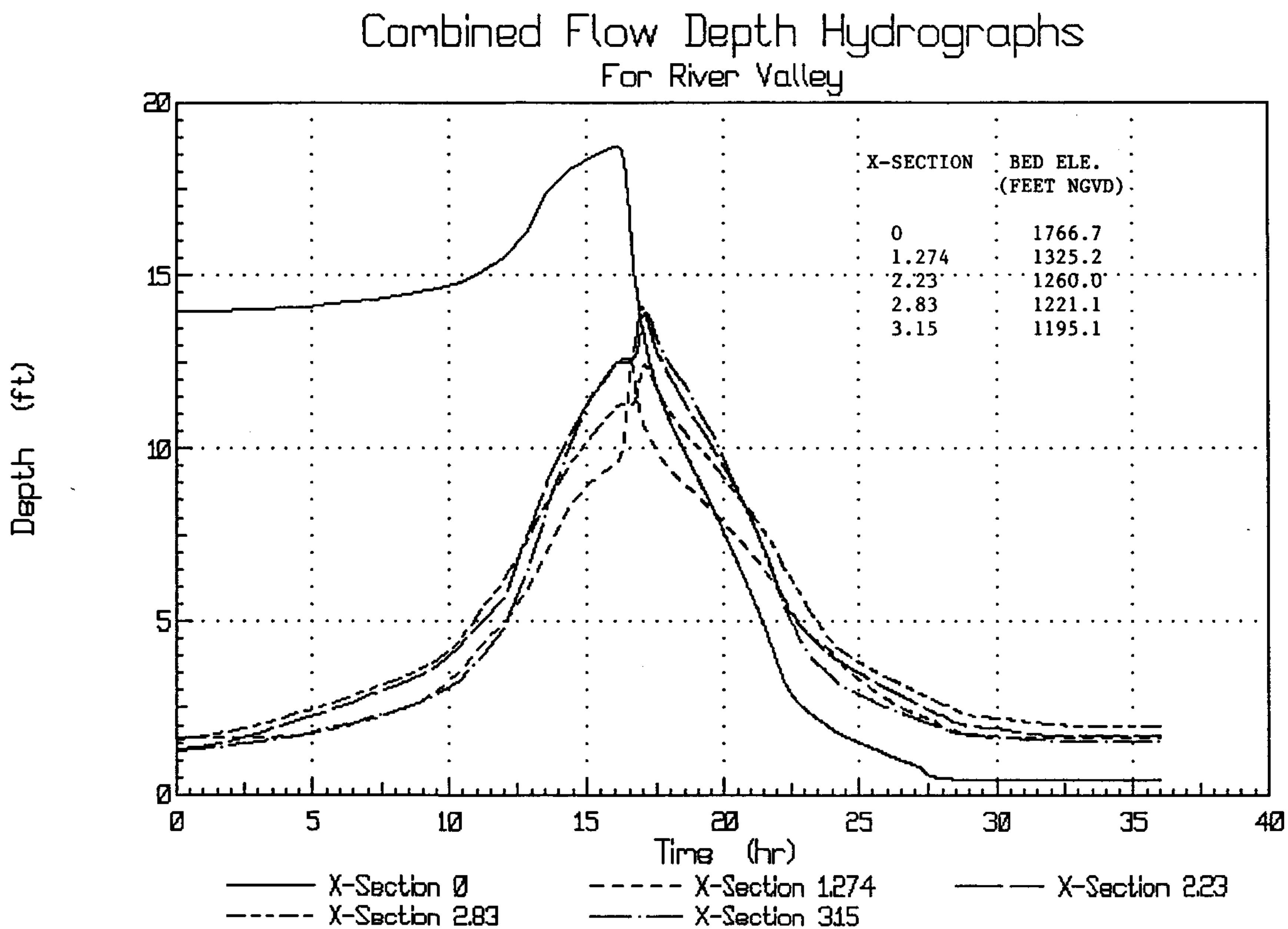
