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Docket No. 50-29
LS05-81-02-013

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US NRC DISTRIBUTION SERVICES BRANCH

Mr. James A. Kay
Senior Engineer - Licensing
Yankee Atomic Electric Company
1671 Worcester Street
Framingham, Mass. 01701

Dear Mr. Kay:

SUBJECT: TOPIC II-3.A, HYDROLOGIC DESCRIPTION AND TOPIC II-3.B. (PARTIAL), FLOODING POTENTIAL AND PROTECTION REQUIREMENTS (YANKEE ROWE)

Enclosed is a copy of our draft evaluation of Systematic Evaluation Program Topics II-3.A and II-3.B. You are requested to examine the facts upon which the staff has based its evaluation and respond either by confirming that the facts are correct, or by identifying errors and supplying the corrected information. We encourage you to supply any other material that might affect the staff's evaluation of these topics or be significant in the integrated assessment of your facility.

Your conclusions regarding the subject topics and your seismic evaluation of the dam should be considered together because of possible interrelationships between the subjects.

Your response is requested within 30 days of receipt of this letter. If no response is received within that time, we will assume that you have no comments or corrections.

In future correspondence regarding Systematic Evaluation Program topics, please refer to the topic numbers in your cover letter.

Sincerely,

Dennis M. Crutchfield, Chief
Operating Reactors Branch #5
Division of Licensing

Enclosure: As stated

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UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D. C. 20555

February 09, 1981

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SEP DRAFT SAFETY TOPIC EVALUATION
YANKEE ROWE NUCLEAR POWER STATION

TOPICS II-3.A, HYDROLOGIC DESCRIPTION
TOPIC II-3.B, (PARTIAL), FLOODING POTENTIAL AND PROTECTION REQUIREMENTS

I. Introduction

It must be assured that the designs of safety-related structures, systems and components have considered appropriate hydrologic conditions. Hydrologic considerations include the interface of the plant with the hydrosphere, the identification of hydrologic causal mechanisms that may require special plant design or operating limitations. The scope of these safety topic evaluations is to assure that appropriate hydrologic factors have been considered and to assess any hydrologic considerations which may have changed since being reviewed during the initial licensing of the plant. Should flooding potential exist, the impact of the flood on the plant will be examined. If flooding protection is required, it must be assured that the protection relied upon is available, appropriate, and that provisions have been made to implement the required protection. The protection will be reviewed to assure that safety-related structures, systems and components are protected against floods.

II. Current Review Criteria

The current NRC criteria applicable to these topics are (1) Standard Review Plans 2.4.1 through 2.4.14, 3.4.1 and 9.2.5; (2) Regulatory Guides 1.102, 1.127, 1.27, 1.59 which includes American National Standards Institute Standard N170-1976, and Regulatory Guide 1.70.

III. Related Safety Topics and Interfaces

The Topic identifies water levels and other hydrologic information that may be pertinent to other review areas for assessment of effects on safety-related buildings and equipment. The related interface Topics are: (1) III-3.A Effects of High Water Level on Structures; (2) II-4E Dam Integrity; (3) III-6 Seismic Design Considerations; (4) VII-3 Systems Required for Safe Shutdown; (5) VIII-2 On Site Emergency Power Systems - Diesel Generators; (6) XI-3 Station Service and Cooling Water Systems; and (7) XVI Technical Specifications.

The categories of "In Service Inspection of Water Control Structures" and "Structural and Other Consequences of Failures of Underdrain Systems" also require hydrologic review and input; however, the hydrologic aspects are addressed in Topics III-3.C and III-3.B, respectively.

IV. Review Guidelines

This report includes: a discussion of the potential flood related problems at the plant site as a result of severe precipitation up to and including the severity of the Probable Maximum Precipitation (PMP) on the Deerfield River Basin; a brief description of the hydrologic features of the site and related surrounding area; a description of the analysis procedures used to predict the flood levels at the site; and a discussion of the study results.

IV. Review Guidelines (cont)

As a result of the predicted possible severe flooding of the Yankee Rowe Nuclear Plant Site, this report was expedited and is therefore limited to the discussion of flooding potential for the Deerfield River Basin and Yankee Rowe Nuclear Plant Site. The review of groundwater, local flooding and safety related water supply will be deferred until the more severe problem of potential Deerfield River flood effects has been assessed.

Regulatory Guides 1.59 and 1.102 have been specifically identified by the NRC's Regulatory Requirements Review Committee as needing consideration for backfit on operating reactors. These guides are utilized in determining whether the facility design complies with current criteria or some equivalent alternatives acceptable to the staff.

This evaluation was performed under the auspices of the Systematic Evaluation Program and is prepared as input to the Integrated Assessment Report.

V. Evaluation

1.0 INTER-AGENCY COORDINATION

The Federal Energy Regulatory Commission (FERC) has the licensing responsibility for Hydroelectric Developments. The NRC has previously met with FERC to apprise them of our preliminary findings. Interagency coordination is also required between NRC and the Federal Emergency Management Agency (FEMA) in accordance with criteria set forth in Appendix E, 10 CFR Part 80, NUREG-0654 and the Inter Agency Steering Committee. FEMA has also been apprised of our preliminary findings, and both agencies will be included on distribution for the completed draft flood study and any subsequent correspondence relating to the potential flood problem at the Yankee Rowe site.

2.0 Discussion of Problem

The Yankee Rowe Nuclear Power Plant is located on the east bank of the Sherman Dam and Reservoir, the fourth dam in a chain of hydroelectric dams on the Upper Deerfield River Basin (See Figures 3.1.1 and 3.1.2). These dams were constructed in the 1920s. The first step in the NRC's SEP review is to compare the existing plant to current licensing criteria for new plants. Thus, in this study it is required to determine if, the Harriman Dam, the first upstream dam from the plant, can safely pass a Probable Maximum Flood (PMF) - the current design basis for new nuclear power plants. Since the failure of Harriman Dam could induce damaging flood levels at the plant site, it is also necessary to estimate the magnitude of these flood levels.

This flood study shows that although Harriman Dam can safely pass about 13 inches of basin rainfall, it does not meet current licensing criteria in that 18.9 inches (the design basis Probable Maximum Precipitation based on current criteria) on the Upper Deerfield River Basin will overtop and fail Harriman and Sherman Dams and produce flood levels at the plant site that are 40 or more feet above plant grade.

Tabulated below are several key evaluation areas that have a significant influence on the determination of flood level at the plant site.

1. Magnitude of precipitation in the basin.

2. Distribution of precipitation within the storm (% of rainfall in each 6 hour period).

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3. Magnitude and timing of antecedent storms.

4. Reduction in Harriman Spillway capacity due to debris or reduced hydraulic performance characteristics at heads greater than the design value.

5. The shape and duration of dam breaches due to overtopping.

6. Early failure of Sherman Dam.

The significance of these items will be discussed further in Section 4.0.

It should also be noted that failure of the Sherman Dam, which impounds the plant's normal and emergency water supply, could affect the plant's safe shutdown capability. This subject will be addressed at a later date under Topic II-3.6, "Safety Related Water Supply."

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3.0 HYDROLOGIC DESCRIPTION

3.1 Yankee Rowe Site and Facilities

3.1.1 Site Description

The Yankee Rowe facility is located in the town of Rowe, in Franklin County, Massachusetts, on the east side of the Deerfield River, three-quarters of a mile south of the Vermont-Massachusetts border. Figure 3.1.1 shows the site location on a general area map.

The site consists of approximately 2,000 acres straddling the Deerfield River in the towns of Rowe and Monroe, Massachusetts. The reactor facility is located on the eastern side of the Deerfield River next to Sherman Dam and adjacent to the Sherman Reservoir, which serves as a source of cooling water for the Yankee plant's once-through condenser and service water cooling system.

Most of the land in the immediate vicinity of the site is heavily forested. At the site, which is in a valley, the elevation is about 1130 feet above mean sea level (feet msl). Within a distance of one mile, however, the hills on both sides of the site rise to above elevation 2000 feet msl. This steep-slope character of the Deerfield River extends from Wilmington, Vermont, 12 miles north, to Charlemont, Massachusetts, 8 miles south-southeast.

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3.1.2 Station Description

The Yankee Nuclear Power Station is a single-unit pressurized water reactor nominally rated at 185 MWe gross generating capacity with a rated net capacity of 176 MWe. The containment, a steel sphere elevated 30 feet above the ground, encloses the entire primary system, including the steam generators. The station has operated since July 1960, under Atomic Energy Commission license number DPR-3, issued under section 104(b) of the Atomic Energy Act of 1954 (as amended).

A once-through open cycle system with an average flow of 310 cfs is used for condenser cooling.

There is an 82.6-acre watershed southeast of the plant that drains across the plant site to Sherman Pond. There is a 994-acre watershed located north and east of the main plant area. This watershed is drained by Wheeler Brook which empties into Sherman Pond just north of the main plant area. Flood potential from these two drainage areas and the plant site will be deferred to a later report.

Yard grade in the vicinity of the plant proper is 1127.7 feet msl and 1119.7 feet msl at the screenhouse. Floor slab elevation of the turbine building is 1128.3 feet msl.

Figure 3.1.2 illustrates the general layout of the station.

3.2 Hydrosphere - The Deerfield River Basin

3.2.1 General Description

The Deerfield River, a tributary of the Connecticut River, has a total drainage area of 664 square miles and extends from southern Vermont into the northwestern corner of Massachusetts. The Yankee Rowe power station, situated on the central portion of the Deerfield River Basin (Figure 3.2.1) is only affected by hydrologic events in the 236 square mile drainage area of the upper basin.

The upper Deerfield River Basin, in general, has fairly steep slopes and comprises four sub-basins - Somerset, Searsburg, Harriman, and Sherman - which are delineated in Figure 3.2.1 by the bold dotted lines. The average Deerfield River flow for the 61-year record at Charlemont, Massachusetts, is 887 cubic feet per second(1). Charlemont is approximately 9 miles downstream from Sherman Dam and has a drainage area of 362 square miles. The maximum flow at Charlemont was 56,300 cubic feet per second recorded during the hurricane of 1938.

3.2.2 Description of Upper Deerfield River Reservoir Developments

The Upper Deerfield Project includes the Somerset, Searsburg, Harriman, and Sherman Developments.

The Deerfield River rises in Southern Vermont and flows generally in a south and east direction through a valley that is narrow at the headwaters but broader as it approaches the entrance to the Connecticut River. At the Somerset Reservoir Dam in the upper reaches of the river, the elevation is 2134 feet msl and at its confluence with the Connecticut River its elevation is 120 feet msl, a drop of 2014 feet.

Somerset Reservoir

The drainage area above the dam is approximately 30 square miles. The reservoir extends upstream for approximately 5.6 miles and has a surface area of about 1623 acres at elevation 2133.6 feet msl.

The spillway structure is located at the west end of the earth embankment. It consists of a trapezoidal concrete section divided into eight bays, each 24 feet wide with a crest at elevation 2133.6 feet msl, and two 10-foot wide sections with crest at elevation 2133.6 feet msl provided with stop logs to elevation 2136.6 feet msl. A creosoted timber bridge spans the spillway on concrete piers. New flashboard stanchions were installed in 1964 to carry three feet of flashboards. The spillway discharges into a channel excavated in ledge. The channel is about 800 feet long, 45 feet in average width, and from 6 to 30 feet deep.

The Somerset Dam is of the semi-hydraulic fill type and is constructed on a curved alignment. The entire upstream slope is protected by riprap. The roadway at the crest is of gravel construction.

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River flow leaving Somerset Reservoir flows south through the East Branch of the Deerfield River to the Searsburg Reservoir.

Searsburg Development

The drainage area above the development is approximately 90 square miles. The pond extends upstream for approximately 1 mile and has a surface area of about 28 acres at elevation 1754.7 feet msl.

The spillway consists of a concrete ogee weir 137 feet long with crest at elevation 1749.7 feet msl and provided with pin type flashboards 5 feet in height. A bypass channel used during construction is located at the northerly end of the spillway, and closure in this area consisted of a vertical concrete wall and deck.

The earth embankment at the northerly end of the dam is of the semi-hydraulic fill type, supported by a gravity concrete retaining wall. The roadway at the crest is of gravel construction.

Because of the limited storage capacity (283 acre-feet), the failure or non-failure of this dam will have no appreciable effect on downstream flood flows. Therefore, this dam and reservoir were ignored in subsequent flood analyses.

Figure 3.2.3 and Table 3.2.1 show some pertinent features of the dam and reservoir.

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The flow continues downstream in a south and east direction into Harriman Reservoir.

Harriman Development

The drainage area above the development is approximately 184 square miles. The reservoir extends upstream for approximately 9 miles and has a surface area of about 2050 acres at elevation 1492.7 feet msl.

The 200-foot high earth dam is of the semi-hydraulic fill type and was raised with rolled earth fill from elevation 1511.7 feet msl to elevation 1521.2 feet msl in 1964 to increase flood retention capability. The upstream slope has riprap protection to elevation 1480.7 feet msl for most of the length of the embankment, varying to elevation 1495.7 feet msl at the northerly end. The roadway at the crest is of gravel covered with a seal coat of asphalt. The downstream slope has a grass cover.

The spillway is of the morning-glory type and is located upstream of the southerly end of the dam. The spillway discharges through a 22.5-foot minimum diameter vertical shaft and 90 degree bend into the concrete bypass conduit that was used for diversion during construction. The bypass conduit is now plugged with concrete upstream of the vertical shaft. This conduit has a cross-sectional area equivalent to a 22.5-foot diameter circle. The spillway has a crest elevation of 1491.7 feet msl and 16 equally spaced piers that can accommodate 7 feet of flashboards. The spillway shaft and crest were resurfaced in 1954.

Figures 3.2.4 and 3.2.5 and Table 3.2.1 show some pertinent features of the dam and reservoir.

Flow leaving the Harriman Dam continues downstream in a southerly direction into Sherman Reservoir.

Sherman Development

The drainage area above the development is approximately 236 square miles. The pond extends upstream approximately 2.2 miles and has a surface area of about 194 acres at elevation 1103.7 feet msl.

The dam is of the semi-hydraulic fill type and was raised 10 feet to elevation 1129.7 feet msl with rolled earth fill in 1964 in order to increase spillway capacity. The embankment is supported at its northerly end by a concrete retaining wall which also was raised in 1964. The upstream slope has riprap protection to elevation 1095.7 feet msl for the full length of the embankment. The downstream slope has grass cover over its entire area.

The spillway structure is located at the north end of the dam. It consists of a gravity concrete ogee weir section with a crest elevation of 1103.7 feet msl and is provided with pin type flashboards 4 feet in height. There is a spillway channel, excavated in ledge, about 360 feet long and 50 feet wide spanned by a plate girder bridge. The spillway channel was deepened in 1964 by the removal of 4600 cubic yards of material to increase discharge capability by reducing backwater effect. An eroding area downstream of

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the spillway bridge on the west side of the channel was graded and riprapped at the same time. A concrete bypass conduit, used during construction, runs through the earth dam. This has a cross-sectional area of 140 square feet and has been plugged at the upstream end with concrete.

Figures 3.2.6, 3.2.7, and 3.2.8 and Table 3.2.1 show some pertinent features of the dam and reservoir.

3.3 Floods

3.3.1 Flood History

The flood history of the Deerfield River during this century is readily available from records which extend back to 1911 when construction began on the hydroelectric facilities. These records and those from the U.S. Geological Survey gaging station at Charlemont clearly document major storm and discharge events which are summarized in Table 3.3.1.

As shown, the most significant event was during the "1938 hurricane," closely followed by the 1948-49 "New Year's Eve" storm, and the 1927 and 1936 events. Although undocumented, a local reference[2] states that the flood of October 1869 was similar in severity to the 1927 event. The rainfall which accompanies tropical storms (including hurricanes) often produces major floods during the summer and early fall. Extratropical storms and/or snow melt produce principal floods during the winter and spring months.

Maximum average depths of rainfall for selected historic storms of record for the region are shown in Table 3.3.2. These actual storm values are presented as an indication of what has occurred in the region historically. Also shown is the controlling storm for the northeast United States (OR 9-23), commonly known as the Smethport, Pa. storm. This storm occurred on the west side of the Appalachian Mountains and is generally not transposed across the mountains.

4.0 Analysis Procedure

4.1 General

In order to determine the Probable Maximum Flood (PMF) elevation at the Yankee Rowe Plant site, the Probable Maximum Precipitation (PMP) is applied to individual subbasins using a unit hydrograph to define the runoff characteristics of the subbasins. The hydrographs thus obtained are routed through the stream channels and reservoirs to account for attenuation due to channel and valley storage. Where dams are overtopped the erosional failures and resulting outflow hydrographs are simulated by synthetic methods.

The PMF is used by many federal, state and local agencies and architectural engineering firms to predict upper limit flood levels for planning and design purposes. The PMF is defined as "The hypothetical flood (peak discharge, volume and hydrograph shape) that is considered to be the most severe reasonably possible, based on comprehensive hydrometeorological application of probable maximum precipitation and other hydrologic factors

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favorable for maximum flood runoff such as sequential storms and snowmelt."(3) The PMP is defined as "The estimated depth for a given duration, drainage area, and time of year for which there is virtually no risk of exceedance. The PMP for a given duration and drainage area approaches and approximates the maximum which is physically possible within the limits of comtemporary hydrometeorological knowledge and techniques."(3)

On a complex river basin such as the Deerfield where multiple water levels and sensitivity analyses are required, a computer model is used. In this case, the HEC-1 Flood Hydrograph Package for Dam Safety Investigations(4) was used.

The basic HEC-1 Flood Hydrograph Package was developed by the U.S. Army Corps of Engineers' Hydrologic Engineering Center for modeling basin stream networks. This computer program is used by many federal, state and local agencies as well as architectural and consultant engineering firms. The dam safety investigation program is a modification to the basic program that allows the estimation of the overtopping potential of a dam and the downstream hydrologic-hydraulic consequences resulting from assumed structural failures of a dam.

