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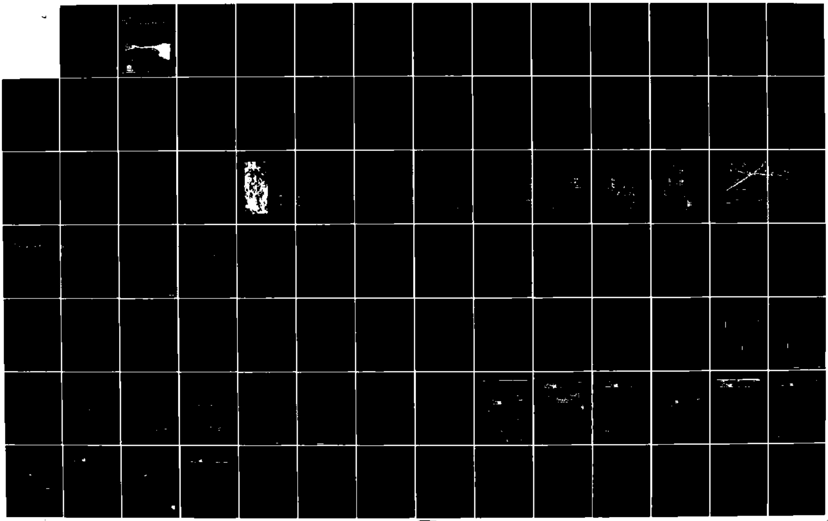

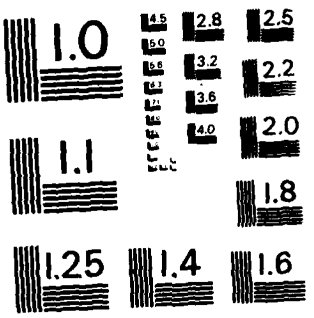

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Detailed Project Report
Beaver Brook, Keene, New Hampshire

(14)

Supporting Documentation

Flood Damage Reduction

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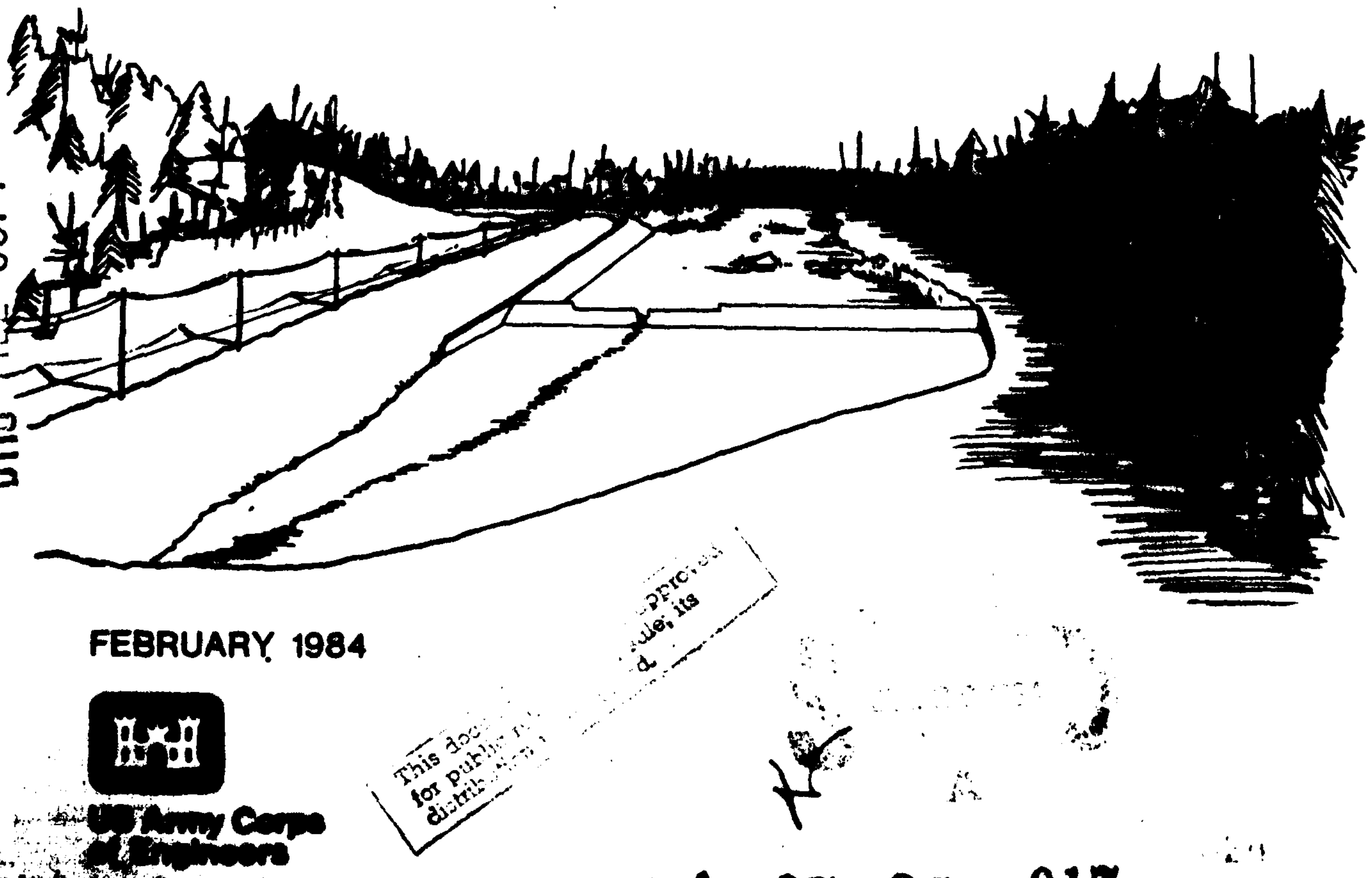

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Report contains fine appendixes

Section A- Hydrologic Analysis
B- Geotechnical and Design Condiderations
C- Economic Analysis
D- Archaeological Reconnaissance Report
E- Real Estate

FLOOD DAMAGE REDUCTION

BEAVER BROOK

KEENE, NEW HAMPSHIRE

SUPPORTING DOCUMENTATION

FOR

DETAILED PROJECT REPORT

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASSACHUSETTS 02254

FEBRUARY 1984

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BY
HYDROLOGIC ENGINEERING SECTION
WATER CONTROL BRANCH
ENGINEERING DIVISION

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION CORPS OF ENGINEERS
WALTHAM, MASSACHUSETTS

BEAVER BROOK FLOOD CONTROL
KEENE, NEW HAMPSHIRE

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HYDROLOGIC ANALYSIS

1. INTRODUCTION

a. Purpose and Scope. This report presents hydrologic information and analysis relative to flood problems on Beaver Brook located in the Ashuelot River watershed in Keene, New Hampshire. The study was performed under authority contained in Section 205 of the 1948 Flood Control Act and analysis was directed specifically to flooding on Beaver Brook in the reach approximately 1,800 feet in length between Marlboro and Water Streets. Also investigated were modifications to the outlet of the upstream "Three Mile Swamp" for purposes of increased flood control storage. Included are sections on watershed description, general climatology, flood history and frequencies, standard project flood, flood profiles, and plans of improvement. Beaver Brook flooding has been the subject of several past studies. Earlier studies by the Corps of Engineers lead to Congressional authorization in 1968 of "Beaver Brook Lake", a multipurpose water supply and flood control dam and reservoir. However, due to escalating costs and lack of local support the project was not constructed. Current studies reported herein, dealt with the hydrologic analysis of selected small scale structural improvements for inclusion, if feasible, in a primarily nonstructural flood control plan for the city of Keene. The token improvements analyzed were identified in a nonstructural flood damage reduction report completed in 1980 for the city of Keene.

b. References. Some earlier reports dealing with flood problems and hydrology of Beaver Brook are as follows:

(1) "Report on Drainage in the Beaver Brook Area", Dec. 1962, by Camp, Dresser and McKee Engrs. for the city of Keene.

(2) "Beaver Brook Dam and Reservoir, Interim Report on Review of Survey", July 1965, Revised December 1966, by New England Division, Corps of Engineers.

(3) "Channel Improvements for Beaver Brook from Beaver Street to Baker Street", Dec. 1967, by Camp, Dresser and McKee Engrs. for city of Keene.

(4) Beaver Brook Lake, Design Memo, No. 1 , "Hydrology", March 1972, by New England Division, Corps of Engineers.

(5) "Beaver Brook Improvements - Harrison Street to Baker Street", May 1975 by Whitman and Howard Engrs. for city of Keene.

(6) "Flood Plain Management Study, Keene, New Hampshire" December 1978, by Anderson-Nichols Engrs. for New England Division, Corps of Engineers.

(7) "Keene, New Hampshire - Formulation, Assessment and Evaluation of Nonstructural Flood Damage Reduction Techniques", May 1980, by Resource Analysis Engrs. for New England Division, Corps of Engineers.

2. WATERSHED DESCRIPTION

a. General. The city of Keene is at approximately 42° 56' north latitude and 72°16' west longitude in the southwest corner of the State of New Hampshire and is located on the Ashuelot River about 27 miles above its confluence with the Connecticut River near Hinsdale. The Ashuelot River drains an area of 421 square miles and the watershed (shown on plate 1) is generally hilly with low mountains in the headwaters.

b. Main tributaries. The two main tributaries of the Ashuelot River are the Branch and the South Branch. The Branch, entering the Ashuelot just below Keene, New Hampshire, about 26.5 miles upstream from the mouth, is formed by the confluence of Minnewawa and Otter Brooks. The South Branch joins the Ashuelot just above Swanzey Station, or about 23.5 miles upstream from the mouth.

c. Keene flood plain. The reach of the Ashuelot River between the Faulkner and Colony Company Dam in Keene and the Dickinson Dam in West Swanzey is referred to as the Keene flood plain. This reach is the principal flood damage area in the Ashuelot basin. The meandering river channel in the flood plain has low discharge capacity due to its small cross sectional area and flat gradient, with the result that floodwaters cause considerable depth of pondage. About 75 percent of the Ashuelot River drainage area empties into this reach of the river.

d. Beaver Brook. Beaver Brook, with a drainage area of about 10 square miles, flows southward through the city of Keene, joining the Branch near its mouth in the Keene flood plain. The watershed is rectangular in shape with a length of about 7 miles and a width of about 1.5 miles. It has basically a single stream pattern with short side tributaries and steep slopes. Although Beaver Brook has a total fall of over 700 feet in about 8 miles of length, the lower 2 miles in the city of Keene and that portion in the vicinity of the "Three Mile Swamp" are relatively flat as indicated by the profile shown on plate 2. The Beaver Brook watershed is hdyrologically flashy due to the mountainous headwater terrain.

A multipurpose water supply and flood control reservoir, "Beaver Brook Lake", was authorized for construction by the Flood Control Act of 13 August 1968. However, due to escalating cost and lack of local support the project was not constructed. "Three Mile Swamp", a natural swamp partially controlled by a low rock dam, is located within the originally authorized reservoir area. Pertinent data on the Three Mile Swamp is listed in Table I.

TABLE I

THREE MILE SWAMP
PERTINENT DATA

Drainage Area (square miles)	6.0
Dam Elevation Approximate (feet NGVD)	790
Outlet Elevation Approximate (feet NGVD)	787
Dam Height Approximate (feet)	5
Dam Length (feet)	220
Swamp Area Approximate (acres)	70

e. Existing Flood Control Projects. Surry Mountain and Otter Brook Lakes are two Corps of Engineers flood control projects located in the Ashuelot River watershed.

Surry Mountain Dam, completed in 1942, is located in the town of Surry on the Ashuelot River about 6 miles upstream of the Faulkner and Colony Dams. It controls a drainage area of 100 square miles, with the reservoir containing a flood control storage equivalent to 5.9 inches of runoff.

Otter Brook Dam, completed in 1958, is in the city of Keene on Otter Brook about 5 miles upstream of the confluence of the Branch and the Ashuelot River. It controls a drainage area of 47 square miles and has storage equivalent to 7.0 inches of runoff.