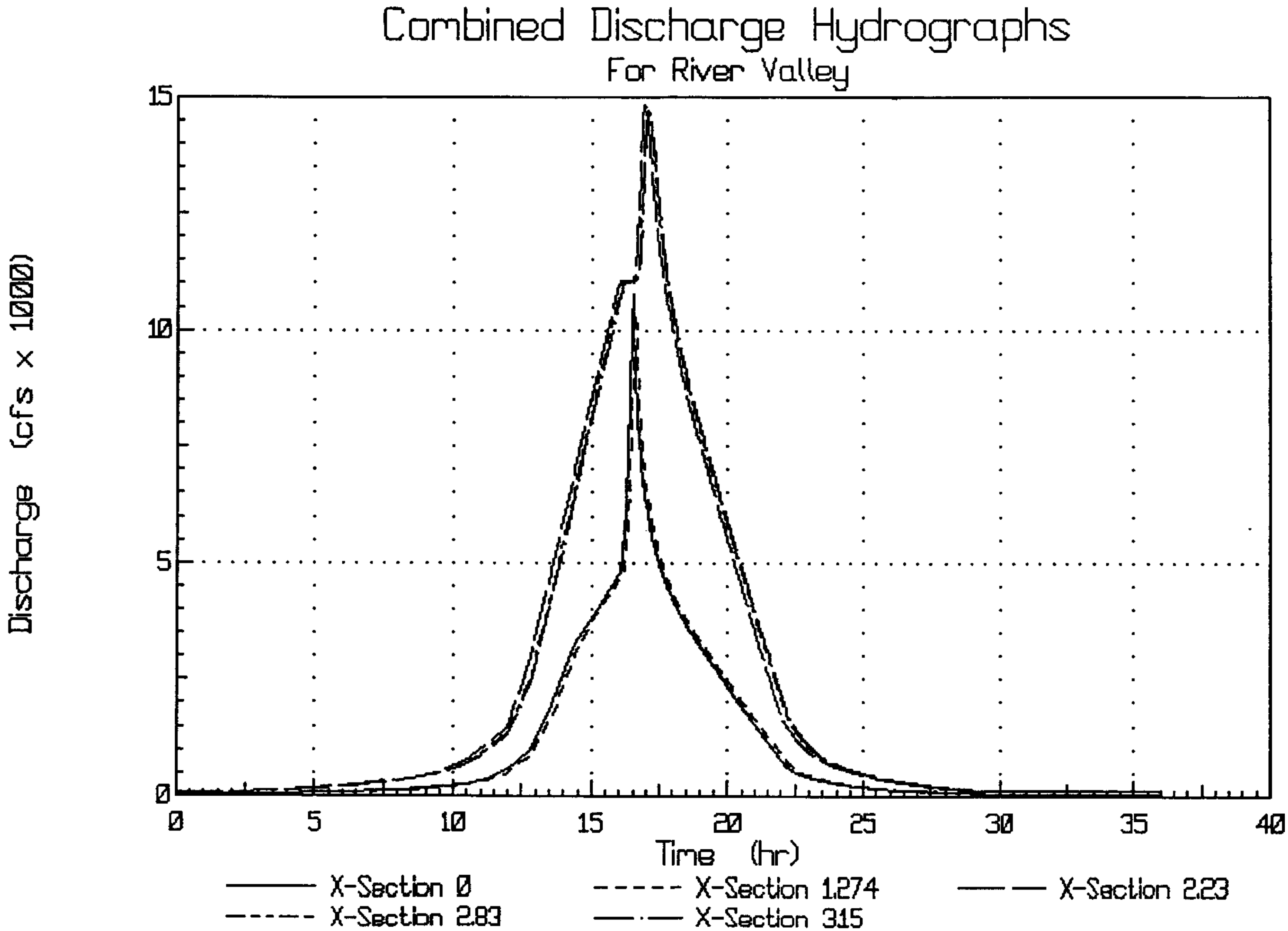