4.2 Rainfall and Runoff

4.2.1 Probable Maximum Precipitation

The staff considered several sources for the PMP for this study. Hydrometeorological Report Number 33 (H.R. #33), April 1956(5), is the source most commonly used for PMP estimates east of the 105th meridian. This report was revised and expanded and released as Hydrometeorological Report Number 51 (H.R. #51)(6) in June 1978. H.R. #51 predicts slightly larger rainfall values for the study area than H.R. #33. Since H.R. #51 has not been fully reviewed by some major federal agencies such as NRC, we have not used precipitation estimates from H.R. #51 for the study.

The licensee, Yankee Atomic Power Company, has recently completed a Draft Probable Maximum Flood Analysis for the Yankee Rowe Nuclear Generating Station.(7) This report attempts to derive a PMP for the Upper Deerfield River Basin by transposing and maximizing several maximum regional storms to the Deerfield Basin. The NRC staff has not accepted this analysis due to the lack of supporting data and apparent erroneous transposed storm rainfall values. The staff assertion of erroneous results is based in part on the staff's independent analysis of the Westfield storm which resulted in rainfall estimates considerably larger than the licensee's estimates.

NRC regulations do not specifically require a PMP for the Design Basis Flood for nuclear power plants. Title 10, Part 50, Appendix A of the Code of Federal Regulations states, "The design bases for these structures, systems and components shall reflect: (1) appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and sourrounding area, with sufficient margin for the limited accuracy, quantity, and period of time in which the historic data have been accumulated." In order to be assured that the licensee

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is not being unduely penalized by the use of Generalized PMP values, the staff made an independent analysis of the August 1955 storm that was centered at Westfield, Mass. (about 50 miles south of the Rowe site). The maximization and transposition of this storm to the upper Deerfield River Basin provides a rainfall estimate that can be considered as a lower limit value for a design basis flood.

The storm was maximized and transposed to the Deerfield Basin using procedures suggested in H.R. #51(6) and the manual for Estimation of Probable Maximum Precipitation(15). The transposed storm had an adjustment factor of 1.17 which includes factors of 110% for moisture maximization, 89% for transposition and elevation and 120% for orographic effects. The 200 square mile, 24-hour adjusted rainfall for the storm is 16.6 inches. Depth area-duration curves for the transposed Westfield Storm are shown on Figure 4.2.2. An idealized isohyetal storm pattern, with this rainfall, was centered on the upper Deerfield River Basin and planimetered to obtain an average 236 square mile basin rainfall of 16.5 inches for 24 hours. The resulting flood runoff would overtop Harriman dam and produce a flood level at the Yankee Rowe site of 1172.4 feet msl. It is noted that this lower limit rainfall value by itself does not qualify as a PMP. It follows that a comprehensive regional PMP study would predict at least 16.6 inches for the 200 square mile, 24-hour value and would probably predict a somewhat larger value but less than the 18.9 inches from H.R. #33. Since this lower limit rainfall does not alter the conclusions of this study, it was concluded that the rainfall from H.R. #33 would be used as the design basis rainfall for this study.

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The depth area-duration curves from H.R. #33 for the upper Deerfield River Basin are shown in Figure 4.2.1.

The distribution of rainfall in the worst 6-hour period has a significant influence on the potential flood levels at the Yankee Rowe site. H.R. #33 suggests that 72.8% of the 236 square mile, 24-hour rainfall occurs in the critical 6-hour period. Referring to Table 3.3.2, the maximum 6-hour values for these historic storms range from 43 to 66 percent of the 200 square mile, 24-hour value. This percentage ranges up to 90% for northeast U.S. storms shown in reference (6). For a sensitivity test, the model was run with 43% of the 24-hour PMP in the maximum 6-hour period. The results of this run show that Harriman Dam would still be overtopped and the level at the Yankee Rowe site would increase by 8.0 feet. The reason for the higher stage at the site is because the higher percentages of rainfall in the critical 6-hour period causes Sherman Dam to overtop and fail about 4 hours before the peak outflow from Harriman Dam reaches the Yankee Rowe site. Whereas with the 43% distribution, Sherman Dam does not fail until the peak outflow from Harriman Dam reaches the site. This higher initial Sherman reservoir level induces a higher peak flood stage at the Yankee Rowe site. The rainfall distribution suggested in H.R. #33 was selected for use in this flood study.

4.2.2 Rainfall Losses

Rainfall loss rates can be derived from historic storm and flood records. The loss rates thus computed would generally be directly applicable to

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maximized storms such as the PMF. The licensee derived loss rates in Reference (7) from two historic Deerfield River storms. The computed rates were 0.03 and 0.06 inches per hour. Other verification evaluations[7] indicated about 0.1 inch per hour. We discussed rainfall losses with personnel from the New England Division of the U.S. Army Corps of Engineers who have studied many storms in the region. They recommend a loss rate of 0.1 inch per hour and no initial loss for a PMF study. Based on our own experience and the information supplied by the licensee and the Corp of Engineers, we selected a loss rate of 0.1 inch per hour and no initial loss. The licensee used an initial loss of 0.5 inch and 0.1 inch per hour for his PMF study.[7] Since the PMF by definition optimizes and maximizes parameters, the use of no initial loss is justifiable and reasonable. Additionally, the losses are a small part of the total rainfall and do not have any significant effect on subsequent flood levels.

4.2.3 Unit Hydrograph Coefficients

The Unit Hydrograph is "the hydrograph of surface runoff (not including groundwater runoff) on a given basin, due to an effective rain falling for a unit of time. The term 'effective rain' means rain producing surface runoff. The unit of time may be one day or preferably a fraction of a day. It must be less than the time of concentration."[8] Unit hydrographs can be derived from actual storms or by synthetic methods using empirical equations. For this study, both methods were used to derive unit hydrograph coefficients. Several synthetic methods from the literature were used to derive coefficients for each of the four subbasins discussed in Section 3.2.2. The methods used were: (1) Snyders Equation (EM 1110.2-1405)[9], (2) Design of small Dams[10], (3) Linsley, Kohler and Paulhus[11], and (4) Standard Project Flood Criteria, Southern California[12]. Another set of coefficients were derived from a historical hydrograph presented in the Licensee's studies[13]. Unit Hydrograph coefficients were also developed from information obtained in discussions with personnel of the New England Division of the U.S. Army Corps of Engineers who have done many similar studies in the region. Another set of coefficients were selected based on personal observation and experience.

Ultimately, a set of Clark Unit Hydrograph Coefficients were selected for each subbasin based on professional judgment, personal experience, and with due consideration of the values obtained from the above methods. Prior to the completion of this study, the licensee also furnished Snyder Unit Hydrograph Coefficients for each subbasin. The Snyder values were derived from actual storms and verified in other storm reconstitutions. The following table shows a comparison of Snyder and Clark coefficients for the NRC and licensee values:

UNIT HYDROGRAPH COEFFICIENTS
DEERFIELD RIVER BASIN
COMPARISON OF NRC AND YANKEE ATOMIC VALUES

Drainage Subbasin	Snyder Coefficients[1/]				Clark Coefficients[1/]			
	NRC		Yankee Atomic		NRC		Yankee Atomic	
	TP	CP	TP	CP	Tc	R	Tc	R
Somerset	2.42	.56	2.68	.81	2.7	2.7	10.37	3.65
Searsburg	3.09	.57	2.98	.81	3.4	3.4	11.50	4.09
Harriman	4.09	.58	4.16	.81	4.4	4.4	16.79	5.14
Sherman	3.17	.57	3.23	.81	3.5	3.5	12.86	4.22

1/ Conversions from Snyder to Clark and visa versa were done with the HEC-1 program (4)

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The somewhat large Yankee Atomic Clark coefficients reflect the slower times of concentration that the licensee obtained in the case studies. The runoff hydrographs developed from the Yankee Atomic coefficients will be broader and flatter (lower peak discharge) than the hydrographs developed with the NRC coefficients.

Table 4.1 shows a comparison of output from sensitivity studies run with the HEC-1 model. This comparison shows that the choice of unit hydrograph coefficients has little effect on resulting flood levels. However, in the interest of conservatism, the staff's final results and conclusions are based on the model runs using Snyder coefficients furnished by the licensee. Additionally, unit hydrograph coefficients derived and verified with actual flood events are generally more acceptable to the technical community.

The HEC-1 model is used to develop runoff hydrographs for each subbasin from the unit hydrographs, the rainfall, and rainfall losses. These subbasin runoff hydrographs are then in turn routed through the channels and reservoirs in the river basin.

4.2.4 Maximum Regional Rainfall

It is common practice, when analyzing potential flood problems, to determine the maximum historic rainfall for the region.

Table 3.3.2 shows the maximum recorded rainfall values for storms that have occurred in the region. The Westfield, mass storm (No. NA 2-22A) is the largest storm in the region by a considerable margin.

This storm was transposed to the Deerfield River Basin for the purpose of determining the potential effect on Harriman Dam and the Yankee Rowe Nuclear Plant. The transposed storm would have a 24 hour average basin (236 square miles) rainfall of about 13 inches.

4.3 Flood Routing

The channel and reservoir routing with the HEC-1 model uses the modified Puls method. This method of routing accounts for hydrograph attenuation due to channel, valley, and reservoir storage, but it does not account for the time required to convey water from one subbasin to the next. Above Harriman Dam this is accounted for in the subbasin hydrographs;

between Harriman and Sherman Dams it is not. We estimate this time to be about 1.5 hours between Harriman and Sherman Dams. Thus, in terms of predicted flood levels at the Yankee Rowe site the peak inflow hydrograph to Sherman Reservoir could lag the predicted arrival time by less than 1.5 hours. In most cases analyzed, Sherman Dam is predicted to fail prior to the arrival of the predicted failure hydrograph from Harriman Dam. Therefore, any delay in the arrival of the Harriman peak flow could result in somewhat lower flood levels at the nuclear plant site. However, any failure hydrograph from Harriman Reservoir is sufficient to cause significant flood levels at the site, regardless of when Sherman Dam fails.

4.3.1 Reservoirs Routing

Reservoir routing was done by the Modified Puls method. Reservoir storage curves were provided by the licensee. The storage curves for Somerset, Harriman, and Sherman Reservoirs are presented in Figures 4.3.1, 4.3.2, and 4.3.3, respectively. The curves have been conservatively extrapolated by the licensee above the top of dam levels using an incremental storage procedure.

Reservoir outflows were of three types: (1) discharge through the normal reservoir spillways or outlet works, (2) discharge over non-eroded portions of the dams, and (3) discharge through the eroded or breached dam sections. The spillway rating curves for Somerset, Harriman, and Sherman Reservoirs were furnished by the licensee and are shown in Figures 4.3.4, 4.3.5 and 4.3.6, respectively.

The staff has some reservations with respect to the spillway for Harriman Reservoir. This spillway is the "Morning Glory" type which is designed to operate with a small change in discharge for a large range in heads. The spillway is located in the corner of the reservoir and very close to the dam and southern bank of the reservoir. This location would be conducive to debris accumulation and potential blockage, especially for rare floods that will carry many large trees and other debris downstream. These spillways are also noted for undesirable discharge characteristics at reservoir levels above the design value. The Harriman spillway was designed for a reservoir elevation of 1498.6 feet msl. The dam has been raised subsequently, and predicted levels could be as high as elevation 1525 feet msl. When these type spillways are subjected to heads above the design value, there is possibility of shifting control between weir, orifice and full pipe (pressure flow) with the associated uncertain discharge capability, slug flow, vortices, cavitation, and vibration. There is another uncertainty with respect to the rather narrow discharge channel immediately downstream of the tunnel outlet. The unknown is whether the narrow channel could submerge the outlet at higher flows, thus forcing a hydraulic jump in the conduit and thus reducing capacity.

During past meetings with the licensee he has demonstrated a willingness to construct booms or other considerations to preclude debris from the spillway if the additional capacity would ensure non-overtopping of Harriman Dam. The spillway was model tested in 1925 prior to construction. The model test does mention some runs at higher heads and the associated vortex formation and some fluctuation in discharge. It is doubtful that the friction in the riser and barrel were properly modeled and even if they were it is very difficult to predict, with a model, the flow characteristics for this type of spillway, for greater than design conditions. In light of the many uncertainties with this spillway, we limited the assumed discharge to the capacity of the riser throat as an orifice. This is about a 25% reduction in capacity at the higher heads.

The spillway for Sherman Dam has a rather narrow discharge channel downstream of the crest. Downwater computations for this spillway indicate the possibility of hydraulic jumps or turbulent flow conditions at very high discharges. Depending on the resolutions of other more serious problems, this issue may be investigated more thoroughly at another time.

Discharge over the non-eroded dam sections is computed by the HEC-1 program using the standard weir equation.

The breach hydrograph is computed by a weir flow equation appropriate for the shape of breach selected.

These various possible outflows are summed by the program and used in the routing procedure.

4.3.2 Channel Routing

The only channel routing required in the model was from Harriman Dam to the Sherman Reservoir. The geometric elements for the routing cross section were obtained from a 1:62,000 topographic map. The cross section was used for a one step Modified Puls routing to attenuate the hydrograph for channel and valley storage between Harriman Dam and Sherman Reservoir.

4.3.3 Antecedant Floods

Our current criteria requires that when analyzing potential single or multiple dam failures during a PMF, that the PMF be preceded 3 to 5 days by a flood equivalent to 40% of the PMF. The purpose of this requirement is to allow for the estimation of the loss of reservoir flood storage capacity by antecedant floods. For this study the antecedant flood was routed separately. This routing indicated that all reservoirs would be at the spillway crest elevation at the start of the PMF. Therefore, all subsequent runs were started with the reservoirs at the spillway crest elevation, except Searsburg, which was not considered as a reservoir as discussed in Section 3.2.2.

4.4 Erosional Dam Failures

As discussed in Section 4.1, the Modified HEC-1 program has provisions to simulate erosional type dam failures. The program allows for user discretion in selecting the shape and duration of the breach. Two breach shapes (trapezoidal and triangular) and three durations (1, 2 and 3 hours) were modeled for both Harriman and Sherman Dams in order to determine which would be critical in terms of water level at the Yankee Rowe site. The side slopes of both triangular and trapezoidal breaches were assumed to be the angle of repose ($\emptyset$) of the embankment material which was assumed to be 35°. The bottom width for trapezoidal sections was assumed to be the width of the natural valley at the toe of the dam, and the lower limit of the eroded sections was assumed to be limited to the elevation of the natural valley at the dam site.

The results of these analyses showed that the trapezoidal shape for a duration of one hour would be the critical assumptions for both dams. Since the duration of the Harriman Dam breach has a significant influence on the depth of flooding at the Yankee Rowe site, the TVA Breach Model(18) was used as a method of quantifying the duration of breach.

The Harriman inflow hydrograph for the TVA model was obtained by routing the PMF with the HEC-1 model and assuming infinite dam heights. Other inputs to the model were a $\emptyset$ angle of 15 degrees and a breach width of 400 feet. The section was eroded to elevation 1320 feet msl which is the approximate natural valley floor elevation. The model results showed a time of about one hour to breach the section.

Unfortunately, there is only a limited amount of information available on methodology for simulating erosional failures of earthen embankments. To the best of our knowledge, the TVA model is the only one available that attempts to predict a rate of failure. There is some historic information available, but at best this only supports the unpredictability of this type of dam failure. Photographic documentation of the recent Teton Dam failure indicates that the rapid failure of a large portion of the embankment section occurred in about 30 minutes. Although this was not an

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overtopping breach, the rapid failure of a major portion of the embankment gives a good indication of the erosional potential of reservoir storage.

4.5 Recurrence Intervals of Natural Phenomena

Recurrence intervals or probabilities of natural phenomena (rainfall or flood events) are often used as a decision-making tool. The staff is reluctant to attempt to associate frequencies with rare natural phenomena due to the large confidence intervals and the tendency to place more reliance on the probability estimates than is justified by the basic data. However, there is some frequency information available for rainfall in the region, and it is included in the following paragraphs.

Technical Paper #40[16], which does not include the last 24 years of records nor the effects of Hurricane Diane, shows a regional 24 hour-100 year point rainfall for the Deerfield River Basin of about 6 inches. The Westfield gage (about 50 miles south of the Rowe site) recorded rainfall during Hurricane Diane; the 24 hour-100 year point rainfall for this gage is 12.4 inches. The Springfield gage, which is about 10 miles east of Westfield, has a 24 hour-100 year point rainfall of 8.5 inches. The licensee has provided some rainfall frequency information in Appendix A of Reference 7. They show a 24 hour-100 year point rainfall for Harriman [illegible]on of about 5.6 inches. The other gages they selected all have [illegible]ur-100 year point rainfalls of between 4 and 6 inches. However, [illegible] did not select any sites in the region that have recorded severe [illegible]ical rainfall events, such as Westfield, Mass., Kinsman Notch, [illegible] or Springfield, Mass.

Based on consideration of the above 100-year rainfalls, it is the staff's judgment that the regional 24 hour-100 year point rainfall for the Upper Deerfield River Basin would be about 7 to 8 inches. NOAA Technical Report NWS 24[17] provides a generalized curve for converting point rainfalls to area rainfalls. The 8-inch point rainfall is equivalent to 7.4 inches on the 236 square mile basin. This then can be compared to the 13-inch basin rainfall that Harriman Dam can safely pass. The only guide we can suggest for putting the 13-inch rainfall into a frequency perspective is that the 13-inch rainfall is approximately a regional record value, and based on past experience of other record storms and their extrapolated frequencies, which introduce significant uncertainty, the recurrence intervals are generally in the 500 to 1000 year range.