3. CLIMATOLOGY

a. General. The Beaver Brook watershed has a variable climate characterized by frequent but generally short periods of heavy precipitation. Some intense rainfalls are produced by local thunderstorms and others by larger weather systems of tropical or extratropical origin moving up the eastern coast. The watershed also lies in the path of prevailing westerlies which traverse the country in an easterly or northeasterly direction producing frequent weather changes. Winters are moderately severe, with subzero temperatures rather common. Spring melting of the winter snow cover generally occurs in late March or April.

b. Temperatures. The mean annual temperature at Keene, New Hampshire is approximately 46° Fahrenheit, with the average monthly temperatures varying from about 70° in July to near 20° in January. Extremes in temperature range from highs slightly in excess of 100°F to lows in the -30 degrees. Table II summarizes mean, maximum and minimum monthly temperatures recorded at Keene, New Hampshire for 94 years of record through 1980.

c. Precipitation. The mean annual precipitation at Keene is 38.9 inches, with the greatest annual precipitation of 52.7 inches recorded in 1975 and the least was 27.1 inches recorded in 1894. Table III summarizes the precipitation at Keene for an 89 year period through 1980.

d. Snowfall. The mean annual snowfall at Keene is about 64 inches. Table IV lists mean monthly values, based on 88 years of record through 1980.

e. Snow Cover. Snow surveys have been taken in the Ashuelot River watershed since December 1948. These surveys indicate that the water equivalent of the snow cover reaches its average maximum of 4.7 inches about mid-March. Some mean, maximum and minimum water equivalents of snow cover in the Ashuelot basin for the later winter months are listed in table V.

4. STREAMFLOW

The US Geological Survey has published records of streamflows at 5 locations in the Ashuelot River watershed; however, there are no gaging stations on Beaver Brook. The two nearest gages recording runoff from uncontrolled areas are the South Branch Ashuelot River at Webb, New Hampshire (D.A. 36 square miles) and the Ashuelot River near Gilsum, New Hampshire (D.A. 71.1 square miles). After analyzing the physical characterics of the drainage areas above these two gages, it was determined that the runoff character of the South Branch Ashuelot was probably most similar to that of Beaver Brook. Mean, maximum and minimum runoff values in cfs and inches for 58 years of record at South Branch are listed in table VI. The mean annual runoff is about 58 percent of the mean annual precipitation.

Peak flows for each water year of record, on the South Branch and estimate peak flows experienced on Beaver Brook are listed in table VII. Flows on Beaver Brook were estimated from high watermark information and developed stage discharge relations. Peak flow rates were also compared with rainfall-runoff computations and recorded runoff rates from other watersheds in the region.

TABLE II

MONTHLY TEMPERATURES
KEENE, NEW HAMPSHIRE
(94 Years of Record Through 1980)

Month	Mean (°F.)	Maximum (°F.)	Minimum (°F.)
January	21.3	66	-32
February	22.5	65	-21
March	32.9	85	-21
April	44.6	91	1
May	56.0	95	21
June	64.7	98	27
July	69.5	104	34
August	67.3	102	27
September	60.0	101	19
October	49.3	90	10
November	37.7	80	-15
December	25.5	64	-29
Annual	45.9	104	-32

TABLE III

MONTHLY PRECIPITATION
KEENE, NEW HAMPSHIRE
(89 Years of Record Through 1980)

Month	Mean (inches)	Maximum (inches)	Minimum (inches)
January	2.96	9.24	0.76
February	2.62	7.02	0.57
March	3.20	7.60	0.40
April	3.15	6.65	0.35
May	3.35	7.02	0.79
June	3.46	7.73	0.41
July	3.72	11.09	1.07
August	3.62	8.96	1.05
September	3.53	10.39	0.20
October	2.84	7.84	0.23
November	3.33	7.67	0.52
December	3.16	8.86	0.51
Annual	38.9	52.7	27.1

TABLE IV

MEAN MONTHLY SNOWFALL
KEENE, NEW HAMPSHIRE
(88 Years of Record Through 1980)

Month	Snowfall (inches)
January	16.4
February	16.3
March	11.4
April	3.2
May	0
June	0
July	0
August	0
September	0
October	.1
November	3.7
December	13.2
Annual	64.3

TABLE V

WATER EQUIVALENT OF SNOW COVER
ASHUELOT RIVER WATERSHED
(December 1948-April 1981)

Date	Minimum	Mean	Maximum
1 February	0.1	2.9	7.7
15 February	0.0	3.6	8.5
1 March	0.0	4.3	9.6
15 March	0.0	4.7	9.4
1 April	0.0	3.3	8.9
15 April	0.0	1.2	6.5

TABLE VI

MONTHLY RUNOFF
SOUTH BRANCH ASHUELOT RIVER
AT WEBB, NEW HAMPSHIRE
(D.A. = 36 square miles)
(58 Years of Record Through 1978)

Month	Average		Maximum		Minimum	
	CFS	Inches	CFS	Inches	CFS	Inches
January	56.6	1.81	161	5.16	6.4	0.20
February	49.8	1.45	153	4.60	9.3	0.27
March	112	3.58	366	11.73	23.1	0.74
April	174	5.39	356	11.05	64.4	1.99
May	81.1	2.60	186	5.97	26.6	0.85
June	44.1	1.37	151	4.69	7.0	0.22
July	23.0	0.73	102	3.25	3.0	0.09
August	17.0	0.54	131	4.21	2.7	0.09
September	22.1	0.69	252	7.81	2.6	0.08
October	25.9	0.83	133	4.26	2.9	0.09
November	52.4	1.62	244	7.55	3.7	0.11
December	59.9	1.92	178	5.70	9.5	0.30
Annual	59.7	22.5	105	39.9	17.3	6.5

TABLE VII

ANNUAL PEAK DISCHARGES

Water Year	South Branch Ashuelot at Webb, N.H. (D.A. = 36.0 sq. mi.)	Beaver Brook Keene, N.H. (D.A. = 9.0 sq. mi.)
1921	1560 cfs	
1922	1160	
1923	1400	
1924	1220	
1925	1680	
1926	630	
1927	575	
1928	3560	
1929	1010	
1930	535	
1931	660	
1932	1150	
1933	879	
1934	1300	
1935	619	
1936	3880	900 (March 36)
1937	628	
1938	5960	2000 (Sept 38)
1939	580	
1940	1910	
1941	370	
1942	1780	
1943	476	
1944	1470	
1945	725	
1946	1020	
1947	485	
1948	1070	
1949	660	
1950	615	500 (Nov 50)
1951	2010	
1952	822	
1953	875	
1954	806	
1955	511	
1956	1700	
1957	475	
1958	766	
1959	2070	
1960	4350	1100 (Oct 59)(600 (Apr 60)
1961	445	
1962	878	
1963	758	
1964	552	
1965	325	
1966	828	
1967	750	
1968	1260	
1969	1030	
1970	983	
1971	545	
1972	681	
1973	1210	900 (Dec 73)
1974	2520	
1975	1870	
1976	1110	
1977	2010	
1978	1900	
Mean Log	= 3.0077	
Standard Deviation	= 0.2732	

5. ANALYSIS OF FLOODS

a. General. Flooding on Beaver Brook has been a recurring problem since the earliest times. Some of the more damaging floods in this century occurred in November 1927, April 1934, March 1936, September 1938, November 1950, October 1959, April 1960 and December 1973. The greatest flood occurred in September 1938 and this event together with the lesser flood of October 1959 was analyzed in developing rainfall-runoff relationships for the stream.

From previous studies it was determined that the lower portion of Beaver Brook is in the flood plain of the Ashuelot River. This flood plain is a large storage reach with its outlet at the dam on the Ashuelot River in West Swanzey. With moderate floods, Beaver Brook stages are a function of the discharge on Beaver Brook. With greater floods the Keene flood plain storage increasingly fills, eventually causing a backwater effect on the lower end of Beaver Brook. The stages on Beaver Brook lower reach then become a function of both the flow in the brook and the coincident level of the flood plain.

b. September 1938. This flood-producing storm was the result of a stationary cold front along the Atlantic coast overrun by a rapidly moving tropical hurricane, producing record breaking rainfall over large areas of Connecticut, Massachusetts, and New Hampshire. The storm which started with light rain, gradually increased in intensity over the 4-day period (17-21 September). The total rainfall was over 10 inches in the Keene area with a resulting runoff in the area of about 6 inches. Three hour rainfall amounts in the Keene area and runoff amoungs as determined by analysis of the South Branch are listed in table VIII. The peak discharge at the South Branch gage during this flood was the maximum recorded to date. The peak discharge on the South Branch (D.A. = 36 square miles) was 5,960 cfs. The 1938 flow on Beaver Brook (D.A. = 10 square miles) was estimated to have been about 2,000 cfs.

The 1938 flood on Beaver Brook was modeled using the Hydrologic Computer Program HEC-1. One-half hour unit graphs were developed for the Three Mile Swamp watershed (6.0 square miles) and the intervening local area (3.0 square miles) to Marlboro Street. Unit graph characteristics were determined using adopted Snyder's coefficients. Pertinent characteristics of the adopted one-half hour unit graphs are listed in table IX. The computer model was tested by applying 1938 excess rainfall and determining the degree to which the experienced peak flows could be reproduced. Once calibrated, the model was used to compute a range of storm runoff hydrographs. Analyses of the 1938 and 100-year synthetic flood are graphically illustrated on plates 3 and 4. Storage-discharge characteristics of Three Mile Swamp were determined from photogrammetric mapping and surveys of the outlet structure. The swamp outlet is presently a 220-foot long stone weir with a crest

TABLE VIII

SEPTEMBER 1938 STORM RAINFALL*

Time (day)	(hrs)	Rainfall (inches)	Loss (inches)	Excess (inches)
19 Sept	3	0.50	0.50	0
19 Sept	6	0.90	0.30	0.60
19 Sept	9	0.30	0.30	0
19 Sept	12M	0.0	0	0
20 Sept	3	0.55	0.55	0
20 Sept	6	0.40	0.40	0
20 Sept	9	0.25	0.25	0
20 Sept	12N	0.75	0.40	0.35
20 Sept	3	0	0	0
20 Sept	6	0	0	0
20 Sept	9	0	0	0
20 Sept	12M	1.00	0.30	0.70
21 Sept	3	1.70	0.40	1.30
21 Sept	6	1.90	0.40	1.50
21 Sept	9	1.30	0.40	0.90
21 Sept	12N	0	0	0
21 Sept	3	1.40	0.40	1.00
Total		10.95	4.60	6.35

*From "Analysis of Design for West Peterborough Dam", November 1947, Plate No. IV-11.

TABLE IX

ONE-HALF HOUR UNIT HYDROGRAPHS
PERTINENT DATA

Subbasins	Drainage Area (sq. mi.)	L (miles)	L_{ca} (miles)	Slope (ft/ft)	C_t	640 C_p	T_p (hrs)	Q_p
10 Year & Oct. 1959								
Three Mile Swamp	6	4.0	2.0	.019	1.6	190	3	390
Local	3	4.0	2.0	.016	1.6	190	3	195
100 Year & Sept. 1938								
Three Mile Swamp	6	4.0	2.0	.019	1.1	190	2	570
Local	3	4.0	2.0	.016	1.1	190	2	285
SPF								
Three Mile Swamp	6	4.0	2.0	.019	1.1	240	2	710
Local	3	4.0	2.0	.016	1.1	240	2	355

elevation of approximately 790 feet NGVD. This long weir results in large incremental increased in outflow with relatively small incremental increases in surcharge storage, thereby minimizing any flood modifying potential of Three Mile Swamp.

c. October 1959. This flood was the result of about 4 inches of rain in 24 hours, with 1.5 inches occurring in one hour in the Keene area, and producing high rates of runoff particularly on smaller tributary streams such as Beaver Brook.

Both the experienced October 1959 flood and the 10-year frequency storm runoff were analyzed using the HEC-1 computer model with adopted rainfall runoff relationships. Pertinent data of the adopted one-half hour unit graphs are listed in table IX. The October 1959 and 10-year synthetic floods are graphically illustrated on plates 3 and 4.

d. Other Floods. The November 1927 event resulted from 4 to 6 inches of rainfall on ground saturated from excessive rains during the previous month. The April 1934 flood was produced by a combination of heavy rains and considerable snowmelt. The March 1936 event resulted from two major rainstorms totaling 6 inches, combined with heavy snowmelt, causing two major rises in river stages about six days apart. The November 1950 flood resulted from 3 to 4 inches of intense rainfall on previously wet ground. The April 1960 event occurred when 3 to 4 inches of rain fell on snow with a high water content. The December 1973 flood was the result of over 3 inches of rainfall on snow covered ground.