Figure 5. Stage and Flow Hydrographs Resulting from Storm-Day Dam-Break Flood.


Combined Discharge Hydrographs
For River Valley
Discharge (cfs x 1000)
15
10
5
0
0
5
10
15
20
25
30
35
40
Time (hr)
X-Section 0
X-Section 1.274
X-Section 2.23
X-Section 2.83
X-Section 3.15

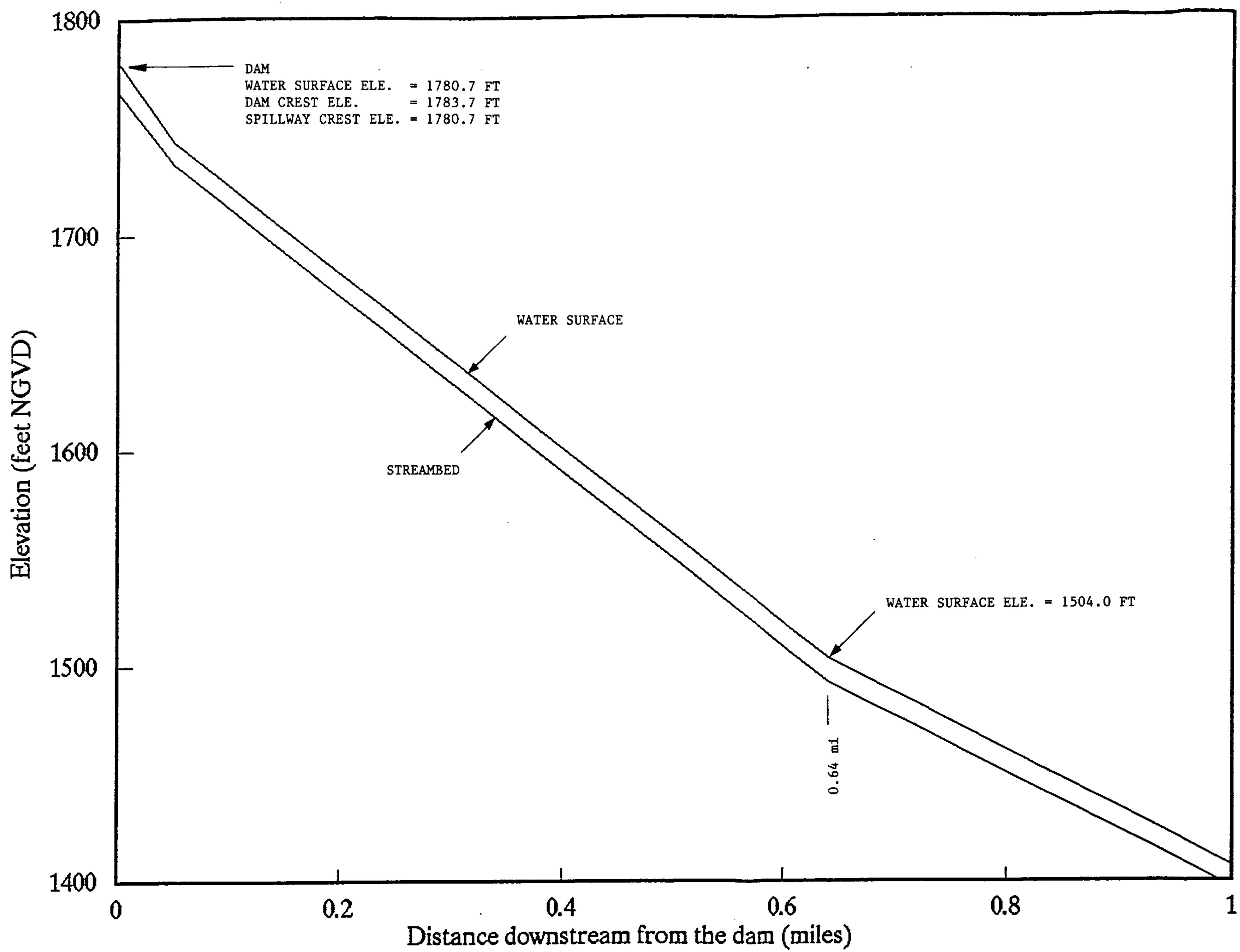


Figure 6. Peak Flood Stages along the Downstream Channel for Sunny-Day Failure Scenario.