It should also be noted that this probability information deals only with the rainfall event and makes no allowance for any conservatism in our methods of determining the PMF, such as locating the storms critically over a specific watershed.

The percent chance of a rainfall value being exceeded in the next 20 years can be determined mathematically. There is an 18% chance of the 100-year (7.4 inch) rainfall being exceeded in the next 20 years. For the 500-year and 1000-year recurrence interval rainfall (13 inches), there is a 4% and 2% chance, respectively, of being exceeded in the next 20 years. Again, it is noted that there is much uncertainty involved in trying to associate frequencies with rare natural phenomena. The above approximate analysis is only intended to provide "ball park" estimates of the likelihood of exceeding the existing capacity of Harriman Dam in the next 20 years.

5.0 Results

This flood study has analyzed floods for a range of rainfalls from 13.0 to 21.3 inches. The resulting flood levels at the Yankee Rowe Nuclear plant site and other outputs are shown in Table 4.1. The 18.9 inches is the 24-hour, 236 square mile PMP from H.R. #33. It is the staff's judgment that, under current criteria, the PMF based on this rainfall or rainfall derived from a detailed regional study as discussed in Section 4.2.1 is the flood that the Yankee Rowe plant should be protected against.

The flood resulting from 13 inches of rainfall on the basin is approximately what Harriman Dam can contain without overtopping. However, the reservoir level for 13 inches of rain would be at the top of the dam and does not include any allowances for coincident wind generated waves which would be about 3 feet high for a 30 to 40 mph wind and the runup would be about 8 feet above the pond level. Additionally, the upper 25 feet of the embankment does not have riprap for erosion protection.

The PMP (18.9 inches) used for the PMF would have a point rainfall of 25.6 inches which compares to a maximum 24-hour point rainfall on the Harriman subbasin of 5.5 inches (Table 3.3.1) that produced the record reservoir level of 1498.1 ft msl. Regionally, a 24-hour, 200 square mile value of 14.2 inches (Table 3.3.2) was recorded at Westfield, Massachusetts, just 48 miles south of the Harriman Dam. This storm when maximized and transposed to the Upper Deerfield River Basin would yield about 16.6 inches of rainfall in 24 hours for a 200 square mile area.

Hydrometeorological Report Number 33 is recognized by major water resources engineering groups throughout the country as the source for Probable Maximum Precipitation for the design of large dams whose failure could result in loss of life and major property damage. In lieu of rainfall values based on a comprehensive regional study, the generalized estimate from HR #33 is the value that should be used for the Probable Maximum Flood for the Yankee River site.

Table 5.1 compares pertinent values of other Deerfield River flood studies to the NRC Flood Study. The values for the C. T. Main (1977) and Yankee Atomic (1977) studies were taken from a report prepared by the Yankee Atomic Electric Company, YAEC-1139[14]. This report was prepared for the Federal Power Commission (FPC) (currently known as the Federal Energy Regulatory Commission - FERC) as part of their licensing requirements.

The C. T. Main values are similar to the NRC PMF values, except for the breach assumptions for the dams which have a significant influence on the resultant predicted maximum reservoir levels. Since the C. T. Main breach assumptions were not provided, we cannot discuss the comparison. The Yankee Atomic (FPC Report) values are considerably lower than ours, and we attribute the difference to the unit hydrographs, rainfall loss rates, and starting pool levels. In all cases, we consider the Yankee Atomic values to be nonconservative.

The values for the 1980 Yankee Atomic flood study were taken from reference (7). The significant difference between these values and the NRC PMF values is the PMP rainfall (see Section 4.2.1). Minor differences are in the Harriman spillway capacity (100%-licensee vs approximately 75%-NRC) (See Section 4.2.1).

The values for the U.S. Army Corps of Engineers (COE) 1963 study were taken from a letter report that is attached as Appendix A. The study resulted from interagency coordination on licensing requirements in 1962 and 1963. The COE study was for a Spillway Design Flood (SDF) for Somerset, Harriman, and Sherman Dams. The SDF is equivalent to the PMF except that the rainfall would be for the drainage area controlled by each dam. Their study also assumed infinite dam heights. The COE values indicate that the results are very close to the NRC PMF study.

In addition to rainfall-induced floods, the site may also be exposed to flood waves from a Harriman Dam failure induced by other causes such as seismic, piping , etc. For these type failures the staff generally assumes instantaneous removal of the dam section and modeling of the flood wave by unsteady flow techniques. Since only 100-foot topographic mapping is available for a portion of the reach and accurate cross sections cannot be prepared, the use of the unsteady flow model is not warranted. Therefore, the staff used approximate methods to estimate a water level of 1148 feet msl at the Yankee Rowe site assuming instantaneous failure of the Harriman Dam with the pool at elevation 1493.0 feet msl and 50% attenuation due to channel and valley storage.

The 13 inches of rainfall that Harriman Dam can contain without overtopping is about 70 percent of the 18.9 inch PMP used by NRC as the design bases PMP under current criteria. It is about 78 percent of the lower limit value of 16.6 inches. The 5.5-inch maximum 24-hour point rainfall at Harriman Station is about 22 percent of the 25.6-inch, 24-hour point rainfall from H.R. #33.

The significance of the difference in assumed Harriman Spillway capacities (100% vs 75%) can be quantified by converting the difference in discharge to equivalent reservoir storage. The difference between 100% and 75% spillway discharge at the top of dam elevation is about 10,000 cfs. This flow for about 6 hours is equivalent to about 2-1/2 feet of reservoir storage. The 13-inch rainfall above Harriman Dam would maintain the reservoir within 2 feet of the top of the dam for about 6 hours.

VI. Conclusions and Recommendations

1.0 Hydrologic Conclusions

Based on the results of the NRC Flood Study, it is the staff's judgment that the Harriman Dam can safely pass the runoff from a storm with an average upper basin rainfall of about 13 inches. It is further concluded that this storm would result in a maximum flood elevation at the Yankee Rowe site of no more than 1132.0 feet msl. This rainfall is about 70 percent of the design basis rainfall under current NRC licensing criteria for new plants.

The staff also concludes that if the upper Deerfield River Basin is subjected to a storm with average 24-hour basin rainfall of much more than 13 inches, that Harriman Dam will be overtopped and will fail. The failure of Harriman Dam for any reason when the pool is above elevation 1490 feet msl will produce a flood level at the Yankee Rowe site that is anywhere from 15 to 70 feet above plant grade.

Current practice by designers and constructors of major dams, especially where there is a potential for loss of life or major property damage, is to provide sufficient storage and spillway capacity to safely pass a Probable Maximum Flood or its equivalent. Harriman Dam is a large dam, and there is potential for loss of life and major property damage in the event of a failure of the dam. If the dam were being constructed today, it would probably be designed to safely pass the PMF.

The Federal Guidelines for Dam Safety[19] state:

> "C. Flood Selection for Design (or Evaluation) - The selection of the design flood should be based on an evaluation of the relative risks and consequences of flooding, under both present and future conditions. Higher risks may have to be accepted for some existing structures because of irreconcilable conditions.
>
> When flooding could cause significant hazards to life or major property damage, the flood selected for design should have virtually no chance of being exceeded. If lesser hazards are involved, a smaller flood may be selected for design. However, all dams should be designed to withstand a relatively large flood without failure even when there is apparently no downstream hazard under present conditions of development."

Therefore, based on the results of this NRC Flood Study and on the Federal Guidelines for Dam Safety, Harriman Dam should probably be considered for upgrading.

2.0 Recommendations

The staff has considered possible remedial measures for the potential flood problem at the Yankee Rowe site. The best "fix" would be either an emergency spillway in the west abutment of the Harriman Dam or a diversion to divert excess flows to an adjoining drainage basin. The emergency spillway would have to be sized to pass the difference in runoff volume between present capacity and the volume for a Spillway Design Flood. The Spillway Design Flood, SDF, is that flood discharge, regardless of other designation or method of computation, which is used to develop the hydrologic and hydraulic design of a spillway and dam. Other possible remedial measures would be to raise the existing dam (about 15 feet) to contain the PMF or remove the dam completely.

Since Harriman Dam can safely pass a flood that is about equivalent to a maximum regional event, which is a rare event (see Sections 4.2.4 and 4.5), it is recommended that continued operation of the Yankee Rowe Plant be allowed provided that the licensee initiate a program of analysis, design and installation or construction of engineered mitigation measures. Such activities should be scheduled for completion in coordination with other influential decisions affecting the plant and the dams; i.e., seismic effects, but in any event, to be completed by January 1, 1984.

7. References

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13. Yankee Atomic Electric Company (YAEC) Report No. YAEC-1139, July 1977, "Yankee Nuclear Station Probable Maximum Flood Analysis."

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15. World Meteorological Organization, Operational Hydrology Report No. 1, 1973, "Manual for Estimation of Probable Maximum Precipitation."

16. U.S. Department of Commerce, Weather Bureau, Technical Paper No. 40, May 1961, "Rainfall Frequency Atlas of the United States."

17. U.S. Department of Commerce, National Oceanic and Atmospheric Administration, National Weather Service, NOAA Technical Report NWS 24, February 1980, "A Methodology for Point-to-Area Rainfall Frequency Ratios."

18. Tennessee Valley Authority, "Dam Breaching Program", November 1973.

19. Federal Coordinating Council for Science, Engineering and Technology, "Federal Guidelines for Dam Safety," June 25, 1979.

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TABLE 3.2.1

CHARACTERISTICS OF UPPER DEERFIELD RIVER DAMS

	Somerset	Searsburg	Harriman	Sherman
Construction Completed	1913	1922	1924	1926
Drainage Area (square miles)	30	90	184	236
Height (feet)	104	50	215.5	110
Length (feet)	2010	475	1250	810
Dam Crest Elevation - USGS (feet)	2146.58	1762.66	1521.16	1129.66
Spillway Crest Elevation USGS (feet)	2133.58	1749.66	1491.66	1103.66
Active Storage at Spillway Crest (Acre feet)	57345	282.5	103375	4561
Surface Area at Spillway Crest (Acres)	1623	25	2050	194
Discharge Capacity, 5 ft. over Spillway Crest (cfs)	4950	5850	15040	6800
Discharge Capacity, 10 ft. over Spillway Crest (cfs)	8930	14890	33520	20700
Generation Capacity (MW)	--	4	33.6	7.2

TABLE 3.3.1

Major Storm Events and Related Information
in the Upper Deerfield River Basin at Harriman Station Since 1924

Year	Harriman Max. 24-Hr. Rainfall (In.)	Storm Duration (Hr.)	Harriman Total Rainfall (In.)	Harriman Max. Water Level (Ft.)	Charlemo[illegible] Max. Flow [illegible]
1927 (Nov.)	5.38	>72	5.77	1482.3	36,000
1936 (Mar.)	1.94	62	3.50	1486.5	32,200
1938 (Sep.)	4.74	82	8.89	1497.8	56,300
1948-49 (Jan.)	5.53	74	9.02	1498.1	42,600
1955 (Aug. 14)	4.27	82	8.08	1491.2	2,980
1955 (Aug. 19)	2.49	42	2.82	1489.1	5,240
1955 (Oct.)	3.87	66	7.76	1494.0	20,200
1960 (Sep.)	3.87	38	5.73	1487.8	12,800
1976 (Aug.)	4.11	86	6.06	1494.8	18,100

Harriman Spillway Crest (USGS Datum) - 1491.7 feet
Harriman Dam Crest (USGS Datum) - 1521.2 feet
Harriman Dam Drainage Area - 184 sq. miles
Charlemont Drainage Area - 362 sq. miles

TABLE 3.3.2

HISTORIC RAINFALL

No. NA 1-21; Storm Center: Elka Park, N.Y.; Oct. 4-6, 1932

MAXIMUM AVERAGE DEPTH OF RAINFALL IN INCHES

Area in Sq. Mi.	Duration of Rainfall in Hours										
	6	12	18	24	30	36	48	66			
Max. Station	4.7	7.5	10.0	10.0	10.0	11.5	11.7	11.7			
10	4.5	7.2	9.8	9.8	9.8	11.2	11.5	11.5			
100	3.9	6.3	8.9	9.1	9.1	10.1	10.6	10.6			
200	3.8	5.9	8.6	8.8	8.8	9.7	10.1	10.1			
500	3.6	5.5	7.8	8.1	8.1	8.8	9.1	9.1			
1,000	3.3	5.2	7.2	7.4	7.4	8.1	8.4	8.4			
2,000	3.1	4.8	6.6	6.8	6.8	7.4	7.7	7.7			
5,000	2.6	4.1	5.5	6.0	6.0	6.5	6.9	6.9			
10,000	2.2	3.5	4.8	5.4	5.4	5.9	6.3	6.3			
20,000	1.9	3.0	4.1	4.8	4.9	5.3	5.7	5.7			
50,000	1.5	2.4	3.3	4.1	4.2	4.6	5.0	5.0			
60,000	1.4	2.3	3.1	3.9	4.1	4.4	4.8	4.8			

No. NA 2-22A; Storm Center: Westfield, Mass.; Aug. 17-20, 1955

MAXIMUM AVERAGE DEPTH OF RAINFALL IN INCHES

Area in Sq. Mi.	Duration of Rainfall in Hours									
	6	12	18	24	30	36	48	60	72	
Max. Station	7.9	11.7	14.3	18.2	19.4	19.5	19.8	19.8	19.8	
10	7.8	11.1	13.0	16.4	18.5	18.9	19.4	19.6	19.6	
100	7.6	10.5	11.6	14.6	17.6	18.1	18.8	19.0	19.0	
200	7.4	10.2	11.4	14.2	17.1	17.6	18.2	18.4	18.4	
500	6.8	9.7	10.8	13.4	16.3	16.8	17.2	17.3	17.3	
1,000	6.2	9.2	10.2	12.4	15.4	15.9	16.2	16.4	16.4	
2,000	5.4	8.0	9.4	11.2	14.0	14.5	14.9	15.2	15.2	
5,000	4.0	6.3	7.9	9.5	11.7	12.1	12.6	13.0	13.0	
10,000	3.1	5.0	6.5	8.0	9.7	10.0	10.6	10.8	10.8	
20,000	2.1	3.6	4.9	6.3	7.6	7.9	8.3	8.5	8.5	
35,000	1.3	2.5	3.6	4.7	5.6	6.0	6.4	6.5	6.5	

No. NA1-3; Storm Center: Paterson, N.J.; Sept. 20-24, 1882

MAXIMUM AVERAGE DEPTH OF RAINFALL IN INCHES

Area in Sq. Mi.	Duration of Rainfall in Hours										
	6	12	18	24	30	36	48	60	72	96	108
10	5.6	9.7	11.3	11.4	11.4	14.3	16.9	16.9	17.6	17.9	17.9
100	4.9	8.7	10.3	10.6	10.6	13.2	15.7	15.9	16.1	16.6	16.6
200	4.6	8.3	9.8	10.1	10.1	12.6	15.0	15.2	15.5	15.9	15.9
500	4.1	7.6	8.8	9.0	9.1	11.6	13.7	14.0	14.3	14.6	14.6
1,000	3.6	6.7	7.7	7.9	8.1	10.6	12.2	12.6	12.9	13.2	13.2
2,000	3.3	5.8	6.6	6.8	6.9	9.2	10.6	11.0	11.3	11.7	11.7
5,000	2.7	4.5	[illegible]	5.4	[illegible]	7.5	[illegible]	9.2	9.4	[illegible]	[illegible]
10,000	2.2	3.7	4.2	4.6	[illegible]	6.2	7.4	[illegible]	8.2	[illegible]	[illegible]
20,000	1.8	3.1	3.7	3.9	4.2	5.4	6.1	[illegible]	6.8	7.3	7.3
40,000	1.3	2.3	2.6	2.9	3.5	4.2	4.7	5.2	5.5	5.9	5.9

TABLE 3.3.2 (Con't)

No. NA 1-17; Storm Center: Kinsman Notch, N.H.; Nov. 2-4, 1927

MAXIMUM AVERAGE DEPTH OF RAINFALL IN INCHES

Area in Sq. Mi.	Duration of Rainfall in Hours									
	6	12	18	24	30	36	48	60		
10	7.8	10.6	11.7	12.0	12.8	13.7	14.0	14.0		
100	5.8	8.3	8.8	9.2	9.5	10.1	10.3	10.3		
200	5.7	8.2	8.6	8.8	9.0	10.0	10.2	10.2		
500	5.5	7.9	8.2	8.3	8.5	9.0	9.2	9.2		
1,000	4.8	7.3	7.7	7.8	8.2	8.8	8.9	8.9		
2,000	4.0	6.4	7.0	7.3	7.9	8.1	8.2	8.2		
5,000	2.7	4.8	6.1	6.7	7.2	7.7	7.9	7.9		
10,000	2.3	4.0	5.5	6.3	6.7	7.0	7.3	7.3		
20,000	2.0	3.5	4.7	5.0	5.8	6.2	6.4	6.4		
50,000	1.6	2.8	3.6	4.1	4.5	4.9	5.1	5.1		
60,000	1.4	2.5	3.3	3.8	4.2	4.6	4.8	4.8		

