CONNECTICUT RIVER FLOOD CONTROL

OTTER BROOK
DAM & RESERVOIR

OTTER BROOK, NEW HAMPSHIRE

DESIGN MEMORANDUM NO. 1

HYDROLOGY & HYDRAULIC ANALYSIS

Corps of Engineers, U.S. Army - Office of the Division Engineer

New England Division - Boston, Mass.

JUNE 1955

26

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5	General Design Memorandum	27 May 1955 ** 30 June 1955
6	Relocations	
7	Embankment Design	27 May 1955 ** 30 June 1955
8	Outlet Works	
9	Reservoir Management	

* Initial submission in Draft to secure approval of spillway design flood and outlet requirements, approved 17 March 1955.

** Submission of Preliminary Draft for discussion and review.

OTTER BROOK DAM AND RESERVOIR

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CONNECTICUT RIVER BASIN

NEW HAMPSHIRE

DESIGN MEMORANDUM NO. 1

HYDROLOGY AND HYDRAULIC ANALYSES

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CORPS OF ENGINEERS, U.S. ARMY
OFFICE OF THE DIVISION ENGINEER
NEW ENGLAND DIVISION
BOSTON 15, MASS.

FLOOD CONTROL PROJECT

OTTER BROOK DAM AND RESERVOIR
OTTER BROOK
CONNECTICUT RIVER BASIN
NEW HAMPSHIRE

DESIGN MEMORANDUM NO. 1

HYDROLOGY AND HYDRAULIC ANALYSIS

30 June 1955

A. PURPOSE

1. Purpose. - The purpose of this memorandum is to describe the hydraulic and hydrologic criteria applicable to the design of the Otter Brook Dam and Reservoir on Otter Brook in the Ashuelot River watershed, Connecticut River Basin, New Hampshire. Part I - Hydrology includes sections on climatology, stream flow and spillway and outlet design criteria. Part II - Hydraulic Design includes the analysis and design of hydraulic structures.

PART I - HYDROLOGY

B. BASIN DESCRIPTION

2. Location. - Otter Brook Dam will be located in New Hampshire on Otter Brook, the principal tributary of The Branch, which in turn is a tributary of the Ashuelot River. The dam site is located about 4.9 miles above the confluence of The Branch with the Ashuelot River. The reservoir at spillway crest elevation will extend about 2.3 miles upstream and will be located in the City of Keene and the Town of Roxbury, Cheshire County, New Hampshire; 70 miles air-line distance northwest of Boston, Massachusetts, and 40 miles southwest of Concord, New Hampshire. The general location of the dam site is shown on Plate No. 1-1.

3. Watershed. - The watershed of Otter Brook above the dam site is roughly diamond-shaped, about 12 miles long and with a maximum width of 7.5 miles. The drainage area above the site is 47 square miles. The elevations at the perimeter of the watershed vary from 1,000 to 2,200 feet with an average of about 1,800 feet. The stream pattern is basically "fan-shaped" with a series of brooks located on the steep side slopes, a condition conducive to rapid run-off. In the steep perimeter of the basin, the main stream has a fall of approximately 500 feet in the first 1.5 miles of its headwaters and then has a fairly uniform slope with a fall of 750 feet in the remaining 11 miles to the dam site. A map of the drainage area and a reservoir profile are shown on Plate Nos. 1-2 and 1-3 respectively.

C. CLIMATOLOGY

4. General. - The Ashuelot River Basin has a variable climate characterized by frequent but short periods of heavy precipitation. It lies in the belt of "prevailing westerlies" and consequently is in the path of cyclonic disturbances that cross the country from the west or southwest. The winters are moderately severe with sub zero temperatures quite common. The spring melting of the winter snow cover occurs generally during the month of April. The summer temperatures average between 60° and 70°F with occasional temperatures in the 90's.

5. Temperature. - The mean annual temperature at Keene, New Hampshire, which is located about two miles southwest of the Otter Brook dam site and about 300 feet lower in elevation is 45°F. Freezing temperatures can be expected from the latter part of September through early May. The mean, maximum, and minimum monthly temperatures recorded each month at Keene, New Hampshire, for the 60-year period ending December 1953 are shown in Table 1-I below.

TABLE 1-I

MONTHLY TEMPERATURES AT KEENE, NEW HAMPSHIRE
Elevation 490 ft., m.s.l.
(Degree Fahrenheit)

MONTH	MEAN	MAXIMUM	MINIMUM
January	21.1	66	- 32
February	21.2	64	- 32
March	32.0	85	- 20
April	43.7	91	1
May	55.3	93	21
June	63.6	98	27
July	68.8	104	34
August	66.3	100	27
September	59.4	101	19
October	48.6	88	10
November	36.7	80	- 15
December	24.8	63	- 29
Annual	45.1	104	- 32

6. Precipitation. - The mean annual precipitation at Keene, New Hampshire, for 61 years of record through 1953 is 38.25 inches. The greatest annual precipitation recorded was 51.20 inches in 1951 and the least annual amount was 27.12 inches recorded in 1894. Table 1-II summarizes the precipitation record at Keene, New Hampshire.

TABLE 1-II

MONTHLY PRECIPITATION AT KEENE, NEW HAMPSHIRE
Elevation 490 ft., m.s.l.
(Depth in Inches)

MONTH	MEAN	MAXIMUM	MINIMUM
January	2.89	5.95	.89
February	2.68	7.02	.60
March	3.21	7.60	.04
April	3.09	6.65	.35
May	3.11	7.02	.79
June	3.23	7.73	.41
July	3.80	11.09	1.07
August	3.86	8.96	1.05
September	3.60	10.39	.20
October	2.76	6.67	.23
November	3.03	7.67	.52
December	2.99	6.70	.51
Annual	38.25	51.20	27.12

7. Snow. - Monthly and annual average snowfall for 58 years at Keene, New Hampshire, and 33 years at Fitzwilliam, New Hampshire, located about 16 miles south of the dam site are tabulated in Table 1-III below.

TABLE 1-III

MEAN MONTHLY SNOWFALL
(Average Depth in Inches)

Month	Keene, N.H. Elev. 500 ft., m.s.l.	Fitzwilliam, N.H. Elev. 1350 ft., m.s.l.
January	16.4	17.3
February	16.0	14.8
March	10.0	11.9
April	3.5	5.9
May	Trace	0.4
June	-	-
July	-	-
August	-	-
September	-	-
October	0.1	0.5
November	3.9	5.6
December	10.4	9.0
Annual	60.3	65.4

8. Snow cover reaches a maximum depth in late March with water content averaging more than three inches and ranging from a minimum of about one-half inch to a maximum of about eight inches. The melting snow results in heavy spring run-off but this alone seldom produces damaging floods. However, the possibility of sudden thaws combined with heavy rains creates a potential flood hazard nearly every spring.

9. Eight snow courses have been maintained in the watersheds above the existing flood control dams at Surry Mountain near Keene, New Hampshire, and Edward MacDowell at West Peterborough, New Hampshire, for the past five years (1950 to 1954) (See Plate No. 1-4). Table 1-IV below summarizes the average values of water content in inches on the dates indicated.

TABLE 1-IV

AVERAGE WATER CONTENT OF SNOW COVER
(Inches)

	February 1	March 1	March 15	April 1
Mean	2.4	3.0	3.1	1.2
Maximum	4.2	8.0	8.0	5.1
Minimum	0.9	0.3	0.6	0

10. Storms. - The rapidly moving cyclonic storms or "lows" that travel over or near the Ashuelot River Basin from the west or southwest produce frequent periods of unsettled but not extremely severe weather. The region is also exposed to occasional coastal storms, some of tropical origin, that travel up the Atlantic coast and move inland over New England. The hurricane of September 1938 was of this type. The precipitation which preceded and accompanied this storm was estimated to average 10 inches over the Otter Brook drainage area. The location of centers of this and other outstanding storms of record in the northeastern United States are shown on Plate No. 1-6 together with their area-depth characteristics.

D. RUN-OFF

11. Discharge Records. - Four U.S. Geological Survey gaging stations are located in the Ashuelot River Basin as indicated on Plate No. 1-4. In addition to these stations there is a recording gage located on the Ashuelot River in the tailwater of the Corps of Engineers flood control dam at Surry Mountain. The discharge hydrographs for the period of record (1923-1953) for the gaging station upstream of the proposed dam site on Otter Brook are shown on Plate No. 1-5.

12. Stream Flow Data. - The annual run-off for the 31 years of record through September 1953 for the Otter Brook gage varied from 12.63 inches to 32.93 inches with a mean of 22.77 inches. The mean annual run-off represents about 60 percent of the mean annual precipitation. Table 1-V is a summary of the maximum, minimum and mean monthly run-off in inches and a relation of run-off to precipitation for the period of record at the Otter Brook gaging station.

TABLE 1-V

MONTHLY RUN-OFF

OTTER BROOK NEAR KEENE, NEW HAMPSHIRE
Oct. 1923 - Sept. 1953

Month	Maximum (inches)	Minimum (inches)	Mean (inches)	Relation of Mean Run-off to Mean Precipitation (percent)
January	3.77	.16	1.78	62
February	3.31	.33	1.06	40
March	12.00	.63	3.64	113
April	11.08	1.79	5.71	184
May	5.99	.88	2.75	88
June	4.67	.33	1.45	45
July	3.98	.10	.78	21
August	2.30	.14	.47	12
September	9.00	.05	.83	23
October	2.32	.08	0.66	24
November	5.83	.19	1.58	52
December	4.20	.38	1.71	57
Annual	32.93	12.63	22.77	60

B. HISTORY OF FLOODS

13. General. - Outstanding floods on the Ashuelot River may result from early spring storms combined with melting snow, such as the flood of March 1936 or from summer or fall storms, such as the record flood of September 1938. In addition, local thunderstorms can cause serious flash floods on the smaller streams.

14. Floods of Record. - The storm of September 1938 resulted in the greatest flood of record on Otter Brook. The next largest flood in recent years was that of March 1936 caused by heavy rainfall on top of snow cover with high water content creating a flood equal in volume to the September 1938 flood but with a peak discharge approximately 60% in magnitude. Table 1-VI following includes floods having peak discharges exceeding 3,000 c.f.s. at the U.S.G.S. gage on Otter Brook.

TABLE 1-VI

FLOODS EXCEEDING A PEAK FLOW OF 3,000 C.F.S.
Otter Brook near Keene, N.H. (1923 - 1953)

Date	Discharges
Sept. 21, 1938	6,130 c.f.s.
Mar. 18, 1936	3,580 "
Nov. 26, 1950	3,540 "
Nov. 4, 1927	3,180 "
Apr. 12, 1934	3,020 "

In addition to the floods mentioned above, historical records indicate that the Ashuelot River Basin suffered damage from major floods in January 1841, October 1869 and April 1895. Minor floods were reported in December 1738, March 1801, July 1813, March 1818, February 1824, September 1828, September 1882, July 1897, February 1900 and March 1913.

F. RESERVOIR STORAGE CAPACITY

15. General. - The reservoir will be utilized basically for flood control purposes but a small permanent conservation pool will be maintained to satisfy local desires for recreation. The conservation pool will also serve as a winter pool to facilitate gate operation during the freezing weather. Storage allocations and water surface areas are shown in Table 1-VII below.

TABLE 1-VII

ALLOCATION OF RESERVOIR STORAGE CAPACITY

	Elevation (ft., m.s.l.)	Water Area (acres)	CAPACITY (acre-feet)	CAPACITY (inches)
River bottom at Dam Site	669	0	0	0
Recreation Pool	701	70	700	0.3
Flood Control	781	374	17,600	7.0
Total			18,300	7.3

16. Flood Control Storage. - In general it is considered that six inches of flood control storage provides a reasonably high degree of flood protection in New England. However, due to location of some reservoirs in respect to damage centers, restrictive channel capacities, or other regulation difficulties, it is

often found desirable to provide more than six inches. Regulation experience at Surry Mountain Reservoir during the past 13 years has demonstrated the need for more than six inches of storage for all reservoirs in the Ashuelot River Basin.

17. Otter Brook will be operated in conjunction with Surry Mountain Reservoir for the primary purpose of protecting the downstream community of Keene, New Hampshire. The City of Keene, New Hampshire, is located on the border of an extensive flood plain which during a major flood extends downstream about four miles. Three principal tributaries, the Ashuelot, The Branch (Otter Brook) and the South Branch, discharge into this flood plain from mountainous areas conducive to rapid run-off. The meandering river channel in the flood plain has insufficient discharge capacity due to its small cross-sectional areas and flat gradients, with the result that flood waters cause considerable depth of pondage in the plain.

18. The limited discharge capacity in this flood plain at Keene, New Hampshire, has created the necessity for frequent regulation at Surry Mountain Reservoir and required utilization of storage for relatively minor floods. The channel capacity also affects the time of emptying following reservoir regulation, and the period required to empty has always been longer than desired. A clearing and snagging project in the Ashuelot River below Keene has been completed recently and has improved the discharge capacity, but it is still low in relation to the size of the watershed.

19. Surcharge storage above the spillway is small and will not be very effective in reducing floods. This is illustrated by the fact that five feet of surcharge on the spillway represents storage of only 0.8 inches of run-off. From the experience gained in the regulation of Surry Mountain Reservoir, and on the basis of regulation studies of past major floods it is concluded that the Otter Brook project should provide seven inches of flood control storage below crest of spillway.

G. UNIT HYDROGRAPH ANALYSIS

20. General. - Excellent stream flow records from October 1923 to date are available for the U.S. Geological Survey gaging station located approximately 1.5 miles upstream of the dam site. The gage with a drainage area of 42.3 square miles measures the flow from approximately 90 percent of the drainage area of the Otter Brook Reservoir. Precipitation records are available for stations at Hillsboro, Wendell, Peterborough, Fitzwilliam and at the flood control dams at Surry Mountain near Keene, New Hampshire, and Edward MacDowell near West Peterborough, New Hampshire. (See Plate No. 1-4.)

21. Unit Hydrograph for U.S.G.S. Gage. - After a review of the stream gaging and precipitation records, three major floods (March 1936, September 1938 and November 1950) and three minor floods (November 1943, September 1944 and June 1945) were selected for detailed unit hydrograph analyses. The results of these studies are summarized on Plate No. 1-9. Detailed analyses for the floods of June 1945 and November 1950 are shown on Plate Nos. 1-10 through 1-13. As a unit duration of three hours was considered desirable and applicable for the small drainage area of 42.3 square miles, all the hydrographs were converted to three-hour unit

hydrographs following the method prescribed in Engineering Manual, Civil Works, Part CXIV, Chapter 5, March 1948. The unit hydrograph derived from analysis of the November 1950 flood, which developed from a short intense storm, was found to have the highest peak discharge and shortest time of concentration. It was, therefore, adopted for the unit hydrograph to be used in derivation of the spillway design flood.

22. Unit Hydrograph for the Dam Site. - The difference in drainage areas between the gaging station and the dam site is 4.7 square miles. Most of this area is from one stream called Ferry Brook. Rather than develop a unit hydrograph for this small area, the unit graph for the gaging station was increased by a straight drainage area relationship. As the U. S. G. S. gage is located practically at the upper end of the reservoir, the unit hydrograph represents inflow to a full pool, which eliminates the necessity of considering the effect of valley storage within the reservoir. Hence all hydrographs derived from application of this unit hydrograph are inflow hydrographs to the reservoir and may be routed directly using the gross reservoir storage. The final selected 3-hour unit hydrograph is shown on Plate No. 1-14.

H. SPILLWAY DESIGN FLOOD

23. General. - The spillway design flood represents the most severe condition of run-off that would result from the maximum possible storm precipitation falling on ground saturated from previous rains. Concurrently, it is assumed that the reservoir is filled to spillway crest as a result of previous storms, and that the outlet is closed and inoperative during the flood run-off.

24. Maximum Possible Precipitation. - Values of rainfall for the spillway design flood were obtained from Hydrometeorological Report No. 28 - Generalized Estimate of Maximum Possible Precipitation Over New England and New York, dated March 1952, as prepared by the Hydrometeorological Section, Division of Hydrologic Services, U. S. Weather Bureau. Losses from infiltration, surface detention, transpiration, and other intangible factors were assumed at a rate of 0.2 inches for three hours which is consistent with minimum losses determined in previous studies for the New England area. Rates of precipitation, infiltration and rainfall excess are listed in Table 1-VIII in the chronological order used to compute the spillway design flood inflow. A depth-duration curve for the maximum possible precipitation for the Otter Brook Reservoir is shown on Plate No. 7.

TABLE 1-VIII

MAXIMUM POSSIBLE PRECIPITATION

Time in Hours	Rainfall in Inches	Infiltration in Inches	Rainfall Excess in Inches
0	0		
3	.05	.05	-
6	.05	.05	-
9	.1	.1	-
12	.1	.1	-
15	.2	.2	-
18	.2	.2	-
21	.3	.2	.1
24	.3	.2	.1
27	.4	.2	.2
30	.8	.2	.6
33	3.7	.2	3.5
36	15.0	.2	14.8
39	1.4	.2	1.2
42	.5	.2	.3
45	.4	.2	.2
48	.3	.2	.1
51	.3	.2	.1
54	.2	.2	-
57	.1	.1	-
60	.1	.1	-
63	.1	.1	-
66	.1	.1	-
69	.05	.05	-
72	.05	.05	-
Total	24.8	3.6	21.2

25. Spillway Design Flood Inflow. - The spillway design flood representing an inflow to full reservoir is shown on Plate No. 1-15. It was derived by applying the rainfall excess of Table 1-VIII to the adopted unit hydrograph (Plate No. 1-14 plus an assumed base flow of 100 c.f.s. The peak of the spillway design flood inflow is 38,000 c.f.s. equivalent to 810 c.f.s. per square mile for the drainage area of 47 square miles. This is comparable to the peak value of 36,300 c.f.s. used for the design of the existing Edward MacDowell Reservoir with a drainage area of 44 square miles.

26. Spillway Design Flood Outflow. - Assuming the reservoir full to spillway crest, the design flood described above was routed through the surcharge storage using the capacity curve (Plate No. 1-8) and the spillway rating curve. (Plate No. 1-21). The resulting maximum spillway discharge was 34,500 c.f.s. with a maximum water surface elevation of 797.1 m.s.l. equivalent to 16.1 ft. of surcharge (See Plate No. 1-15).

27. Spillway Design Flood with Snowmelt. - Consideration was given to a spillway design flood which would include run-off from snowmelt. Table 1-V indicates that the month of April is generally the month with the greatest snowmelt run-off. Hydrometeorological Report No. 29 prepared by the U.S. Weather Bureau shows that standard project flood rainfall varies seasonably and that a factor of 60 percent should be applied to the maximum standard storms when used for the month of April. With this same factor the maximum possible rainfall that could occur during the month of April over Otter Brook is approximately 15 inches. Assuming a heavy snow cover in April containing a water content of 12 inches at the start of this storm and concurrent high temperatures, it is conceivable that the volume of run-off from the rainfall and snowmelt would exceed the volume obtained from the summer type storm. However, it is very unlikely that the rate of snowmelt would compensate for the higher rates of precipitation occurring during the spillway design storm (Table 1-VIII) hence the peak of the spring flood would be less than the summer type. Since the surcharge storage in the reservoir is small, the magnitude of the spillway discharge is related to the peak of the inflow rather than the volume. Hence the summer type spillway design flood, as illustrated on Plate No. 1-15 was considered the most critical criterion for determining the required spillway capacity. Furthermore, should a spring-type flood be developed for analysis, it would be logical to assume that, if the heavy snow cover was on the ground, there had been no antecedent run-off and the reservoir would be empty at the start of the spillway design flood. This assumption would further decrease the severity of the spring flood in respect to design requirements of the spillway.

I. TOP OF DAM ELEVATION

28. Freeboard. - The term freeboard as used in this memorandum refers to the difference in elevation between the maximum water surface elevation and the top of the earth embankment constituting the dam. The freeboard is provided to avoid any possibility of water going over the top of the dam by wind or wave action when the spillway is operating at maximum surcharge.

Wind and wave action include vertical wave height, wave ride-up and wind set-up. Maximum wave height was estimated to be 3.3 feet by the formula

$$h = 0.17 \sqrt{VF} + 2.5 - \sqrt[4]{F}$$

in which h = vertical wave height in feet

V = wind velocity, assumed to be 80 miles per hour

F = Fetch, equal to 1.7 statute miles

Wave height plus ride-up on the sloped surface of the dam was computed at 1.4 times wave height alone or 4.6 feet.

Wind set-up was computed to be 0.3 feet by the formula.

$$S = 0.00125 \frac{V^2 F \cos A}{D}$$

in which S = set-up in feet

V & F = same as for wave height

D = mean depth of water taken as 50 feet

A = angle between wind and fetch taken as 0°.

Summarizing the above computations the minimum computed freeboard requirement is 4.9 feet.

29. Selected Elevation, Top of Dam. - The top elevation of the Otter Brook Dam is determined by the following data:

Elevation, crest of spillway	781.0
Maximum spillway surcharge	16.1
Minimum freeboard	4.9
Adopted elevation for top of dam	802.0

J. RESERVOIR OUTLET CAPACITY

30. General. - The capacity of the outlet must be adequate (1) to pass the normal flow of the stream and minor flood rises without imposing restrictions on the right of riparian owners below the dam and without using more than a minor portion of the storage capacity of the reservoir; (2) to pass discharges required for operation of the reservoir during floods; (3) to pass the channel capacity of the stream below the dam with the reservoir only partially filled so as to permit draining of the reservoir within a reasonable time after a flood and (4) to pass a flood of reasonable size during construction without requiring cofferdams of excessive height. As indicated in the following paragraphs a 6' diameter horseshoe shaped conduit with three gates, each 2'-6" by 4'-6", was found to meet the above criteria and was adopted.

31. Channel Capacity. - It is estimated that the channel capacity of Otter Brook below the dam site is approximately 1,000 c.f.s. However, as mentioned in paragraph 17 the operation of the proposed reservoir will be limited to the restricted channel capacity of the Ashuelot River in Keene, New Hampshire, which is also approximately 1,000 c.f.s., but has a contributing drainage area of 310 square miles.

32. Outlet Discharge During Floods. - The Otter Brook Reservoir will be regulated in conjunction with Surry Mountain Reservoir whenever the total flow through Keene, New Hampshire, exceeds the channel capacity of the Ashuelot River. A Telemark gage, presently located at the junction of the Ashuelot River and The Branch will be the chief index point. When necessary, the gates will be closed and remain closed until river stages at the Telemark gage permit their reopening. The gates will also be closed if the main Connecticut River is in flood even though the conditions in Keene do not warrant the closure.

33. Time of Emptying. - Assuming a reservoir outflow of 1,000 c.f.s., equivalent to the channel capacity of Otter Brook, and allowing for an inflow of 200 c.f.s. the total flood control storage could be emptied in approximately 11 days. During the emptying period the regulated discharges from Surry Mountain and Otter Brook reservoirs will be proportional to the total storage utilized by each reservoir and in actual regulation the time to empty a full pool is likely to be longer due to the restricted channel capacity in the Ashuelot River. This is demonstrated on Plate No. 1-17.

34. Stream Diversion During Construction. - The construction schedule requires that a cofferdam be constructed to divert the stream through the outlet during the second construction season. From an examination of the stream flow records for Otter Brook gaging station from October 1923 to September 1953, it was determined that a 10-year flood for the construction season would have a peak rate of inflow of about 2,000 c.f.s. or 43 c.f.s. per square mile. By routing a typical flood of this magnitude through the reservoir storage, it was determined that the maximum pool elevation would be 709.5. The maximum discharge through the 6 foot diameter horseshoe conduit for this pool elevation would be 800 c.f.s.

35. Tailwater Rating Curve. - The tailwater rating curve, shown on Plate No. 1-16, was developed from observations made at a staff gage installed in the vicinity of the proposed stilling basin. Concurrent discharges were obtained from the records of the upstream U. S. Geological Survey's gaging station, and adjusted on a drainage area relationship.

K. EFFECT OF RESERVOIR REGULATION

36. General. - The proposed regulation of the Otter Brook Reservoir as described in paragraph 32 will be primarily for the protection of the City of Keene, New Hampshire. This method of regulation was tested on the two major floods of record and a Standard Project Flood developed for the City of Keene, New Hampshire. These results are described in the following paragraph.

37. Flood of September 1938. - As mentioned in paragraphs 10 and 14, the storm of September 1938 resulted in the greatest flood of record on Otter Brook. This flood was also the maximum of record at Keene, New Hampshire, where the stage equalled elevation 480.6 (Keene datum which is 5.3 feet above M.S.L.). A recurring flood of equal magnitude would be reduced to a maximum elevation of 477.0 with Surry Mountain and Otter Brook Reservoirs. This regulation is shown on Plate No. 1-17. Approximately 95 percent of the storage at both reservoirs would be

utilized and would require about 30 days to empty. This duration of the emptying period is necessary due to the limited channel capacity in Keene where the combined regulated outflow cannot safely exceed 1,000 c.f.s.