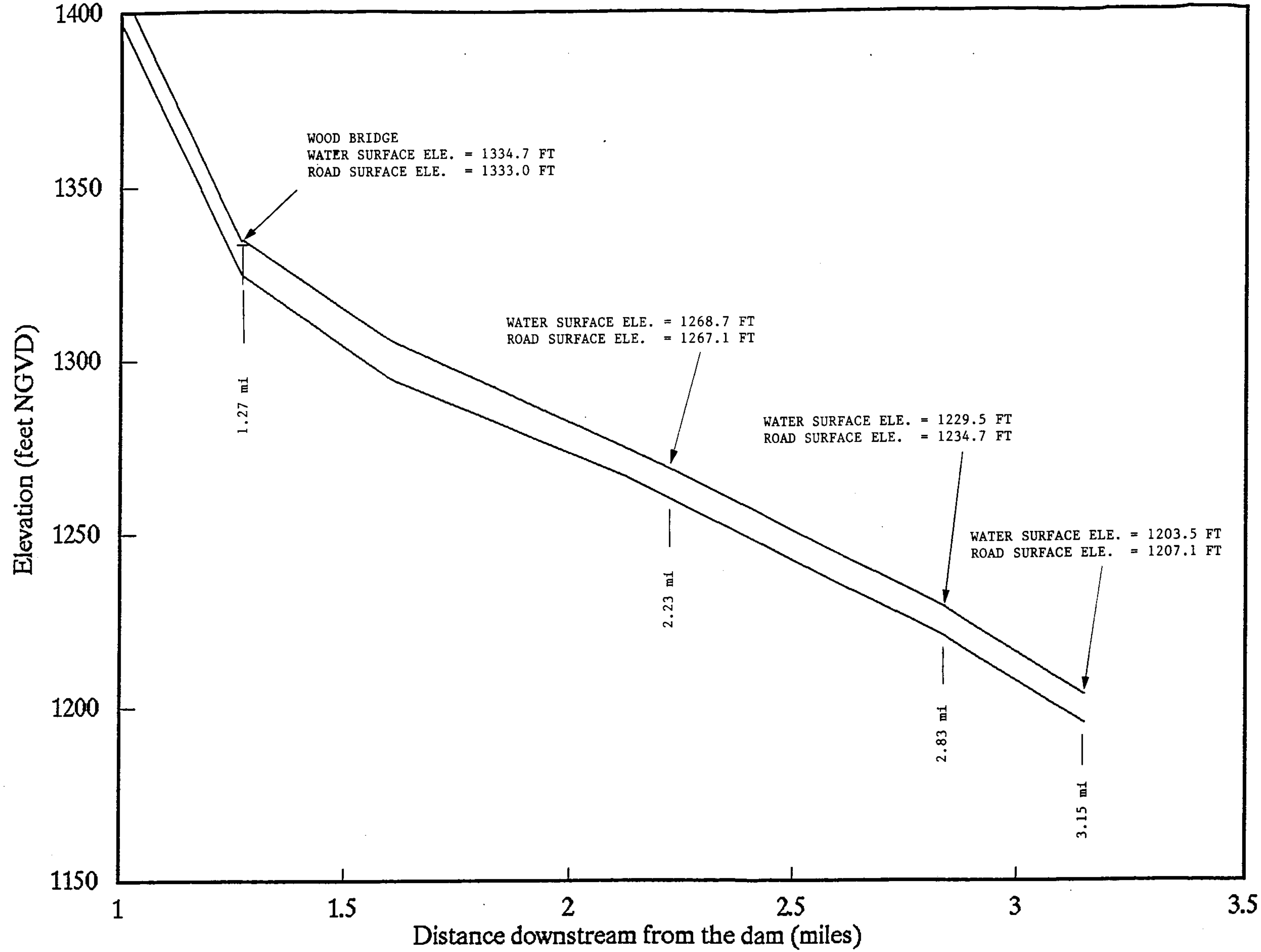


Figure 6. (continued)