No. NA 1-27; Storm Center: Hector, N.Y.; Jul. 6-10, 1935

MAXIMUM AVERAGE DEPTH OF RAINFALL IN INCHES

Area in Sq. Mi.	Duration of Rainfall in Hours									
	6	12	18	24	30	36	48	60	72	90
10	5.2	10.2	11.4	11.8	12.0	13.4	14.2	14.2	14.2	14.2
20	5.1	9.7	11.1	11.5	11.6	12.9	13.9	13.9	14.0	14.1
100	4.9	8.6	10.1	10.5	10.7	11.5	13.0	13.1	13.4	13.6
200	4.7	8.0	9.6	10.0	10.3	10.9	12.5	12.6	12.9	13.2
500	4.3	7.3	8.8	9.3	9.5	9.8	11.6	11.8	12.0	12.4
1,000	4.0	6.7	8.2	8.6	8.8	9.0	10.6	10.8	11.1	11.5
2,000	3.5	6.0	7.3	7.8	8.0	8.2	9.5	9.8	10.0	10.4
5,000	2.7	4.8	5.9	6.4	6.6	6.8	7.7	8.2	8.5	8.7
10,000	2.1	3.7	4.6	5.1	5.4	5.7	6.4	7.0	7.2	7.5
20,000	1.3	2.6	3.2	3.7	4.1	4.5	5.1	5.6	5.9	6.2
30,000	0.9	1.9	2.4	2.9	3.4	3.8	4.3	4.8	5.2	5.5
38,500	0.7	1.4	1.9	2.4	2.9	3.3	3.8	4.3	4.8	5.1

No. NA 2-2; Storm Center: Barre, Mass; Sept. 17-22, 1938

MAXIMUM AVERAGE DEPTH OF RAINFALL IN INCHES

Area in Sq. Mi.	Duration of Rainfall in Hours										
	6	12	18	24	30	36	48	60	72	96	120
10	6.4	8.2	9.6	11.3	12.2	13.2	14.3	15.0	15.8	16.9	17.1
100	5.0	6.8	8.3	9.5	10.4	11.4	13.0	14.0	15.1	16.5	16.8
200	4.6	6.3	7.8	9.0	9.8	10.9	12.4	13.4	14.8	16.1	16.4
500	4.1	5.6	7.1	8.3	9.0	10.2	11.6	12.6	14.2	15.4	15.7
1000	3.7	5.1	6.6	7.7	8.4	9.6	11.0	12.0	13.8	14.6	15.0
2000	3.3	4.5	6.0	7.2	7.8	9.0	10.4	11.3	13.2	13.8	14.2
5000	2.7	3.9	5.1	6.3	6.9	8.2	9.5	10.3	12.0	12.4	12.8
10000	2.3	3.3	4.4	5.7	6.2	7.4	8.6	9.6	10.9	11.3	11.6
20000	1.9	2.8	3.8	4.9	5.5	6.4	7.5	8.6	9.6	10.0	10.3
50000	1.4	2.1	2.8	3.7	4.2	4.8	5.8	6.6	7.3	7.7	8.0
67000	1.2	1.9	2.5	3.3	3.7	4.2	5.1	5.9	6.9	6.8	7.1

TABLE 3.3.2(Con't)

No. OR9-23; Storm Center: Port Alleghany, Pa.; Jul. 17-18, 1942

"Smethport'

MAXIMUM AVERAGE DEPTH OF RAINFALL IN INCHES

Area in Sq. Mi.	Duration of Rainfall in Hours										
	6	12	18	24							
Max.Station	30.7	34.3	35.5	35.5							
1	29.3	32.0	33.8	34.2							
5	26.4	28.6	30.5	31.0							
10	24.7	26.7	28.7	29.2							
20	22.8	24.8	26.8	27.4							
50	19.7	21.9	24.1	24.6							
100	16.4	19.4	21.8	22.4							
200	13.1	16.8	19.3	19.9							
500	9.1	13.2	15.7	16.3							
1,000	6.4	10.3	12.6	13.3							
2,000	3.9	7.2	9.2	10.2							
4,300	2.5	4.6	6.1	7.1							

TABLE 4.1

SENSITIVITY ANALYSIS

	HARRIMAN SPILLWAY ≡ 75% CAPACITY 4/										HARRIMAN SPILLWAY = 100% CAPACITY							
	Licensee Snyder Coefficients 1/						NRC Clark Coefficients 2/				Licensee Snyder Coefficients 2/				NRC Clark Coefficients 2/			
Index Rainfall (inches)	13.0	16.0	16.5	18.9 3/	18.9	21.3	16.0	16.5	18.9	21.3	16.0	16.5	18.9	21.3	16.0	16.5	18.9	21.3
HARRIMAN RESERVOIR																		
Max. Pool (elev)	1520.7	1524.6	1524.0	1524.3	1525.7	1526.4	1523.9	1523.6	1524.4	1525.2	1524.1	1523.9	1524.9	1525.6	1522.6	1523.2	1524.5	1525.2
Peak Q (1000 cfs)	27	2,463	2,500	2,454	2,547	2,596	2,413	2,443	2,474	2,522	2,433	2,479	2,526	2,578	41	2,408	2,462	2,514
Time of Failure	-	69.33	68.67	69.33	68.0	67.33	73.0	71.67	69.67	68.67	69.67	69.00	68.0	67.33	-	73.67	70.33	69.0
Time of Qp (hrs)	72.33	70.33	69.67	70.33	69.0	68.33	74.0	72.67	70.67	69.67	70.67	70.00	69.0	68.33	75.0	74.67	71.33	70.0
Duration over top (hrs)	0	1.55	1.24	1.53	1.29	1.33	2.49	2.18	1.55	1.27	1.51	1.55	1.25	1.29	6.0	3.47	1.88	1.59
SHERMAN RESERVOIR																		
Max. Pool (elev)	1131.6	1171.3	1172.4	1182.5	1174.6	1176.5	1196.6	1197.4	1172.5	1174.1	1170.9	1171.7	1174.0	1175.9	1132.4	1169.7	1172.0	1173.7
Peak Q (1000 cfs)	80	2,151	2,193	2,146	2,273	2,352	2,065	2,096	2,183	2,250	2,117	2,164	2,248	2,329	325	2,078	2,161	2,23[illegible]
Time of Failure (hrs)	-	66.00	66.33	69.67	65.67	65.33	73.67	72.33	66.33	66.0	65.67	65.67	65.0	64.67	66.67	66.67	66.0	65.67
Time of Qp (hrs)	67.33	70.67	70.00	70.65	69.33	68.67	74.35	73.02	71.00	70.0	71.00	70.33	69.33	68.67	67.65	75.00	71.67	70.33
Duration over top (hrs)	2.33	2.43	2.76	3.00	2.82	2.51	3.67	3.33	2.41	2.45	2.47	2.45	2.47	2.47	1.41	2.41	2.47	2.8[illegible]

1/ NRC Flood Study Using Adopted Values

2/ Sensitivity Study for Comparison Purposes

3/ 43% of 24-Hour Rainfall in Worst 6-Hour Period

4/ Spillway Capacity Limited to Orifice Control in the Riser - Elev. 1414

TABLE 5.1

COMPARISON OF OTHER FLOOD STUDIES TO THE NRC FLOOD STUDY

	C. T. MAIN (FPC Report)[12] (1977)	YANKEE ROWE (FPC Report) (1977)	YANKEE ROWE (1980 Report)	NUCLEAR REGULATORY COMM. PMF	NUCLEAR REGULATORY COMM. Lower Limit	Corps of Engineers 1963 4/
SOMERSET RESERVOIR						
Peak Inflow (cfs)	40,000	30 300	33,000	54,489	39,000	55,900
Peak Outflow (cfs)	7,600	7,100	7,500	8960	6,600	10,800
Max. Pool Elev. (ft msl)	2143.7	2141.4	2141.9	2145.4	2142.5	2144.7
Top of Dam (ft msl)	2146.58					
Starting Pool Elev. (ft msl)		2133.58	2133.58	2133.6	2133.6	
SubBasin Rainfall (24 hr-200 S.M.)	HR#33	19.5	14.1	18.9 2/	14.0 2/	25.1 3/
HARRIMAN RESERVOIR						
Peak Inflow (cfs)	210,600	161,500	149,900	240,570	172,100	170,000
Peak Outflow (cfs)	216,300	40,700	37,000	2,547,000	32,100	38,600
Max. Pool Elev. (ft msl)	1523.2 1/	1521.9	1519.2	1525.7 1/	1522.9	1527±
Top of Dam (ft msl)	1521.16					
Starting Pool (ft msl)		1488.96	1491.66	1491.6	1491.6	
Sub Basin Rainfall (24 hr-200 S.M.)	HR#33	19.5	14.1	18.9 2/	14.0 2/	20.6 3/
SHERMAN RESERVOIR						
Peak Inflow (cfs)	265,300	74,140	87,300	2,121,600	86,700	113,000
Peak Outflow (cfs)	241,800	71,000	287,000	2,273,300	312,600	110,000±
Max. Pool Elev. (ft msl)	1131.66 1/	1128.6	1131.7	1174.6 1/	1132.2 1/	1135±
Top of Dam (ft msl)	1129.66					
Starting Pool (ft msl)		1103.66	1103.66	1103.6	1103.6	
Sub Basin Rainfall (24 hr-200 S.M.)	HR#33	19.5	14.1	18.9 2/	14.0 2/	25.1 3/

1/ Dam Assumed to fail when overtopped by 2.0 feet.

2/ 24 hour average rainfall for 236 square mile drainage area

3/ The Corps study was for a spillway design flood, so their rainfall is probably based on the reservoir subbasin drainage area.

4/ No flow over dams.

APPENDIX A

[illegible]

8 February 1963

Chairman
Federal Power Commission
Washington 25, D. C.

Dear Mr. Chairman:

Reference is made to the Commission's letter dated 15 November 1962 concerning the application filed by New England Power Company for license for constructed hydroelectric project (No. 2323) located on the Deerfield River in Massachusetts and Vermont.

The applicant's project consists of seven dams with hydroelectric power stations located on the Deerfield River and one storage dam and reservoir on the East Branch of the Deerfield River. The projects are located between points 11 miles and 59 miles of the Deerfield River upstream of its junction with the Connecticut River.

There is an existing Federal navigation project on the Connecticut River from its mouth to Hartford, Connecticut, 68 miles below the mouth of the Deerfield River. The Deerfield River itself is not currently used for commercial navigation nor is there any indication that improvements of the river for navigation are desired or warranted at this time. The plans of the structures affecting navigation are satisfactory and the project would not affect the interests of navigation. Therefore, special terms and conditions for insertion in the license, if issued, are not considered necessary insofar as the interests of navigation are concerned.

The Somerset, Harriman and Sherman dams are considered major earth structures with the Somerset and Harriman dams having significant effect on reducing flood flows of record. The Sherman dam is a run of river project and would not reduce flood discharges. The applicant has studied the adequacy of existing spillways of all dams of the project and selected a design flood that is considered a rare event. However, computations made by the Corps of Engineers based on recent criteria indicate that the Harriman and Sherman dams would not pass the Corps' spillway design flood with spillway capacities and heights of dams as

[illegible]
Chairman, Federal Power Commission 8 February 1963

now proposed by the applicant. Computations indicate that the Somerset dam would pass the spillway design flood using either the Corps' or the Company's criteria but there would be a reduction in freeboard associated with the Corps' criteria. The Commission may wish to review the spillway design features of two of the major structures and data used by the Corps of Engineers will be furnished upon request.

One copy of the application is returned as requested.

Sincerely yours,

1 Incl
Application

ROBERT C. MARSHALL
Colonel, Corps of Engineers
Assistant Director of Civil Works
for Eastern Divisions

CC: U. S. Army Engineer Division, New England

Mr. Coh /Mr. Childs/mbc/546

Mr. Cohen

NEDGW (15 Nov 62) 2nd Ind
SUBJECT: New England Power Company, Project No. 2323

U. S. Army Engr Div, New England, Waltham, Mass. 30 January 1963

TO: Chief of Engineers, ATTN: ENGCW-BP, DA, Washington, D. C.

1. The application of the New England Power Company (NEPCO) of Boston, Massachusetts to the Federal Power Commission for a license for the constructed hydroelectric project (No. 2323) located on the Deerfield River in Massachusetts and Vermont, together with supplementary data submitted with your letter of 18 January 1963, has been reviewed and the following report is submitted herewith.

2. Project. - The project consists of seven dams with hydroelectric power stations located on the Deerfield River between points 11 miles and 59 miles upstream of its junction with the Connecticut River and one dam and reservoir on the East Branch of the Deerfield River used to regulate flows. The seven dams with power stations are:

Deerfield No. 2 near Conway and Shelburne, Mass. with three generating units totalling 4,800 KW.

Deerfield No. 3 near Shelburne and Buckland, Mass. with three generating units totalling 4,800 KW.

Deerfield No. 4 near Shelburne and Buckland, Mass. with three generating units totalling 4,800 KW.

Deerfield No. 5 near Monroe and Florida, Mass. with three generating units totalling 15,000 KW.

Sherman near Monroe, Mass. with one generating unit of 7,200 KW.

Harriman near Whitingham and Wilmington, Vt., with three generating units totalling 33,600 KW.

Searsburg near Searsburg, Vermont with one generating unit of 4,000 KW.

These seven power stations contain a total of 17 hydroelectric generating units with a combined capacity of 74,200 KW. One of the dams (Harriman) also provides a storage reservoir to regulate river flows and one (Somerset) provides storage only with no hydroelectric generating facilities. The remaining dams are used for pondage only so that water use is essentially run of river flow.

3. Effect of Project on Flood Flows. -

a. Of the eight NEPCO dams on the Deerfield River and its East Branch, three are considered major earth structures. Two of these dams have a significant effect on reducing flood flows. Somerset dam, located in the upper portion of the Deerfield River, controls the runoff from 30 square miles of drainage area and contains a usable capacity of 57,300 acre-feet or almost 36 inches of runoff. Harriman dam, located downstream of Somerset dam has a net drainage area of 154 square miles and contains 116,000 acre-feet of usable storage which is equivalent to about 14 inches of runoff. Sherman dam, a run of river plant is a high earth dam and is located about two miles downstream of Harriman dam.

b. Somerset Reservoir is a storage reservoir and in general is drawn down during the summer, fall and winter months to augment the natural river flow. The reservoir refills during the spring snow melt season. Harriman Reservoir is a storage reservoir which also contains a penstock for power development. The reservoir draw down and filling in general follows the same pattern as Somerset Reservoir. The remaining dams on the Deerfield River are run of the river plants with no appreciable storage.

c. During the March 1936 flood, both Somerset and Harriman dams stored almost the entire storm runoff. During the September 1938 flood, the record flood in this area, Somerset dam stored almost the entire inflow and reduced the computed peak inflow of 7,500 cfs to a discharge of about 1,000 cfs. At Harriman dam, the computed peak inflow of 44,400 cfs was reduced to about 15,000 cfs.

d. Somerset and Harriman dams are considered major earth structures and both have been designed to pass a major flood. Following the August 1955 flood in southern New England, the Power Company performed studies to determine the adequacy of existing spillways at all dams on the Deerfield River. The design storm was patterned after the August 1955 storm, which at its center near Westfield, Massachusetts, produced 18.2 inches of rainfall in 48 hours. The runoff from the design storm is 14 inches. Unit hydrographs were developed from the 1948 - 1949 year-end flood.

e. The design flood, routed through the reservoirs, indicated that remedial measures would be required at Harriman and Sherman dams. Somerset dam is capable of handling the design flood with about four feet of freeboard. At Harriman dam, the Power Company proposes to raise the top of the earth embankment seven feet to elevation 1518.66 feet, msl. This would allow 5.2 feet of freeboard during the design flood. At Sherman dam, the Power Company plans to raise the dam 20 feet to elevation 1129.66 feet, msl and to enlarge the spillway discharge channel. No major alterations are planned for the remaining dams.

f. The New England Division has built several flood control dams in the Connecticut River basin just north and south of the Deerfield River. The more recent reservoirs have been designed to pass the latest Corps spillway design flood with five feet of freeboard. Some NED dams built prior to 1955 do not have adequate spillway capacity to pass a spillway design flood computed from the latest criteria. A spillway design flood was computed for Somerset, Harriman and Sherman dams from data used to derive the recent spillway design floods at Ball Mountain Dam on the West River and Littleville Dam on the Middle Branch, Westfield River. This analysis indicated that Somerset dam could pass the spillway design flood with about two feet of freeboard but additional spillway capacity would be required at both Harriman and Sherman dams.

g. At Harriman dam, if additional spillway capacity were not provided to supplement the limited capacity of the existing morning glory, the dam w[illegible]e to be raised an additional 13 feet more than the present p[illegible] in order to store the excess volume of runoff.

h. At Sherman dam, the additional spillway capacity required to pass the spillway design flood would be at least double the proposed capacity and could be even greater if additional spillway capacity were provided at Harriman dam. The following table compares the results derived from the Power Company studies and the Corps criteria for Somerset, Harriman and Sherman dams.

Description	Somerset Dam Corps Criteria	Somerset Dam Power Co. Criteria	Harriman Dam Corps Criteria	Harriman Dam Power Co. Criteria	Sherman Dam Corps Criteria	Sherman Dam Power Co. Criteria
Drainage Area - sq. mi.	30.0	30.0	154 net	154 net	52 net	52 net
Design Storm Rainfall-inches	25.1	18.2	20.6	18.2	25.1	18.2
Design Storm Runoff-inches	22.4	14.0	18.1	14.0	22.4	14.0
Net Peak Inflow - cfs	55,900	21,000	159,000	77,000	96,000	32,500
Total Peak Inflow - cfs	55,900	21,000	170,000	80,000	113,000	63,000

DRAFT

NEDGW (15 Nov 62) 2nd Ind 30 January 1963
SUBJECT: New England Power Company, Project No. 2323

(Table continued)

Description	Somerset Dam Corps Criteria	Somerset Dam Power Co. Criteria	Harriman Dam Corps Criteria	Harriman Dam Power Co. Criteria	Sherman Dam Corps Criteria	Sherman Dam Power Co. Criteria
Peak Outflow - cfs	10,300	5,700	39,600	36,000	110,000±	61,500
Maximum Pool Stage	2144.7	2142.68	1,527±	1513.46	1,135±	1125.46
Power Company - Proposed Top of Dam		2146.58		1518.66		1129.66

i. It is recognized that the design flood selected by the Power Company is a rare event and will provide a much higher degree of protection against failure than the present design. Somerset dam can pass the spillway design flood with two feet of freeboard and is considered satisfactory. Harriman dam, with a fixed spillway capacity of about 36,000 cfs would be vulnerable to a flood approaching the magnitude of a Corps spillway design flood. Similar conditions occur at Sherman dam.

j. Inspection of the remaining five dams indicates that they are run of river plants with no appreciable storage. The lesser design criteria used for these dams is reasonable since failure of any of them should not release the tremendous volumes of water required to cause a catastrophe.