38. Although the flood stage at Keene, New Hampshire, is elevation 472.5 on the Telemark gage, significant damage does not begin until the river reaches elevation 474.0. Therefore in a major flood it is anticipated that the releases from the dams will begin as soon as the receding stages at Keene equal 474.0. As shown on Plate No. 1-17 this river elevation occurred at noon on the 23rd of September. As the effect of the regulation would not be reflected on the Connecticut River at Montague City, Massachusetts, until 24 hours later it was safe to begin discharging from the reservoirs at noon on the 23rd of September without contributing to the flood stages at Montague City.

39. Flood of March 1936. - The second largest flood of record at Keene, New Hampshire, was the flood of March 1936 which developed from heavy rains falling on a snow cover with high water content. The effectiveness of the Otter Brook and Surry Mountain reservoirs on a recurring flood of this magnitude is shown on Plate No. 1-18. It should be noted that following the first peak, dropping stages on the Ashuelot River would permit releases from the reservoirs on the 13th at noon but it was necessary to wait until the Connecticut River at Montague City, Massachusetts, had peaked. As a result, the gates at Surry Mountain were opened on the 14th at noon with Otter Brook Reservoir remaining closed. At this time the available storage at Otter Brook was 5.2 inches and at Surry Mountain was 4.1 inches. In accordance with proposed regulation procedure initial releases will come from the reservoir with the least amount of available storage until such time that the available storage in both reservoirs are equal. This condition did not develop in this particular flood since gate closures were required again on the 16th at midnight, at which time the available storage equalled 4.4 inches at Otter Brook and 3.6 inches at Surry Mountain.

40. The second rise in the river caused spillway discharge at Surry Mountain Reservoir, and run-off from a third period of rainfall on the 21st and 22nd of March produced spillway discharge from both reservoirs. All gates were kept closed until the 26th at noon when the combined spillway discharges had receded to a total of 1,000 c.f.s. At that time gate regulation was begun to maintain a total discharge of 1,000 c.f.s. (700 c.f.s. from Surry Mountain and 300 c.f.s. from Otter Brook). Complete control of the discharge by gate operation did not occur until the 28th at noon when the pools receded to spillway crests. An additional forty days was required to empty the reservoirs as the inflows to the reservoirs continued to remain relatively high.

41. Standard Project Flood. - A Standard Project Flood for Keene, New Hampshire, was developed as a demonstration flood to measure the effectiveness of the proposed Otter Brook Reservoir in combination with the existing Surry Mountain Reservoir. Since the flood was to be used for demonstration rather than design purposes it was determined by an approximate method. A rainfall value of 11.3 inches, which equals approximately 60 percent of the maximum possible rainfall for 310 square miles, was selected for the standard project storm. (See Plate Nos. 1-6

and 1-7.) Rather than develop unit hydrographs for the component areas the hydrographs of the September 1938 flood were selected as typical and were adjusted to equal the selected project storm run-off. The Standard Project Flood for Keene, representing the total of the composite inflows to the Keene flood plain, had a maximum discharge of 42,000 c.f.s. which is 35 percent greater than the flood of September 1938, the flood of record.

42. The effect of the proposed Otter Brook Reservoir and the existing Surry Mountain Reservoir on the Standard Project Flood is shown on Plate No. 1-19. Neither reservoir has sufficient capacity to contain the entire flood but the spillway discharges occur on the recession side of the downstream hydrographs and do not contribute to the maximum river stages during the flood. The total stage reduction at Keene, New Hampshire, is 4.5 feet.

43. Frequency of Filling. - Plate No. 1-20 is a curve denoting the estimated frequency of filling of the proposed Otter Brook Reservoir. It was developed from an analysis of stream flow records and the regulation experience of the existing flood control reservoirs. This data was used to determine the Real Estate requirements for land acquisition. Elevation 755, approximately the five-year frequency, has been recommended as the guide taking line for acquisition in fee.

PART II - HYDRAULIC DESIGN

L. SPILLWAY

44. General. - The natural topography at the dam site and the favorable location of rock in the right abutment led to the selection of a chute spillway. The spillway structure consists of a shallow approach channel, a low ogee concrete weir, and a discharge chute excavated in rock. Details of the spillway are shown on Plate No. 5-8 in the General Design Memorandum (Design Memorandum No. 5).

45. Length of Crest and Maximum Surcharge. - Assuming various lengths of spillway from 100 to 300 feet, the spillway design flood was routed through the surcharge storage to determine the effect on surcharge head and corresponding elevation of top of dam. From economic studies it was concluded that a spillway length of 145 feet provided the most practical layout. The reservoir pool for this selected spillway length was elevation 797.1. The spillway design discharge was determined to be 34,500 c.f.s.

46. Approach Channel. - The approach channel to the spillway weir will be approximately 300 feet in length. Bottom of the channel at the weir will be elevation 776, or five feet below spillway crest. The channel will slope one percent towards the reservoir for drainage purposes. Head loss in the approach channel for the spillway design discharge was determined to be 0.3 feet. The maximum velocity in approach channel was 12.3 feet per second.

47. Discharge Rating Curve. - The discharge rating curve, shown on Plate No. 1-21 was determined by the conventional weir formula. The total head on the spillway crest, including the velocity of approach, amounted to 15.8 feet for the design

discharge. The toe of the spillway weir is elevation 775.1 which is 5.9 feet below the crest. During the spillway design discharge there is a small submergence of the weir which slightly affects the coefficient of discharge. The spillway rating curve, which relates discharge to the reservoir pool, also provided for the head loss in the approach channel.

48. Discharge Coefficient. - To effect economy in the design of the spillway, a design head on the crest of 13.0 feet was used, equal to approximately 80 percent of the maximum head. Determination of the spillway coefficients for free flow, shown on Plate No. 1-22, was dependent on the ratio of the five-foot approach depth to the 13-foot design head with application of the relationships shown for a low ogee crest on Plate 6 in Engineering Manual, Part CXVI, Chapter 3. For the maximum energy head of 15.8 feet on the crest, "C" for free flow is 3.96. Modification of the discharge coefficient for free flow was affected by use of Plate 10 of the above Manual which reduced the discharge coefficient due to submergence and the interference of the downstream floor. Relations for the maximum discharge of 34,500 c.f.s., give a decrease in coefficient of discharge modifying the coefficient to 3.80. Similarly, other points of the coefficient curve were determined, except that below a total operating head of nine feet, there is no decrease in "C", and free flow conditions exist.

49. Crest Shape. - The shape of the weir was determined from data for a low ogee crest provided in Plate No. 2-4 of the Manual, and is shown on Plate No. 1-23. Fundamental values in determination of the shape are the maximum head on the crest of 13.4 feet and the head due to velocity of approach of 2.4 feet. With these values, it was possible to determine the radii for curvature of the upstream portion of the ogee crest, 7.22 feet and 3.19 feet, respectively, as well as the formula, $y = .0492x^{1.83}$, for the coordinates of the downstream profile. A bucket of 20-foot radius was selected with point of curvature at the toe elevation 775.1, and tangent to the downstream profile of the chute.

M. SPILLWAY CHUTE

50. General. - The most economical spillway chute which satisfies hydraulic requirements is shown on Plate No. 1-24. It is the result of a number of trials involving variations of toe elevation, bottom slope, angle of convergence, and width of the channel downstream of the contraction. The analyses were based on the spillway design discharge of 34,500 c.f.s. starting from the control section at the spillway weir and working a drop-down curve for determination of velocities and water surface elevations. Consideration was given to designing the contraction to minimize the standing waves in the channel downstream from the contraction, but the design was found to be impractical, both hydraulically and economically.

51. Design Criteria. - Criteria for the adopted design are:

(1) Toe of weir elevation 775.1.

(2) Value of "n" in Manning's formula assumed to be 0.040.

(3) Computed velocity should not show any obvious tendency to decelerate.

(4) Due to infrequency of occurrence of floods of spillway design magnitude and the location of the low point in the rock profile in respect to the dam, it is considered that a minimum freeboard of two feet between the water surface profile and the rock is adequate.

(5) Minimum slope of channel downstream of contraction shall be sufficient to maintain normal flow.

52. Water Surface Profile. - Location of the hydraulic control point on the weir for 34,500 c.f.s. for starting the drop-down curve was determined by computing the minimum energy head, 18.1 feet, equal to the critical depth 12.1 feet, plus the corresponding velocity head. The minimum energy head was subtracted from the energy gradient elevation 796.8 at the weir, noted on Plate No. 1-23, to obtain the invert elevation of the control point. This point is a short distance downstream from the ogee weir crest, at approximately station 5 + 08, where the invert elevation is 778.7, and the water surface is elevation 790.8.

53. Several different chutes were assumed for hydraulic and economic analyses. The invert slopes were varied, different widths of the channel below the contraction were assumed between 50 and 75 feet, and the length of the contraction was varied considerably. Calculations of water surface, velocity and energy gradient were made in the spillway chute at sections approximately 200 feet apart. Contraction head losses due to turbulence equivalent to 10 percent of the difference in velocity heads between reaches were applied. The slope downstream of the transition was computed by Manning's formula and is adequate to produce the same velocity and depth of water obtained at end of the contraction.

54. Selected Chute. - The selected chute has a long contraction, varying from 142 feet wide at the toe of the weir at station 5 + 18.7 to a minimum of 60 feet at station 11 + 00. Slope of the chute in this section will be a constant 8.2 percent and will accelerate the water from 29.5 feet per second at the toe of the weir, to 37.8 feet per second at the narrow section of the contraction. Downstream of this section the channel width of 60 feet is maintained and the slope is reduced to 4.7 percent, which is sufficient to maintain normal flow. The selected slope and degree of contraction will provide the stipulated two feet of freeboard below the minimum rock elevation as indicated on the profile. During construction, when the rock is uncovered, the freeboard will be checked and, if necessary, adjustments in the profile will be made.

N. OUTLETS

55. Design Criteria. - The outlet works are illustrated on Plate No. 5-6 of the General Design Memorandum (Design Memorandum No. 5). To satisfy criteria in paragraphs 30-34, the controlling factor in the design of the outlet is a discharge capacity of 1,000 c.f.s. at approximately elevation 725, equivalent to about 15 percent of the flood control storage available above elevation 701. This requirement also provides adequate discharge capacity needed for diversion through the

conduit during construction without an excessive height of cofferdam. (Refer to paragraph 35.) Three gates are desirable to give flexibility in operation, so that in case of the failure of one gate there would still be sufficient discharge capacity. It was considered that the area of these gates should exceed the conduit cross sectional area by 10 to 15 percent.

56. Outlet and Gates. - A horseshoe-shaped conduit, 540.5 feet in length with a diameter of six feet satisfied the above discharge criteria. The total cross-sectional area of the three 2'-6" by 4'-6" sluice gates is 33.75 square feet, or 13 percent larger than the cross-sectional area of 29.85 square feet for the conduit. With one of the three gates inoperative, the discharge through two gates is 92 percent of the maximum discharge with three gates open.

57. Invert Elevations. - The invert of the conduit from the intake to a point immediately downstream of the gates is level at elevation 683.0. The bottom of the transition consists of a reverse curve and drops from elevation 683.0 to 681.5. The vertical curve was checked with the discharge trajectory based on maximum velocity of flow plus 25 percent. The horseshoe conduit, which begins at the end of the transition, slopes 0.833 percent from elevation 681.5 to 677.0 at the portal.

58. Conservation Weir. - The conduit intake has been designed to satisfy the requirements of a small conservation pool with a water surface elevation of 701, or 18 feet above the gate-sill. Plate No. 5-7 of the General Design Memorandum shows the intake structure. A concrete weir and stop logs having a total effective length of 34'-4" in the five stop log openings will be constructed at the entrance to the center gate. The top of stop logs will be adjustable to hold the conservation pool at the desired elevation during normal range of summer flows. For high flows the outside gates can be opened if necessary. Augmenting the weir discharge during the summer months, is a 6-inch pipe through the conservation weir at an elevation just above the floor elevation 683. This pipe will permit cooler water to be drawn from the bottom of the reservoir to satisfy fishing interests.

59. Intake. - The walls of the approach channel and conservation weir suppress the approach flow to each gate passage eliminating the need of elliptical side curves in the conduit entrance. The intake roof curve of each of the three passageways is based on the equation

$$\frac{x^2}{D^2} + \frac{y^2}{\left(\frac{2D}{3}\right)^2} = 1$$ where D is 4'-6", the vertical dimension of each passageway.

60. Air Vent. - Three 6" x 2'-6" air vents located immediately downstream from each sluice gate will tap a 24-inch header, in turn connected to a 24-inch riser. Entrance to the 24-inch riser is located in the freeboard range above maximum surcharge elevation in an opening in the wall of the intake structure. The size of the air vent is based on the design curve of "Air Demand", Plate No. 4 of the Manual, Part CXVI, Chapter 2, February 1953. The air requirement of the outlet conduit was determined from the design curve by assuming that maximum demand occurs at a gate opening of 80 percent. The Froude number was based on the velocity and depth at the vena contracta, with the velocity based on maximum

reservoir head, and with the depth based on a gate contraction coefficient of 0.80 for a rounded gate lip. Air demand was computed as 140 cubic feet per second. Using a criteria of 150 feet per second as the maximum air vent velocity the required cross-sectional area is 135 square inches while the selected 6" x 2'-6" air vent size provides 180 square inches. The 24-inch diameter supply main is approximately equal to the cross-sectional area of the three 6" x 2'-6" vents. With the air vent designed for the maximum velocity of 150 feet per second, the pressure drop in the conduit, or head loss in the vents, will equal 1.2 feet of water.

61. Conduit Characteristics. - Hydraulic elements of the 6-foot horseshoe-shaped conduit are shown on Plate No. 1-25. The illustrated characteristics include the shape, area, hydraulic radius, discharge at both normal and critical depth, and velocity. The velocity and discharge for normal conditions were based on Manning's formula with "n" of 0.012 and the invert slope of 0.833 percent.

62. Outlet Rating Curves. - Although comparison between discharge at normal and critical depth for the conduit flowing partly full is shown, in which critical flow is the limiting condition up to a depth of 5.5 feet, in the final analysis this relation was not utilized in determination of the outlet rating curves of Plate No. 1-26. The control was found to be at the end of the piers in the gate passageway for conduit flowing partly full. When flowing under pressure the hydraulic control with one gate open for any pool elevation is at the end of the piers. It is also at the end of the piers for two gates fully open up to approximate pool elevation 705. Due to the effect of the conservation weir, the hydraulic control stays at the end of the piers, with three gates fully open, to approximately pool elevation 703. When the pool is above these elevations for the two and three gates fully open, computations indicated that the hydraulic control shifts to the conduit portal.

63. On Plate No. 1-26 are shown the outlet rating curves, and the conservation weir rating with transition of flow from weir discharge to one gate opening. When the flow over the weir exceeds approximately two feet, equivalent to a discharge of approximately 300 c.f.s., the hydraulic control shifts from the weir to the gate opening.

64. Analysis of Flow Conditions. - Plate No. 1-27 shows water surface, pressure, and energy gradient profiles for two conditions of flow; namely, for 1,000 c.f.s., which is the downstream channel capacity discharge, and for 1,500 c.f.s., which is the capacity of the conduit with the three sluice gates fully open. The minimum reservoir pool required to discharge 1,000 c.f.s. with the three gates fully open is elevation 725, while for 1,500 c.f.s. discharge, the pool is at spillway crest elevation 781. A regulated discharge of 1,000 c.f.s. can be maintained when the pool elevation is above elevation 725 by throttling the sluice gate openings. For structural design the greatest static pressure in the conduit downstream of the gates will exist where the pressure gradient is elevation 748 for the 1,500 c.f.s. flow. Upstream of the gates the highest pressure would be for the static condition with gates closed and water surface at maximum spillway surcharge at elevation 797.1. With the gates wide open and with a discharge of 1,500 c.f.s., the pressure gradient varies from elevation 748 at the gates to 680 at the portal.

O. OUTLET STILLING BASIN

65. Design Criteria. - The stilling basin design is illustrated on Plate No. 1-28. Inasmuch as 1,000 c.f.s. will be the maximum regulated discharge, the stilling basin is designed for this condition in accordance with the criteria described in the Engineering Manual. With the conduit friction coefficient reduced to 0.010, the conduit would discharge 1,670 c.f.s. with reservoir pool at spillway crest. Although this discharge would be permitted only during emergency circumstances, the stilling basin is also designed to produce a hydraulic jump for this higher flow, but using less rigorous design criteria. The following summarizes the minimum criteria adopted for the design of the stilling basin.

	1000 c.f.s.	1670 c.f.s.
Trajectory	1.25 portal vel.	0.90 portal vel.
Flare	1 on 5, to 1 on 6	same
Tailwater depth	0.90 D_2	0.75 D_2
Basin length	3 D_2	$2\frac{1}{4}$ D_2
Basin width	25 ft.	25 ft.

66. Trajectory. - To satisfy wall flare criteria, tailwater depth, and selected basin width, it was necessary to provide a trajectory curve that satisfies the theoretical trajectory for 1,670 c.f.s. The 0.833 percent slope of the conduit introduces a small value in addition to the usual trajectory formula. Coordinates of the trajectory, therefore, are based on the expression:

$$y = .00833X + .00513X^2$$

Fillets are required between the portal of the horseshoe conduit and the downstream rectangular section. The fillets start at the portal at elevation 677.5 and extend 15 feet. Coordinates of the trajectory for the fillets are based on:

$$y = .00791X^2$$

67. Flare. - The horizontal distance to satisfy the trajectory for 1,670 c.f.s. and the selected basin width of 25 feet resulted in a flare of 1 to 5.53. Based on results of hydraulic model studies for other projects it was considered this flare was adequate.

68. Stilling Basin Floor. - The elevation of the stilling basin floor was based on the 0.75 D_2 for the discharge of 1,670 c.f.s. The theoretical D_2 is 15.8 and reduced to 0.75 D_2 is 11.8 feet. Tailwater for this discharge is elevation 670.3 which establishes the floor grade at elevation 658.5. As the tailwater

rating curve has been extrapolated in this discharge range, the floor of the stilling basin was set at elevation 658.0. The theoretical D_2 for 1,000 c.f.s. is 10.4 feet. With tailwater at elevation 669.2 the actual D_2 provided for this discharge is more than adequate.

69. Length of Stilling Basin. - The length of the stilling basin, 35 feet, was selected to satisfy both the 1,000 and 1,670 c.f.s. discharge conditions. For 1,000 c.f.s. the length is about 3.4 D_2, and for 1,670 c.f.s. the length is 2.2 D_2. Considering the ledge rock to be left at the end of the basin, the rockfill to be provided at the left side of the channel, and the natural stony nature of the streambed, this selected length is adequate.

70. Structural Detail. - Two rows of baffles and an end sill will be provided. The baffles will be staggered, and will be 2.5 feet high. The end sill will be at elevation 662.0 which is 4.0 feet above the stilling basin floor. The top of walls in the flare section and the stilling basin will provide 3.0 feet and 3.7 feet of freeboard, respectively.

71. Analysis of Flow Conditions. - Velocities for the discharge of 1,670 c.f.s. are 56.0 feet per second at the portal, 63.7 feet per second at the beginning of the hydraulic jump, and 7.2 feet per second at the end sill. For the regulated discharge of 1,000 c.f.s., similar velocities are 33.5, 48.2, and 5.5 feet per second, respectively.

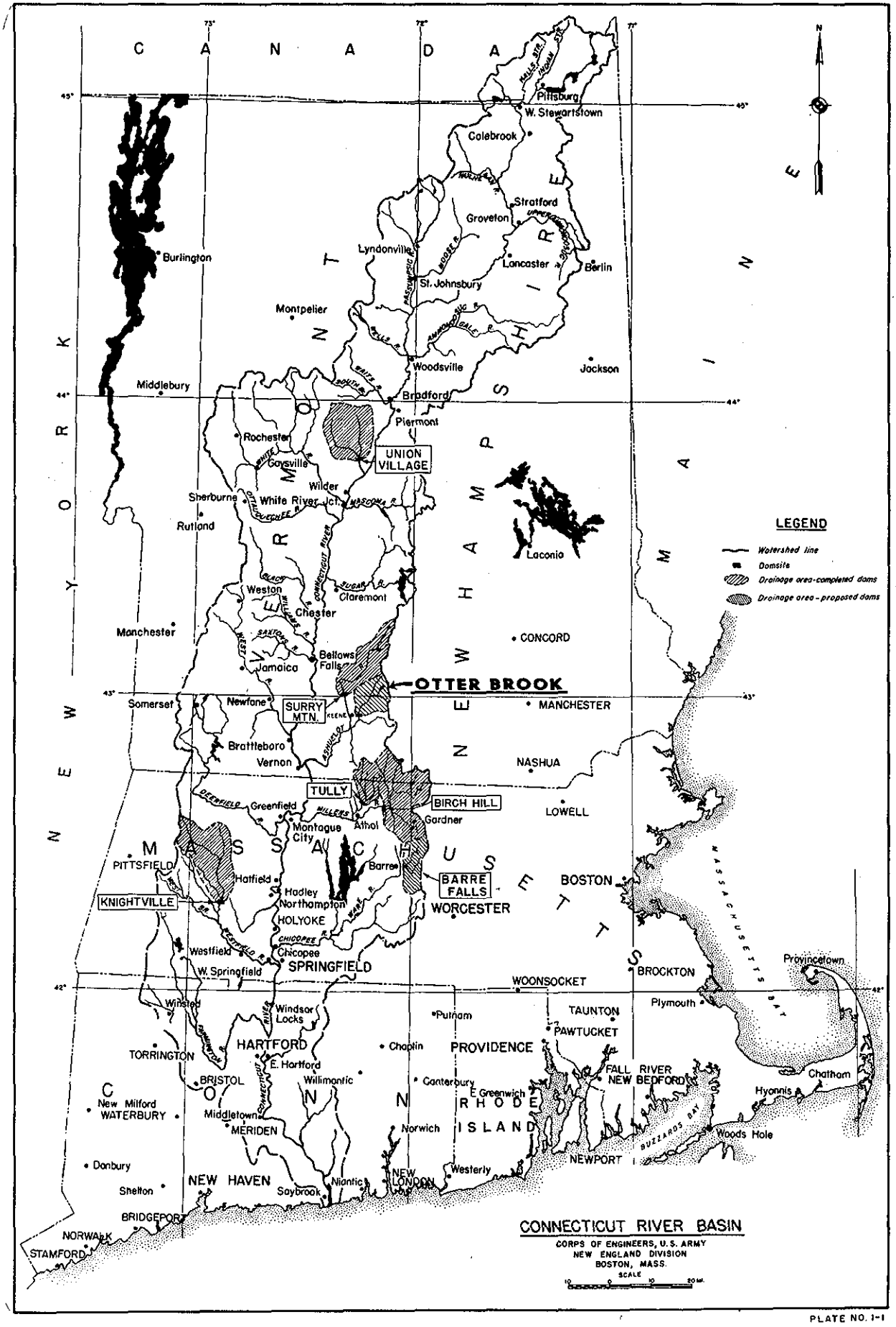
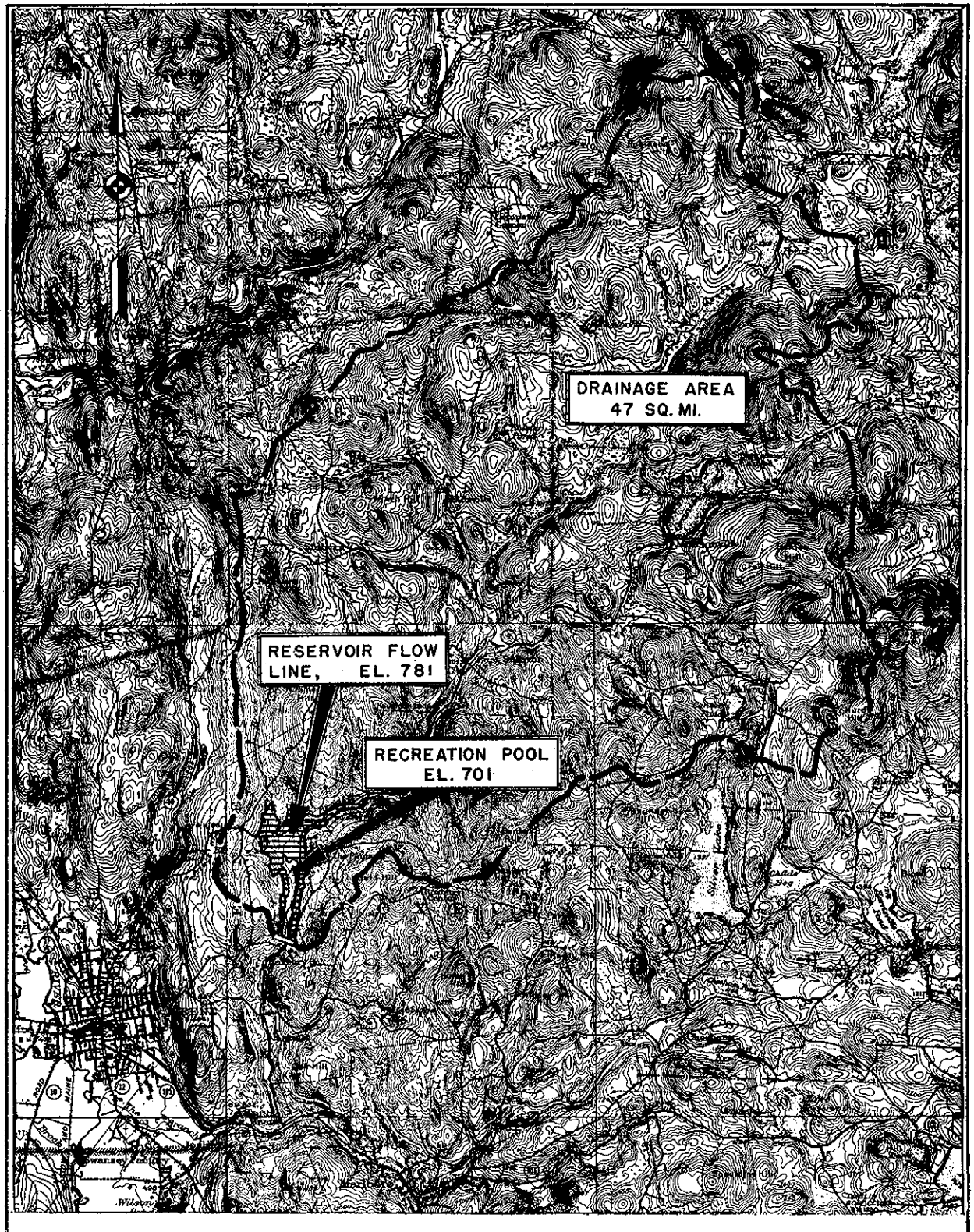