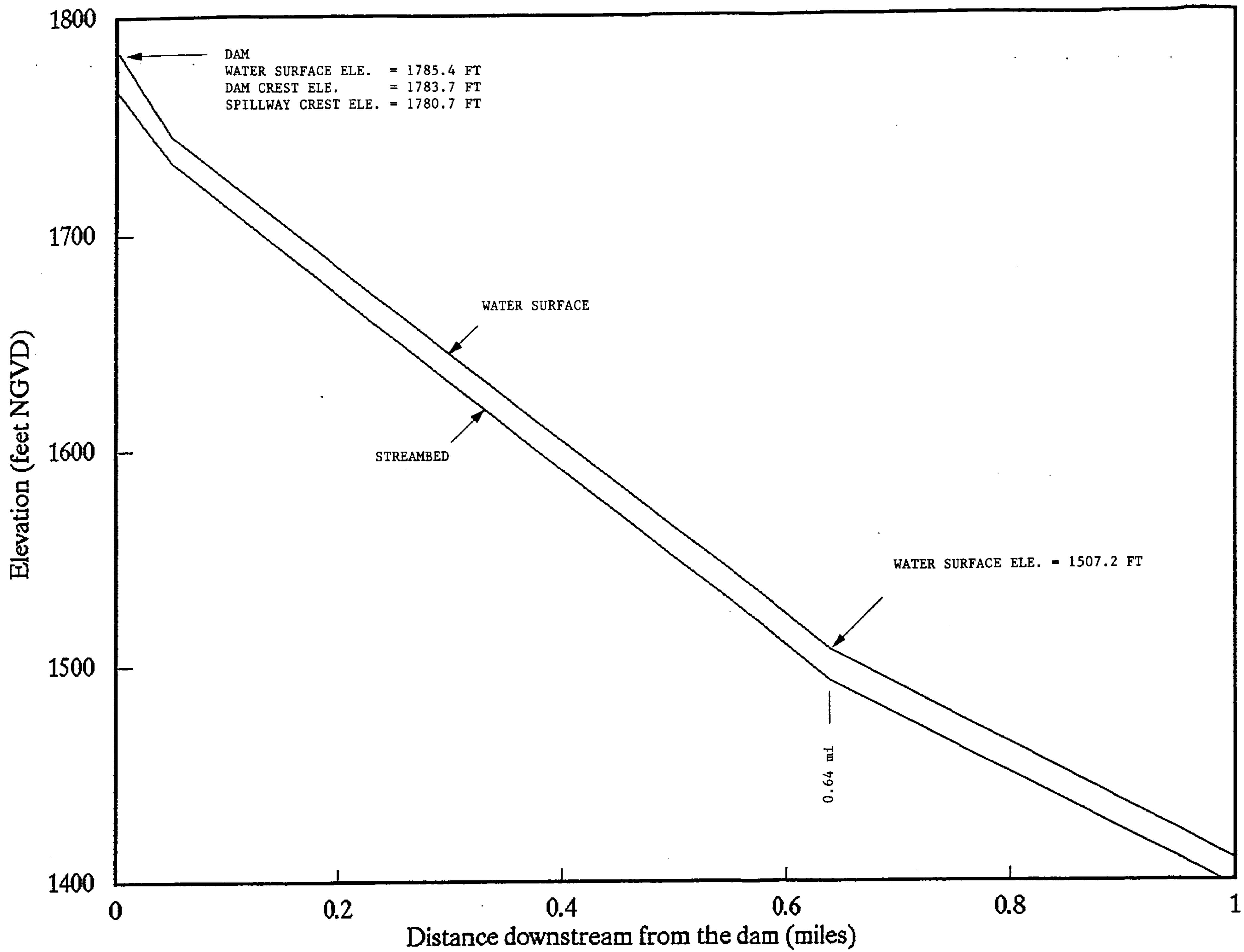


Figure 7. Peak Flood Stages along the Downstream Channel for Storm-Day Failure Scenario.