4. Effect of Project on Navigation. - There is an existing Federal navigation project on the Connecticut River from its mouth to Hartford, Connecticut, 68 miles below the mouth of the Deerfield River. The Deerfield River itself is not currently used for commercial navigation, nor is there any indication that improvements of the river for navigation are desired or warranted at this time.

5. Conclusion. - It is concluded that the two existing storage dams, Somerset and Harriman, have a significant effect in reducing flood flows. Harriman and Sherman dams, as modified by proposals of the Power Company, will be safe against failure in a large flood. However, these two high earth dams have spillways which will not pass a Corps spillway design flood, even after proposed modifications. The project would not

NEDGW (15 Nov 62) 2nd Ind 30 January 1963
SUBJECT: New England Power Company, Project No. 2323

affect present or anticipated commercial navigation and special terms and conditions for insertion in the license, if issued, are not considered necessary.

1 Incl
  1. nc
  wd 1 incl-2

P. C. HYZER
Colonel, Corps of Engineers
Division Engineer

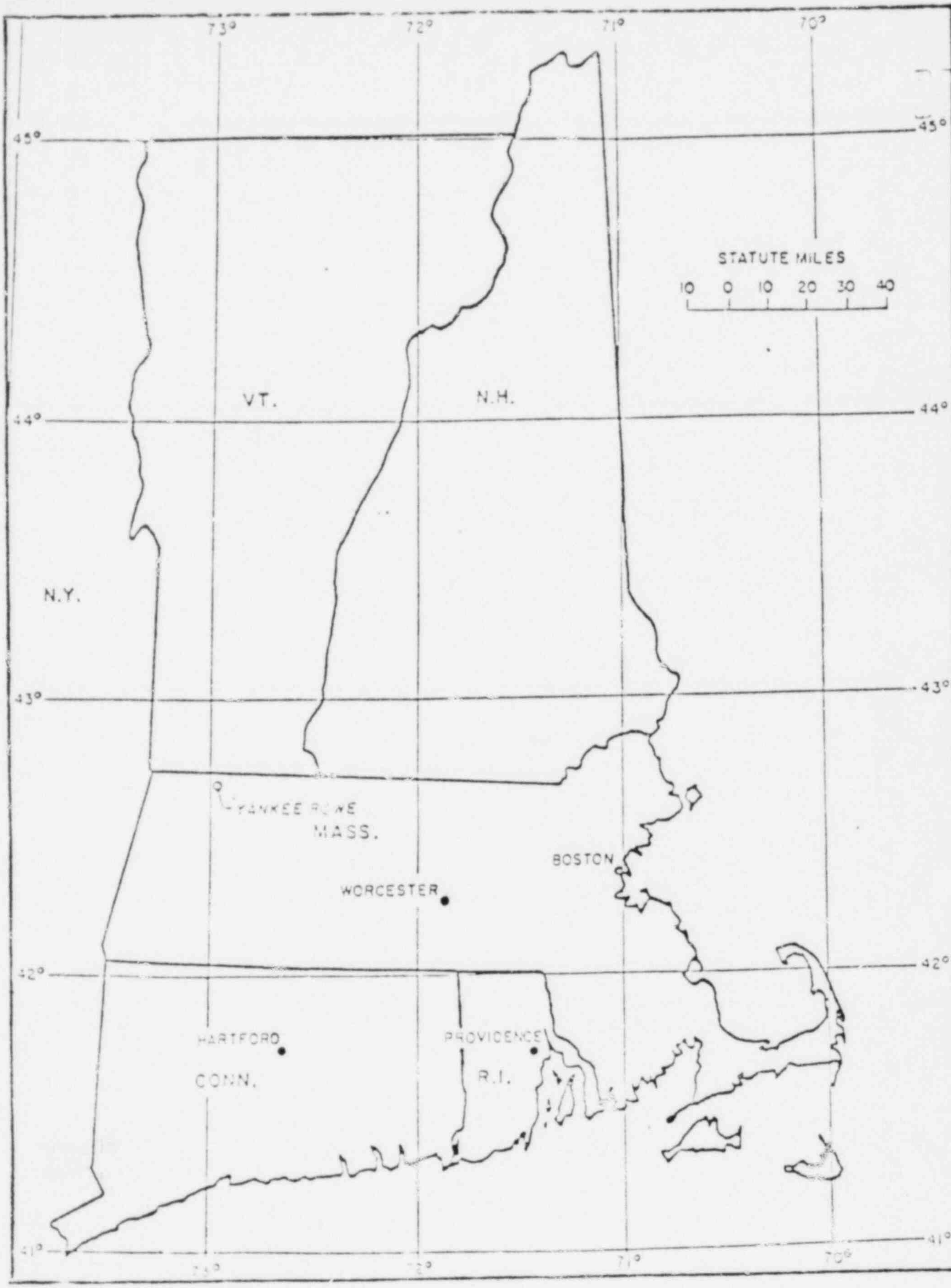

Figure 3.1.1 — General Area and Site Location Map

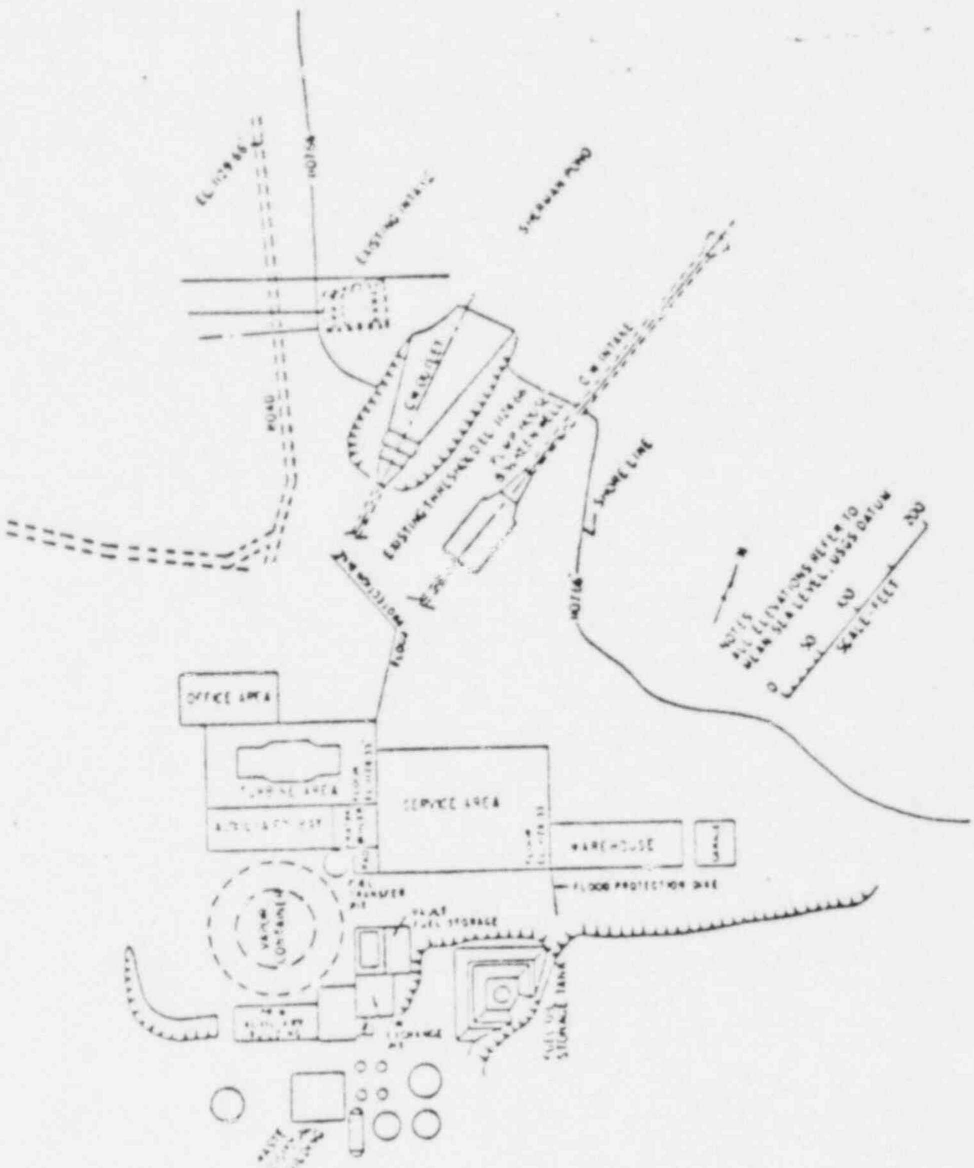

Figure 3.1.2 — General Station Layout

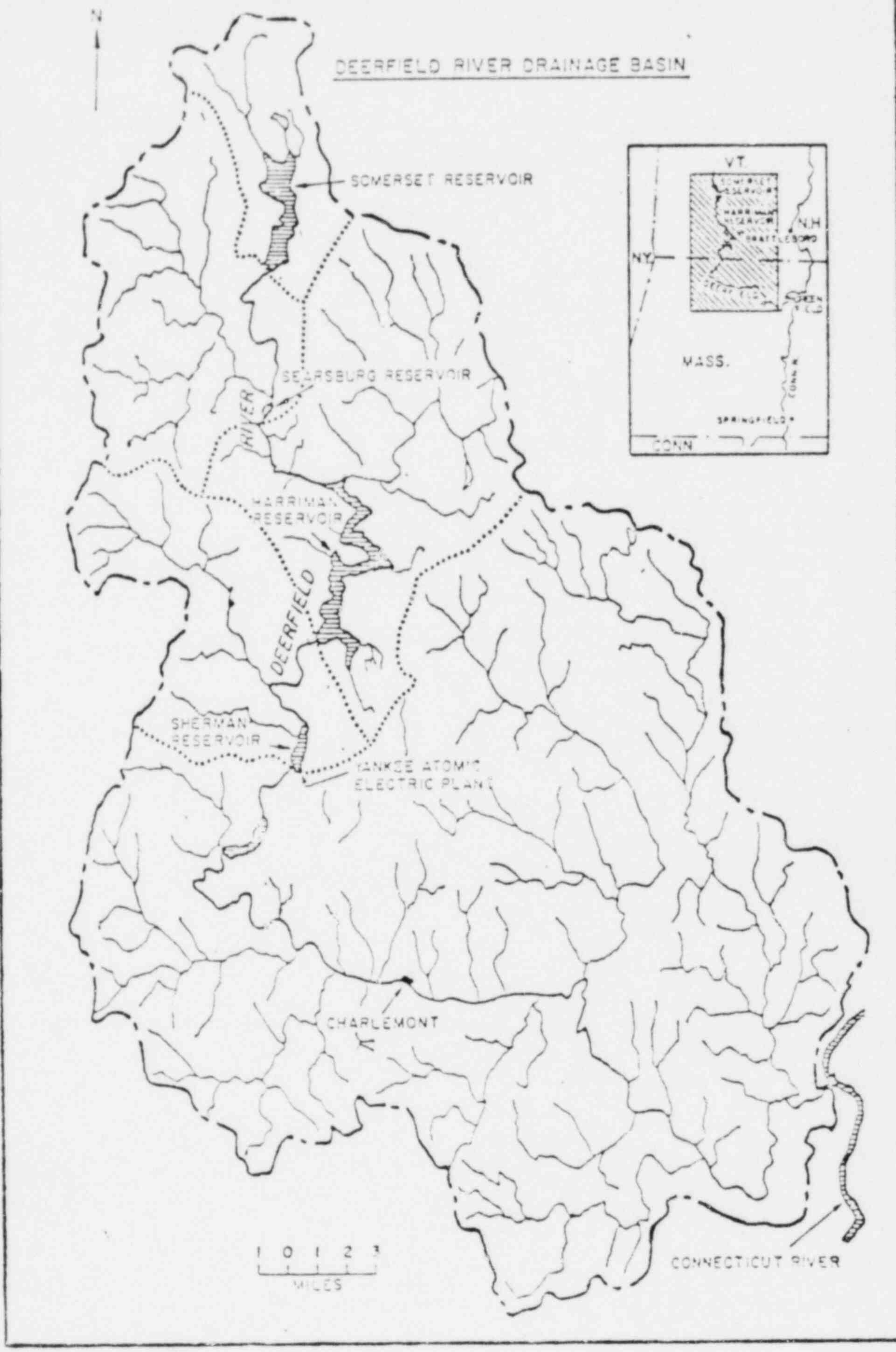