CANADA
NEW YORK
NEW HAMPSHIRE
MAINE
VERMONT
MASSACHUSETTS
CONNECTICUT
RHODE ISLAND
Pittsburg
W. Stewartstown
Colebrook
Stratford
Groveton
Lyndonville
Lancaster
Berlin
Burlington
St. Johnsburg
Montpelier
Woodsville
Jackson
Middlebury
Bradford
Piermont
Rochester
Gaysville
UNION VILLAGE
Wilder
Sherburne
White River Jct.
Rutland
Laconia
Weston
Claremont
Chester
Manchester
CONCORD
Jamaica
Bellows Falls
OTTER BROOK
Somerset
Newfane
SURRY MTN.
MANCHESTER
Brattleboro
Vernon
NASHUA
TULLY
BIRCH HILL
Greenfield
Montague City
Athol
Gardner
LOWELL
PITTSFIELD
Hatfield
Barre
BARRE FALLS
BOSTON
KNIGHTVILLE
Hadley
Northampton
WORCESTER
HOLYOKE
Westfield
Chicopee
W. Springfield
SPRINGFIELD
BROCKTON
Provincetown
WOONSOCKET
Plymouth
Winsted
Windsor Locks
Putnam
TAUNTON
PAWTUCKET
HARTFORD
Chaplin
PROVIDENCE
TORRINGTON
E. Hartford
FALL RIVER
NEW BEDFORD
BRISTOL
Willimantic
Canterbury
E. Greenwich
Chatham
Hyannis
New Milford
WATERBURY
Middletown
MERIDEN
Norwich
Woods Hole
Danbury
NEWPORT
Westerly
Shelton
NEW HAVEN
Saybrook
Niantic
NEW LONDON
BRIDGEPORT
NORWALK
STAMFORD
LEGEND
Watershed line
Damsite
Drainage area-completed dams
Drainage area-proposed dams
CONNECTICUT RIVER BASIN
CORPS OF ENGINEERS, U.S. ARMY
NEW ENGLAND DIVISION
BOSTON, MASS.
SCALE

PLATE NO. I-1



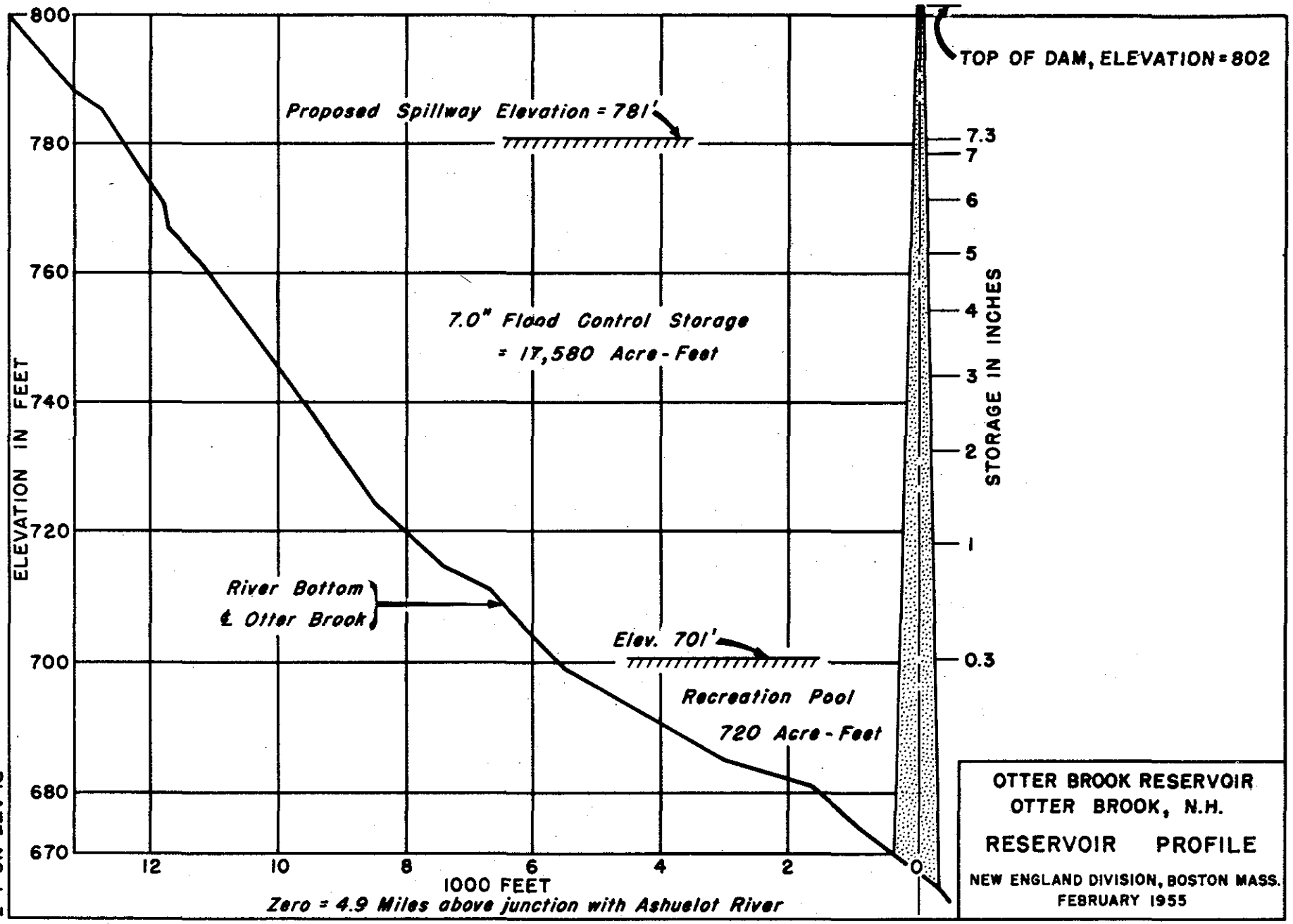
CONNECTICUT RIVER FLOOD CONTROL
OTTER BROOK RESERVOIR
OTTER BROOK, N. H.

DRAINAGE AREA

SCALE IN MILES

1 0 2

PLATE NO. 1-2



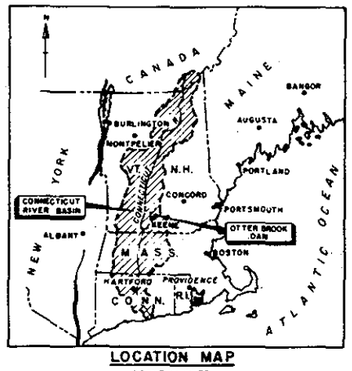
OTTER BROOK RESERVOIR
OTTER BROOK, N.H.

RESERVOIR PROFILE

NEW ENGLAND DIVISION, BOSTON MASS.
FEBRUARY 1955

PLATE NO. I-3

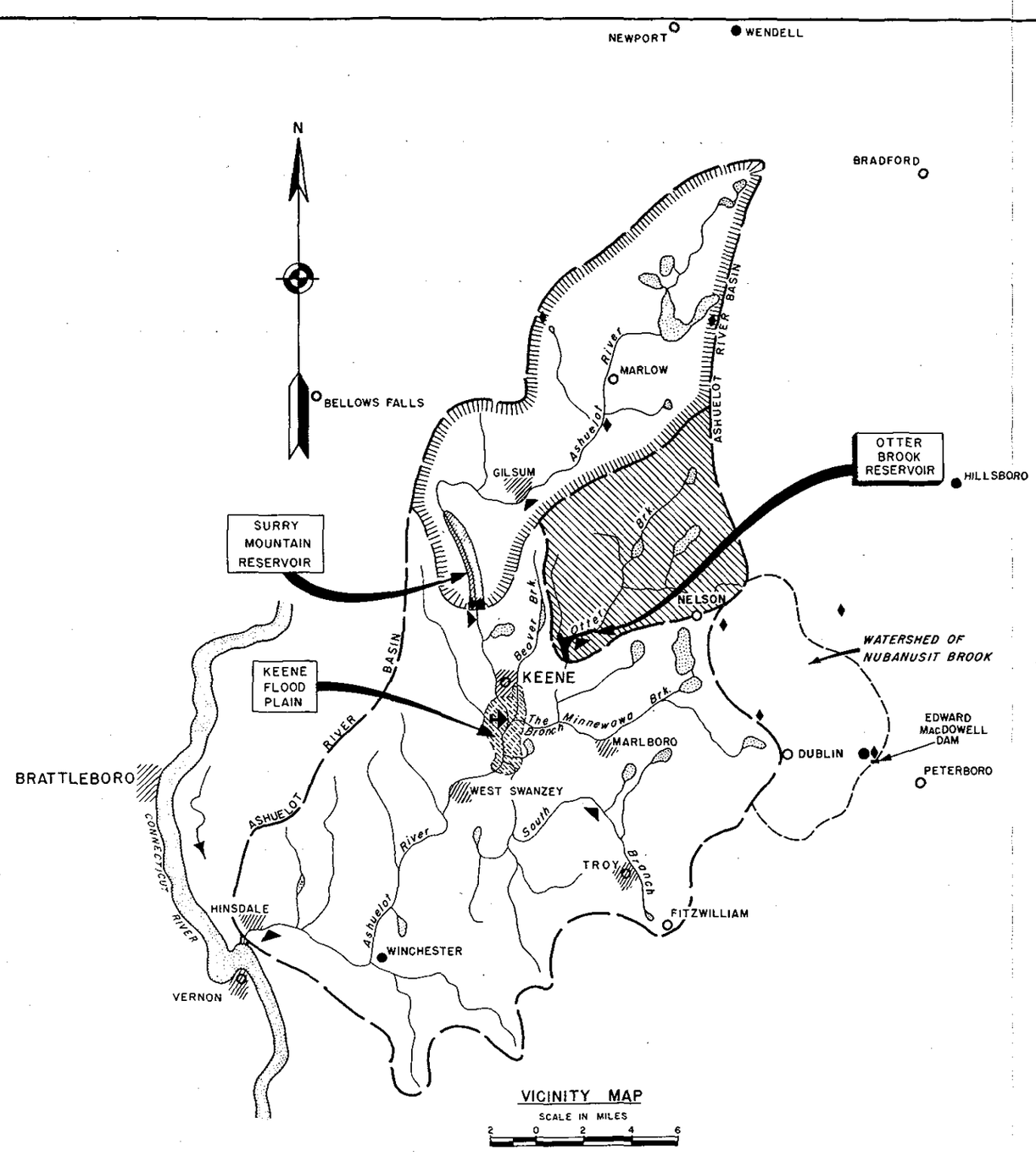
CORPS OF ENGINEERS U. S. ARMY



LOCATION MAP

SCALE IN MILES

40 0 40 80



VICINITY MAP

SCALE IN MILES

2 0 2 4 6

LEGEND

- TELEMARK GAGE
- U. S. G. S. GAGE
- RECORDING RAINFALL GAGE
- NON-RECORDING RAINFALL GAGE
- SNOW SURVEY COURSE

REVISION	DATE	DESCRIPTION	BY

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DIVISION ENGINEER
NEW ENGLAND DIVISION
BOSTON MASS

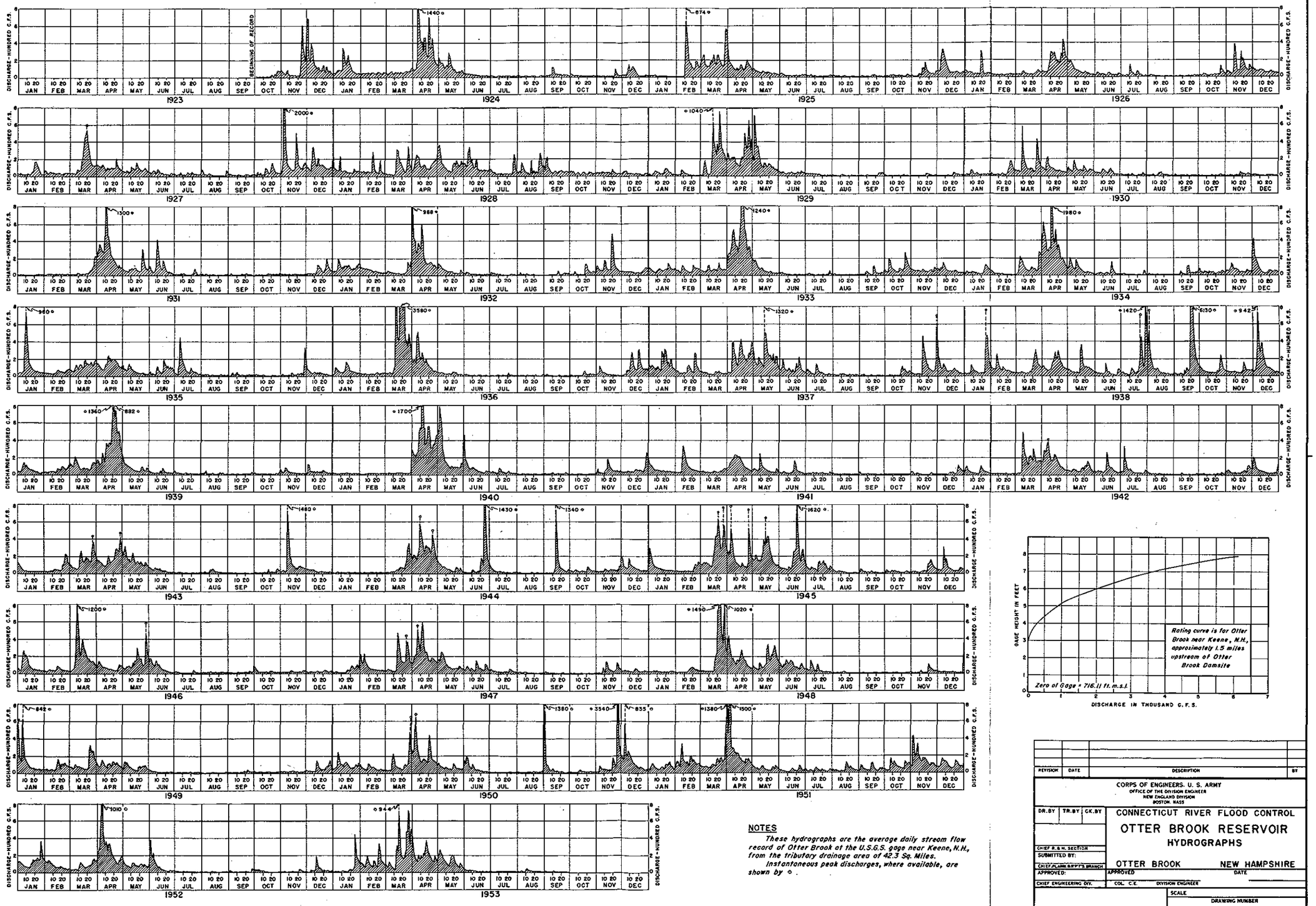
DR. BY | TR. BY | CK. BY

CONNECTICUT RIVER FLOOD CONTROL

OTTER BROOK DAM
PRECIPITATION AND STREAM GAGING LOCATIONS
OTTER BROOK, NEW HAMPSHIRE

CHIEF R & H SECTION
SUBMITTED BY
CHIEF, PLANN. & RPT'S BRANCH
APPROVED
CHIEF ENGINEERING DIV.
APPROVED
COL. C.E. DIVISION ENGINEER
DATE
SCALE
DRAWING NUMBER
SHEET OF

PLATE NO. 1-4



NOTES

These hydrographs are the average daily stream flow record of Otter Brook at the U.S.G.S. gage near Keene, N.H., from the tributary drainage area of 42.3 Sq. Miles.

Instantaneous peak discharges, where available, are shown by ◇.

REVISION	DATE	DESCRIPTION	BY

CORPS OF ENGINEERS. U. S. ARMY
OFFICE OF THE DIVISION ENGINEER
NEW ENGLAND DIVISION
BOSTON, MASS

DR. BY | TR. BY | CK. BY

CHIEF R.&H. SECTION
SUBMITTED BY:
CHIEF PLANN. & RPT'S. BRANCH
APPROVED:
CHIEF ENGINEERING DIV.