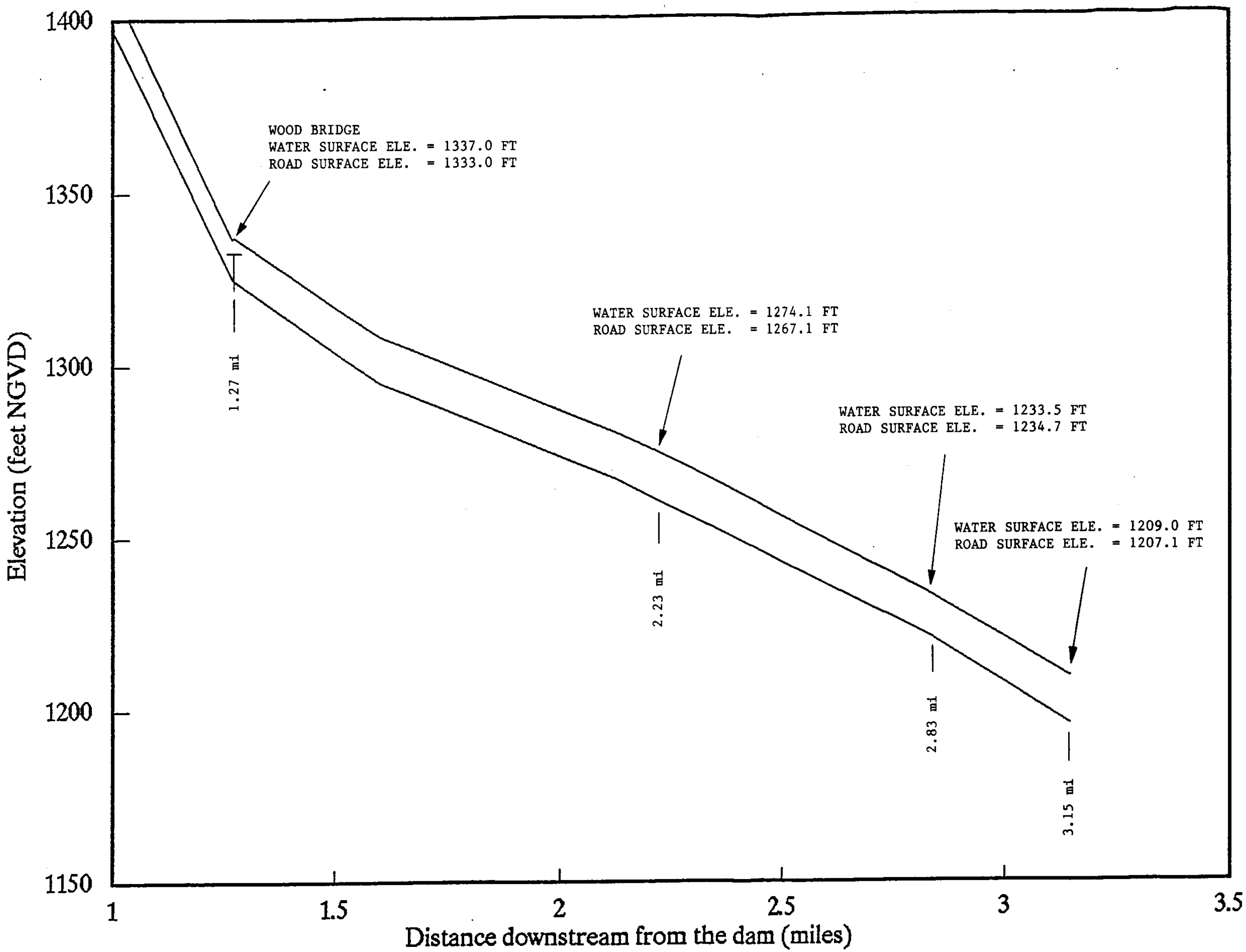


Figure 7. (continued)

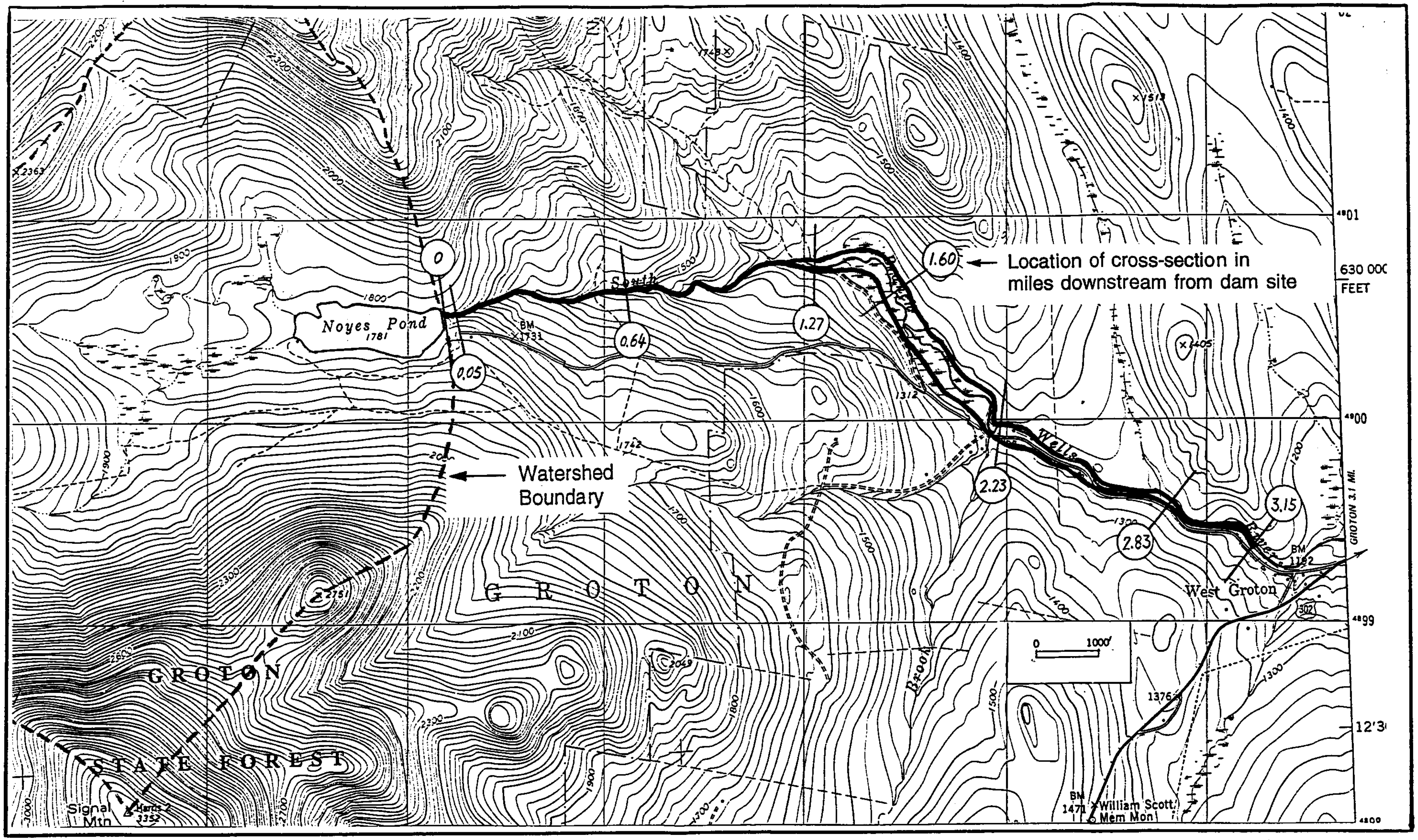


Figure 8. Inundation Map for Sunny-Day Scenario.