6. FLOOD FREQUENCIES

Peak discharge frequencies for ungaged Beaver Brook were developed utilizing the 58-years of South Branch flow records and computed Beaver Brook flows for selected floods. Flows on Beaver Brook were based on high watermark information and developed stage-discharge relations plus recorded runoff rates from other watersheds in the region. Annual peak discharges for the 58-year period of record on the South Branch plus selected Beaver Brook flood discharges are listed in table VII. Three different procedural approaches, or steps, were taken in developing the adopted frequency curve for Beaver Brook. A frequency curve was first computed for the South Branch and transferred to Beaver Brook using a transfer factor equal to the ratio of drainage areas to the 0.7 power. The curve for the South Branch was computed using a Log Pearson Type III statistical analysis with "expected probability" adjustment. The computed frequency curve for the South Branch, with a drainage are of 36 square miles, had a mean log of 3.008, a standard deviation of 0.2732 and an adopted skew of 0.7. The curve transferred to Beaver Brook (D.A. = 9.0 square miles) had the same standard deviation and skew with an adjusted mean log of 2.588.

Secondly, the limited number of flood discharges for Beaver Brook were assigned Weibull plotting positions based on their relative magnitude and positioning in the 58-year historical South Branch record. These plotted points were then compared with the transferred curve for reasonableness of fit. There was reasonably good agreement between the plotted data and computed curve.

Lastly, as a further comparative analysis, the 100-year and 10-year storm rainfall excess as determined from US Weather Bureau Technical Paper No. 40, was applied to the developed HEC-1 computer model and the resulting Beaver Brook discharges were checked for relative agreement with the adopted frequency curve. Again there was reasonably good agreement. The developed frequency curves and plotted data are shown on plate 5.

7. STANDARD PROJECT FLOOD

a. General. The Standard Project Flood (SPF) represents the flood discharge that may be expected from the most severe combination of meteorologic and hydrologic conditions that are considered reasonably characteristic of the region, excluding extremely rare combinations. The SPF represents a "Standard" against which the flood potential of a river can be judged, as contrasted to an analysis of flood records which may be misleading due to abnormal sequences of events during the period of record. The SPF for Beaver Brook was developed by applying standard project rainfall to adopted subbasin unit hydrographs and combining and routing the resulting component hydrographs.

b. Rainfall. Standard project storm rainfall was determined in accordance with Civil Engineer Bulletin 52-8 and EM 1110-2-1411. The 24-hour index rainfall for the 9 square mile Beaver Brook watershed was 10.4 inches. Losses were assumed to be .15 inch per 2 hours, resulting in a 24-hour rainfall excess of 8.7 inches. Two hour rainfall values are listed in table X. Rainfall rate was assumed uniform within each 2 hour interval.

c. Unit Hydrographs. The watershed was divided into two subbasins, the area above Three Mile Swamp and the remaining downstream local. Unit hydrographs for the two subbasins were "peaked" 25 percent to reflect the more rapid runoff rates expected under standard project storm conditions. Pertinent data on the adopted SPF unit graphs are listed in table IX.

d. Standard Project Discharges. Rainfall excess was applied to the unit hydrographs and the SPF hydrographs computed using the HEC-1 computer model. The resulting hydrograph for the area above Three Mile Swamp was routed through the swamp by "modified puls" and the computed outflow was combined with the downstream local to produce the total SPF for Beaver Brook.

Development of the SPF for the Beaver Brook watershed is graphically illustrated on plate 4.

8. FLOOD PROFILES

Flood profiles and stage-discharge relationships for Beaver Brook were computed utilizing the backwater computer program, HEC-2, and conventional hydraulic formulae. Backwater computations were made using cross sectional surveys developed by Anderson Nichols Company during the "Keene Flood Plain Management Study". Analysis extended from downstream of Marlboro Street to upstream of Harrison Street. Backwater computations were made for both natural and modified conditions using a Manning's "n" of 0.032 for the channel and ranging from .045 to .06 for overbank flow areas. Assumed contraction and expansion loss coefficients were, respectively, 0.3 and 0.5. A starting stage-discharge relationship was developed downstream of Marlboro Street using backwater data developed in the earlier "Keene Flood Plain Management Study". Computed natural and modified profiles plus established December 1973 high water elevations are shown on plate 7.

9. PLANS OF IMPROVEMENT

a. General. Hydrologic analyses were made of two structural plans of improvement for flood damage reduction on Beaver Brook. One plan increased channel capacity by improving the brook channel and the second reduced floodflows by increasing upstream flood storage. Neither plan provided standard project flood protection but were geared more to providing some flood reduction at least for the more frequent flood events. Any such structural plans of improvement would be considered only as one component in a comprehensive flood control plan including nonstructural flood plain zoning and flood insurance. Analyses were performed to determine the potential magnitude and frequency of flood level reductions for use in determining project feasibility.

b. Channel Improvements. Channel improvements were investigated for the damage reach extending from Marlboro to Water Streets, a total distance of approximately 1,800 feet. Both an 8-foot deep by 20-foot wide rectangular channel (160 square feet) and an equivalent trapezoidal channel were investigated. The improvements would be quite comparable hydraulically, to an improved channel section already built at one location in the reach by the Kingsbury Tool Company. Past improvements to Beaver Brook were generally designed for a flow of about 600 cfs. The improvement would provide some stage reduction for floods in the 10 to 20 year frequency range but would provide negligible reduction for the larger but rarer floods. As stated previously, the lower reach of Beaver Brook is affected by backwater from the Ashuelot "Keene flood plain"; also the gradient of the brook in this reach is extremely flat.

TABLE X

STANDARD PROJECT STORM RAINFALL

Time (hours)	Rainfall (inches)	Loss (inches)	Excess (inches)
2	.1	.1	0
4	.2	.15	.05
6	.3	.15	.15
8	.4	.15	.25
10	1.5	.15	1.35
12	5.4	.15	5.25
14	.9	.15	.75
16	.6	.15	.45
18	.4	.15	.25
20	.3	.15	.15
22	.2	.15	.05
24	.1	.10	0
Total	10.4	1.70	8.70

These two features limit the extent of flood reduction possibilities via channel improvements. Also, any type of channel improvement in this lower reach would have to be continually maintained free of debris and sediment build-up in order to remain effective.

Natural and modified stage frequency curves in the reach were determined by performing backwater computations for a range of flows. The resulting curves are shown on plate 9.

c. Three Mile Swamp Storage.

(1) General. Three Mile Swamp is a natural retention wetland located about 4.5 miles upstream on Beaver Brook. An analysis was made of the outlet structure of this swamp with respect to increasing the flood storage effectiveness of this swamp. Such improvements would not, or should not, be viewed as a flood control dam and reservoir but simply structural modifications to increase the effectiveness of the natural flood retention area.

(2) Dam Modifications. The existing swamp outlet structure is an old stone masonry dam with a 220-foot long stone weir at approximately elevation 790 feet NGVD. This weir results in large incremental increases in outflow with relatively small increases in surcharge storage, thereby minimizing the flood modifying potential of the Three Mile Swamp.

The alternate weir investigated would consist of a stepped weir, thus varying delta storage versus delta flow, i.e., "time of storage", with changing magnitude of flow. The alternate outlet would be a self-regulating structure with an 8-foot width at elevation 787, 50 feet at elevation 792, and 200 feet at elevation 794 feet NGVD. Discharge rating for the weir was computed using a weir coefficient of 3.0. Storage capacity of the swamp was computed by planimetering available 2 foot contour maps, assuming a horizontal swamp profile. Plan and profile of the swamp, in the area of the outlet structure, and outlet rating curves (existing and modified) and area-capacity curves of the swamp are shown on plate 8.

(3) Flood Discharge Reductions. The HEC-1 computer model, previously developed, was used to route a range of floods through Three Mile Swamp using the developed storage data and the revised outlet rating for the swamp.

Plate 3 graphically illustrates the modifying effects of the alternate outlet structure on the September 1938 and October 1959 floods, while plate 4 illustrates the SPF, 100- and 10-year flood analysis. Plate 6 shows the comparative natural and modified discharge frequency curves. The modified curve was determined by analyzing the above floods for both natural and modified conditions and drawing a representative modified frequency curve.

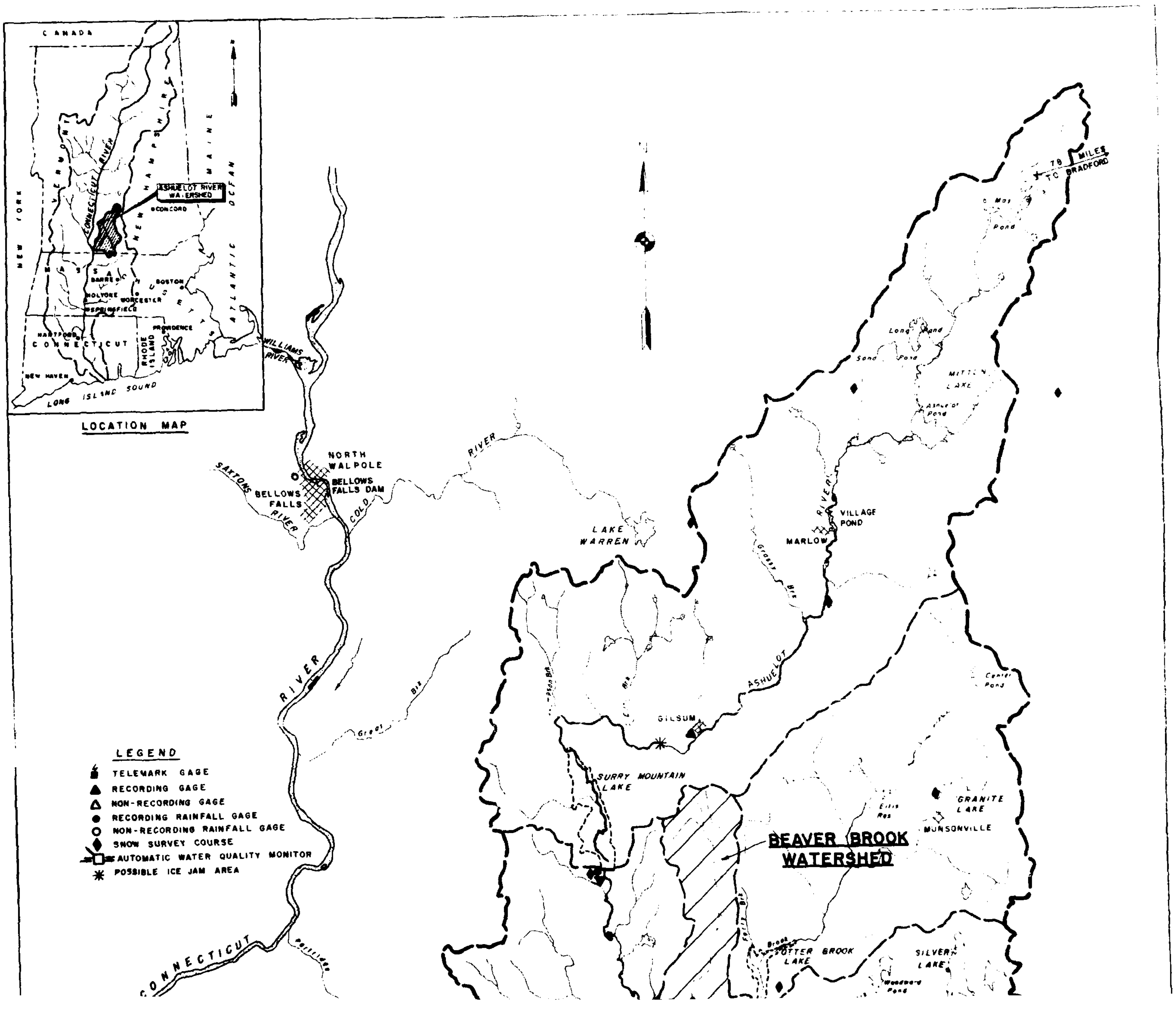
(4) Design Flood. The modified Three Mile Swamp outlet would in effect be a relatively low (generally less than 10 feet in height) run-of-river type overflow dam. Therefore, due to the relatively low height, the standard project flood was adopted as the spillway design flood and the structure would be designed to safely pass the SPF discharge of 4,300 cfs with a minimum of 2 feet of freeboard. A low dike would be required on the east side of Route 10 to prevent ponding on that highway during flood storage periods. Top elevation of the dike would be 799 feet NGVD providing 2 feet of freeboard above the SPF level of 797 feet NGVD. Local drainage west of the dike would discharge to Beaver Brook downstream of the dam structure. A listing of computed inflows, outflows, and storages in the swamp are presented in table XI.