CONNECTICUT RIVER FLOOD CONTROL

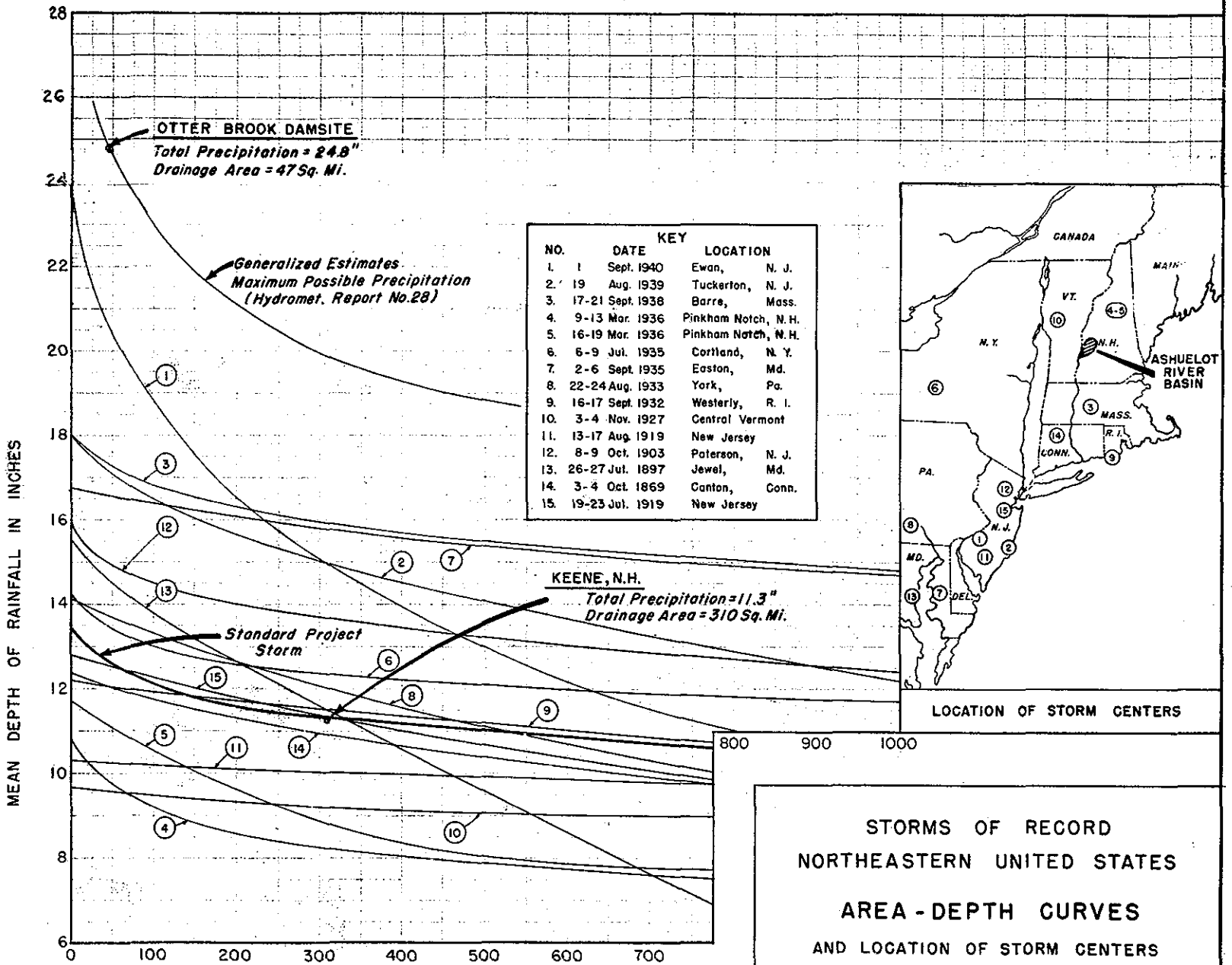
OTTER BROOK RESERVOIR

HYDROGRAPHS

OTTER BROOK NEW HAMPSHIRE

APPROVED DATE
COL. C.E. DIVISION ENGINEER

SCALE
DRAWING NUMBER
SHEET OF



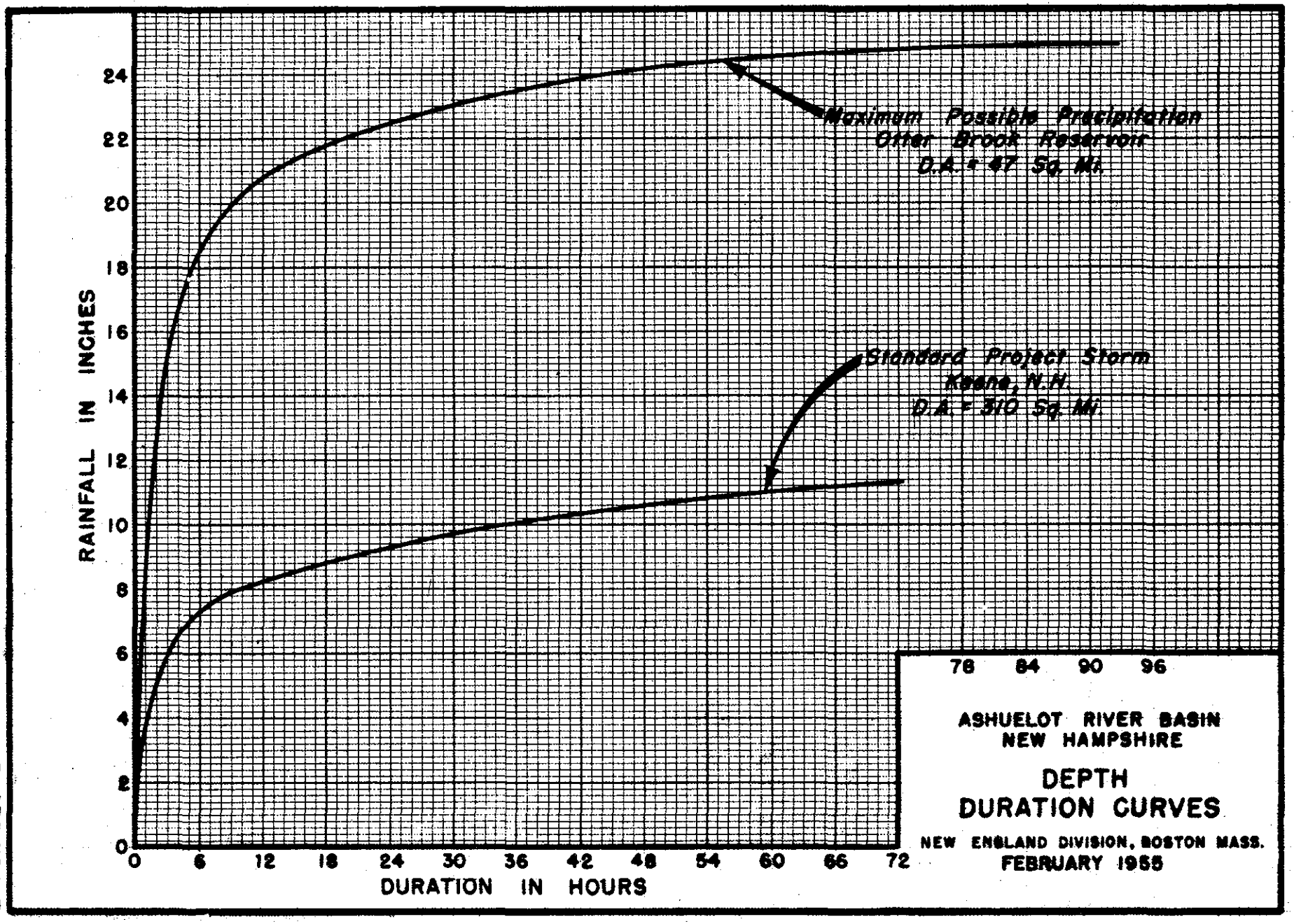
STORMS OF RECORD
NORTHEASTERN UNITED STATES

AREA - DEPTH CURVES

AND LOCATION OF STORM CENTERS

NEW ENGLAND DIVISION BOSTON, MASS.
FEBRUARY 1955

PLATE NO. I-6



ASHUELOT RIVER BASIN
NEW HAMPSHIRE

DEPTH
DURATION CURVES

NEW ENGLAND DIVISION, BOSTON MASS.
FEBRUARY 1955

PLATE NO. I-7

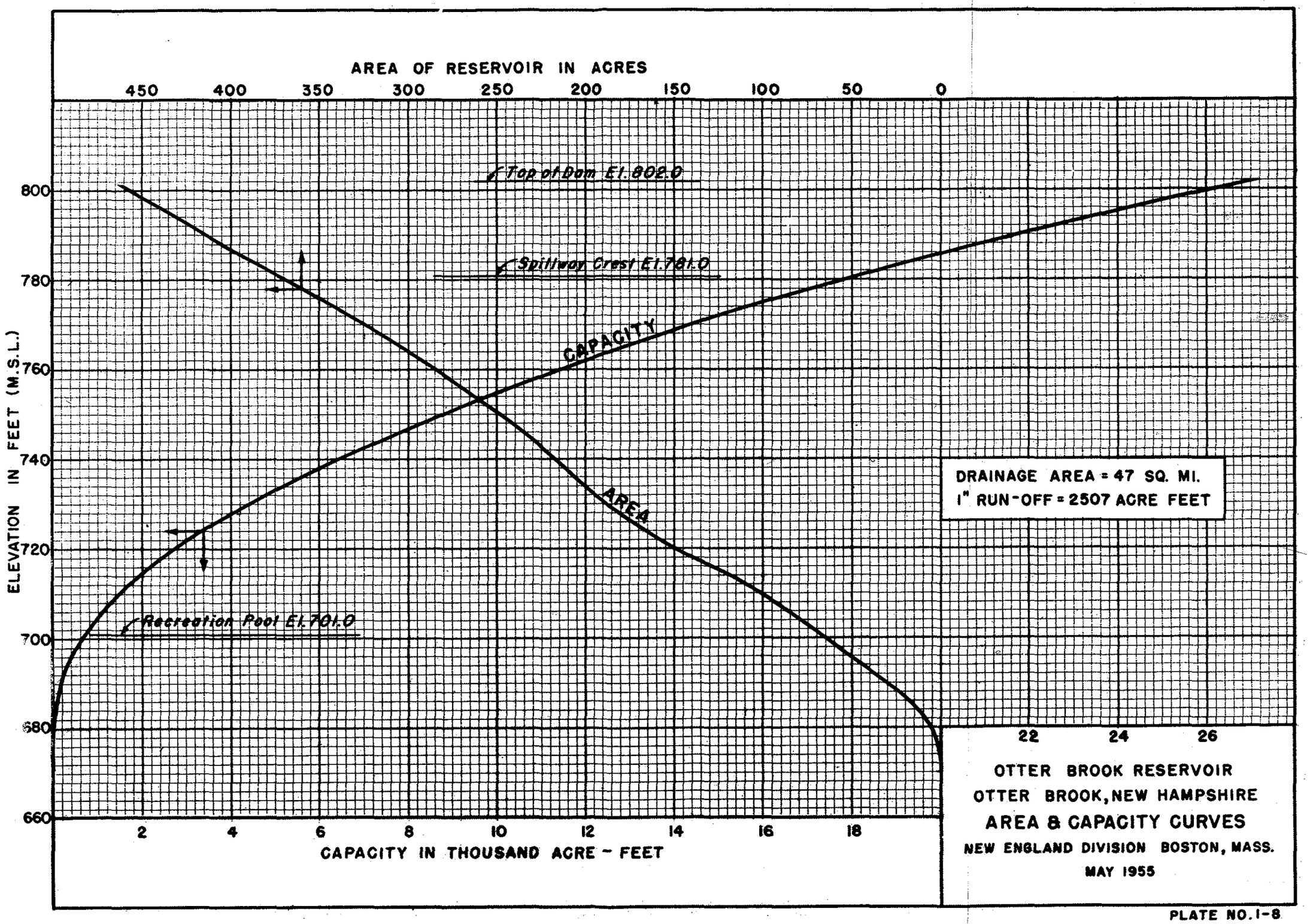
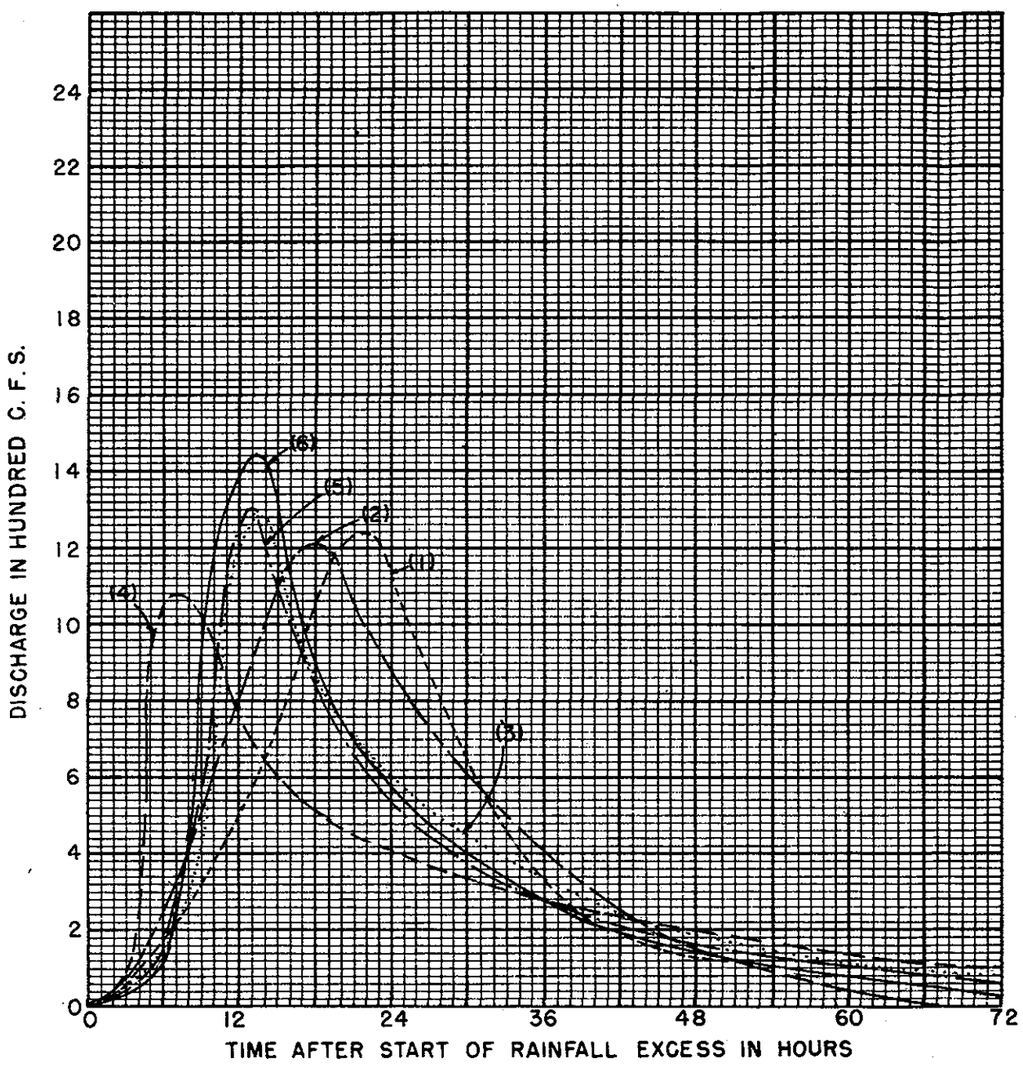
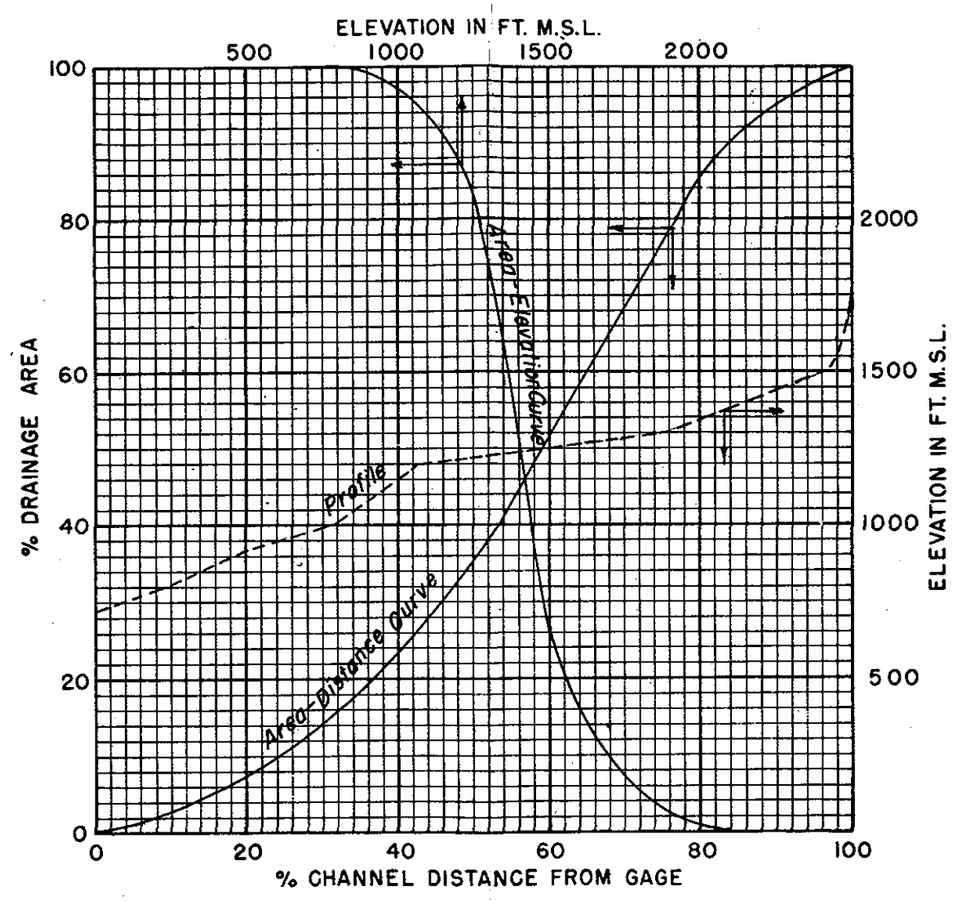

AREA OF RESERVOIR IN ACRES
450
400
350
300
250
200
150
100
50
0
Top of Dam El. 802.0
Spillway Crest El. 781.0
Recreation Pool El. 701.0
CAPACITY
AREA
ELEVATION IN FEET (M.S.L.)
800
780
760
740
720
700
680
660
2
4
6
8
10
12
14
16
18
22
24
26
CAPACITY IN THOUSAND ACRE - FEET
DRAINAGE AREA = 47 SQ. MI.
1" RUN-OFF = 2507 ACRE FEET
OTTER BROOK RESERVOIR
OTTER BROOK, NEW HAMPSHIRE
AREA & CAPACITY CURVES
NEW ENGLAND DIVISION BOSTON, MASS.
MAY 1955

PLATE NO. I-8

OBSERVED UNIT HYDROGRAPHS



DRAINAGE AREA		CHARACTERISTICS	
DRAINAGE AREA	42.3 sq. mi.	L	12 mi.
MAXIMUM ELEVATION	2150 ft. m.s.l.	L_{ca}	5.5 mi.
MINIMUM ELEVATION	700 ft. m.s.l.	$(LL_{ca})^{0.3}$	3.51
MEAN ELEVATION (weighted)	1400 ft. m.s.l.	DRAINAGE DENSITY	— mi/sq.mi.
LAND SLOPE	— ft./mi.	MAP SCALE	1: 62,500
MAIN STREAM SLOPE	62 ft./mi.	METHOD OF FLOW SEPARATION	TYPE A
		BASIN SHAPE FACTOR	3.4



DATA FROM OBSERVED UNIT HYDROGRAPHS

DATE OF RAINFALL	LEGEND	AVE. P (In.)	RAINFALL EXCESS DURATION (hr.)	RAINFALL EXCESS AMOUNT (In.)	L_{cP} (mi.)	STAGE RECORD	Q_{pR} (cfs.)	Q_p tr=3 hrs. (cfs.)	t_{pR} (hr.)	t_p (hr.)	t_v (hr.)	C_{tR}	C_p640	K_m (hr.)	T_c (hr.)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
18 March 1936	------ (1)	4.25	12	2.15	Unifor.	Rec.	1230	1530	12	15	15	3.42	349	9.9	15
20-21 Septembr 1938	——— (2)	5.92	12	4.50	Unifor.	Rec.	1220	1480	8.5	7.5	13	2.42	245	16.6	7
8-9 Novembr 1943	····· (3)	2.33	9	1.06	Unifor.	Rec.	1300	1770	9	10	16	2.56	276	5.6	8
14-15 Septembr 1944	——— (4)	1.85	3	1.07	Unifor.	Rec.	1080	—	5.5	—	21	1.57	140	10.0	9
19-20 June 1945	—·—· (5)	2.13	9	1.10	Unifor.	Rec.	1300	1850	6.5	10	13.5	1.85	200	12.8	6
25-26 Novembr 1950	—— (6)	4.00	9	2.20	Unifor.	Rec.	1440	1900	9	8	12.8	2.56	306	6.5	6

CONNECTICUT RIVER FLOOD CONTROL

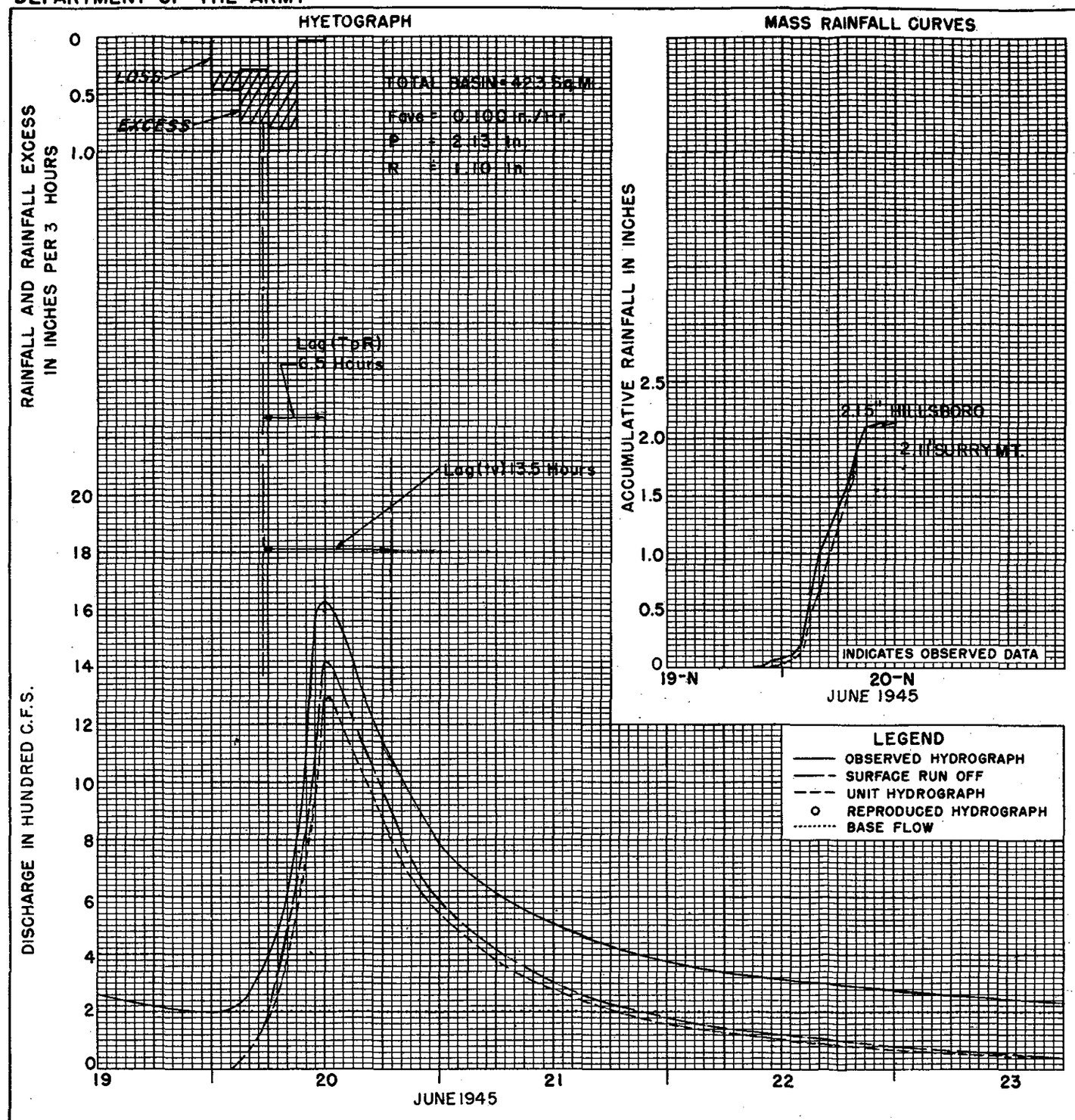
OTTER BROOK RESERVOIR
UNIT HYDROGRAPHS
OTTER BROOK NEAR KEENE, N.H.
PERTINENT DATA

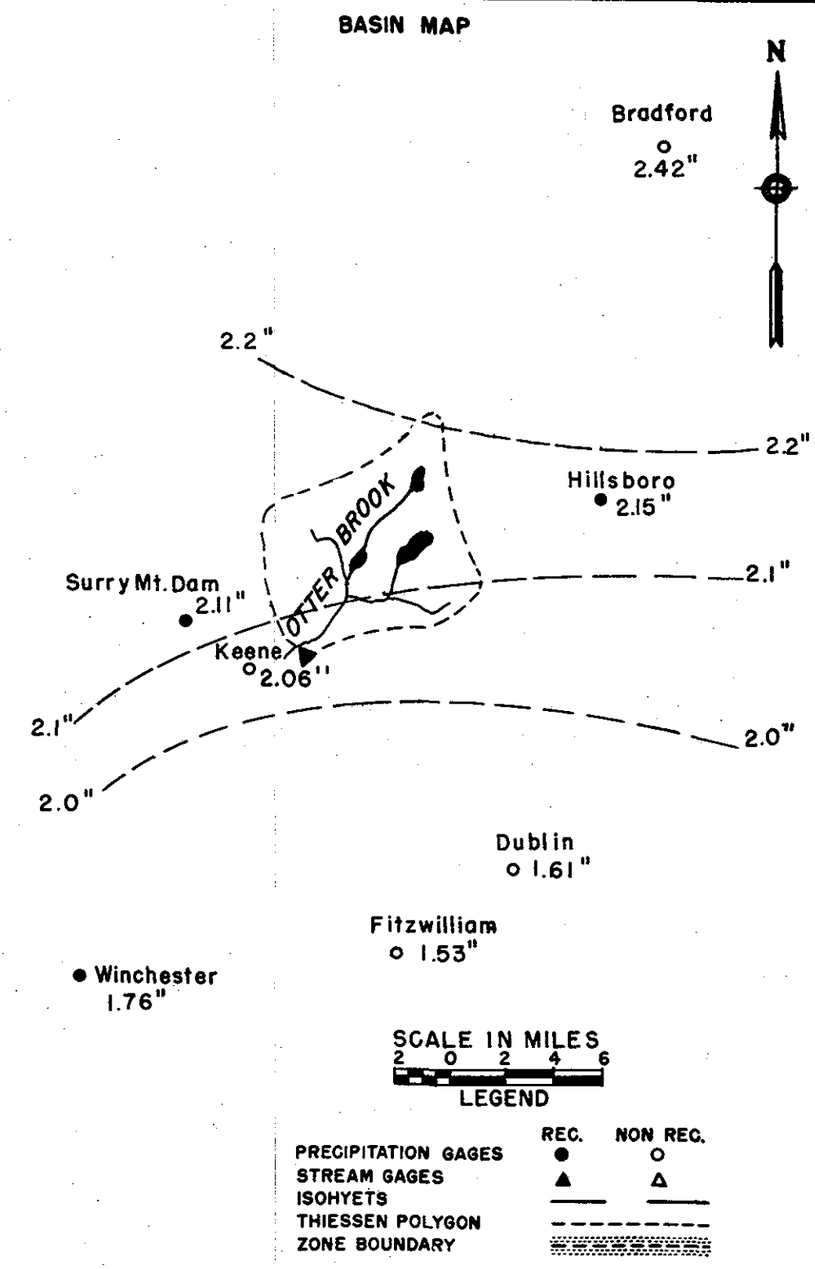
NEW ENGLAND DIVISION BOSTON, MASS.

FEBRUARY 1955

DEPARTMENT OF THE ARMY

CORPS OF ENGINEERS





CONNECTICUT RIVER FLOOD CONTROL

OTTER BROOK RESERVOIR UNIT HYDROGRAPHS

OTTER BROOK NEAR KEENE, N.H.

STORM OF 19 -20 JUNE 1945

BASIC DATA

(Sheet 1 of 2)

NEW ENGLAND DIVISION BOSTON, MASS.

FEBRUARY 1955

ENG FORM 1822 1 NOV 49

PLATE NO. 1-10

DEPARTMENT OF THE ARMY

UNIT HYDROGRAPH BASIC DATA SHEET

CORPS OF ENGINEERS

(SHEET 2 OF 2)

(7) STREAM AND STATION OTTER BROOK NR. KEENE, N.H. LAT. 42°57'55" LONG. 72°14'00"

(8) DATE OF STORM 19-20 June 1945 (9) OFFICE NEW ENGLAND DIVISION

(10) DRAINAGE AREA 42.3 SQ.MI. (11) L 12 MI. (12) L_{ca} 5.5 MI. (13) $(LL_{ca})^{0.3}$ 3.51

(14) AVERAGE RAINFALL 2.13 IN. (15) t_R 9 HRS. (16) DIRECT RUNOFF 1.10 IN.

(17) Q_{pR} 1300 CFS. (18) q_{pR} 30.7 CFS/SQ.MI. (19) Q_p 1850 CFS. (20) t_{pR} 6.5 HRS.

(21) t_p 10 HRS. (22) t_v 13.5 HRS. (23) C_{tR} 1.85 (24) C_{p640} 200 W_{50} 12 HRS. W_{75} 6 HRS.