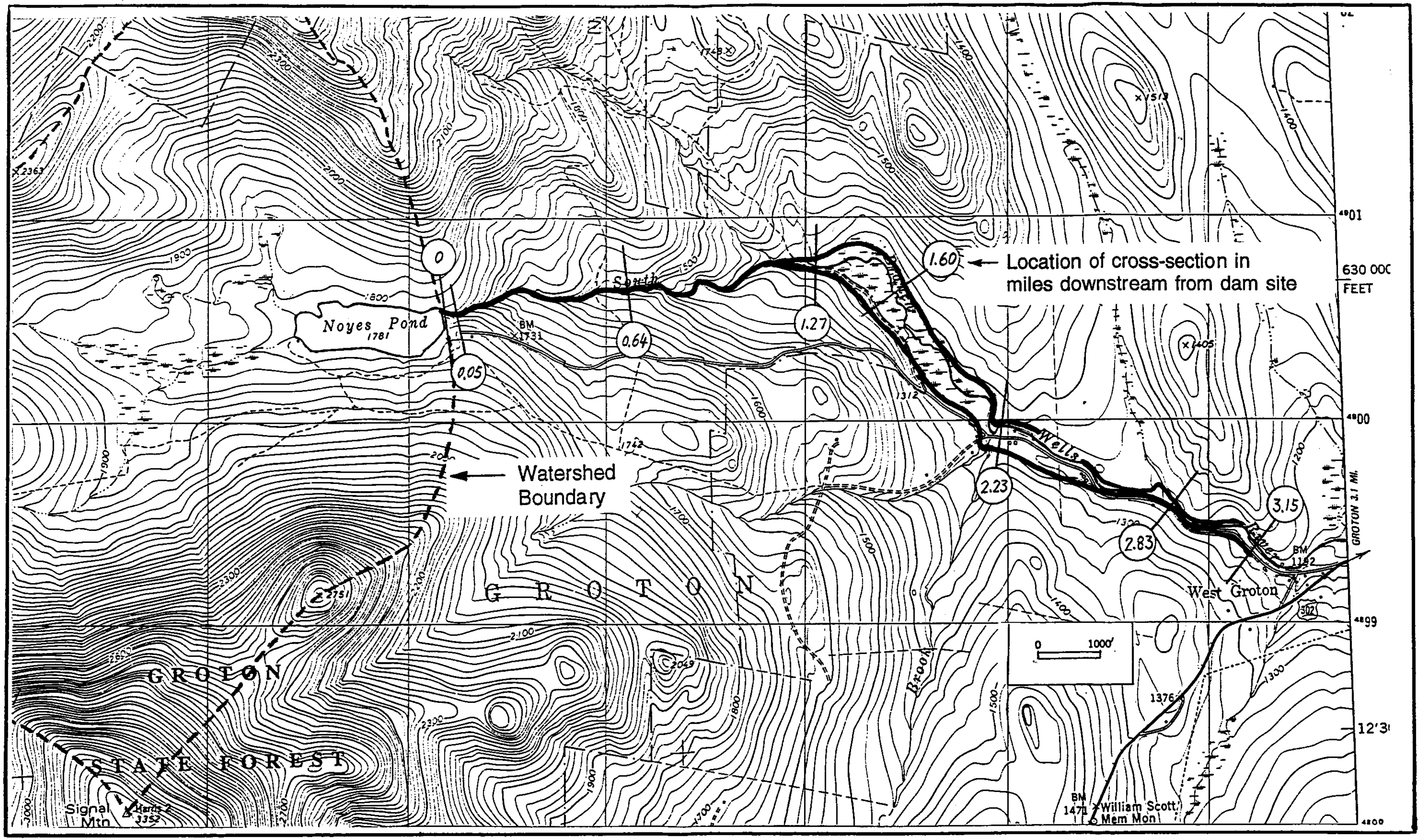


Figure 9. Inundation Map for Storm-Day Scenario.

B. EMERGENCY ACTION PLAN

1. INTRODUCTION

The Emergency Action Plan (EAP) is a suggested procedural outline (Ref. 1) indicating appropriate steps to be taken in the event of an impending failure of the Noyes Pond Dam. Also, this EAP lists phone numbers of certain local and state officials to contact in case of an emergency.