(5) Guide Taking Elevation. Since the modified outlet structure would cause an increase in the magnitude and frequency of flooding in the swamp, land taking or easements would be required. The recommended guide taking elevation for land taking has been set at elevation 797, standard project flood level, encompassing approximately 120 acres of land. A plan and profile of the Three Mile Swamp area, including flood levels, are shown on plate 8.

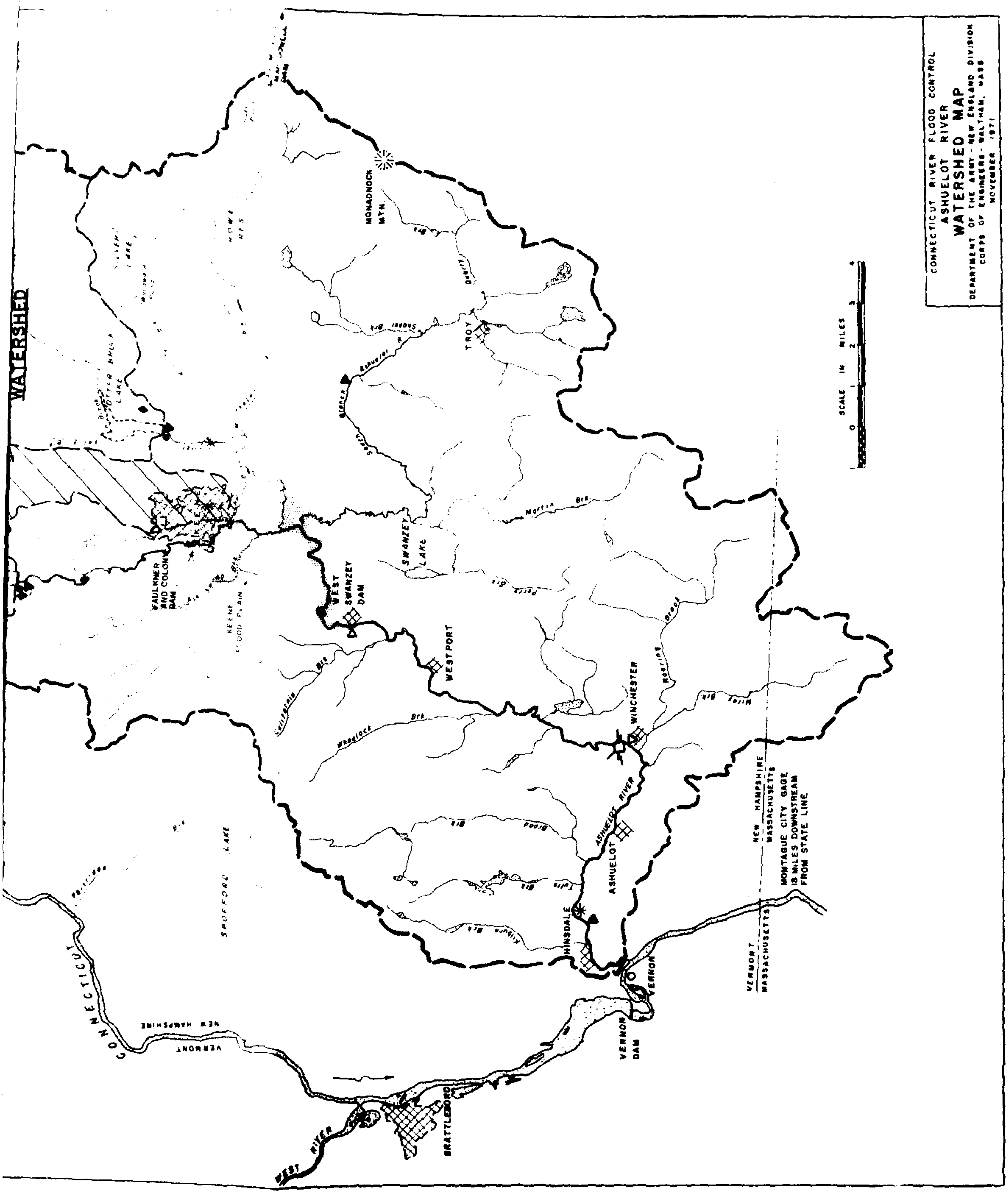
TABLE XI

BEAVER BROOK
THREE MILE SWAMP FLOOD DATA

Flood	Natural		Modified		
	Outflow (cfs)	Stage (ft NGVD)	Outflow (cfs)	Stage (ft NGVD)	Storage (inches(R.O.)
Sept 1938	1340	791	1230	795	1.0
Oct 1959	720	790.6	410	793.1	.6
10 Year	680	790.6	370	792.9	.5
100 Year	1730	791.3	1170	794.9	1.0
SPF	4300	792.7	3800	797.0	1.6



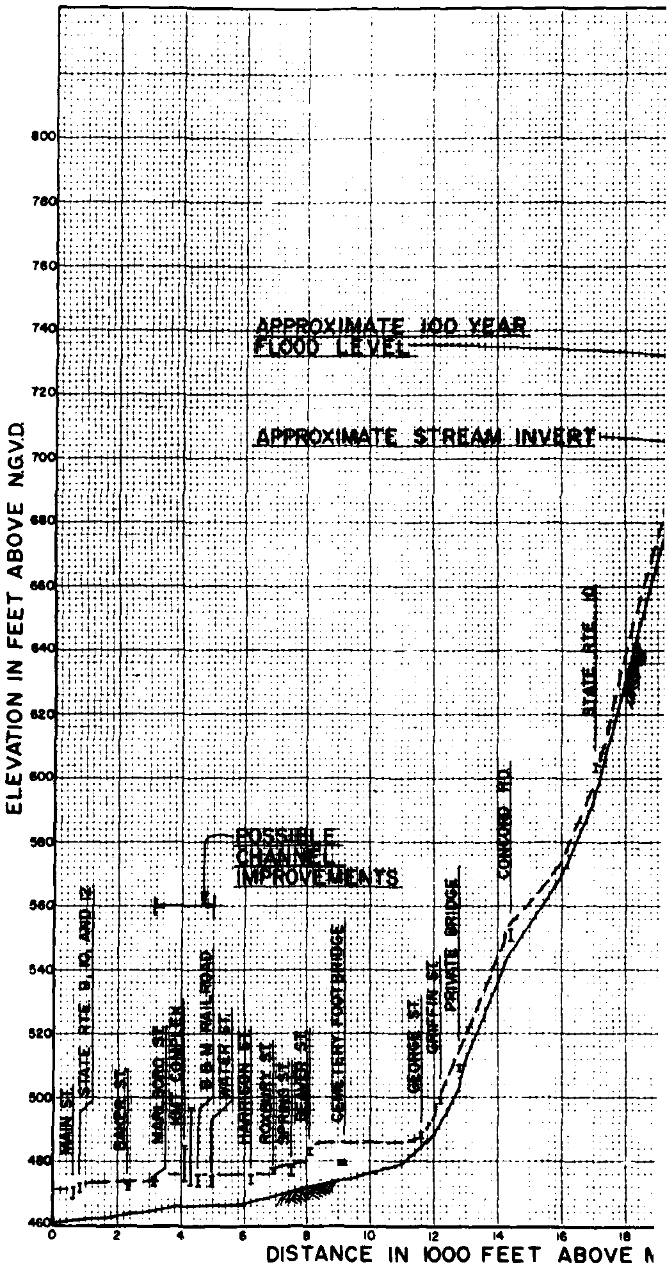
LOCATION MAP
CANADA
NEW YORK
VERMONT
NEW HAMPSHIRE
MAINE
CONNECTICUT RIVER
ASHUELOT RIVER WATERSHED
CONCORD
MASSACHUSETTS
BARRE
BOSTON
HOLYOKE
WORCESTER
SPRINGFIELD
HARTFORD
CONNECTICUT
RHODE ISLAND
PROVIDENCE
NEW HAVEN
LONG ISLAND SOUND
ATLANTIC OCEAN
WILLIAMS RIVER
NORTH WALPOLE
BELLOWS FALLS DAM
BELLOWS FALLS
SAXTONS RIVER
COLD RIVER
LAKE WARREN
RIVER
CONNECTICUT
Partridge
Great Brk
78 MILES TO BRADFORD
Long Pond
Sand Pond
MITTEN LAKE
Ashuelot Pond
VILLAGE POND
MARLOW
RIVER
Grassy Brk
ASHUELOT
GILSUM
SURRY MOUNTAIN LAKE
Center Pond
Ellis Res
GRANITE LAKE
MUNSONVILLE
BEAVER BROOK WATERSHED
OTTER BROOK LAKE
SILVER LAKE
Woodward Pond
LEGEND
TELEMARK GAGE
RECORDING GAGE
NON-RECORDING GAGE
RECORDING RAINFALL GAGE
NON-RECORDING RAINFALL GAGE
SNOW SURVEY COURSE
AUTOMATIC WATER QUALITY MONITOR
POSSIBLE ICE JAM AREA

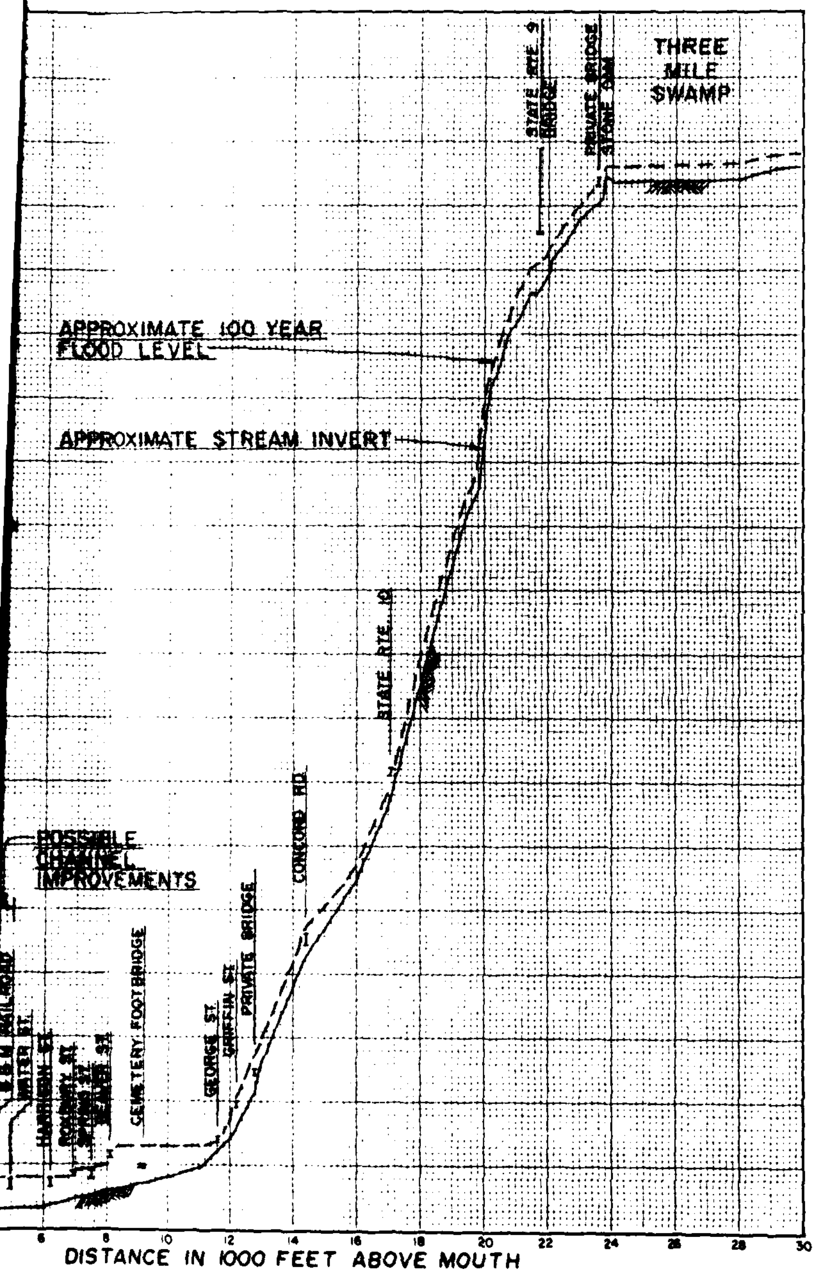

CONNECTICUT RIVER FLOOD CONTROL
ASHUELOT RIVER
WATERSHED MAP
DEPARTMENT OF THE ARMY - NEW ENGLAND DIVISION
CORPS OF ENGINEERS - WALTHAM, MASS
NOVEMBER 1971
SCALE IN MILES
0 1 2 3 4
WATERSHED
MONADNOCK MTN.
TROY
Ashuelot R
Martin Brk
SWANZEY LAKE
WEST SWANZEY DAM
WESTPORT
WINCHESTER
Roaring Brook
Wheelock Brk
KEENE
FAULKNER AND COLONY DAM
KEENE FLOOD PLAIN
ASHUELOT RIVER
HINSDALE
VERNON
VERNON DAM
NEW HAMPSHIRE
MASSACHUSETTS
MONTAGUE CITY GAGE 18 MILES DOWNSTREAM FROM STATE LINE
VERMONT
MASSACHUSETTS
SPOFFORD LAKE
CONNECTICUT
NEW HAMPSHIRE
VERMONT
BRATTLEBORO
WEST RIVER