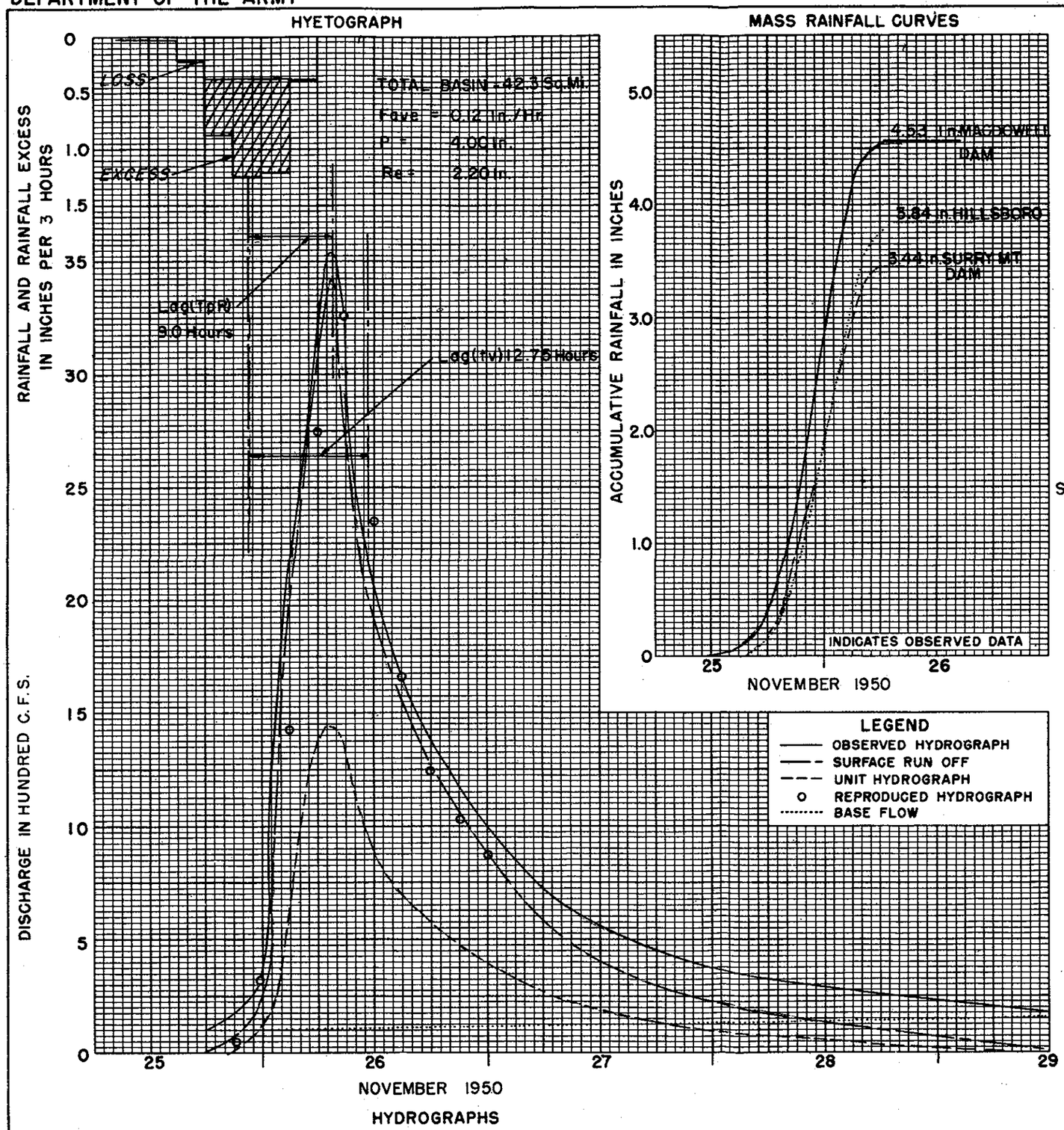
TIME June 1945 (25)	OBSERVED DISCHARGE (~~1000~~ CFS) (26)	ESTIMATED BASE FLOW (~~1000~~ CFS) (27)	DIRECT RUNOFF (~~1000~~ CFS) (28)	OBSERVED 9-HR UNIT HYDROGRAPH (~~1000~~ CFS) (29)	ADJUSTED 3-HR UNIT HYDROGRAPH (~~1000~~ CFS) (30)	REPRODUCED STORM HYDROGRAPH (~~1000~~ CFS) (31)	(32)	(33)
19-N	260	260	0					
3	240	240	0					
6P	220	220	0					
9	210	210	0					
M	200	200	0	0	0			
3	230	200	30	30	120			
6A	400	200	200	180	420			
9	840	200	640	580	1160			
20-N	1620	200	1420	1290	1820			
3	1410	200	1210	1100	1020			
6P	1150	200	950	860	690			
9	930	200	730	660	510			
M	780	200	580	530	420			
3	690	200	490	450	360			
6A	620	200	420	380	315			
9	550	200	350	320	270			
21-N	500	200	300	270	240			
3	460	200	260	240	210			
6P	430	200	230	210	190			
9	400	200	200	180	170			
M	380	200	180	160	155			
3	360	200	160	150	140			
6A	340	200	140	130	125			
9	330	200	130	120	110			
22-N	320	200	120	110	100			
3	310	200	110	100	90			
6P	300	200	100	90	80			
9	290	200	90	80	70			
M	280	200	80	70	60			
3	270	200	70	60	50			
6A	260	200	60	50	40			
9	260	200	60	50	30			
23-N	250	200	50	50	20			
3	240	200	40	40	10			
6P	240	200	40	40	0			
9	230	200	30	30				
M	230	200	30	30				
3P	200	200	0	0				
cfs/3 hrs. Total	17,230	7,730	9,490	8,640	8,995			

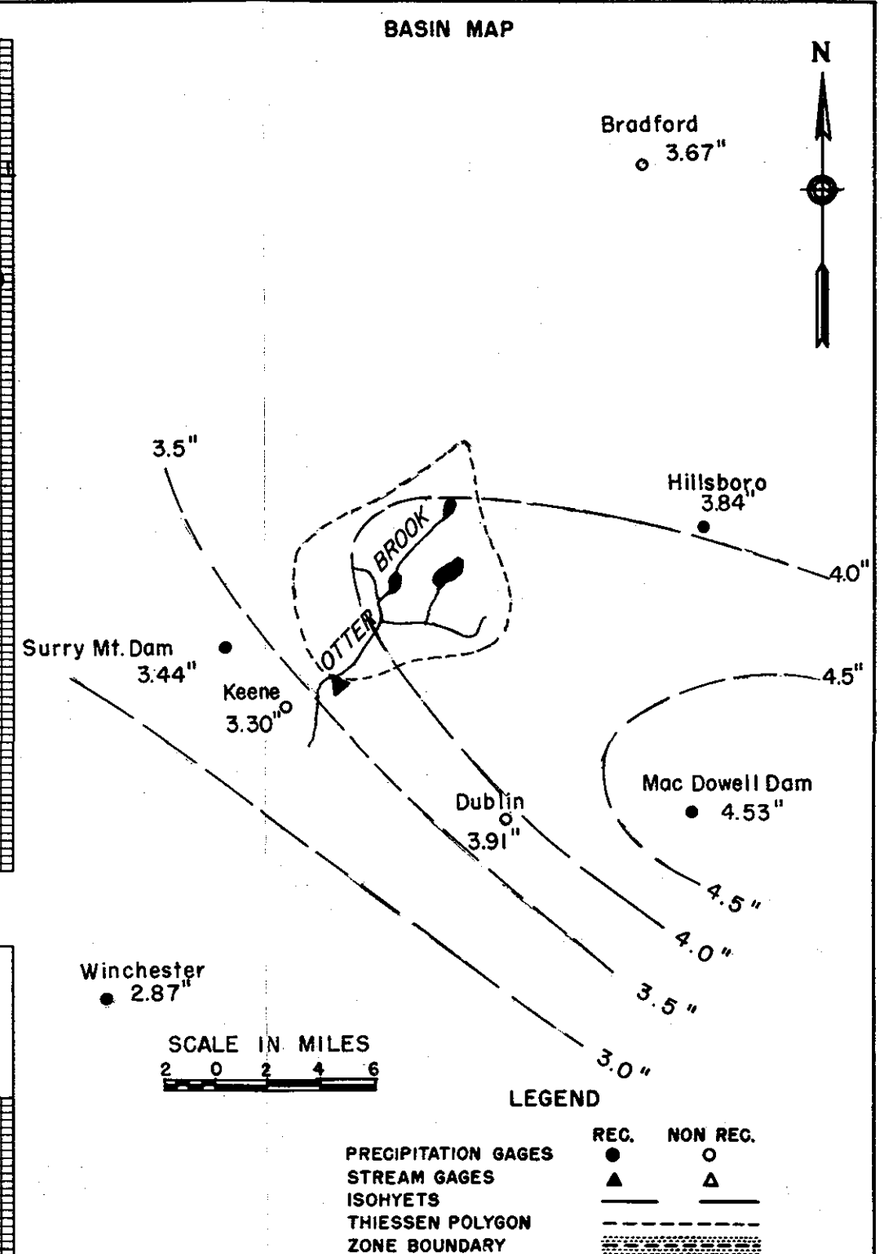
DATE 1-26-55 COMPUTED BY BSJ

ENG FORM 1 NOV 49 1823

50 6617

PLATE NO. I-11





BASIC DATA
(Sheet 1 of 2)

CONNECTICUT RIVER FLOOD CONTROL
OTTER BROOK RESERVOIR
UNIT HYDROGRAPHS
OTTER BROOK NEAR KEENE, N.H.
STORM OF 25-26 NOVEMBER 1950

NEW ENGLAND DIVISION — BOSTON, MASS.
FEBRUARY 1955

DEPARTMENT OF THE ARMY

UNIT HYDROGRAPH BASIC DATA SHEET

CORPS OF ENGINEERS

(SHEET 2 OF 2)

(7) STREAM AND STATION OTTER BROOK NR. KEENE, N.H. LAT. 42°57'55" LONG. 72°14'00"

(8) DATE OF STORM 25-26 November 1950 (9) OFFICE NEW ENGLAND DIVISION

(10) DRAINAGE AREA 42.3 SQ.MI. (11) L 12 MI. (12) L_{ca} 5.5 MI. (13) $(LL_{ca})^{0.3}$ 3.51

(14) AVERAGE RAINFALL 4.00 IN. (15) t_R 9 HRS. (16) DIRECT RUNOFF 2.20 IN.

(17) Q_{pR} 1440 CFS. (18) q_{pR} 34.0 CFS/SQ.MI. (19) Q_p 1900 CFS. (20) t_{pR} 9.0 HRS.

(21) t_p 8 HRS. (22) t_v 12.8 RS. (23) C_{tR} 2.56 (24) C_{p640} 306 W_{50} 11 HRS. W_{75} 6.2 HRS.

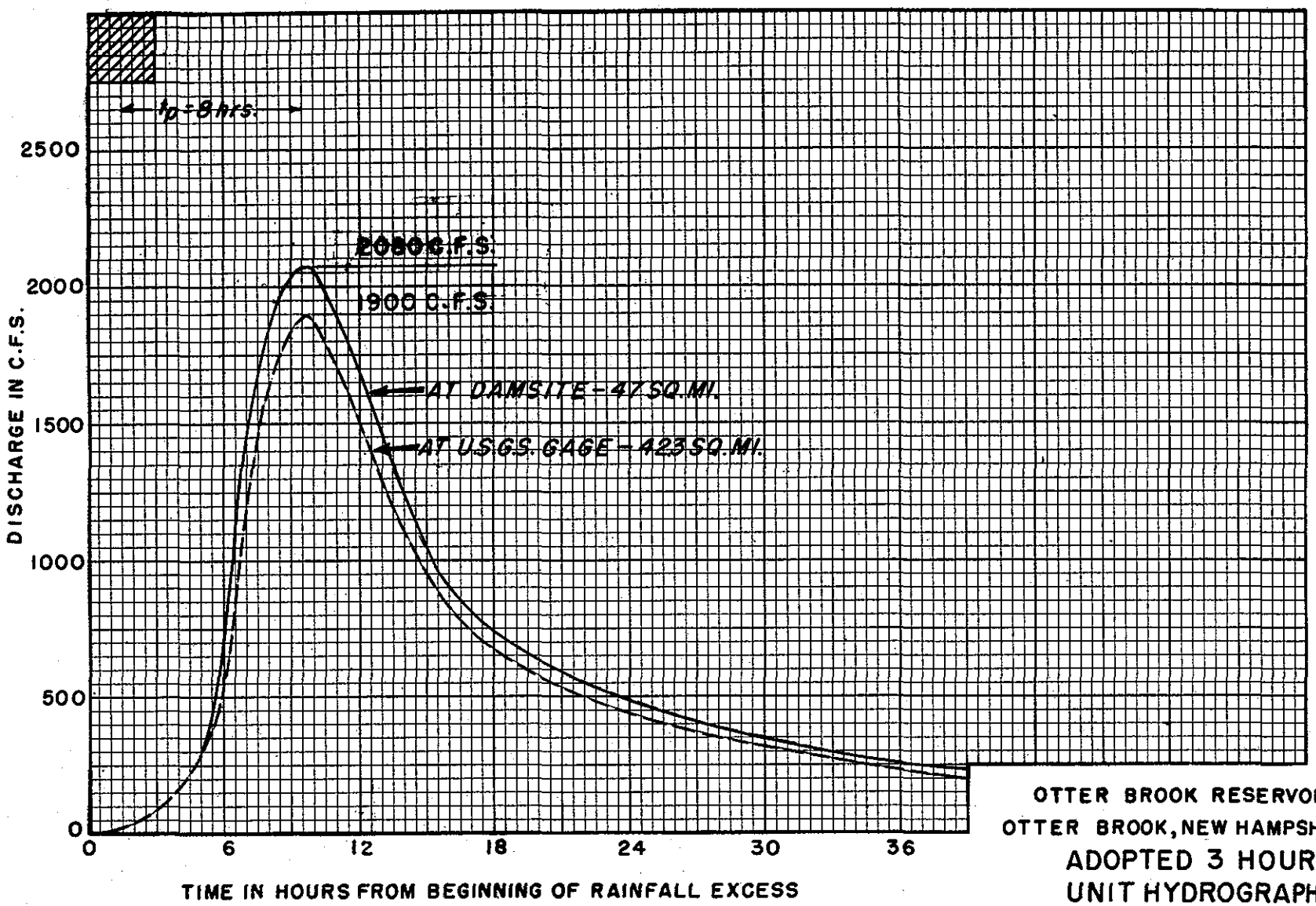
TIME November 1950 (25)	OBSERVED DISCHARGE (~~XXXX~~ CFS) (26)	ESTIMATED BASE FLOW (~~XXXX~~ CFS) (27)	DIRECT RUNOFF (~~XXXX~~ CFS) (28)	9-HR UNIT HYDROGRAPH (~~XXXX~~ CFS) (29)	3-HR UNIT HYDROGRAPH (~~XXXX~~ CFS) (30)	REPRODUCED STORM HYDROGRAPH (~~XXXX~~ CFS) (31)	(32)	(33)
25-N								
3P								
6P	100	100	0	0	0	0		
9	170	100	70	30	90	45		
M	350	100	250	115	520	330		
26-3A	2150	100	2050	930	1850	1435		
6	3150	110	3040	1380	1450	2755		
9	3100	110	2990	1360	930	3270		
N	1920	110	1810	820	690	2365		
3P	1650	110	1540	700	540	1645		
6	1410	120	1290	585	450	1270		
9	1200	120	1080	490	380	1030		
M	1020	120	900	410	320	865		
27-3A	850	120	730	330	270	730		
6	720	120	600	270	230	615		
9	630	130	500	230	195	520		
N	550	130	420	190	165	440		
3P	500	130	370	170	145	380		
6	450	130	320	145	125	325		
9	420	130	290	130	110	285		
M	390	140	250	115	95	245		
28-3A	370	140	230	105	80	215		
6	350	140	210	95	70	185		
9	330	140	190	85	60	155		
N	310	140	170	75	50	135		
3P	290	150	140	65	40	115		
6	270	150	120	55	30	90		
9	250	150	100	50	25	70		
M	230	150	80	40	20	55		
29-3A	210	150	60	30	15	45		
6	200	160	40	20	10	35		
9	190	160	30	15	5	25		
N	180	160	20	10	0	12		
3P	170	160	10	5		4		
6	160	160	0	0		0		
cfs/3 hrs.								
Totals	24,240	4,340	19,900	9,050	8,960	19,691		

DATE 1-28-55 COMPUTED BY BSJ

ENG FORM 1 NOV 49 1823

50 6617

PLATE NO. I-13

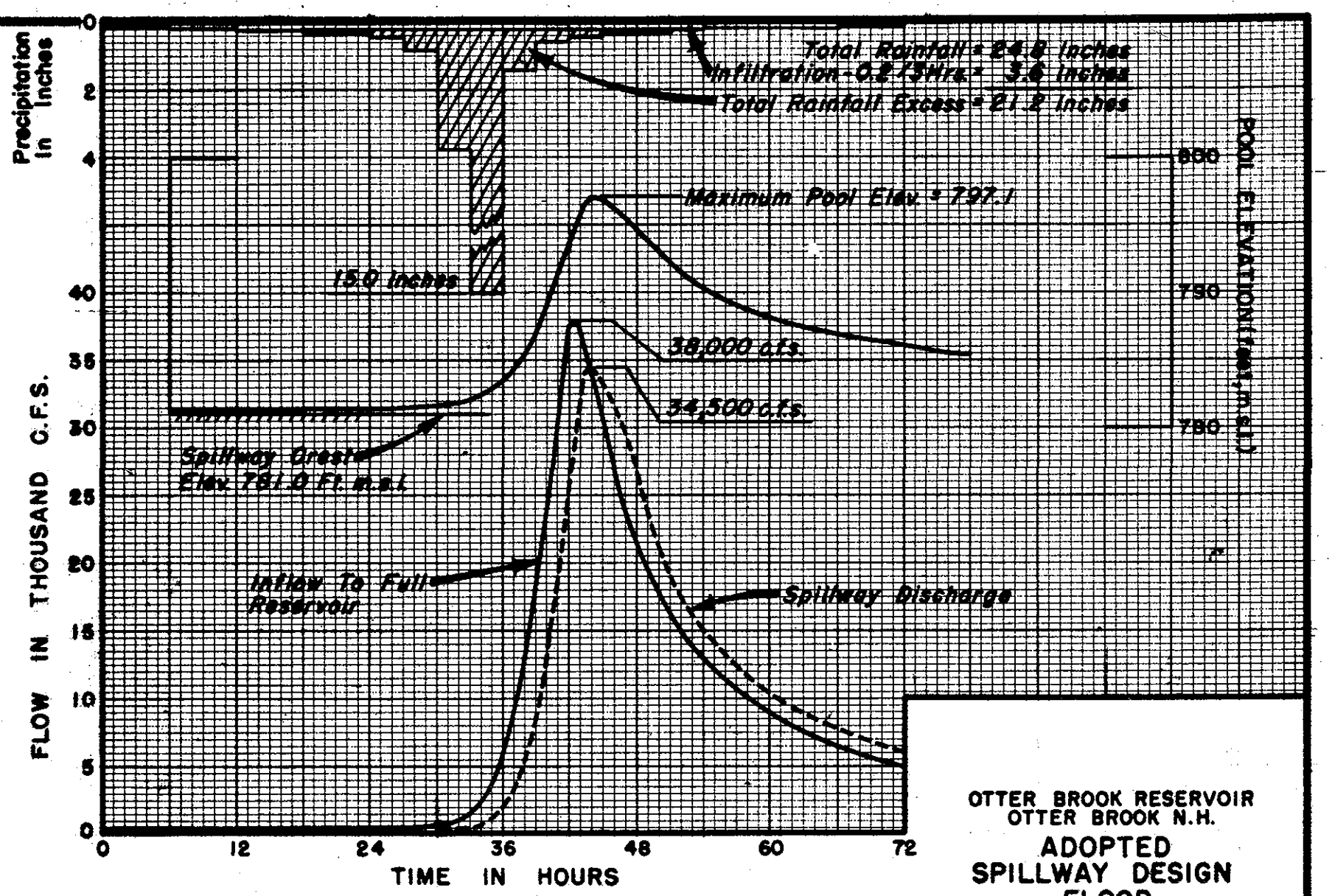


OTTER BROOK RESERVOIR
OTTER BROOK, NEW HAMPSHIRE

ADOPTED 3 HOUR
UNIT HYDROGRAPH

NEW ENGLAND DIVISION - BOSTON, MASS.
FEBRUARY 1955

PLATE NO. I-14

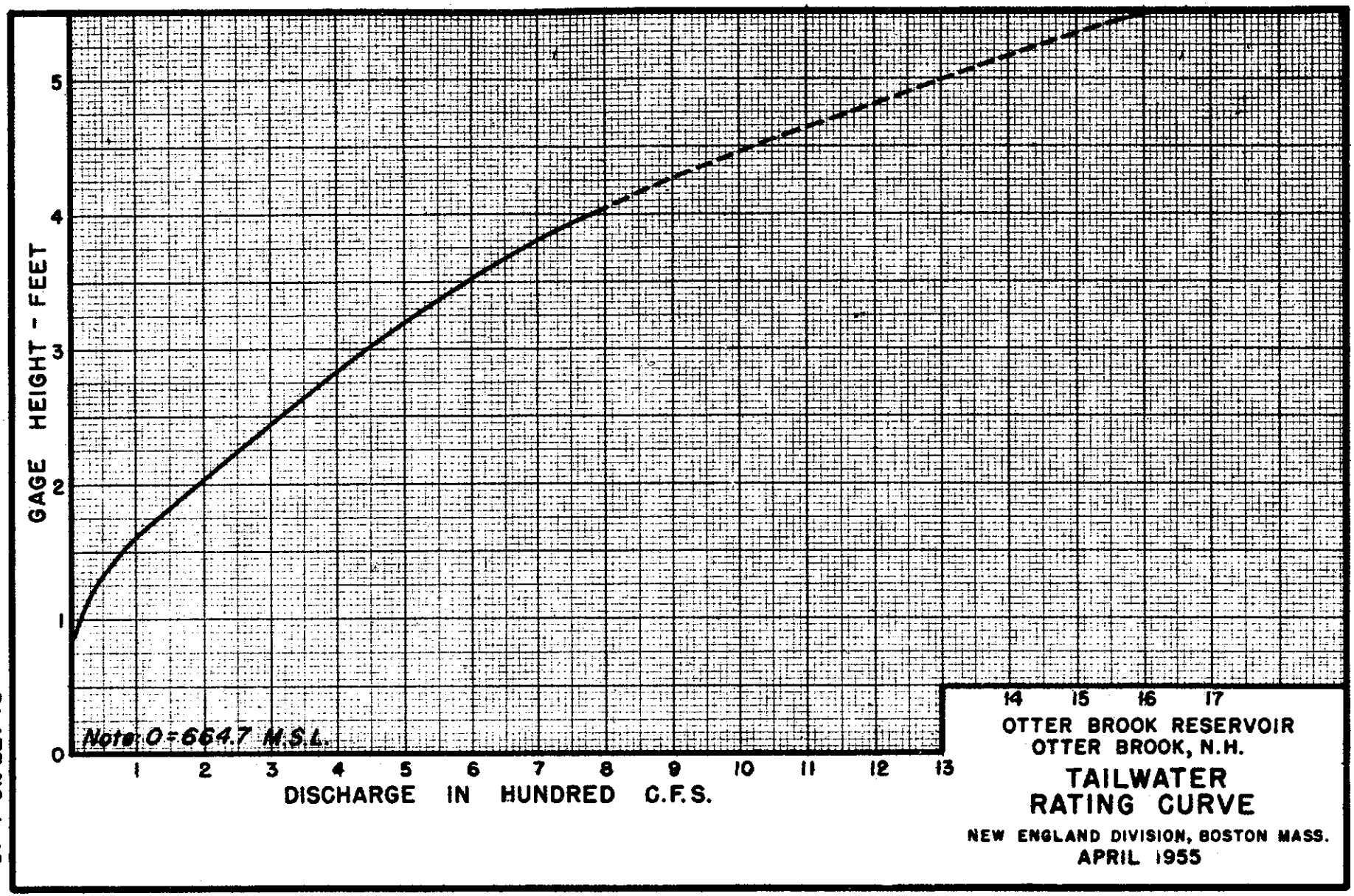


OTTER BROOK RESERVOIR
OTTER BROOK N.H.