Figure 3.2.1 — Deerfield River Drainage Basin Map

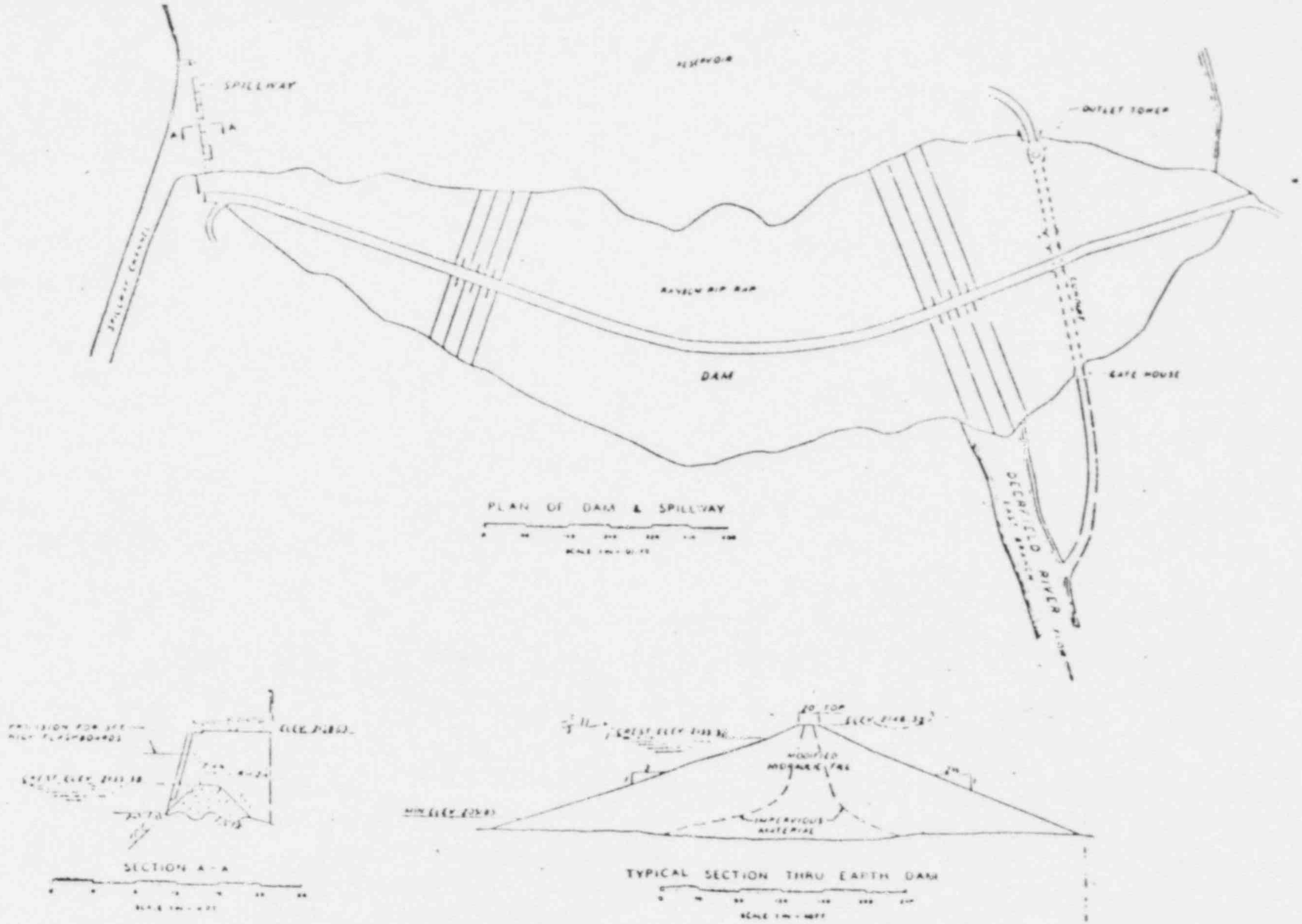

Figure 3.2.2 Somerset Dam and Spillway — Plan and Sections

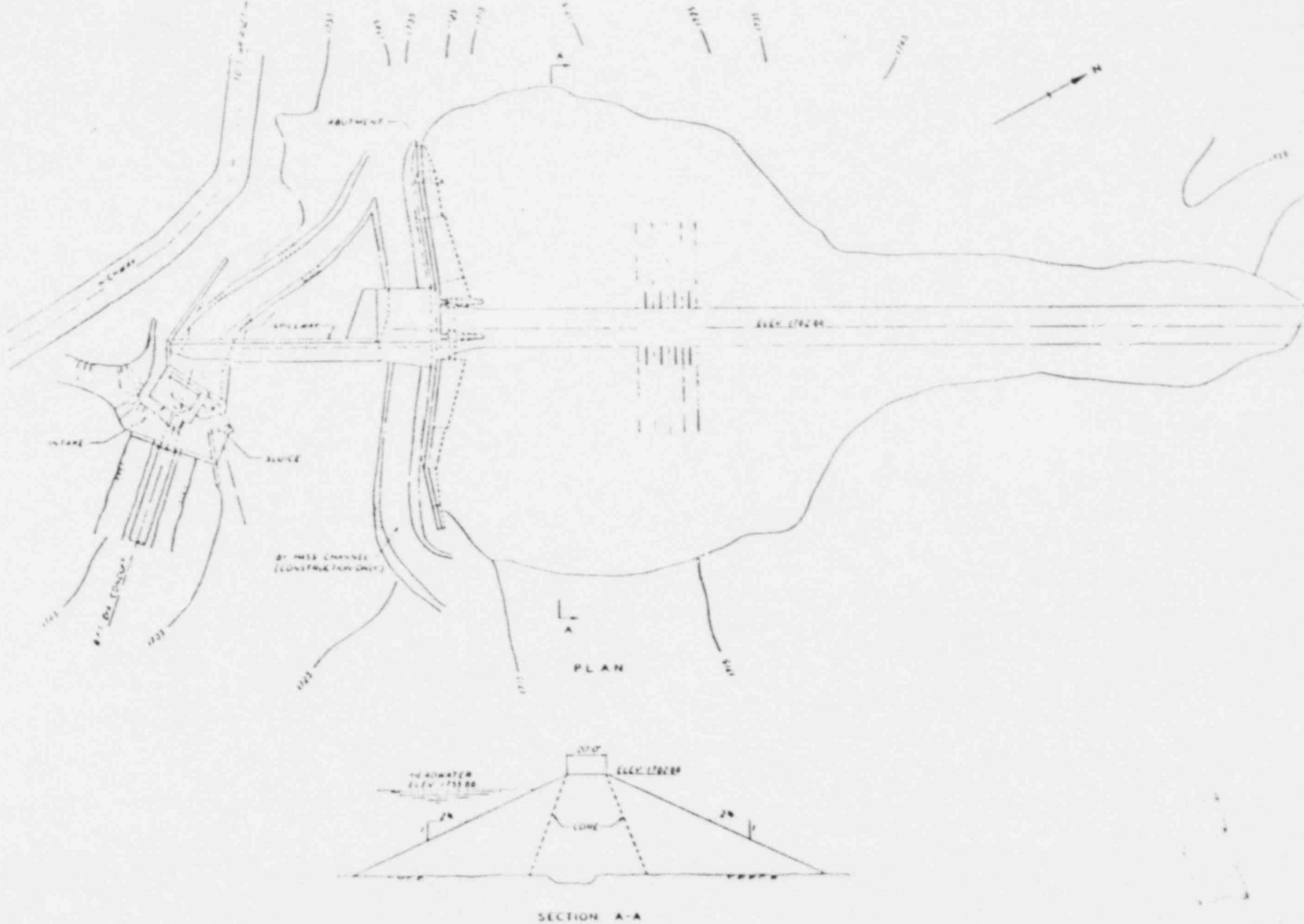

Figure 3.2.3 Searsburg Dam — Plan and Section

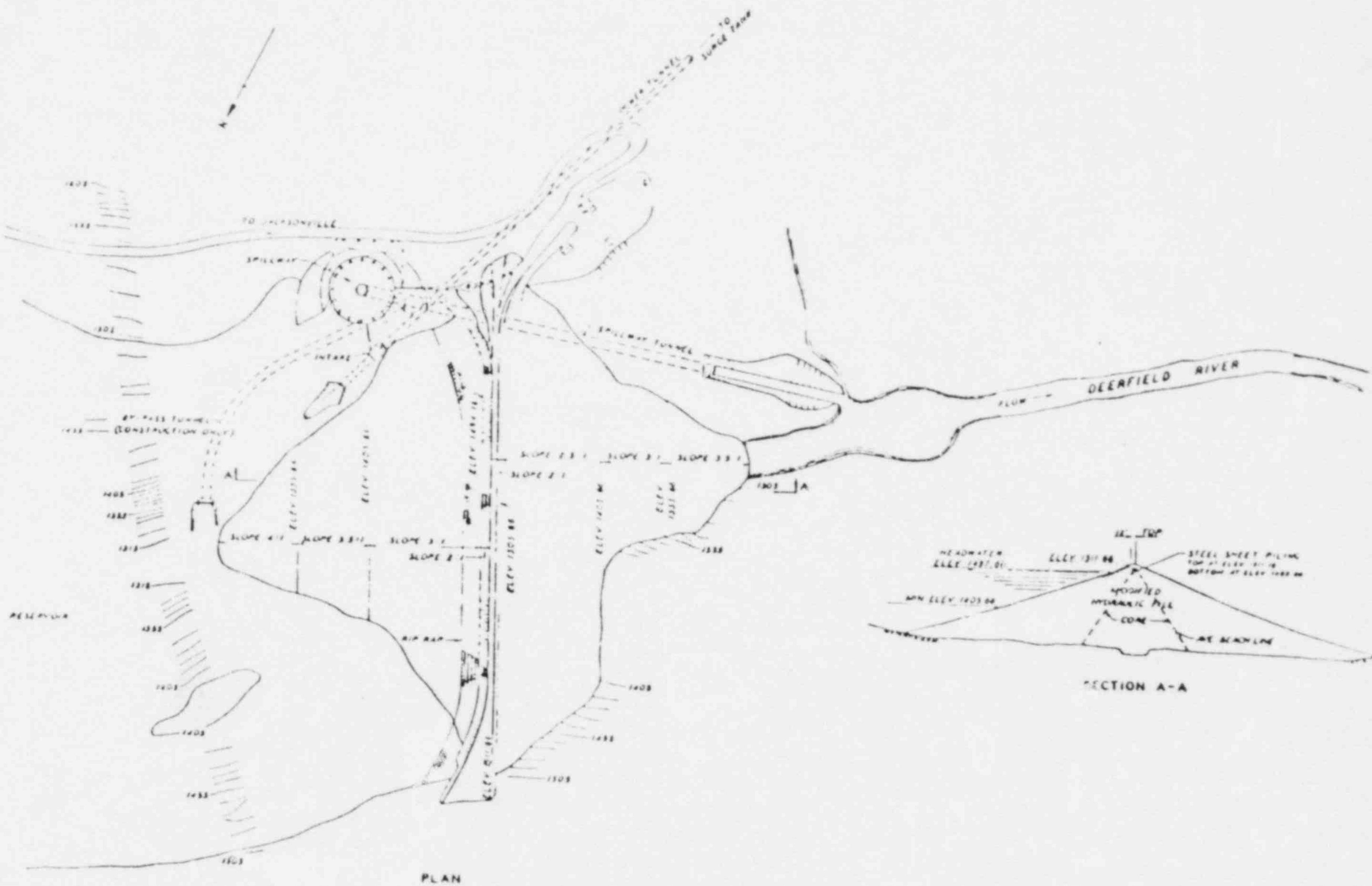

Figure 3.2.4 Harriman Dam — Plan and Section

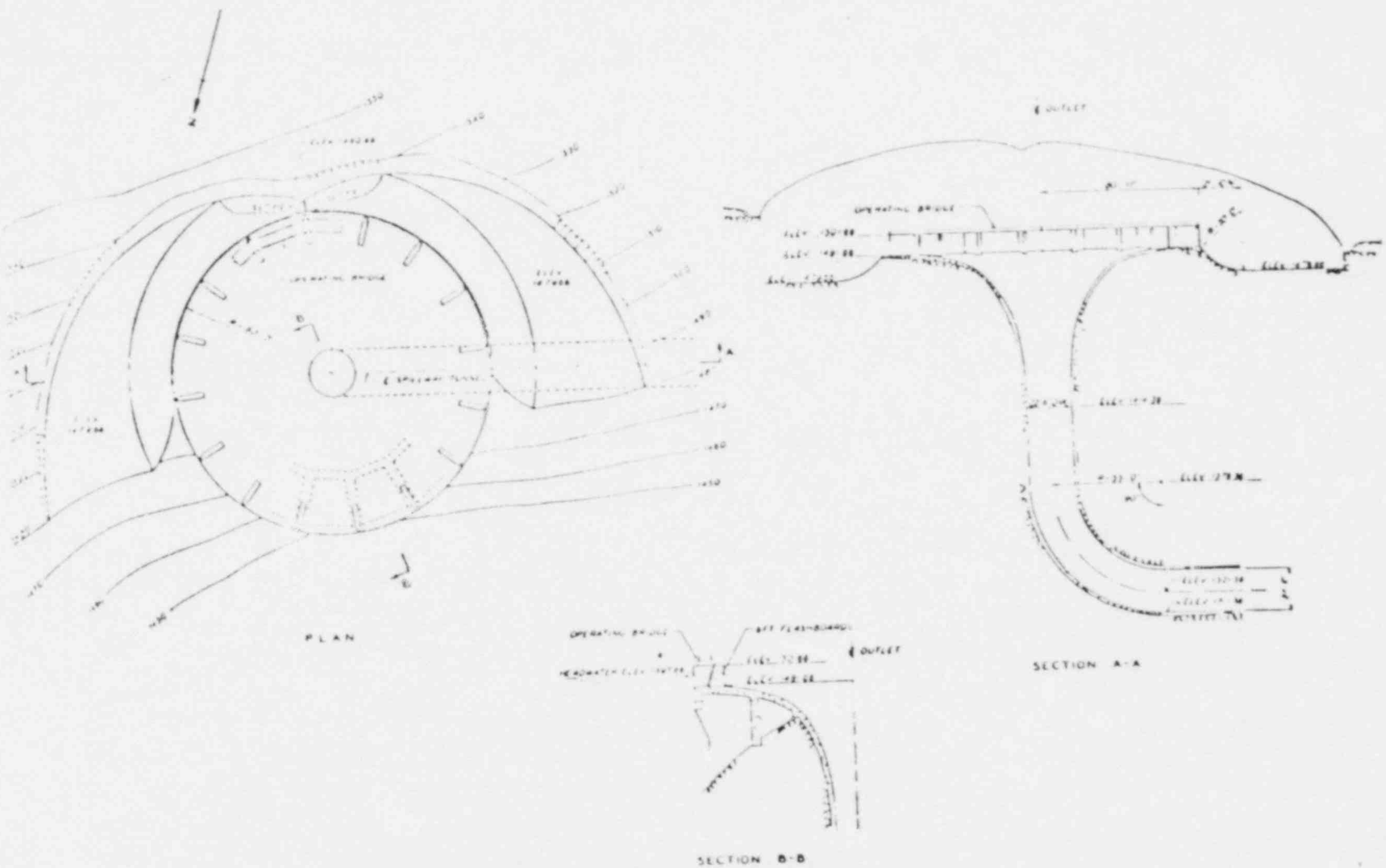

Figure 3.2.5 Harriman Spillway — Plan and Sections

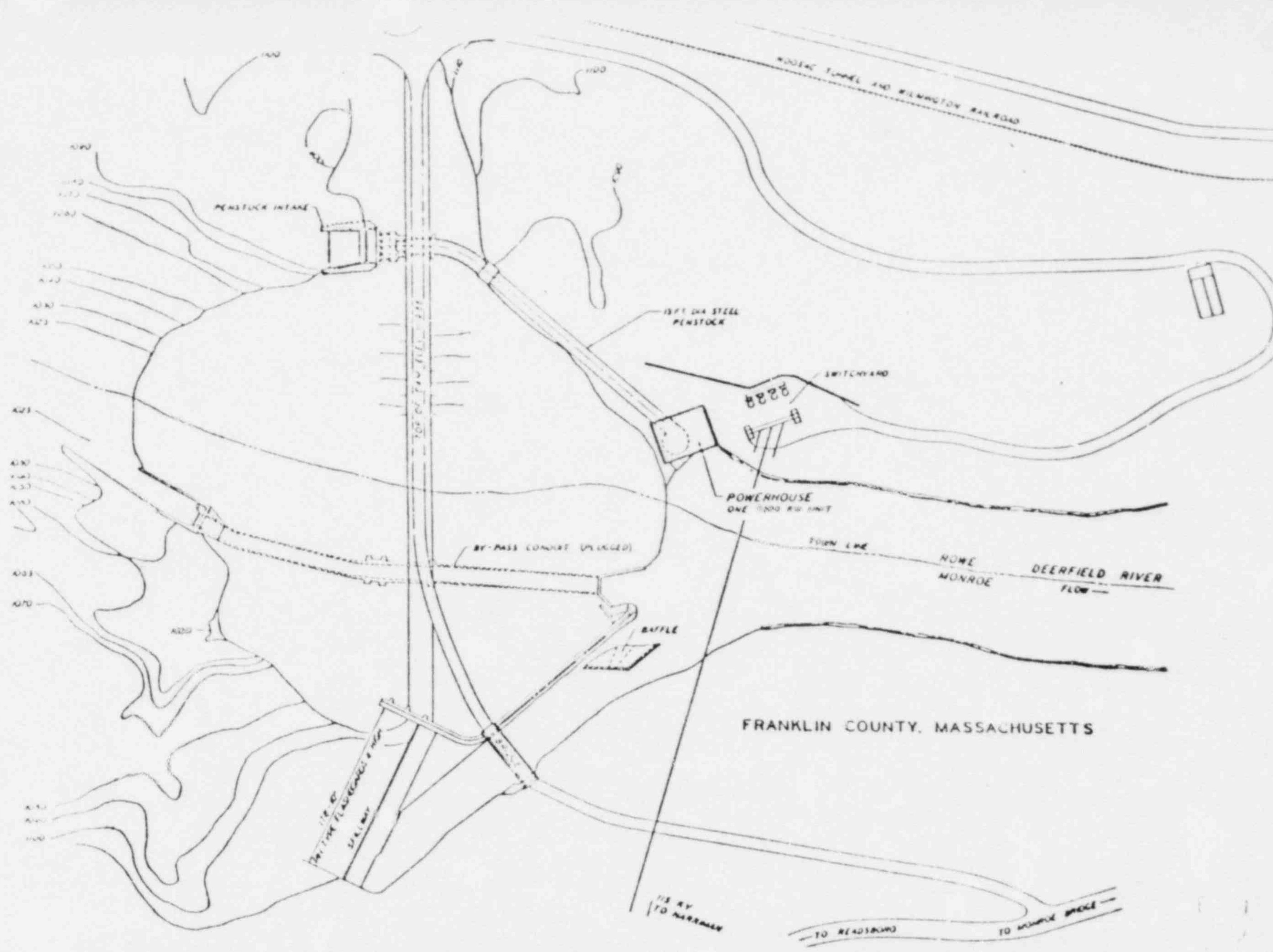

Figure 3.2.6 Sherman Dam — General Plan

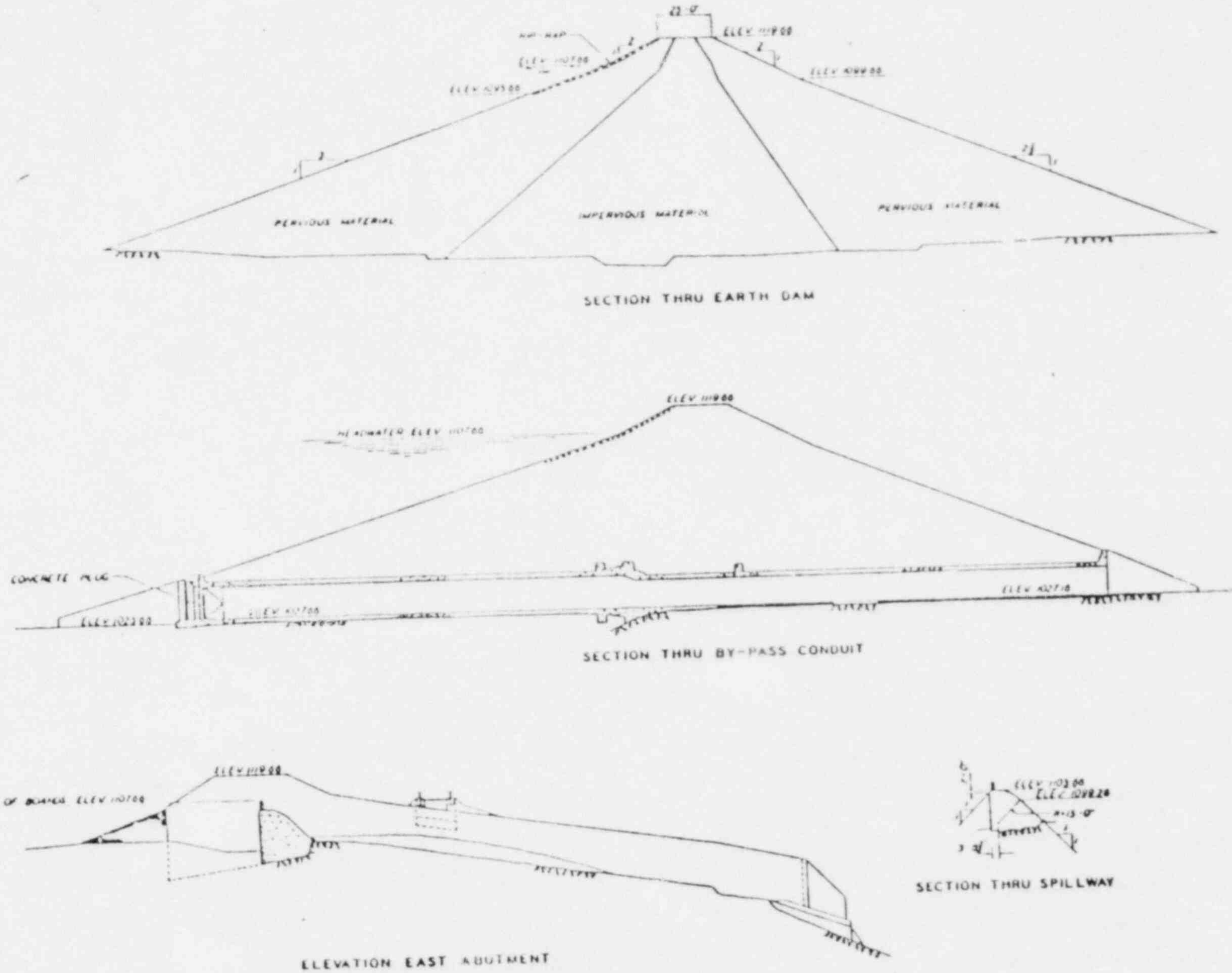

Figure 3.2.7 Sherman Dam — Sections

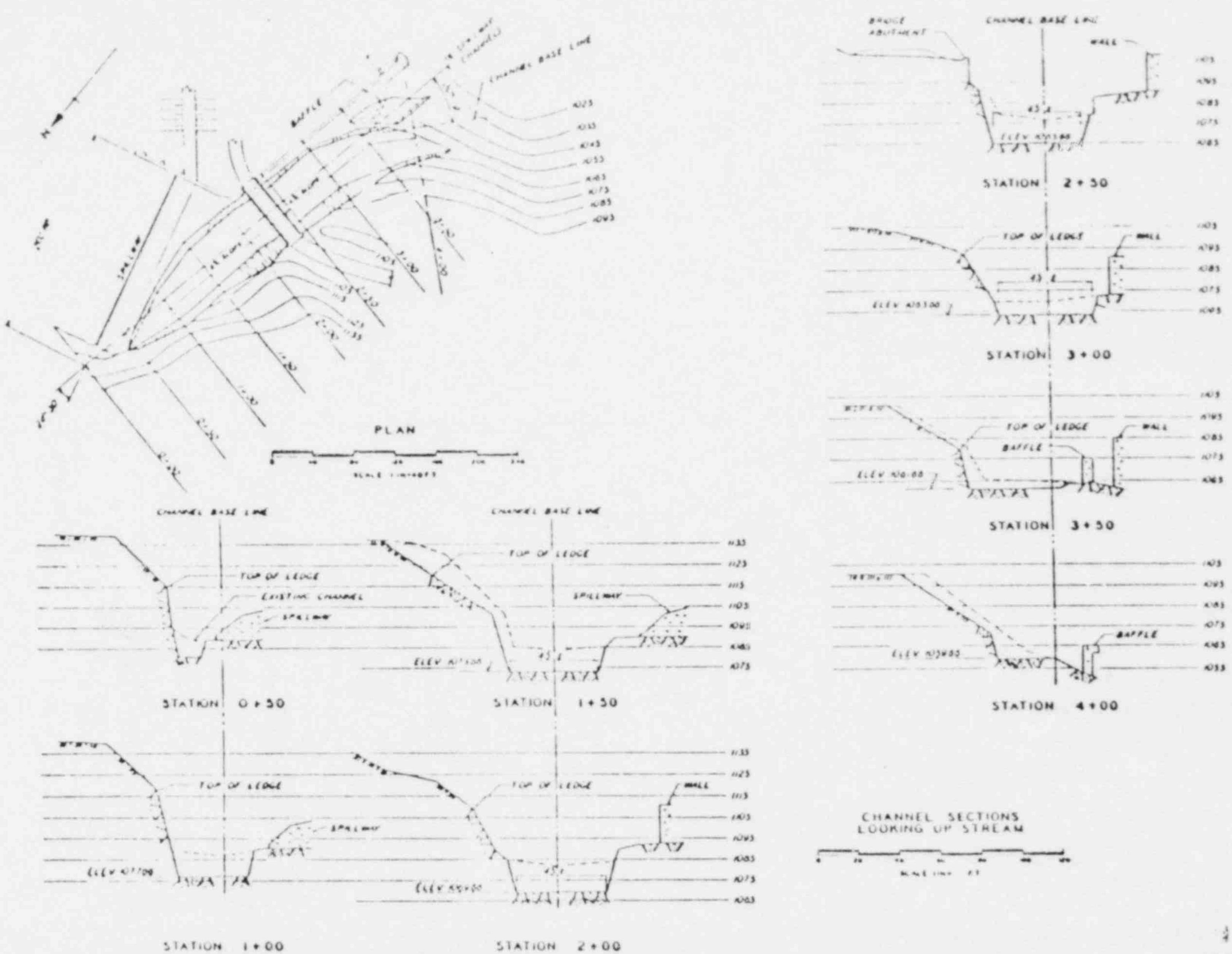

Figure 3.2.8 Sherman Spillway — Plan and Sections

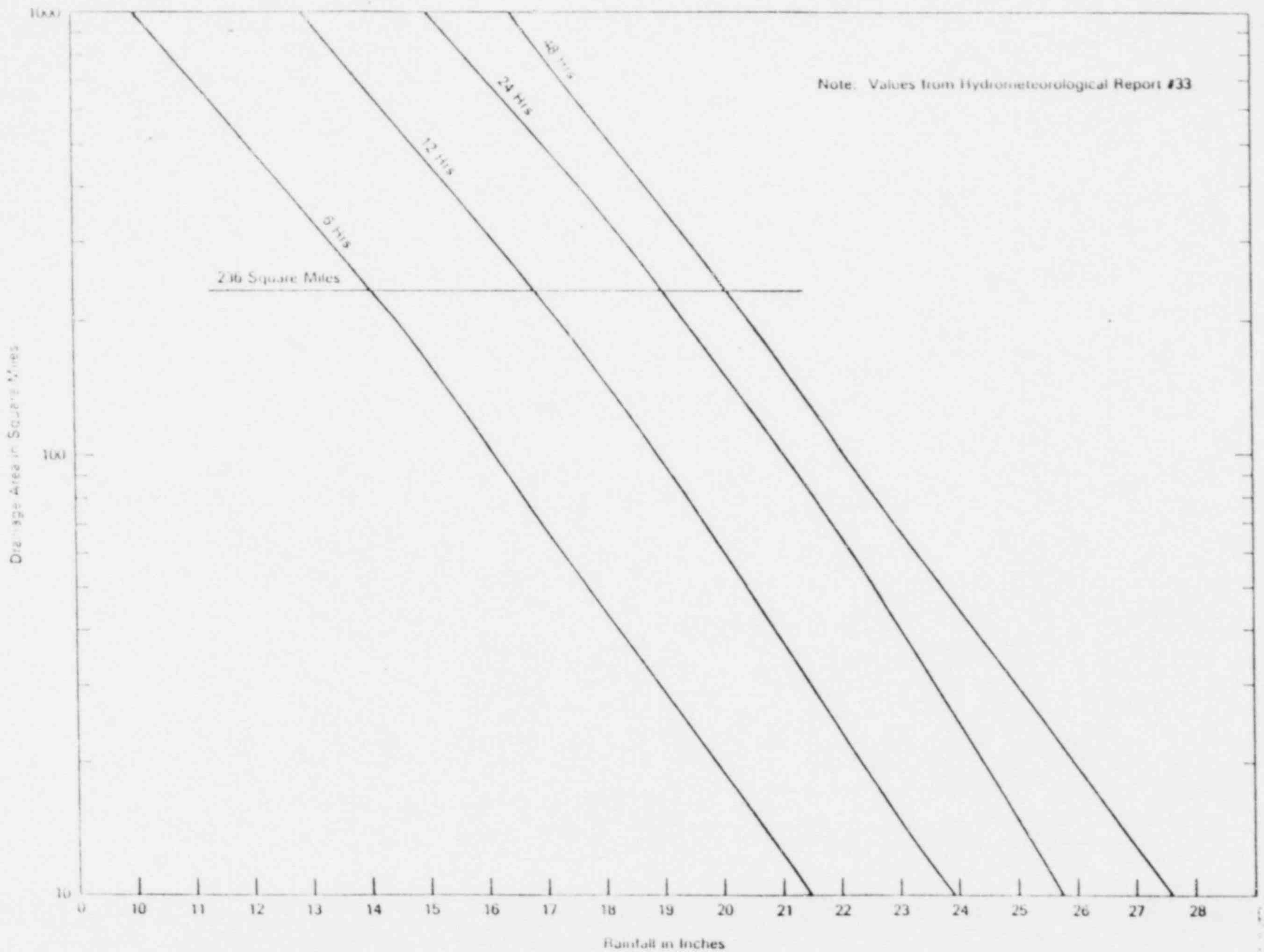