2 PLATE 1

CORPS OF ENGINEERS

PLAN
SCALE: 1" = 2000'
0 2000 4000





DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

ASHUELOT RIVER BASIN

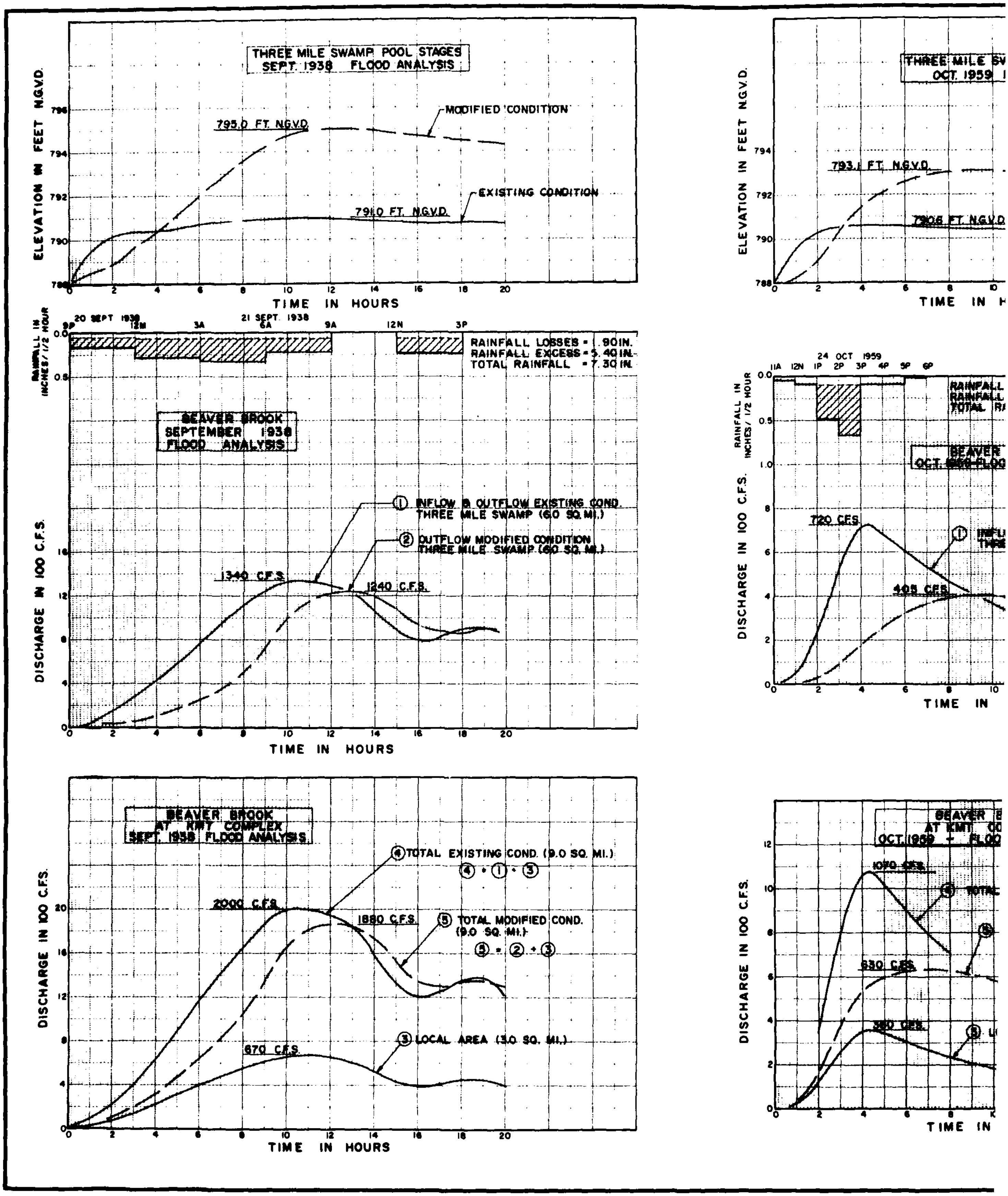
BEAVER BROOK KEENE, N.H.

BEAVER BROOK WATERSHED MAP AND PROFILE

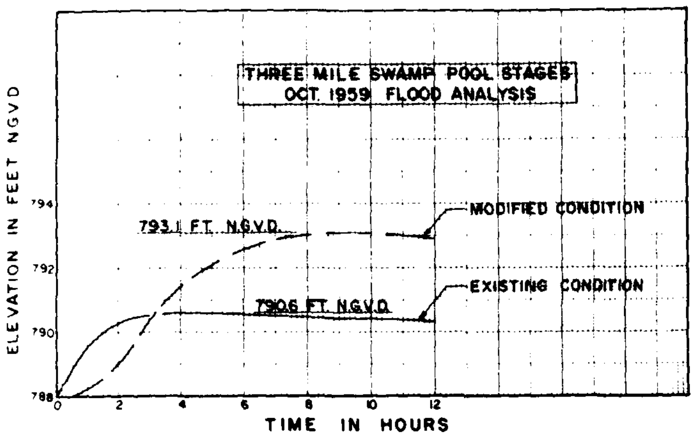
HYDRO ENG SECT FEB 1983

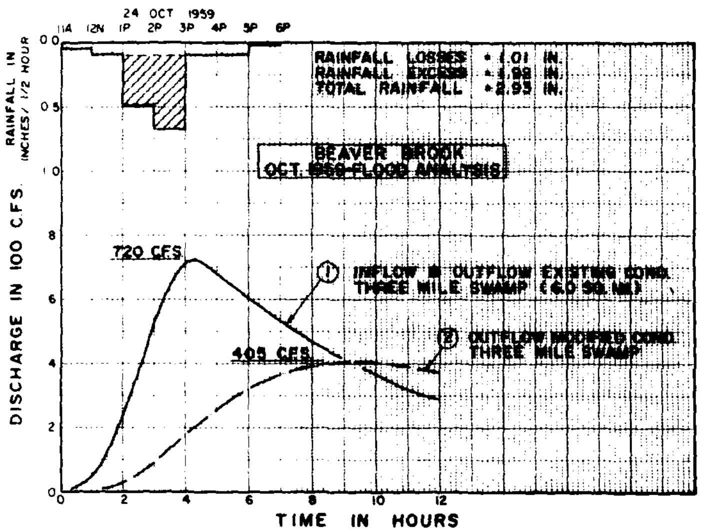
PLATE A-2

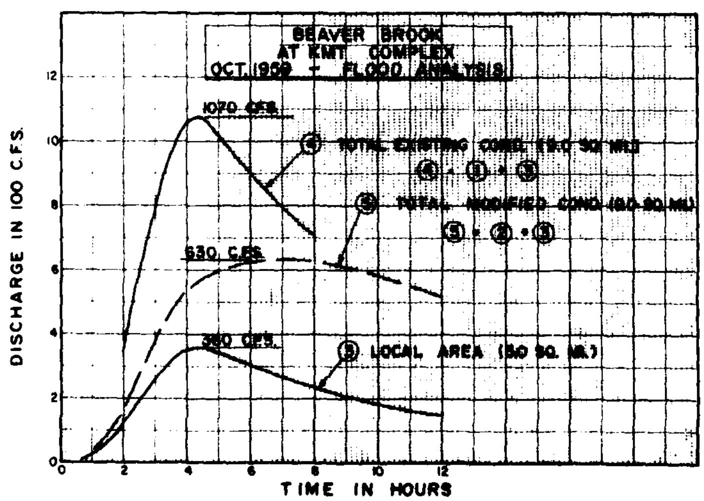
2



THREE MILE SWAMP POOL STAGES
SEPT. 1938 FLOOD ANALYSIS
ELEVATION IN FEET N.G.V.D.
795.0 FT. N.G.V.D.
MODIFIED CONDITION
791.0 FT. N.G.V.D.
EXISTING CONDITION
TIME IN HOURS
20 SEPT 1938
21 SEPT. 1938
RAINFALL IN INCHES/ 1/2 HOUR
RAINFALL LOSSES = 1.90 IN.
RAINFALL EXCESS = 5.40 IN.
TOTAL RAINFALL = 7.30 IN.
BEAVER BROOK
SEPTEMBER 1938
FLOOD ANALYSIS
DISCHARGE IN 100 C.F.S.
① INFLOW & OUTFLOW EXISTING COND. THREE MILE SWAMP (6.0 SQ. MI.)
② OUTFLOW MODIFIED CONDITION THREE MILE SWAMP (6.0 SQ. MI.)
1340 C.F.S.
1240 C.F.S.
TIME IN HOURS
BEAVER BROOK
AT KMT COMPLEX
SEPT. 1938 FLOOD ANALYSIS
DISCHARGE IN 100 C.F.S.
④ TOTAL EXISTING COND. (9.0 SQ. MI.)
④ = ① + ③
2000 C.F.S.
1880 C.F.S.
⑤ TOTAL MODIFIED COND. (9.0 SQ. MI.)
⑤ = ② + ③
670 C.F.S.
③ LOCAL AREA (3.0 SQ. MI.)
TIME IN HOURS
THREE MILE SW
OCT. 1959
ELEVATION IN FEET N.G.V.D.
793.1 FT. N.G.V.D.
790.6 FT. N.G.V.D.
TIME IN H
24 OCT 1959
RAINFALL IN INCHES/ 1/2 HOUR
BEAVER
OCT. 1959 FLOO
DISCHARGE IN 100 C.F.S.
720 CFS
405 CFS
TIME IN
BEAVER B
AT KMT CO
OCT. 1959 FLOO
DISCHARGE IN 100 C.F.S.
1070 CFS
630 CFS
360 CFS
TIME IN







NOTES:
MINIMUM INVERT OF EXISTING AND MODIFIED OUTLET OF THREE MILE SWAMP AT ELEVATION 787 FT. N.G.V.D.