ADOPTED
SPILLWAY DESIGN
FLOOD

NEW ENGLAND DIVISION, BOSTON MASS.
FEBRUARY 1955

PLATE NO I-15

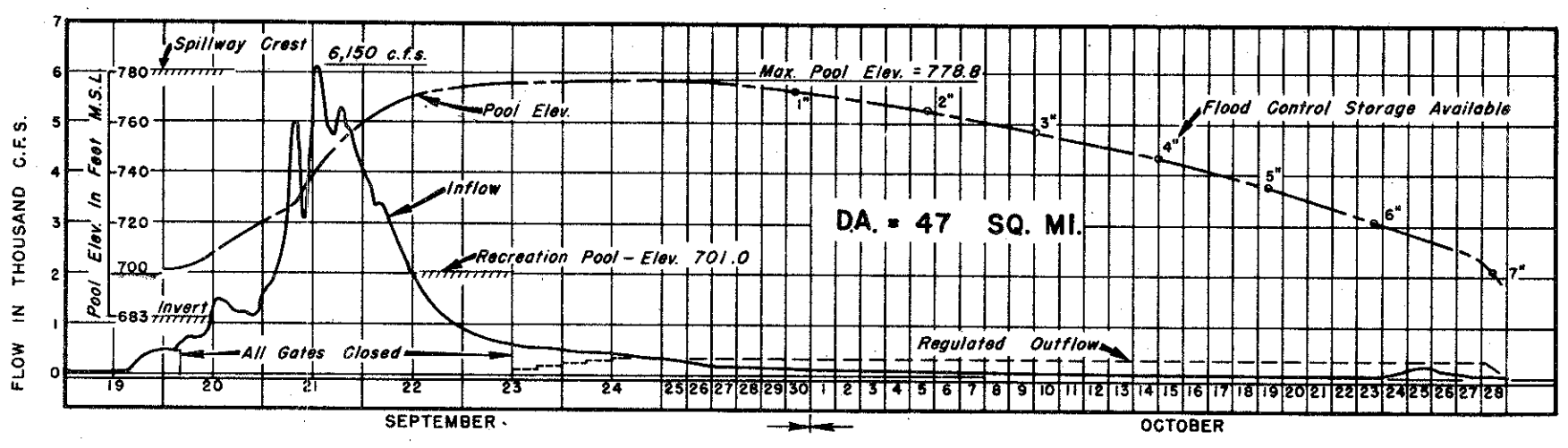


OTTER BROOK RESERVOIR
OTTER BROOK, N.H.