Figure 4.2.1 Rainfall Depth — Area — Duration

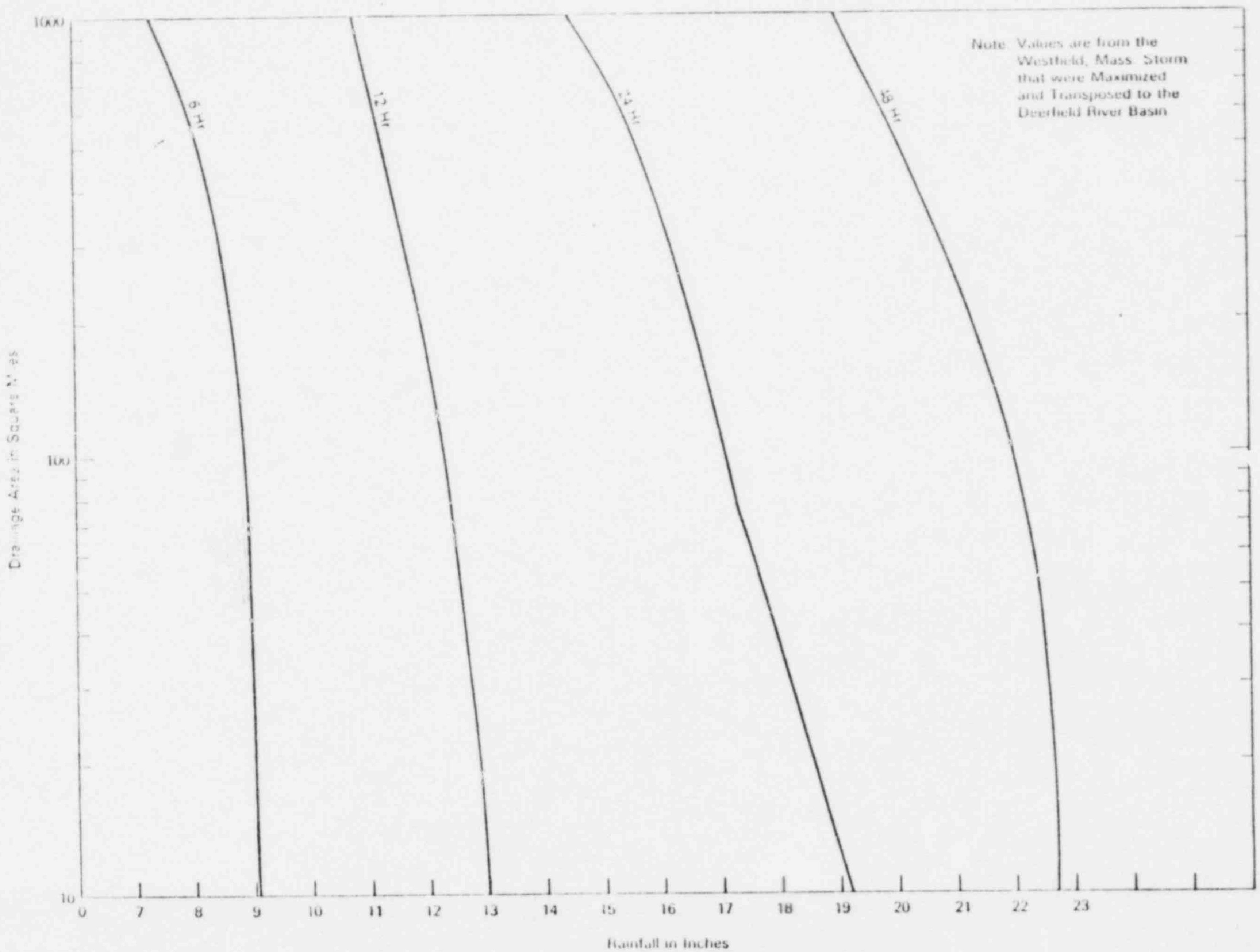

Figure 4.2.2 Rainfall Depth – Area – Duration

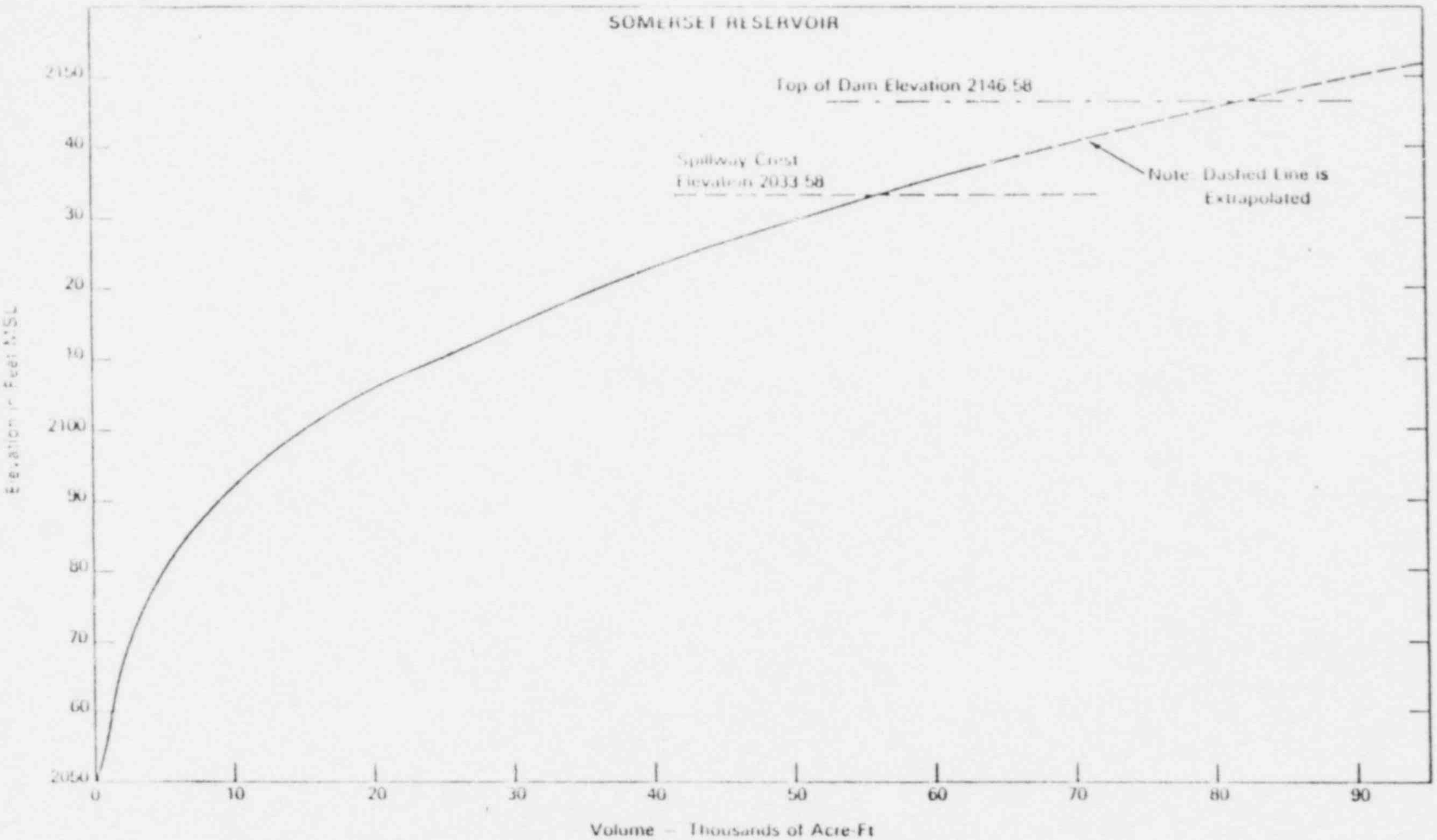

Figure 4.3.1 Somerset Reservoir Capacity Curve

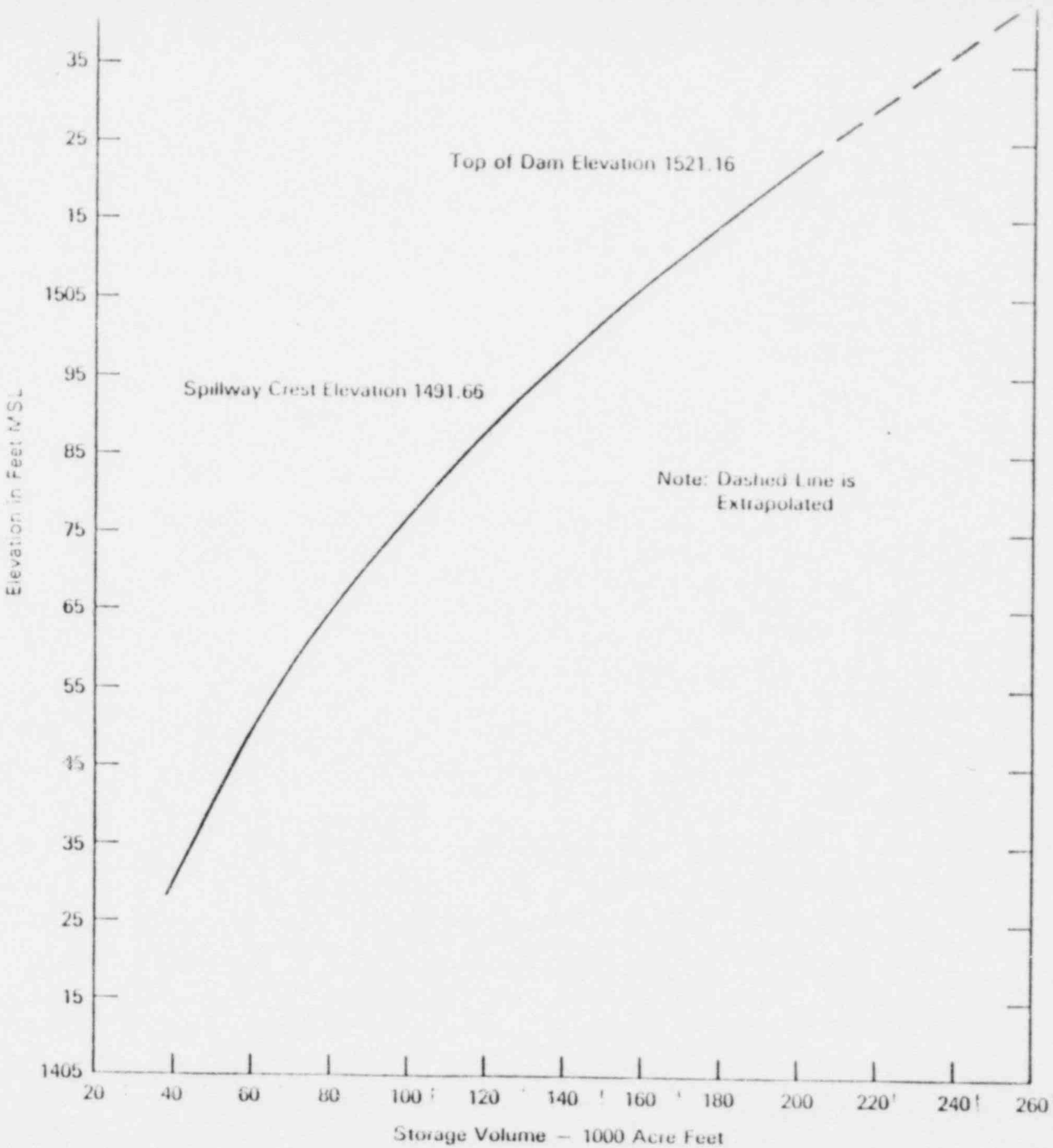

Figure 4.3.2 Harriman Reservoir Capacity Curve

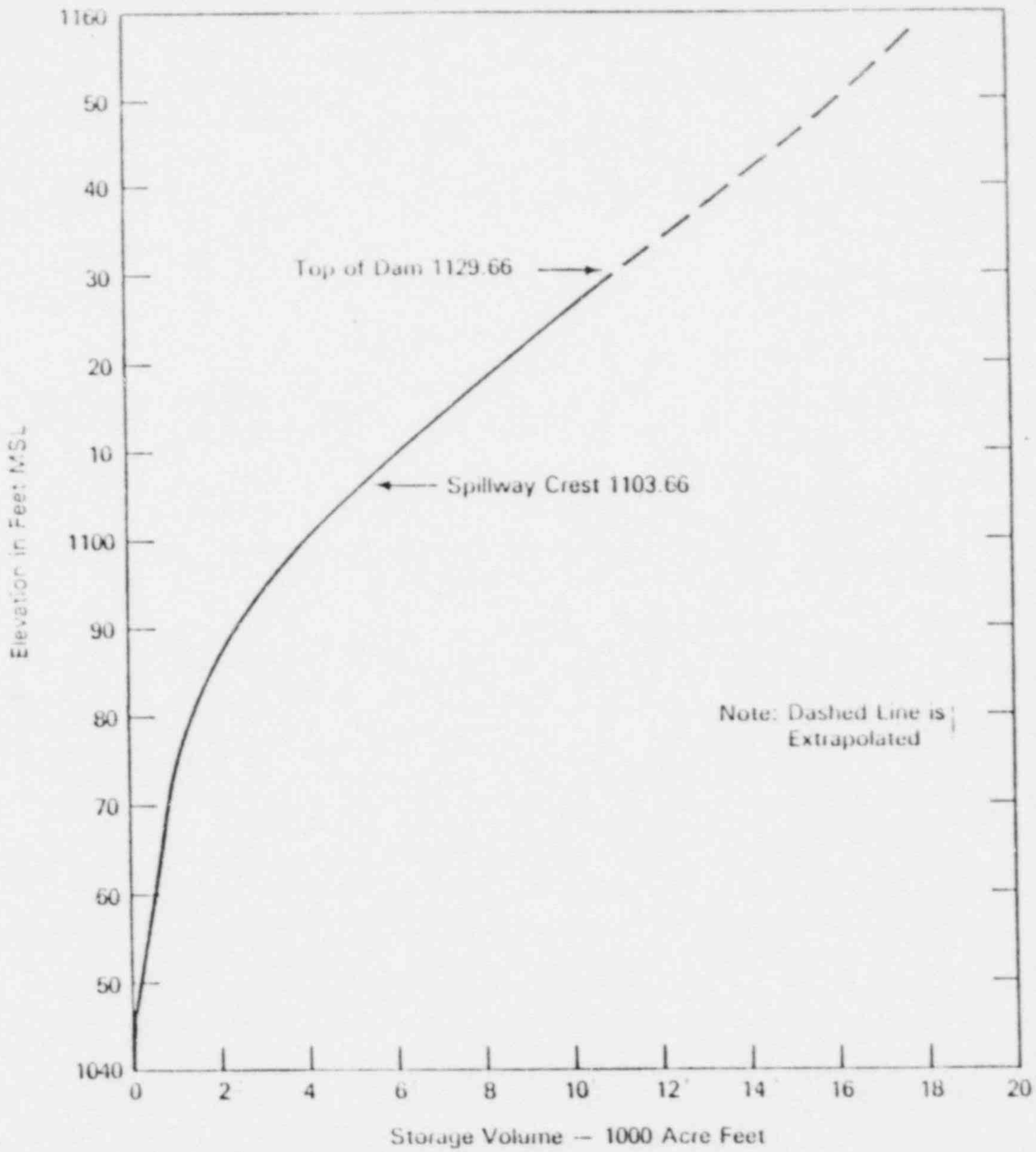

Figure 4.3.3 Sherman Reservoir Capacity Curve

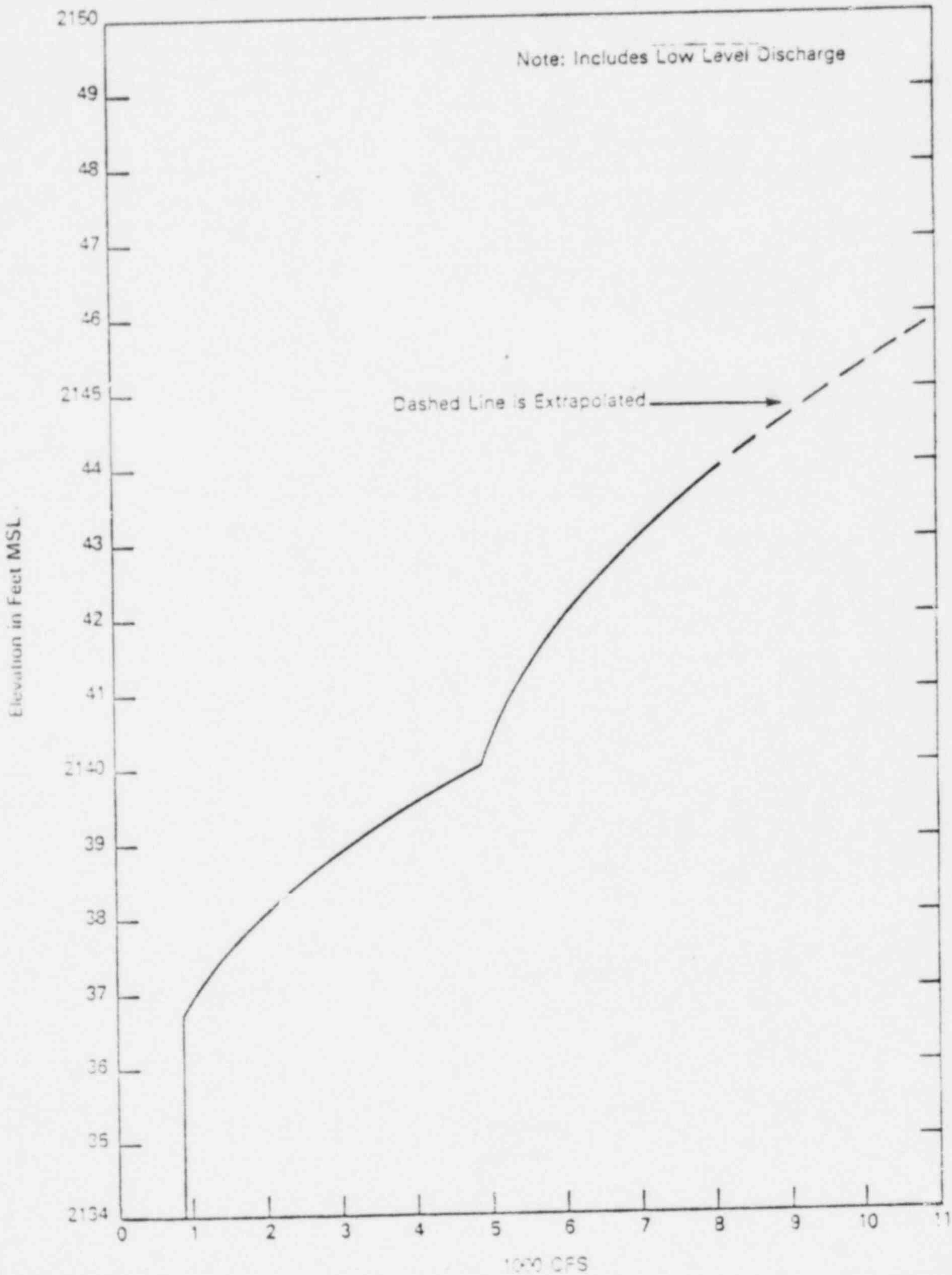

Figure 4.3.4 Somerset Dam Spillway Rating Curve

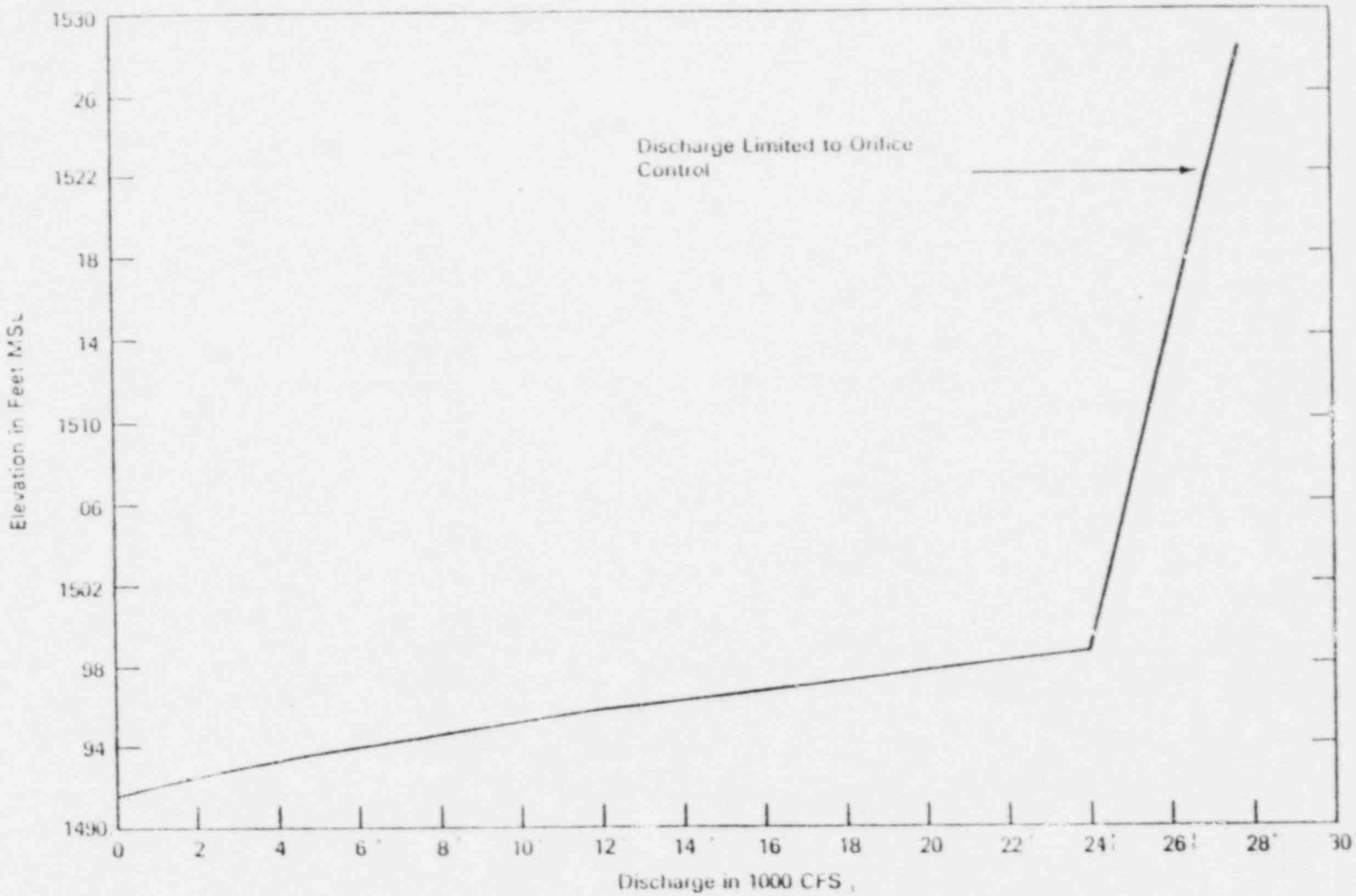

Figure 4.3.5 Harriman Dam Spillway Rating Curve

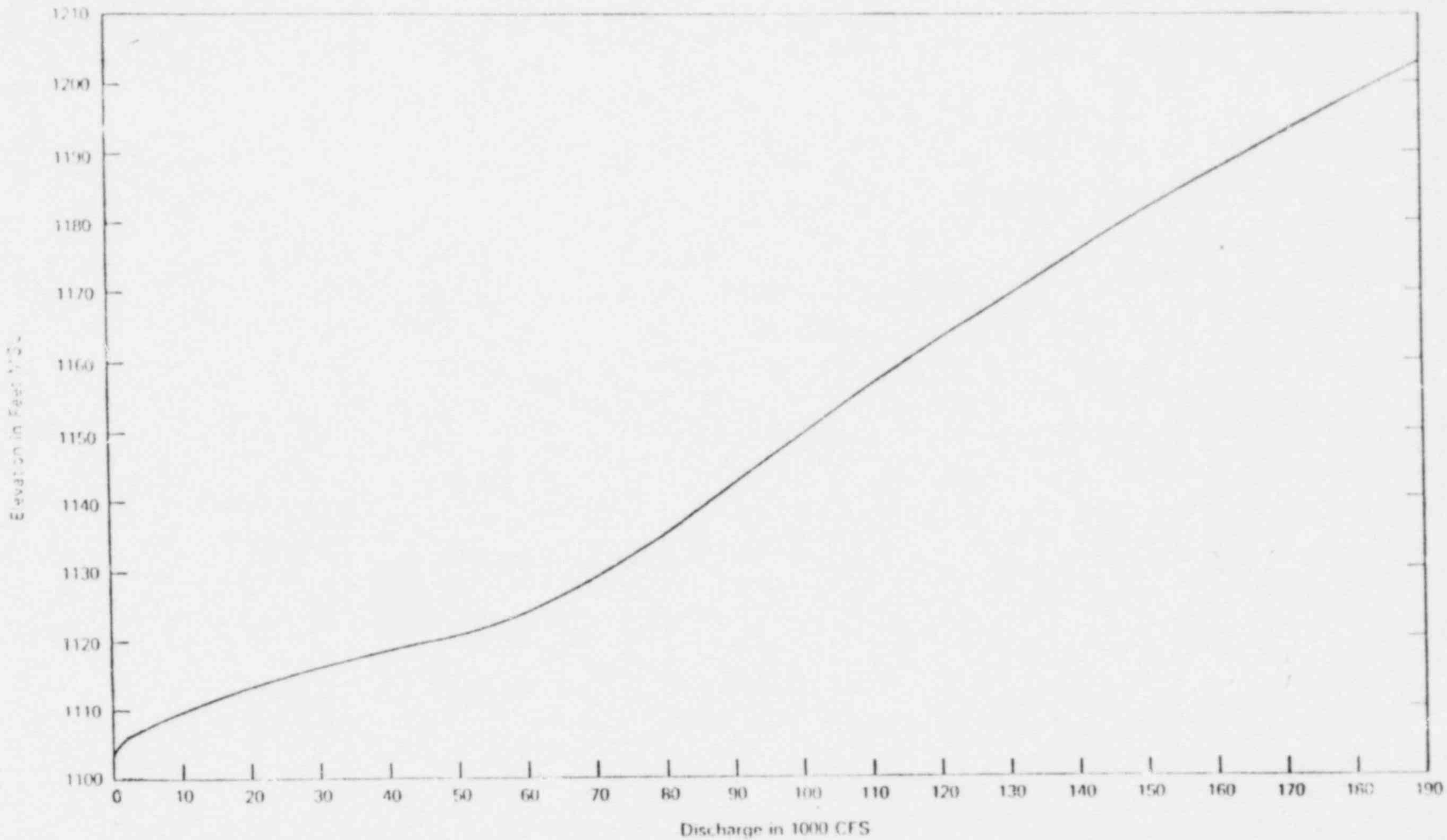

Figure 4.3.6 Sherman Dam Spillway Rating Curve