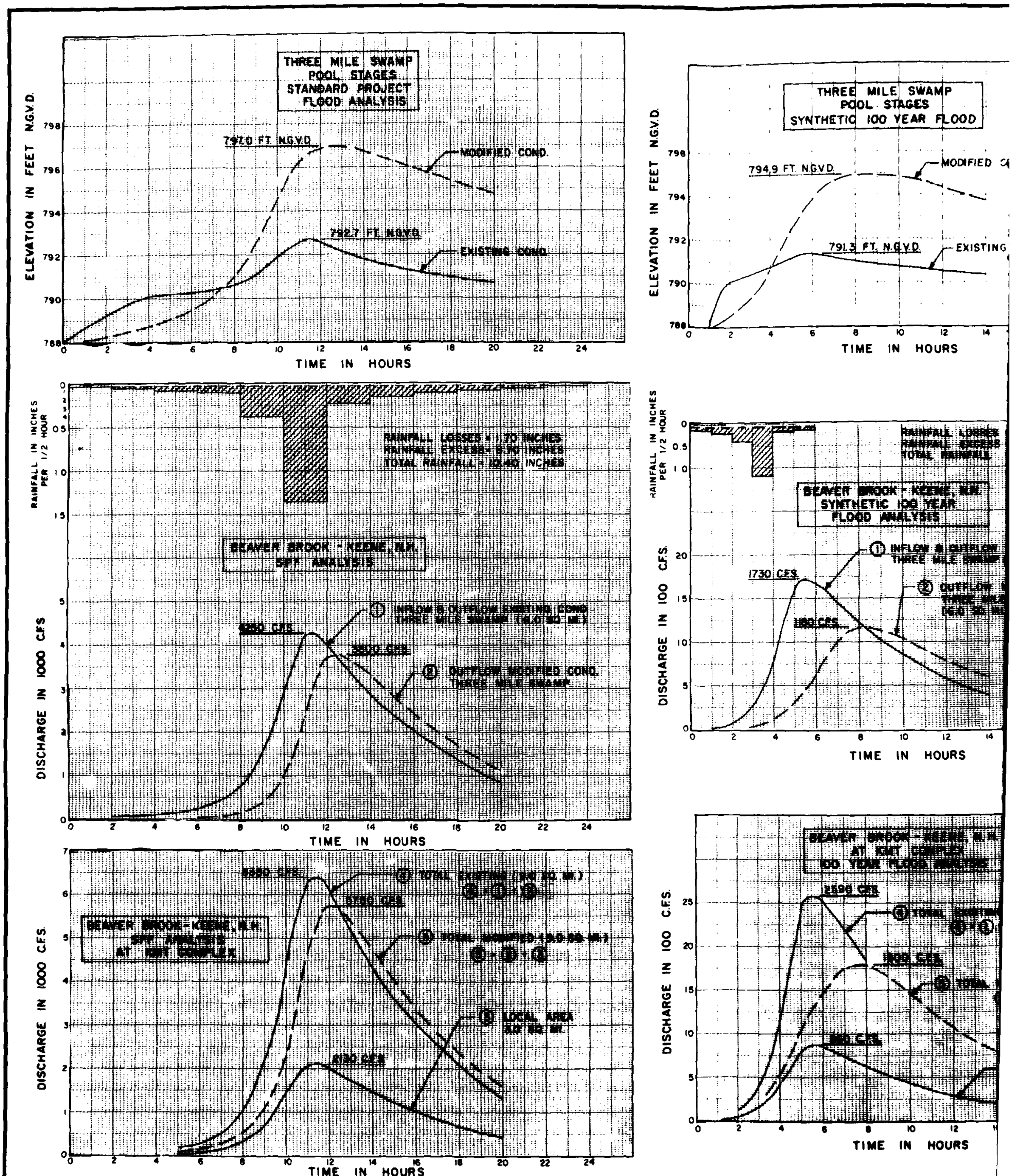
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

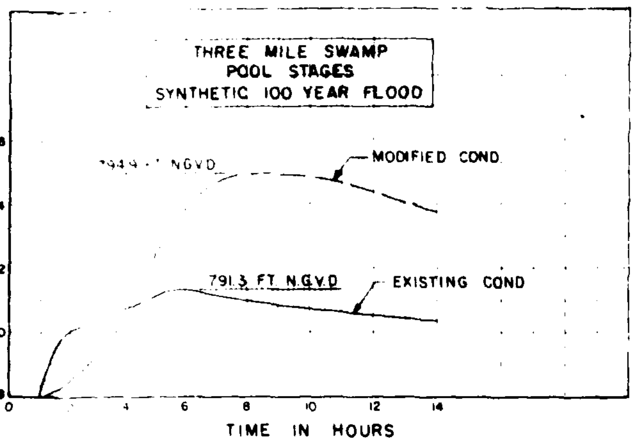
ASHUELOT RIVER BASIN
BEAVER BROOK - KEENE, N.H.
ANALYSIS OF FLOODS
SEPT. 1938 & OCT. 1959

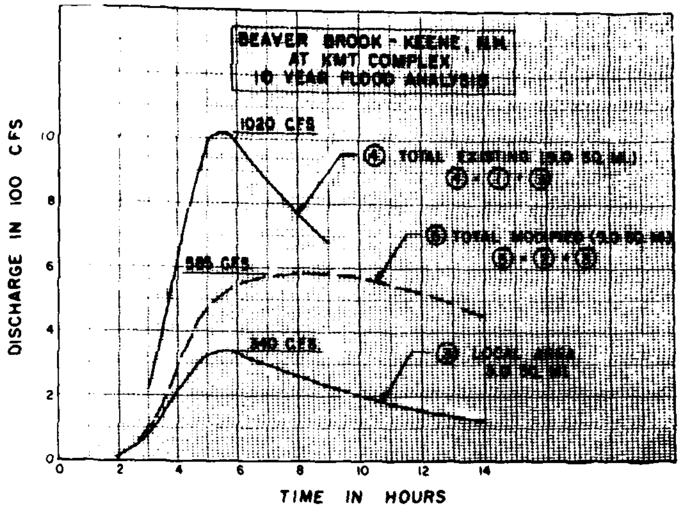
HYDRO. ENG. SECT. FEB. 1983

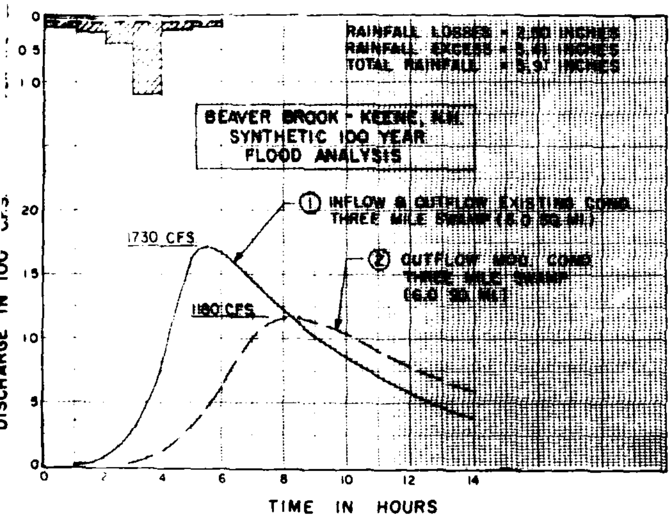
PLATE A-3

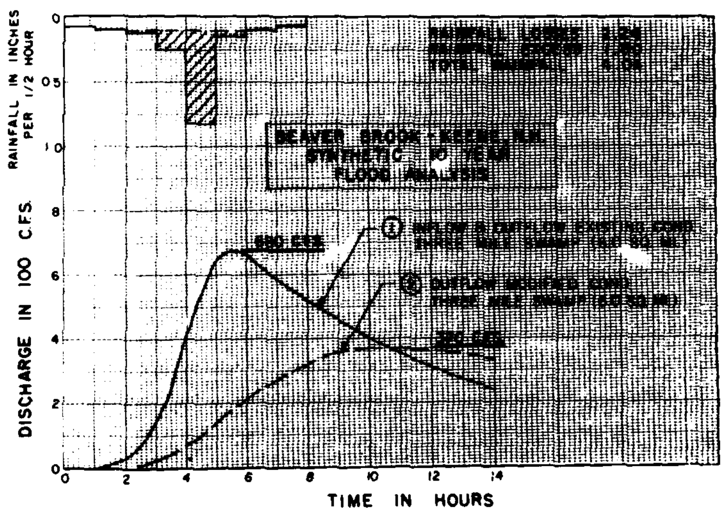
2

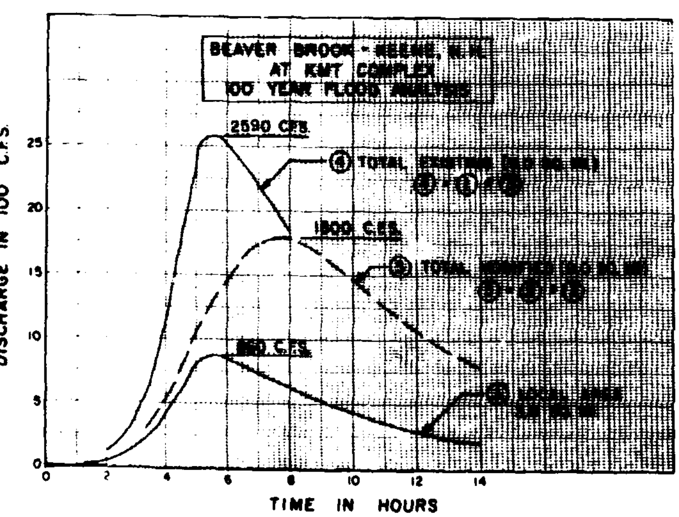

THREE MILE SWAMP
POOL STAGES
STANDARD PROJECT
FLOOD ANALYSIS
797.0 FT. N.G.V.D.
MODIFIED COND.
792.7 FT. N.G.V.D.
EXISTING COND.
ELEVATION IN FEET N.G.V.D.
TIME IN HOURS
THREE MILE SWAMP
POOL STAGES
SYNTHETIC 100 YEAR FLOOD
794.9 FT. N.G.V.D.
791.3 FT. N.G.V.D.
RAINFALL IN INCHES PER 1/2 HOUR
RAINFALL LOSSES = 1.70 INCHES
RAINFALL EXCESS = 8.70 INCHES
TOTAL RAINFALL = 10.40 INCHES
BEAVER BROOK - KEENE, N.H.
SPF ANALYSIS
INFLOW & OUTFLOW EXISTING COND.
THREE MILE SWAMP
OUTFLOW MODIFIED COND.
THREE MILE SWAMP
3800 C.F.S.
DISCHARGE IN 1000 C.F.S.
BEAVER BROOK - KEENE, N.H.
SYNTHETIC 100 YEAR
FLOOD ANALYSES
1730 C.F.S.
DISCHARGE IN 100 C.F.S.
BEAVER BROOK - KEENE, N.H.
SPF ANALYSIS
AT KMT COMPLEX
TOTAL EXISTING
TOTAL MODIFIED
LOCAL AREA
2130 C.F.S.
BEAVER BROOK - KEENE, N.H.
AT KMT COMPLEX
100 YEAR FLOOD ANALYSIS
2590 C.F.S.

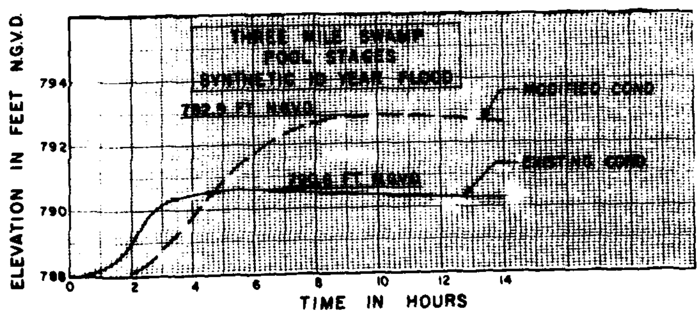












NOTES:
MIN. INVERT OF EXISTING AND MODIFIED OUTLET OF THREE MILE SWAMP AT ELEVATION 787 FT. N.G.V.D.