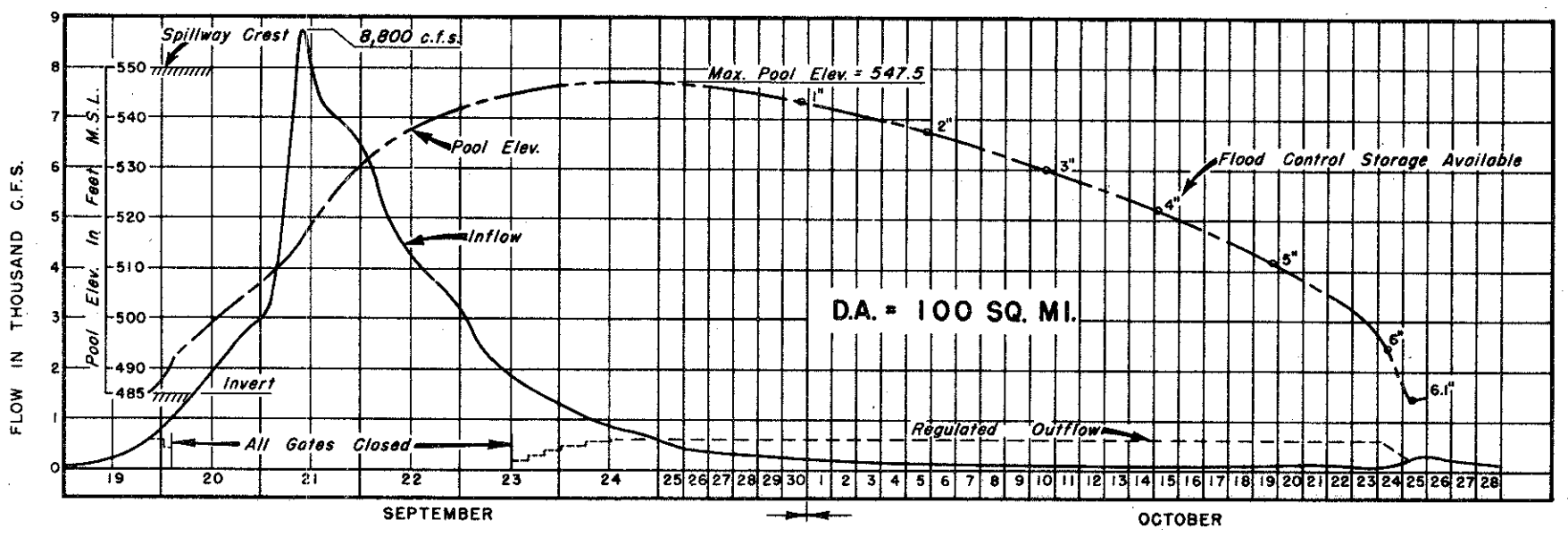
TAILWATER RATING CURVE

NEW ENGLAND DIVISION, BOSTON MASS.
APRIL 1955

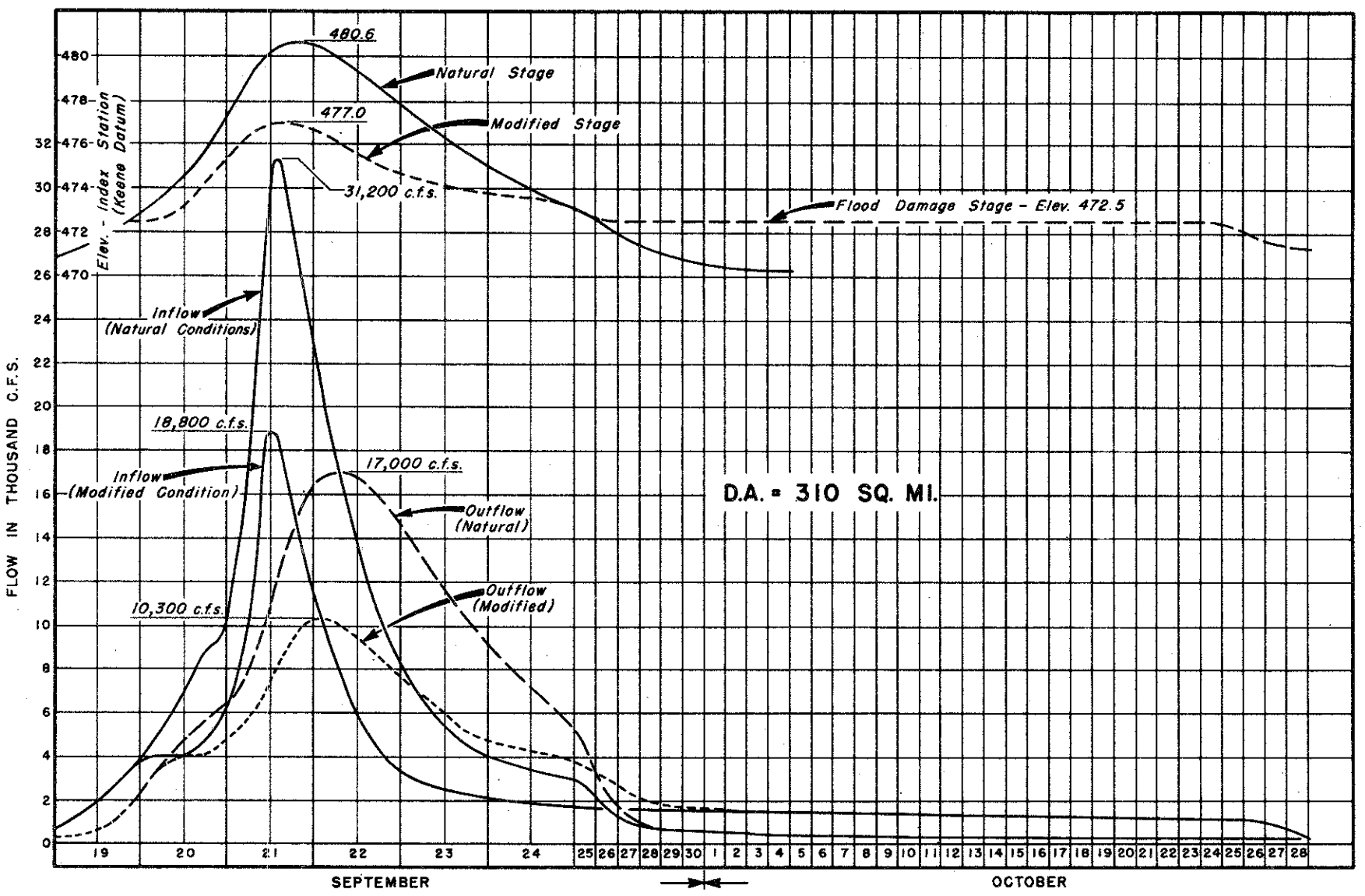
PLATE NO. I-16



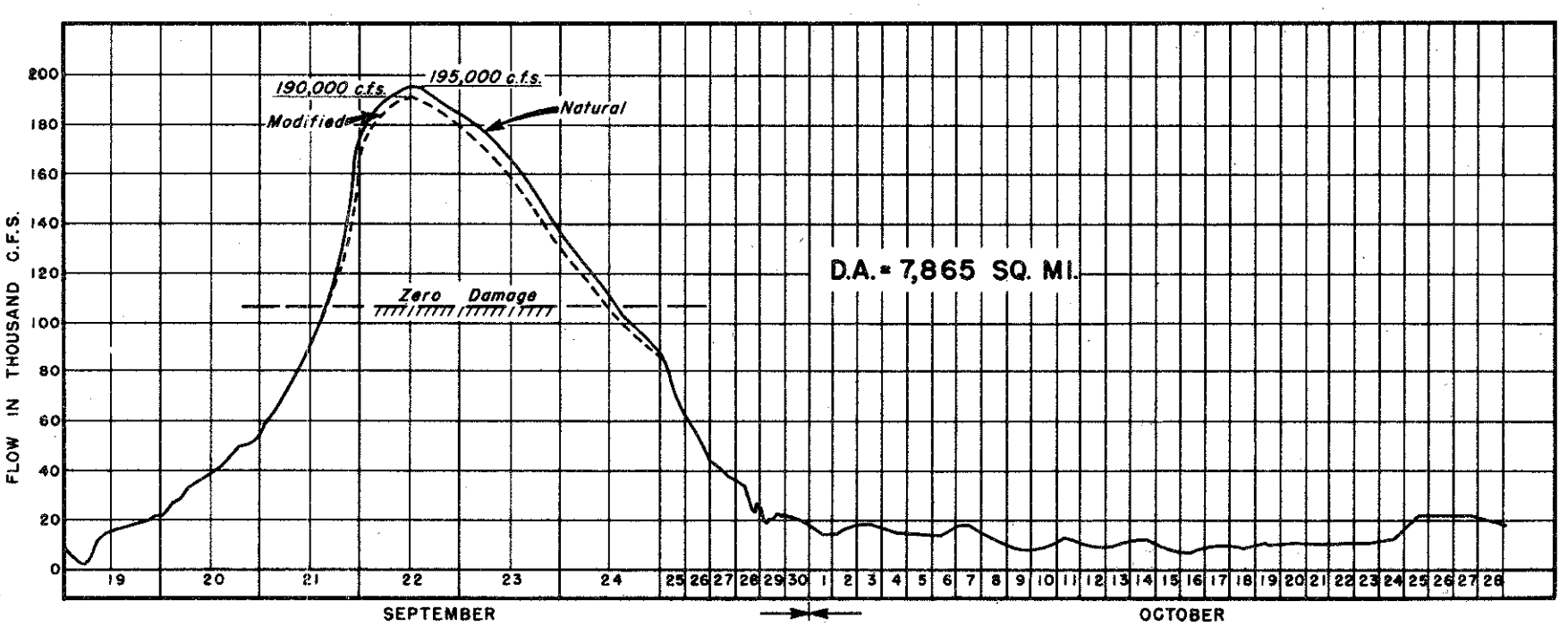
OTTER BROOK AT OTTER BROOK RESERVOIR



ASHUELOT RIVER AT SURRY MT. RESERVOIR



ASHUELOT RIVER AT KEENE, NEW HAMPSHIRE

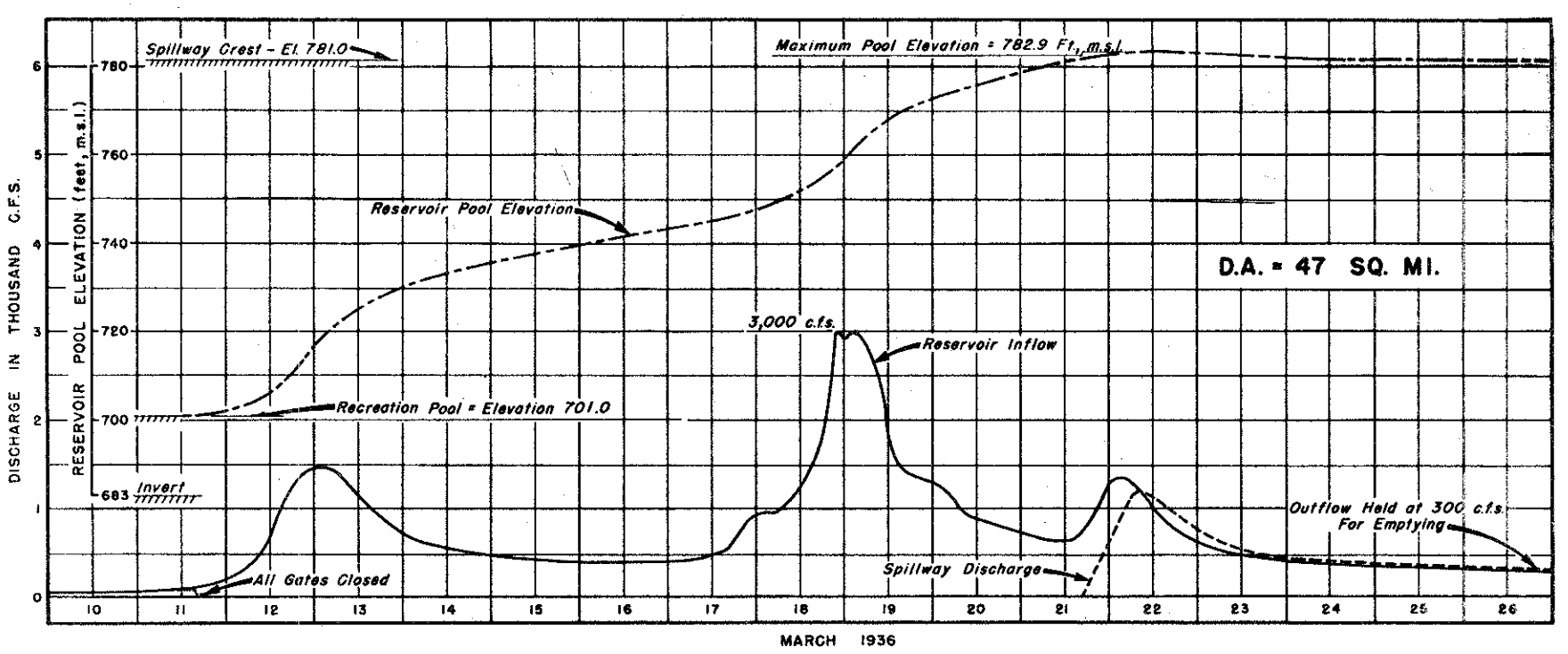


CONNECTICUT RIVER AT MONTAGUE CITY, MASSACHUSETTS

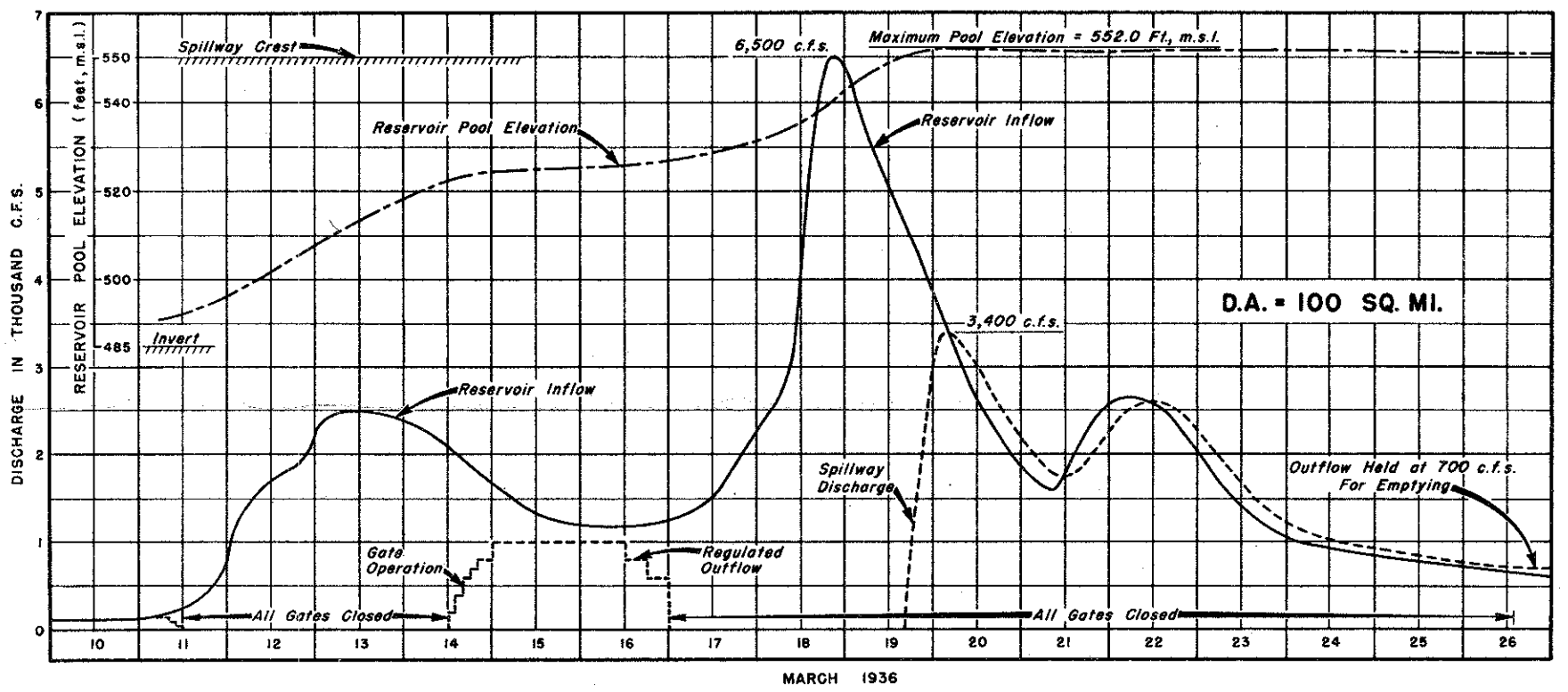
CONNECTICUT RIVER FLOOD CONTROL
ASHUELOT RIVER BASIN

EFFECT OF SURRY MOUNTAIN AND OTTER BROOK RESERVOIRS ON FLOOD OF SEPTEMBER 1938

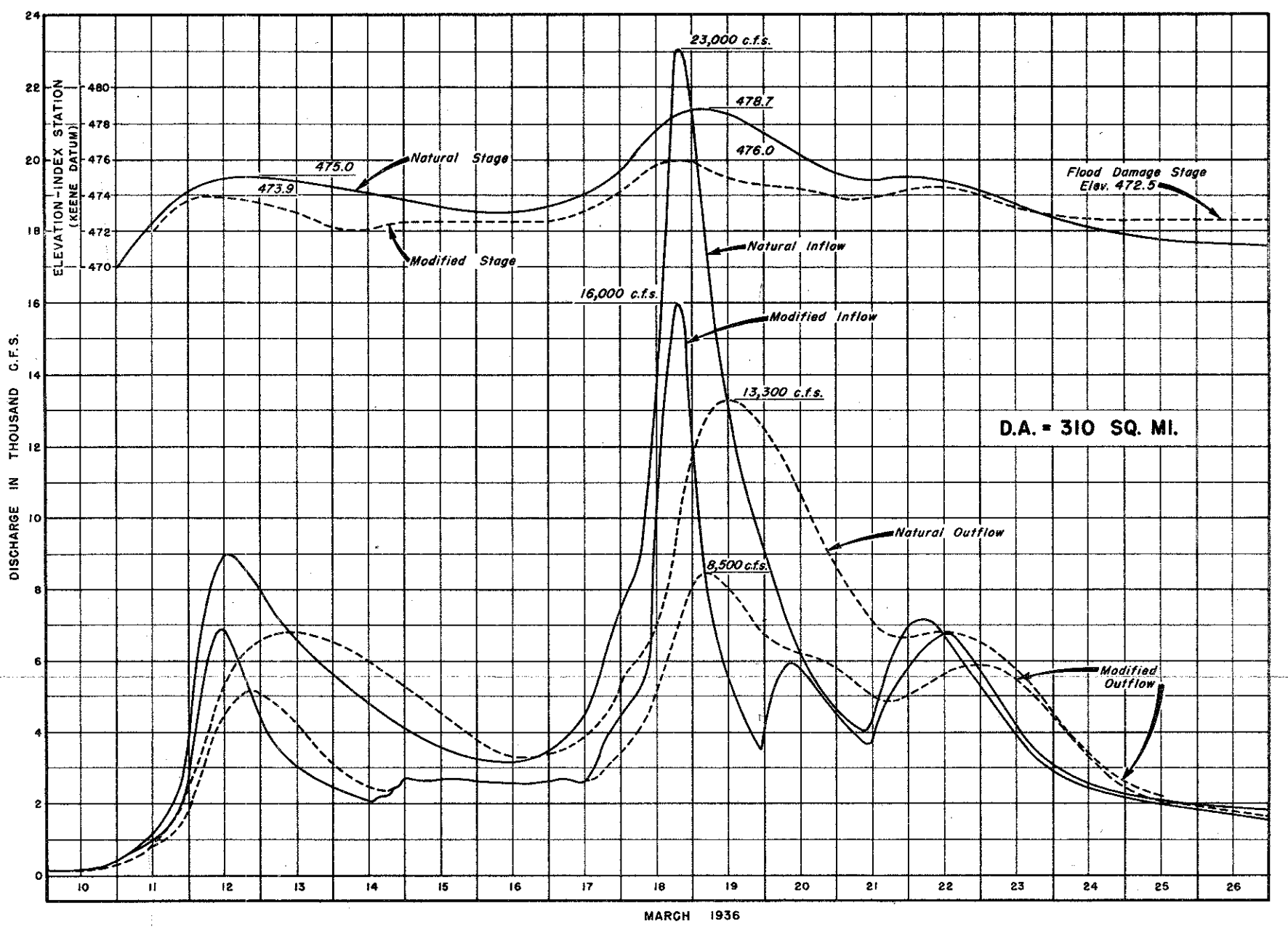
NEW ENGLAND DIVISION, BOSTON MASS.
APRIL 1955



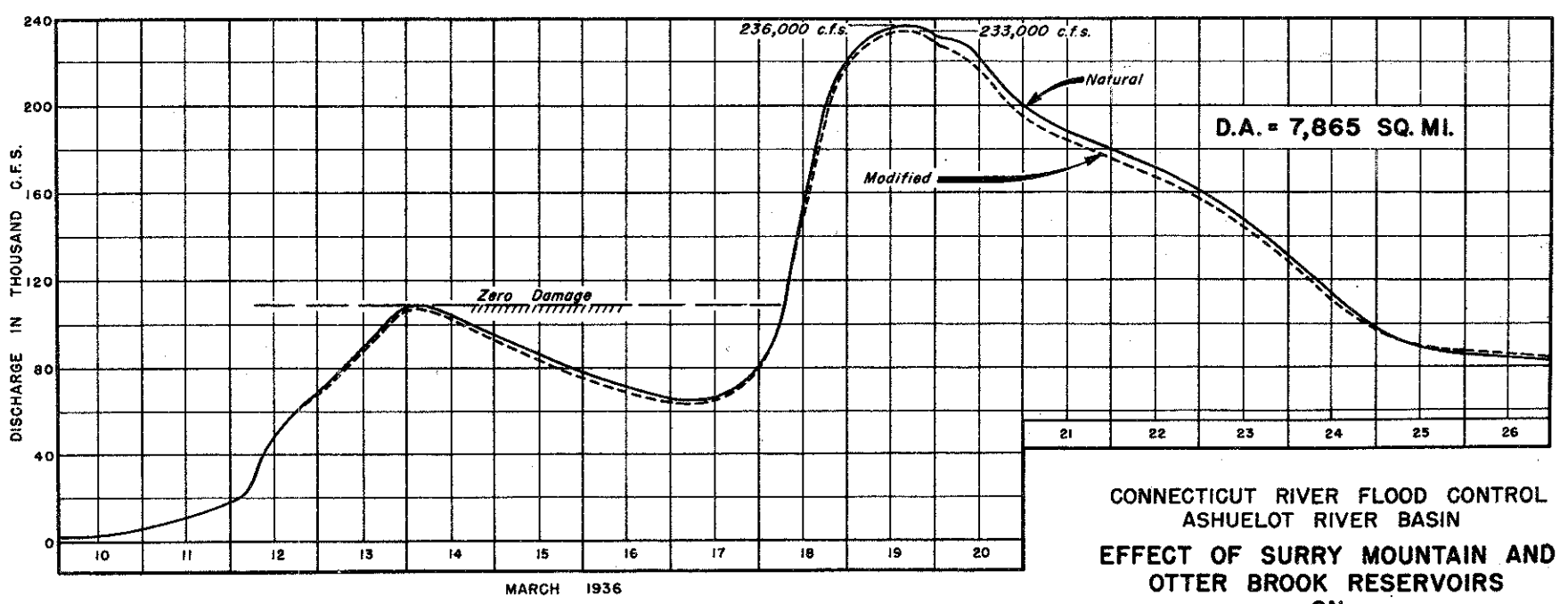
OTTER BROOK AT OTTER BROOK RESERVOIR



ASHUELOT RIVER AT SURRY MT. RESERVOIR



ASHUELOT RIVER AT KEENE, NEW HAMPSHIRE



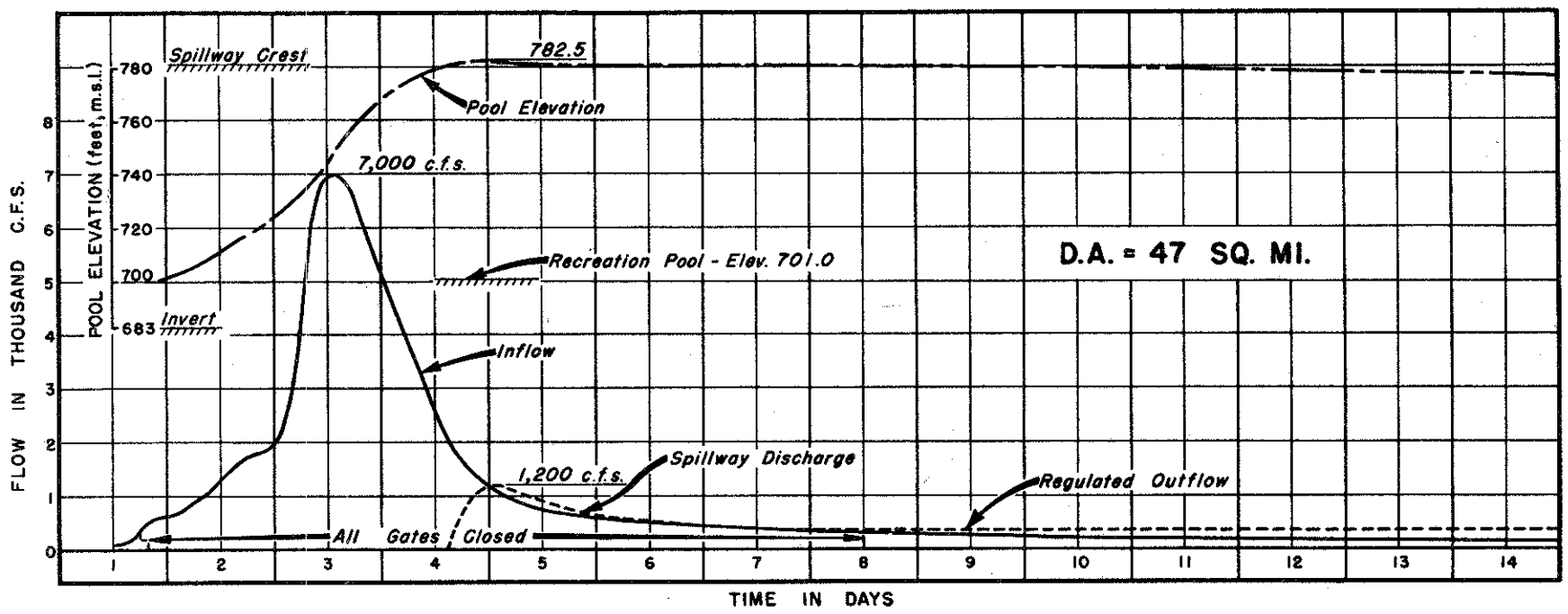
CONNECTICUT RIVER AT MONTAGUE CITY, MASSACHUSETTS

CONNECTICUT RIVER FLOOD CONTROL
ASHUELOT RIVER BASIN

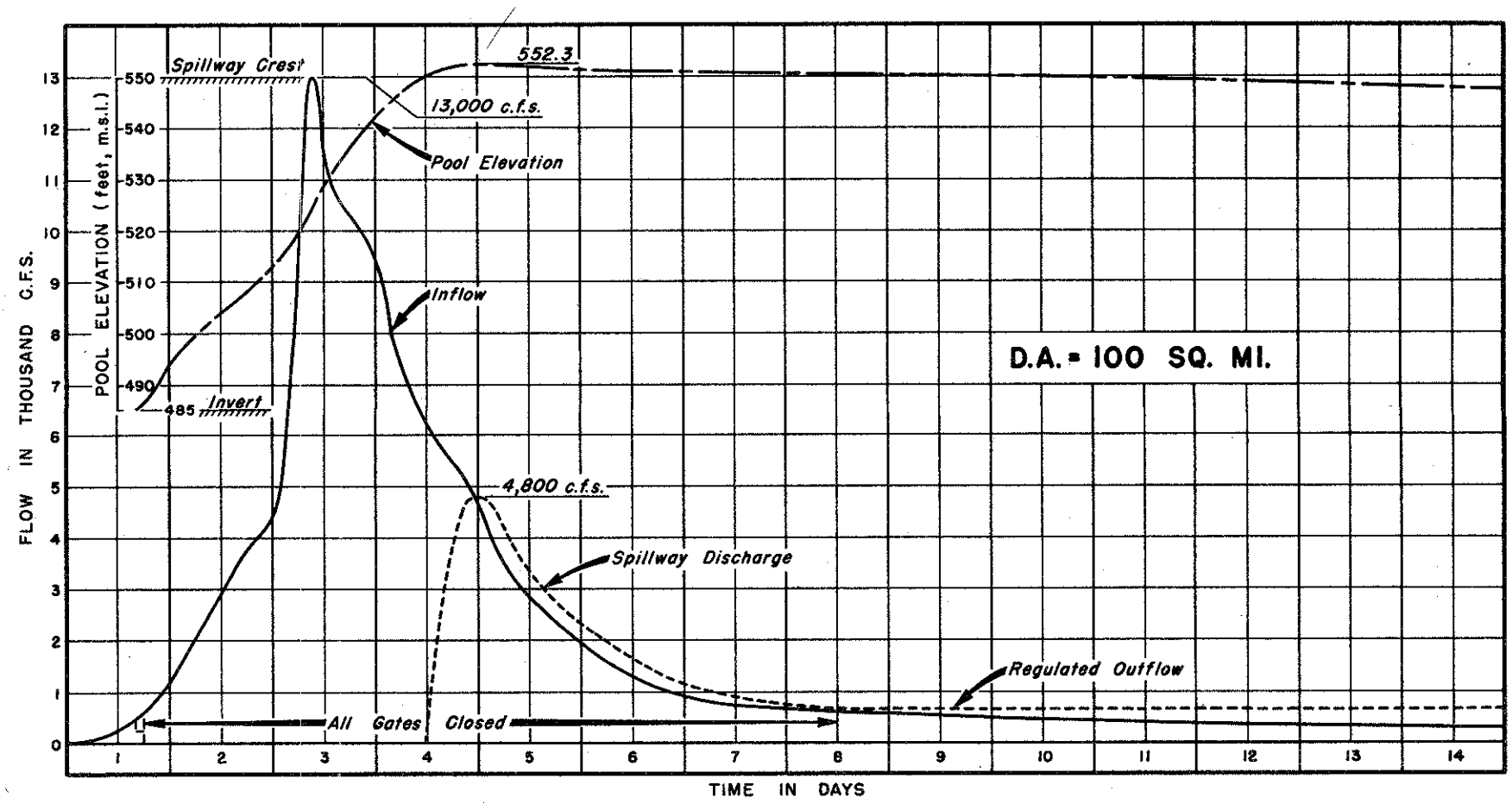
EFFECT OF SURRY MOUNTAIN AND OTTER BROOK RESERVOIRS ON FLOOD OF MARCH 1936

NEW ENGLAND DIVISION, BOSTON MASS.
APRIL 1955

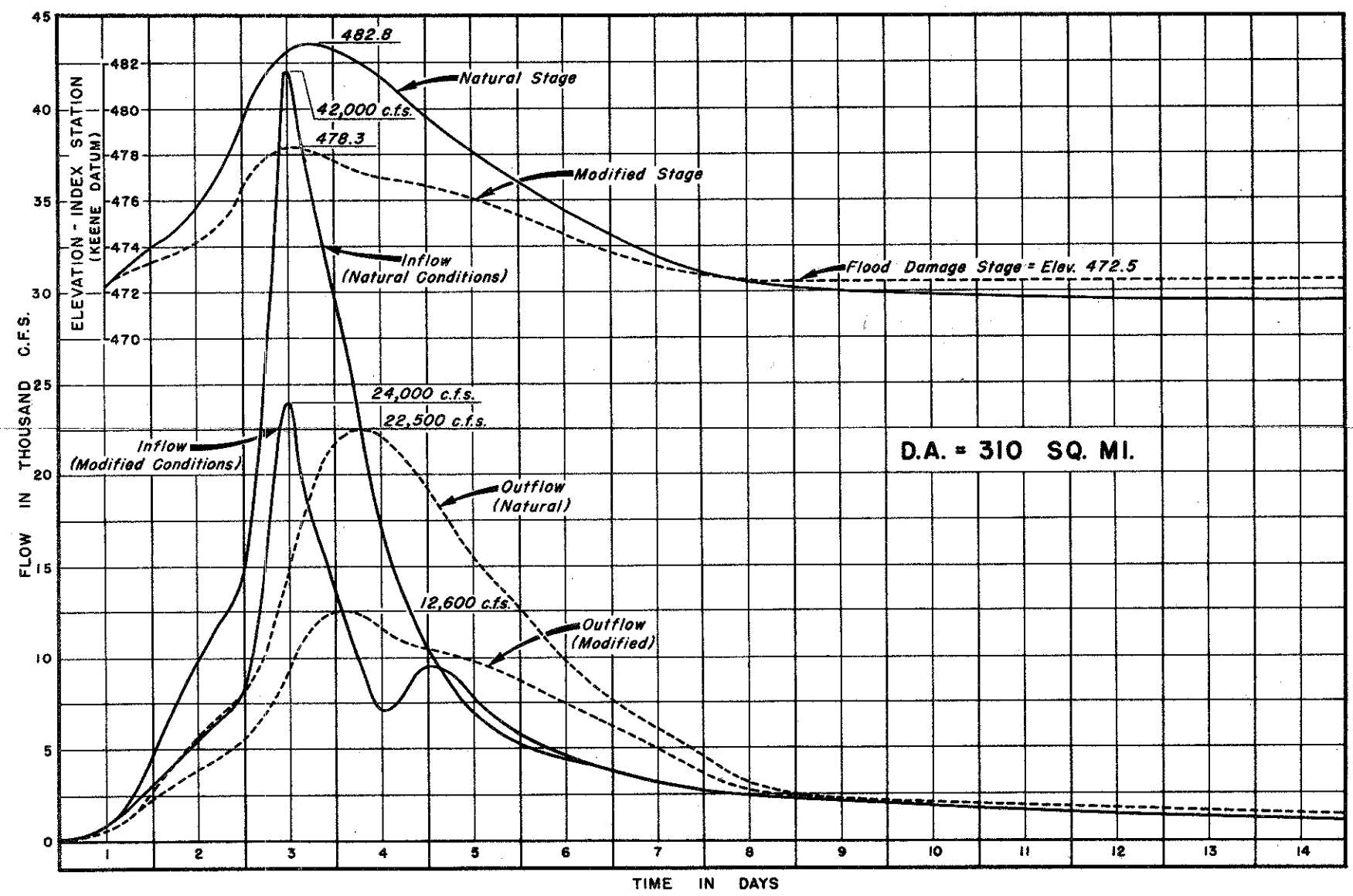
PLATE NO. I-18



OTTER BROOK AT OTTER BROOK RESERVOIR



ASHUELOT RIVER AT SURRY MT. RESERVOIR

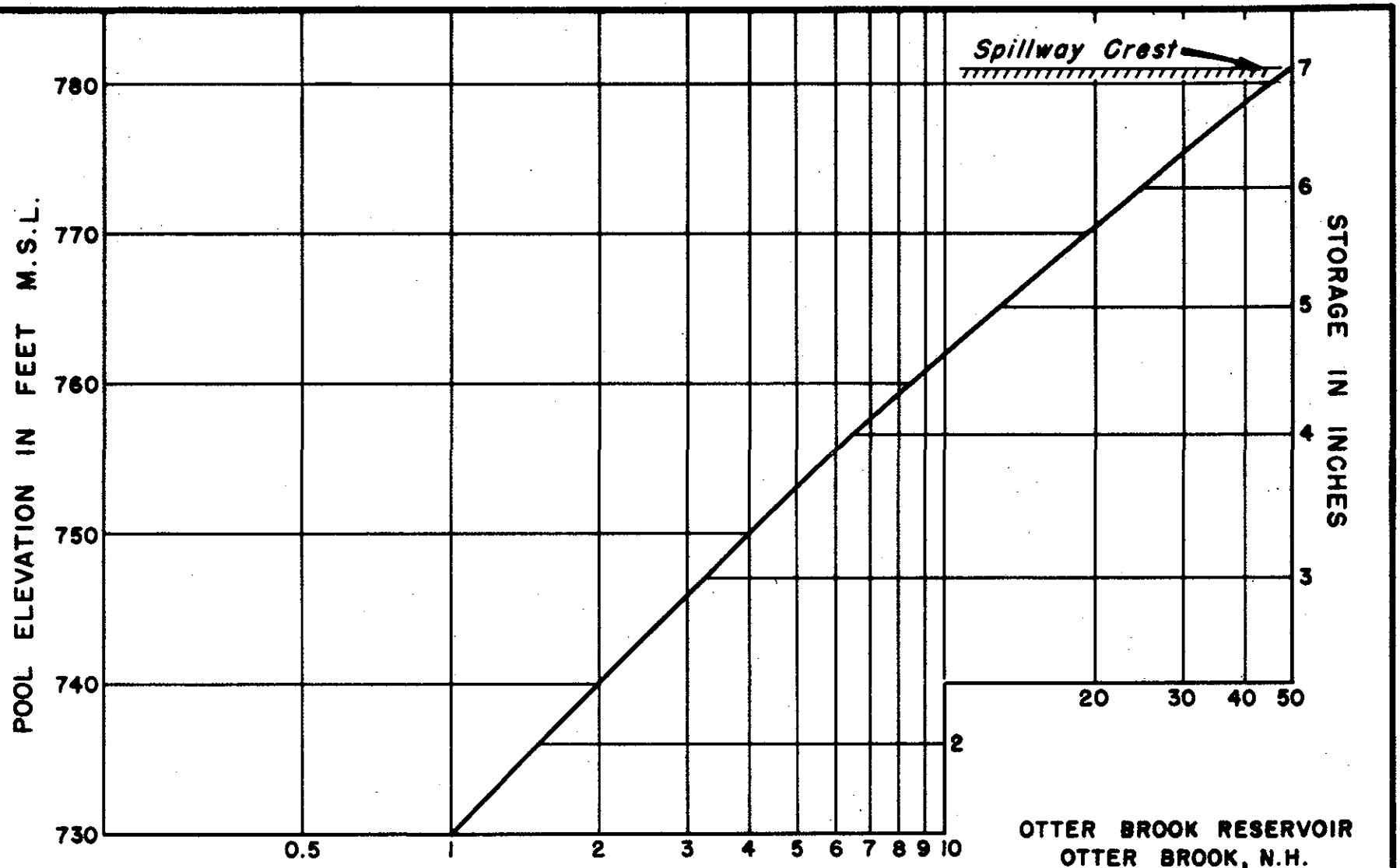


ASHUELOT RIVER AT KEENE, NEW HAMPSHIRE

CONNECTICUT RIVER FLOOD CONTROL
ASHUELOT RIVER BASIN

EFFECT OF SURRY MOUNTAIN AND OTTER BROOK RESERVOIRS ON STANDARD PROJECT FLOOD

NEW ENGLAND DIVISION, BOSTON MASS.
APRIL 1955

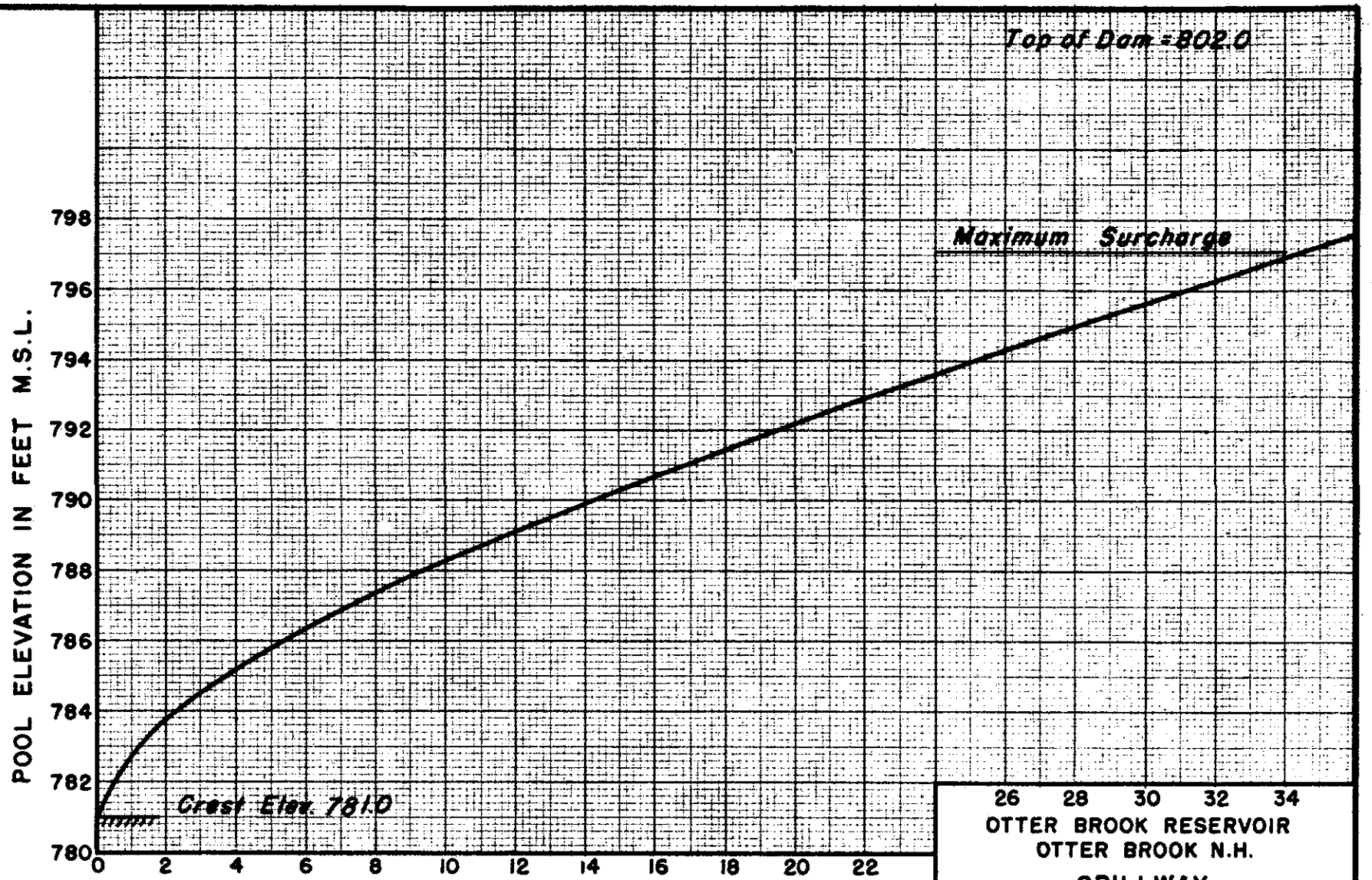


OTTER BROOK RESERVOIR
OTTER BROOK, N.H.

FREQUENCY OF FILLING

NEW ENGLAND DIVISION, BOSTON MASS.
APRIL 1955

PLATE NO. I-20



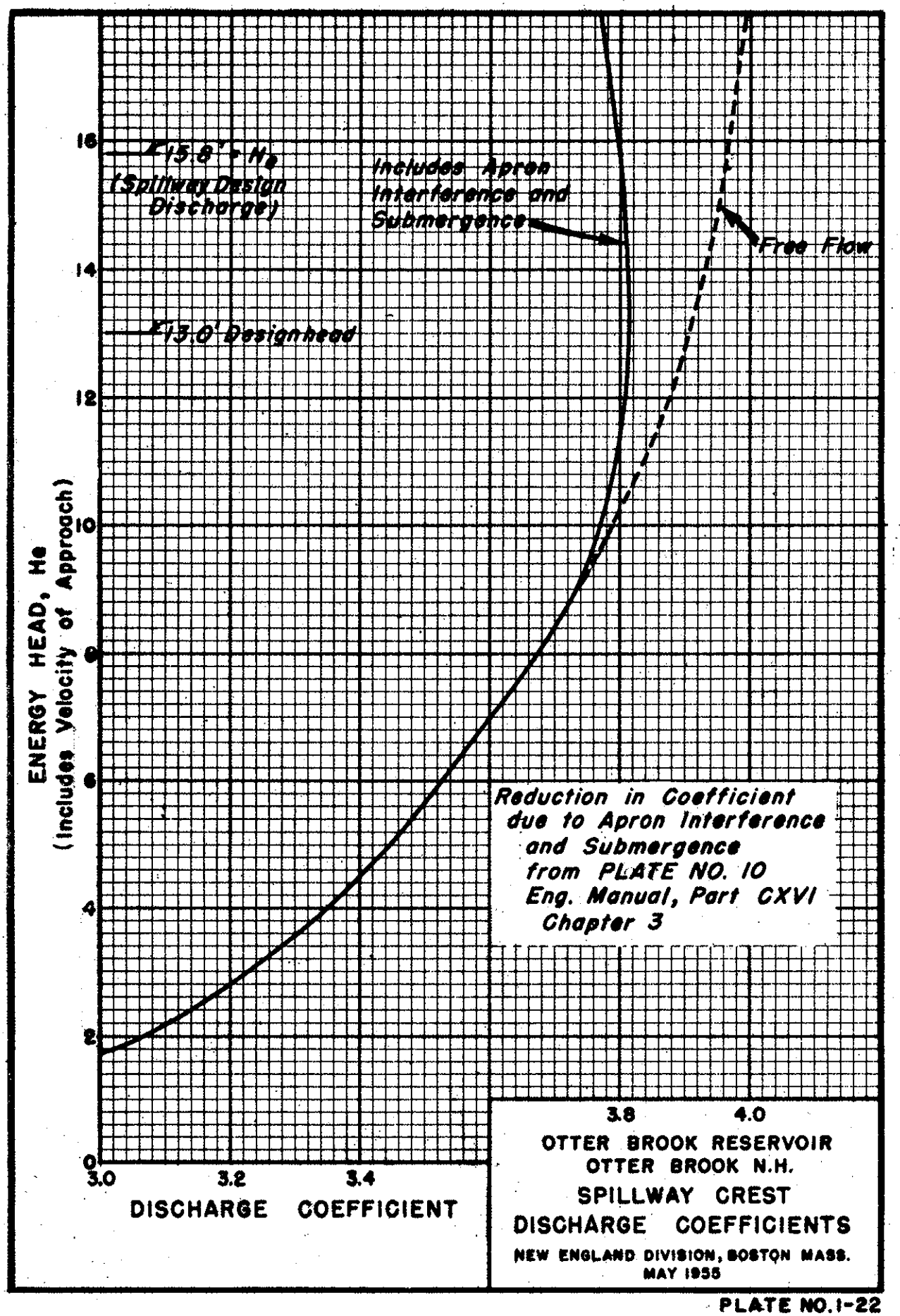
26 28 30 32 34

OTTER BROOK RESERVOIR
OTTER BROOK N.H.