2. ITEMS IN THE EAP

The following are major items which should be addressed by the owner of the dam:

- Monitoring
- Evaluation
- Preventive Action
- Warning

3. MONITORING

a. Purpose

Having a person monitor the dam in the event of an impending dam failure is the first step in implementing the EAP. During periods of heavy precipitation, flooding or any unusual hydrologic events that might cause structural damage to the dam, the owner should have qualified personnel monitor the dam. The owner should assume responsibility for having the monitor at the dam within a reasonable time and for providing an adequate communication system between monitor and local officials.

b. Designated Owner Contact

Name:	Mr. Angelo Incerpi	
	Director of Operations	
	Department of Fish & Wildlife	
Phone:	Home:	(802) 684-3809
	Work:	(802) 241-3700

c. Training

The owner should provide proper training such that the monitor will have sufficient ability to recognize the condition of the dam and be able to identify and evaluate specific problem areas. This training should be extensive enough to allow the monitor to describe conditions to local officials.

d. Communication System

The owner should provide primary and secondary communications systems between the dam monitor and local officials.

Primary System:	Normal telephone communication. The monitor should have access to the nearest available telephone and should have on his person the telephone numbers of all appropriate local officials.
Secondary System:	Shortwave radio: If the phone system is out of service, the monitor should have access to a shortwave radio that can be monitored by local officials with scanners.

As an alternative to this system, if any local officials live within a short distance of the dam, the monitor could drive to one of their residences if the roads are passable.

4. EVALUATION

a. Purpose

In conjunction with the ability to assess the condition of the dam, the monitor should have the ability to determine and evaluate the nature of any existing problem. This evaluation is a crucial step, because failure to accurately and promptly identify problems may adversely affect the EAP warning system.

b. Checklist items

The following is a checklist of items that the monitor should use for assistance in preparing a safety assessment of the dam.

(1) Water Surface Level

Elevation:

a) Normal
b) High (if So, how high, with respect to the top of dam?)

(2) Principal Spillway

Condition upon arrival:

a) Clear
b) Blocked (if so, to what extent?)

(3) Auxiliary Spillways

Condition upon arrival:

a) Clear
b) Blocked (if so, to what extent)

(4) Top of Dam

a) Cover
b) Erosion

(5) Downstream Face

a) Cover
b) Erosion
c) Evidence of piping

5. PREVENTIVE ACTION

The monitor should ensure that the principal and auxiliary spillways are kept clear of debris during normal conditions. In the event of flood conditions, the monitor should also take reasonable steps to ensure that the spillways do not become blocked with debris so that they can carry their full capacity. The monitor's safety should not be jeopardized.

6. WARNING

a. Purpose

If the monitor feels that possible failure of the Noyes Pond Dam is imminent, he should immediately notify the designated parties by utilizing previously established communication systems. The monitor should notify the following officials and the downstream residents. Others can be contacted if determined necessary by the monitor.

b. Notification Chart (As of October 1993)

(1) Jeanne Partington
Town Clerk
Home: (802) 584-3074
Work: (802) 584-3276

(2) Mary Grant
Chairman - Board of Selectmen
Home: (802) 584-3151
Work: (802) 584-3276

(3) Milton Camberton
Constable
Home: (802) 584-3818

(4) Brent Smith
Fire Chief
Town of Groton
Home: (802) 584-3765

(5) Wayne Knott
Civil Defense
Town of Groton
Home: (802) 584-3243

(6) Vermont Emergency Management Agency
24 Hour Duty Officer
1-800-422-8606 or (802) 244-8721

(7) A. Peter Barranco
Department of Environmental Conservation
Owner of Dam
(802) 875-2173

Officials at the Vermont Emergency Management Office can be reached 24 hours a day. During normal business hours, the receptionist at the office will locate the current duty officer. During all other hours the phone connects to the Vermont State Police Department in Guilford, Vermont, which will locate the duty officer. In the event that the phone system has failed, any Vermont State Police barracks or cruiser can reach the duty officer through its radio system. Any available shortwave radio or CB radio could be utilized to contact the nearest police barracks.

c. Downstream Residents

(To be filled out and periodically updated by Dam owner)

Name	Phone Number

C. REFERENCES

1. Recommended Guidelines for Safety Inspection of Dams, U.S. Dept. of Army, Office of the Chief of Engineers, Washington, D.C., Sept. 1979.

2. U.S. Weather Bureau Rainfall Frequency Atlas of the United States, May 1961, Technical Paper 40.

3. Hydrometeorological Report 51, the U.S. Weather Bureau Probable Maximum Precipitation Estimates, June 1978.

4. Hydrometoerological Report 52, the U.S. Weather Bureau Application of Probable Maximum Precipitation Estimates, August 1982.

5. HEC-1, Flood Hydrograph Package, User's Manual, September 1990.

6. DAMBRK, the NWS Dam-Break Flood Forecasting Model, Users Manual, November 1981.

7. Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains. USGS Water-Supply Paper 2339, 1989.