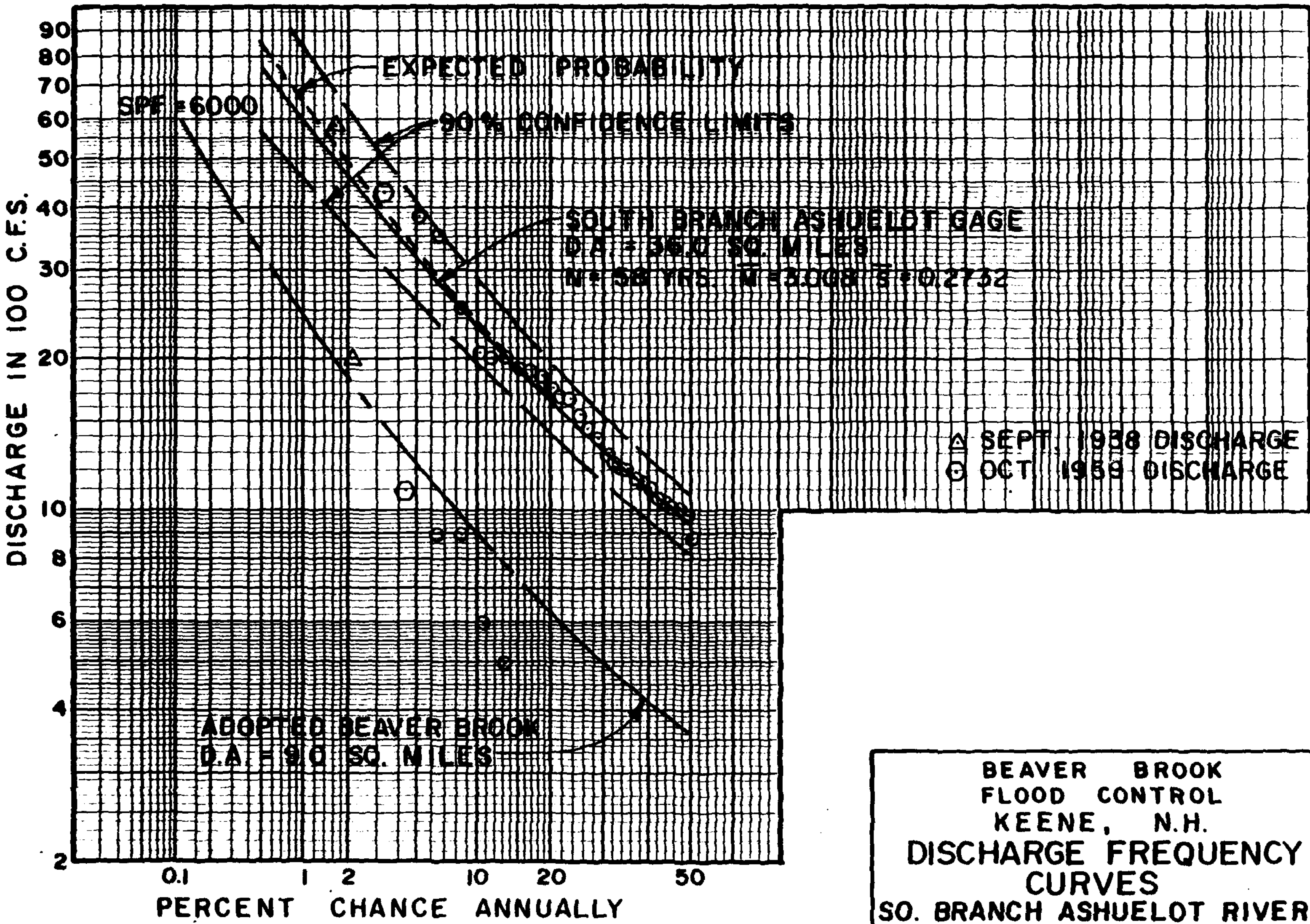
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

ASHUELOT RIVER BASIN
BEAVER BROOK - KEENE, N.H.
ANALYSIS OF FLOODS
SPF 100 YR. - 10 YR.

HYDRO. ENGR. SECT FEB. 1983

PLATE A-4

2

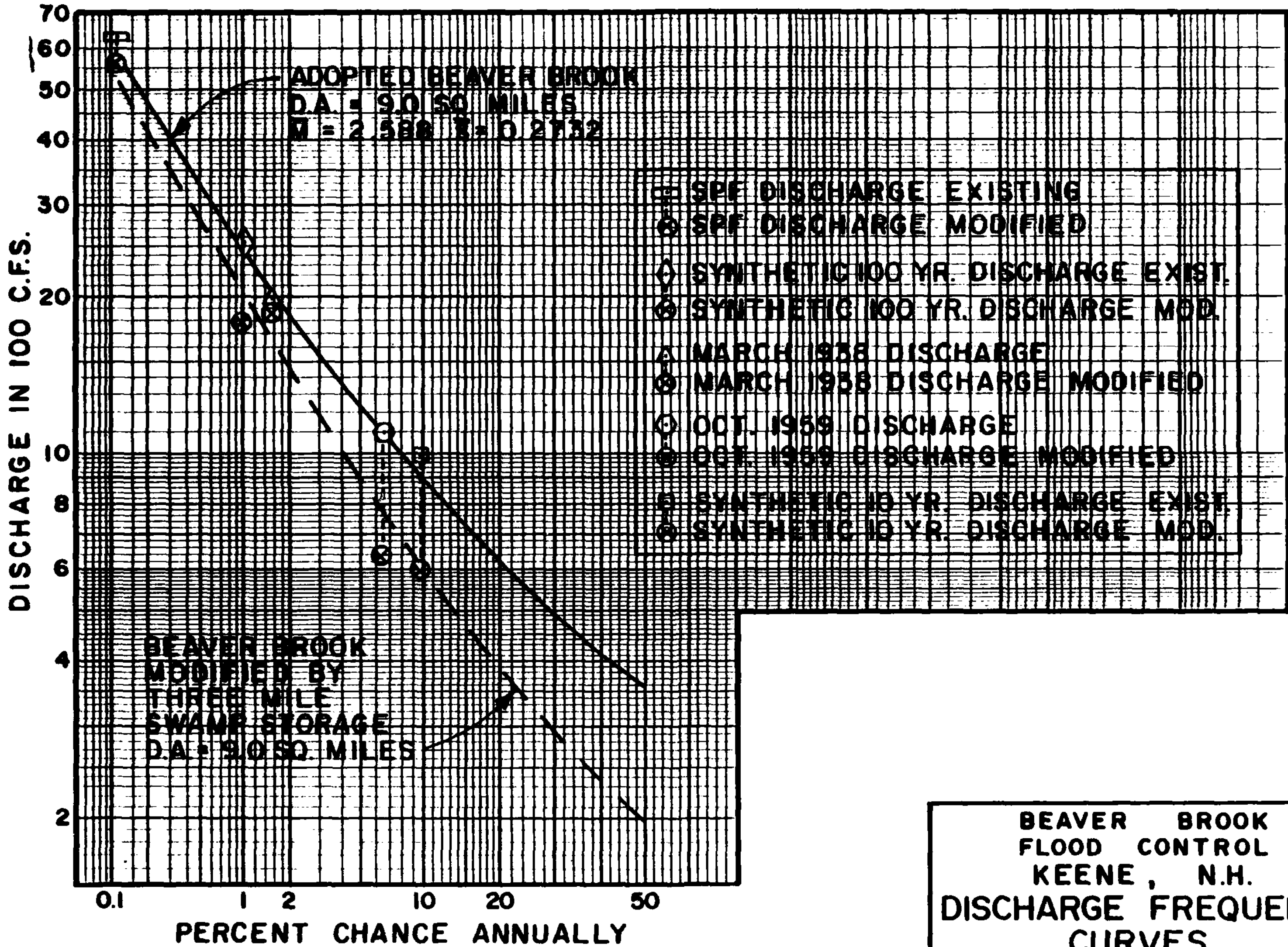


BEAVER BROOK
FLOOD CONTROL
KEENE, N.H.

DISCHARGE FREQUENCY CURVES

SO. BRANCH ASHUELOT RIVER & BEAVER BROOK.

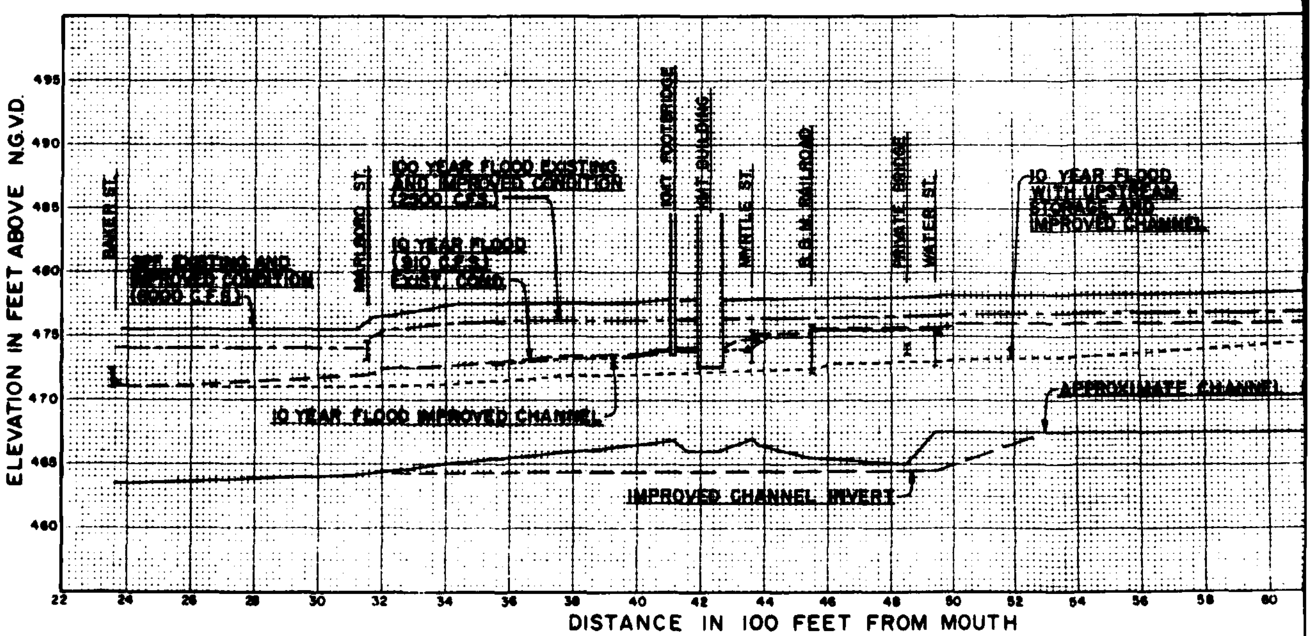
PLATE A-5



BEAVER BROOK
FLOOD CONTROL
KEENE, N.H.
DISCHARGE FREQUENCY
CURVES
BEAVER BROOK
NATURAL & MODIFIED

PLATE A-6

CORPS OF ENGINEERS

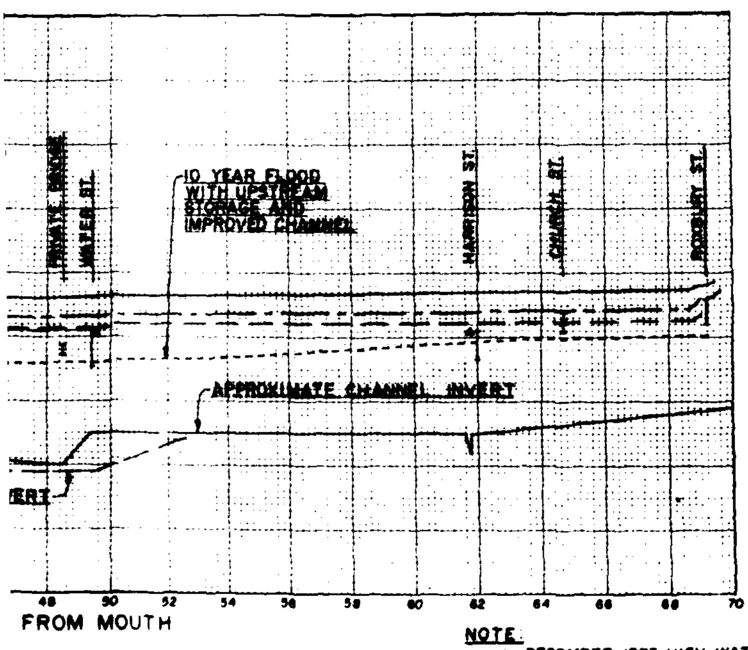


U. S. ARMY

LEGEND

100 YEAR FLOOD LIMITS

SCALE IN FEET
0 200 400



NOTE:
▲ DECEMBER 1973 HIGH WATER MARK

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

ASHUELOT RIVER BASIN

BEAVER BROOK KEENE, N.H.