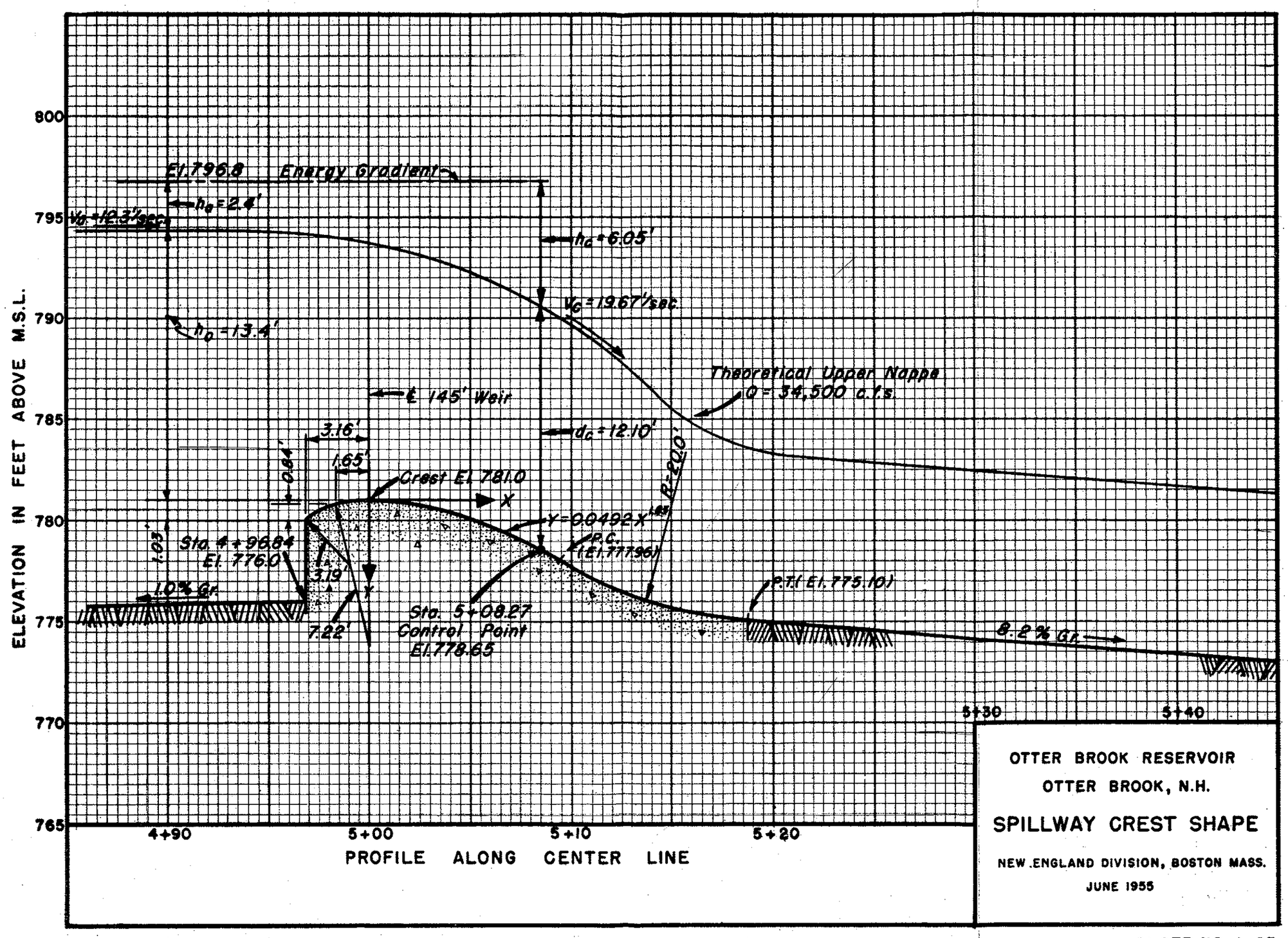
SPILLWAY
RATING CURVE

NEW ENGLAND DIVISION, BOSTON MASS.
MAY 1955

PLATE NO. I-21



OTTER BROOK RESERVOIR
OTTER BROOK N.H.
SPILLWAY CREST
DISCHARGE COEFFICIENTS
NEW ENGLAND DIVISION, BOSTON MASS.
MAY 1955



OTTER BROOK RESERVOIR

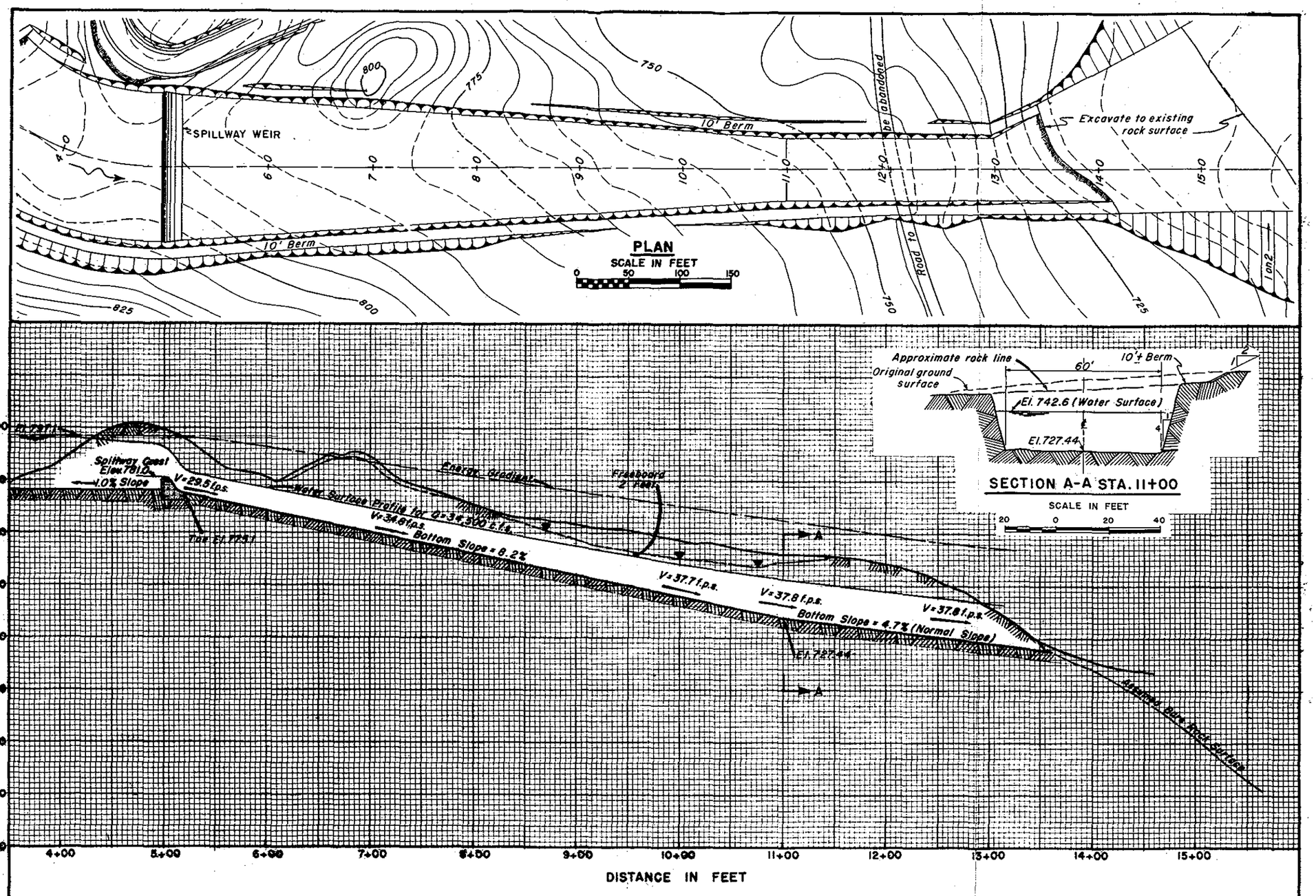
OTTER BROOK, N.H.

SPILLWAY CREST SHAPE

NEW ENGLAND DIVISION, BOSTON MASS.

JUNE 1955

PLATE NO. I-23



SECTION OF CHUTE - SPILLWAY

(LEFT BANK - MINIMUM ROCK ELEVATION IS ON LEFT SIDE OF BANK CHUTE)

LEGEND

▼ *Indicates rock elevation on left bank, 42 feet from center line. (Obtained by test pit excavations)*

OTTER BROOK RESERVOIR

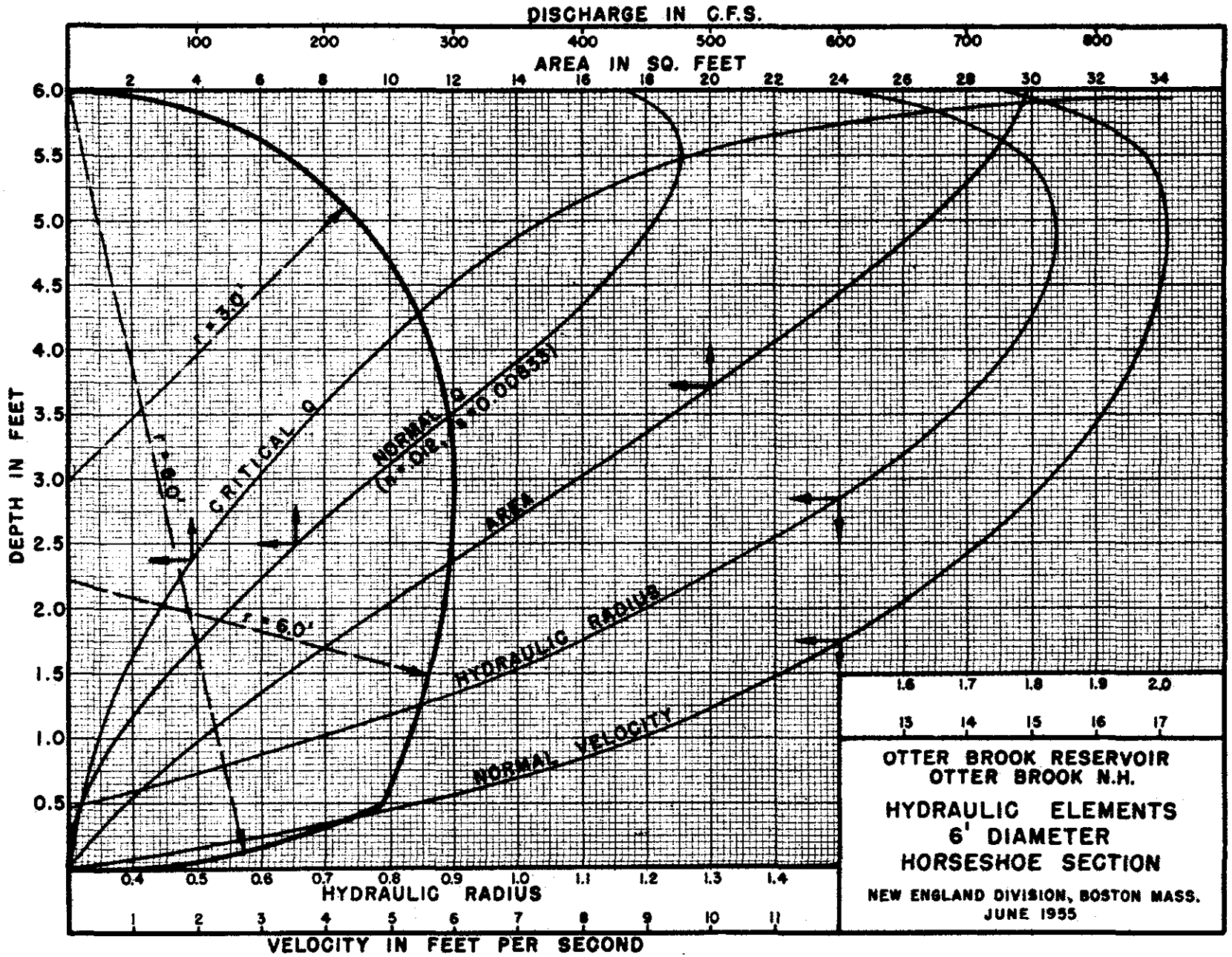
OTTER BROOK, NEW HAMPSHIRE

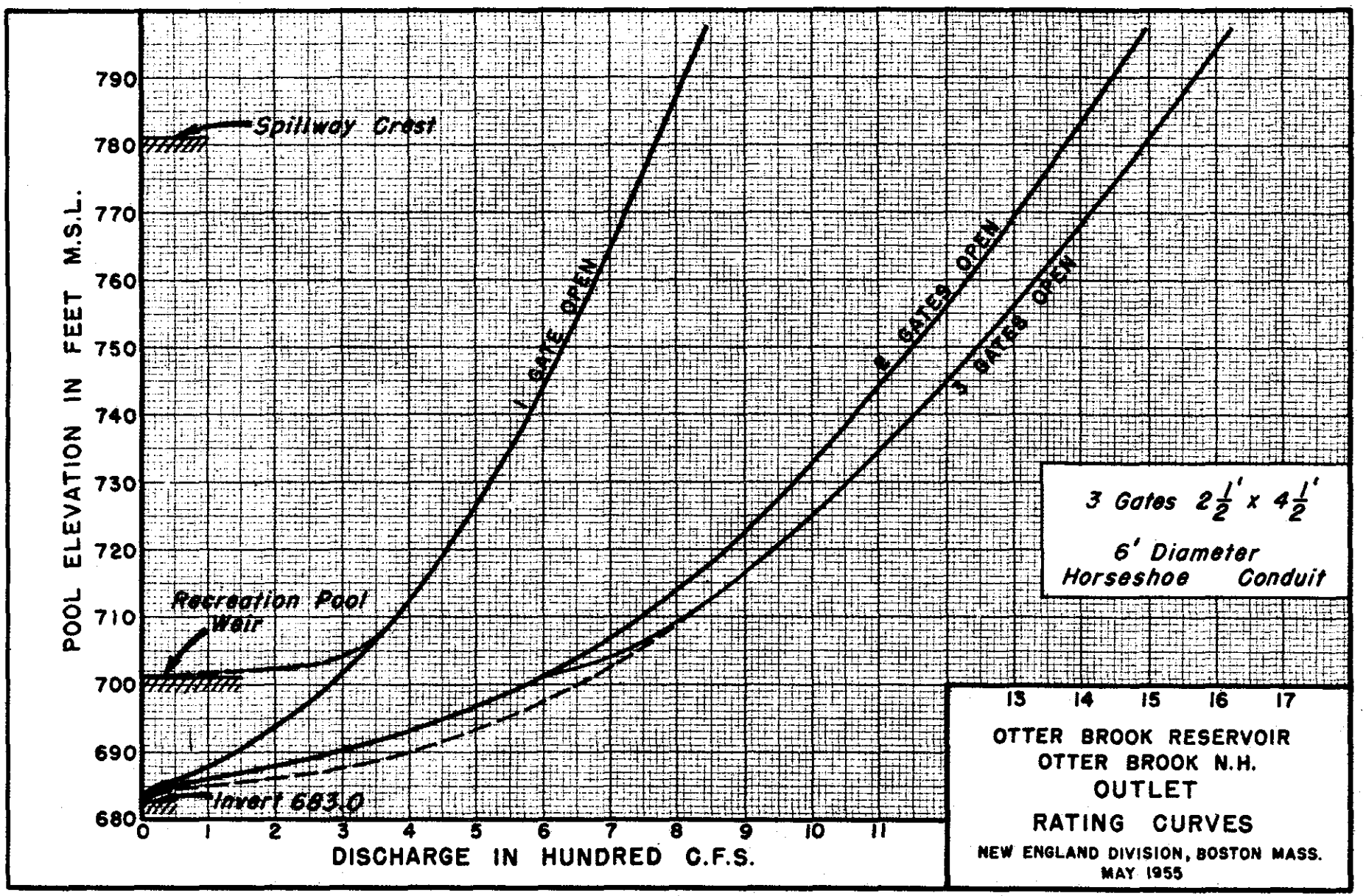
ADOPTED SPILLWAY CHUTE

NEW ENGLAND DIVISION, BOSTON MASS.

JUNE 1955

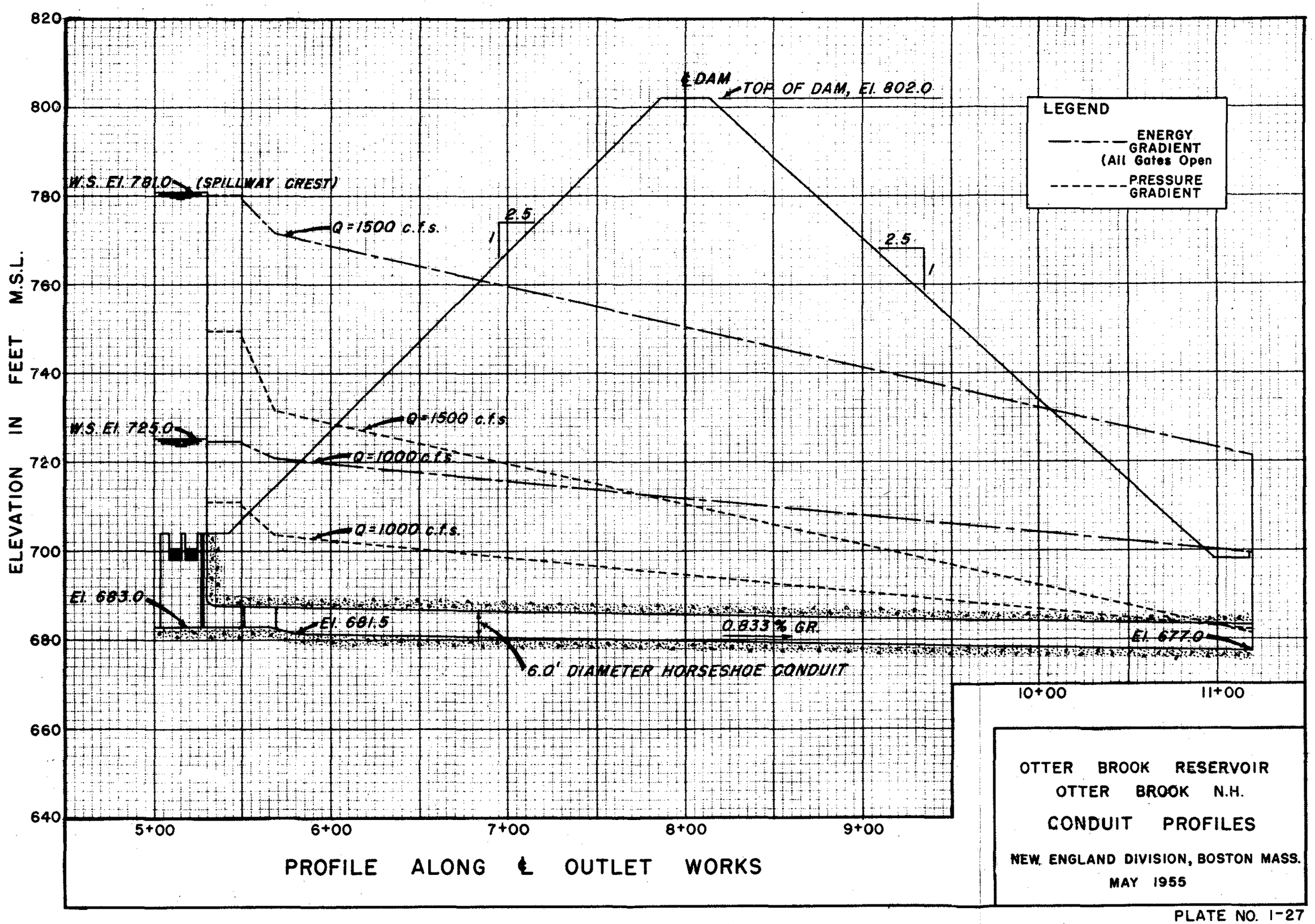
PLATE NO. I-24


DISCHARGE IN C.F.S.
100 200 300 400 500 600 700 800
AREA IN SQ. FEET
2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34
DEPTH IN FEET
6.0 5.5 5.0 4.5 4.0 3.5 3.0 2.5 2.0 1.5 1.0 0.5
CRITICAL Q
NORMAL Q
AREA
HYDRAULIC RADIUS
NORMAL VELOCITY
1.6 1.7 1.8 1.9 2.0
13 14 15 16 17
0.4 0.5 0.6 0.7 0.8 0.9 1.0 1.1 1.2 1.3 1.4
HYDRAULIC RADIUS
1 2 3 4 5 6 7 8 9 10 11
VELOCITY IN FEET PER SECOND
OTTER BROOK RESERVOIR
OTTER BROOK N.H.
HYDRAULIC ELEMENTS
6' DIAMETER
HORSESHOE SECTION
NEW ENGLAND DIVISION, BOSTON MASS.
JUNE 1955



OTTER BROOK RESERVOIR
OTTER BROOK N.H.
OUTLET
RATING CURVES
NEW ENGLAND DIVISION, BOSTON MASS.
MAY 1955

PLATE NO. I-26

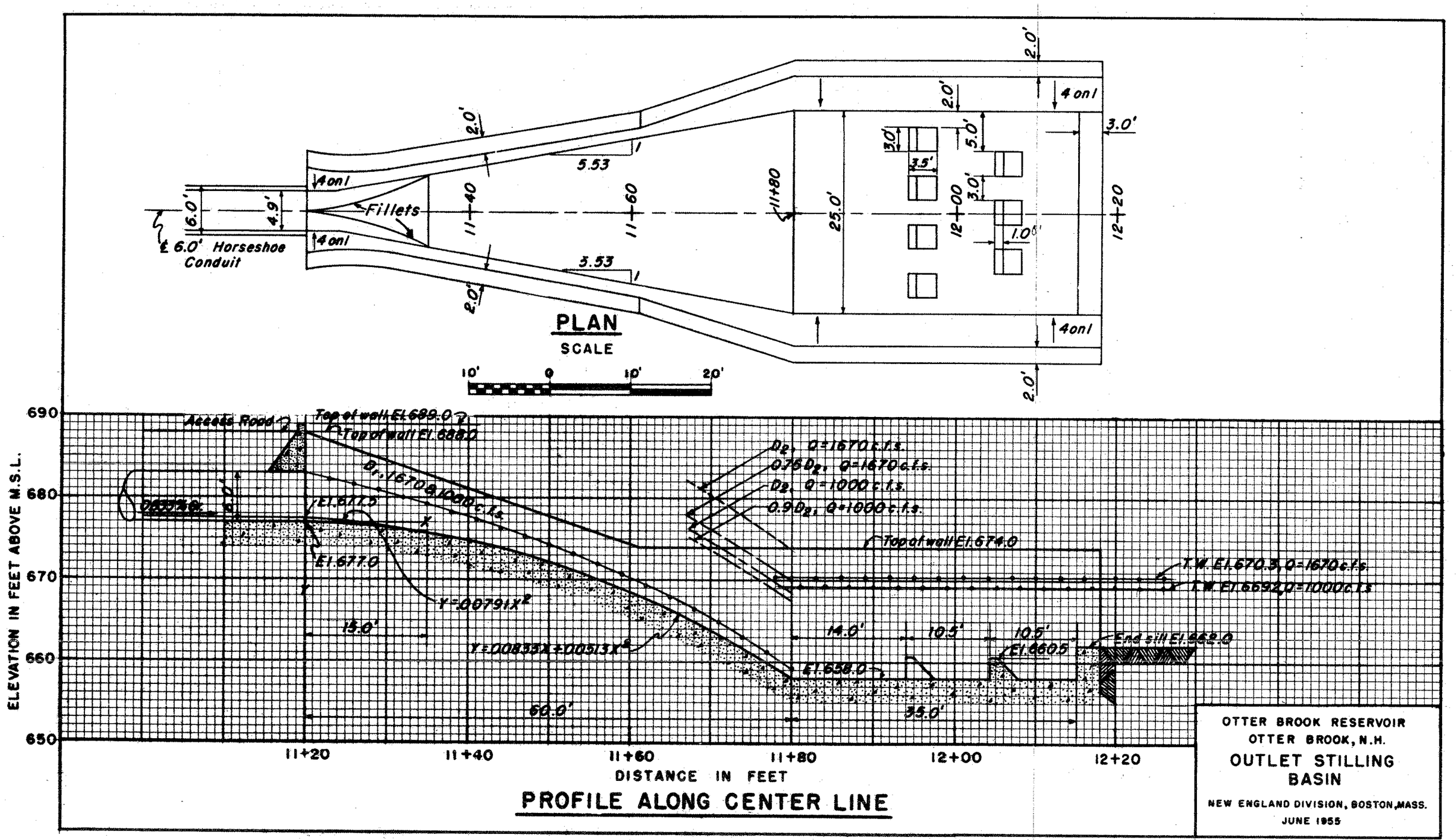


OTTER BROOK RESERVOIR
OTTER BROOK N.H.

CONDUIT PROFILES

NEW ENGLAND DIVISION, BOSTON MASS.
MAY 1955

PLATE NO. 1-27



OTTER BROOK RESERVOIR
OTTER BROOK, N.H.
OUTLET STILLING BASIN
NEW ENGLAND DIVISION, BOSTON, MASS.
JUNE 1955

PLATE NO. 1-28