PLAN & PROFILE

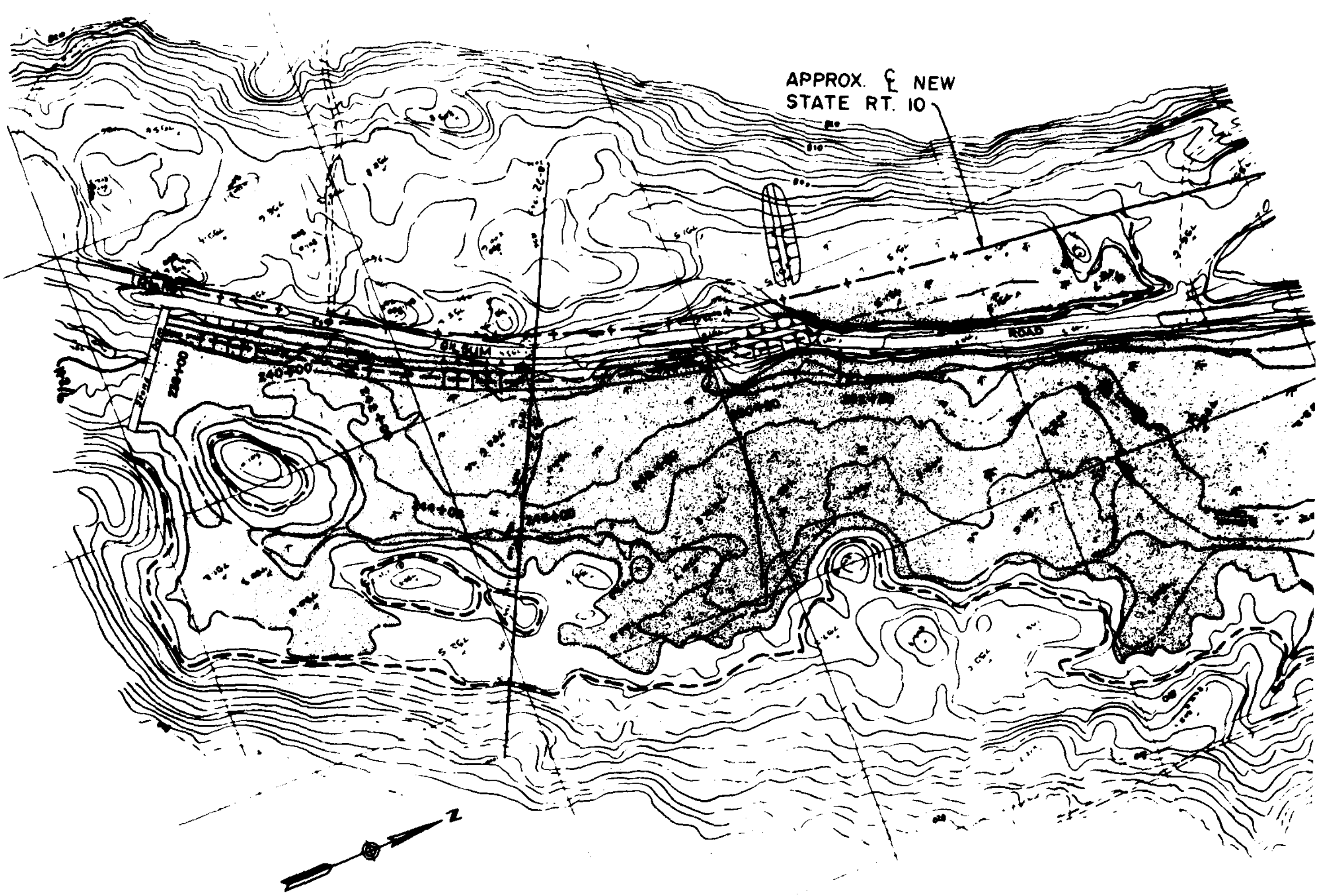
EXISTING & IMPROVED CONDITIONS

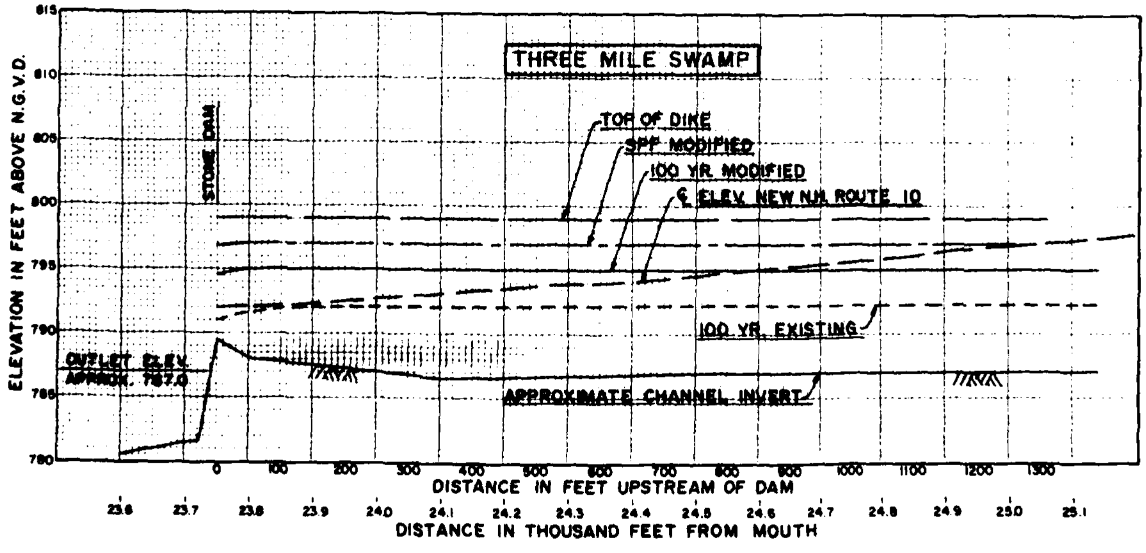
HYDRO. ENG. SECT. FEB. 1983

PLATE A-7

2

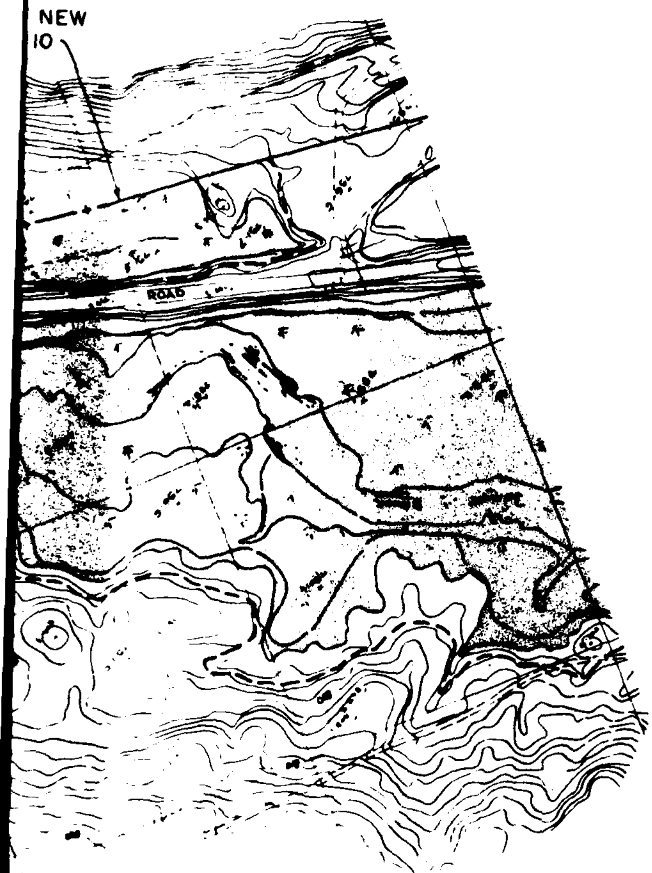
CORPS OF ENGINEERS

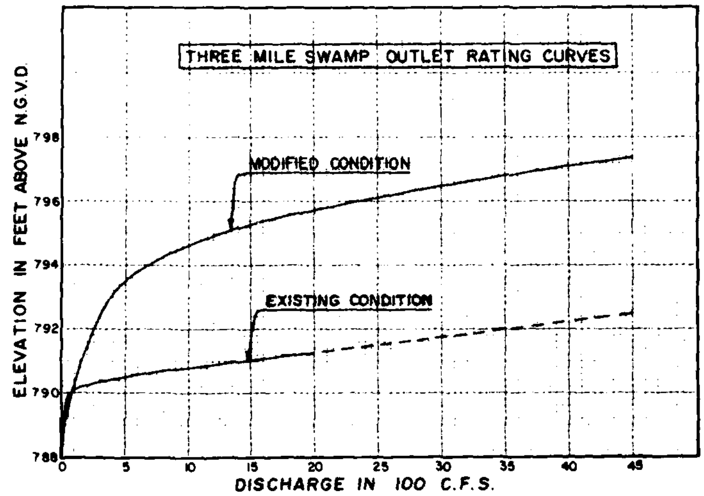


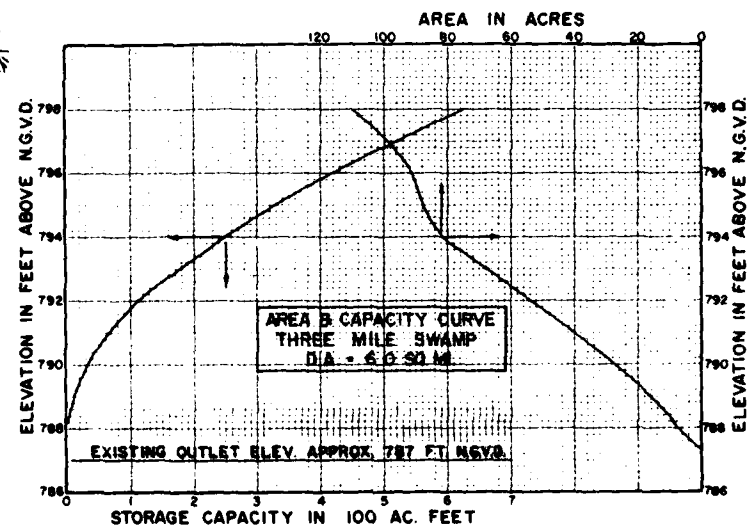


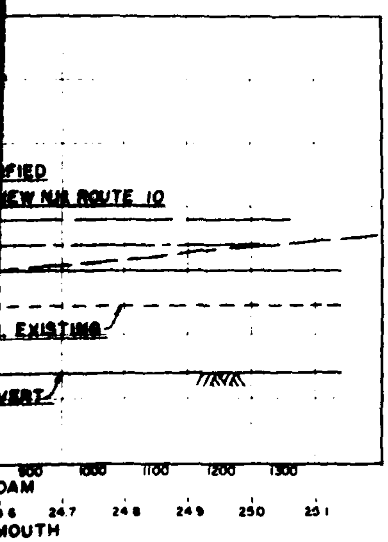
D E F G H

U. S. ARMY









LEGEND

TING 100 YEAR FLOOD LIMITS

MODIFIED 100 YEAR FLOOD LIMITS

DEPARTMENT OF THE ARMY

ASHUELOT RIVER BASIN

BEAVER BROOK KEENE, N.H.

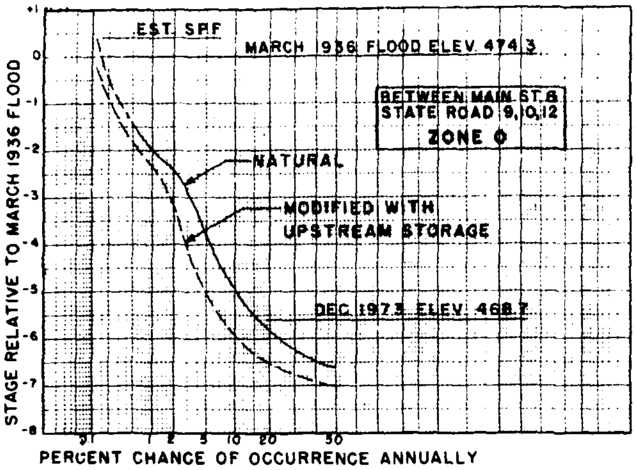
THREE MILE SWAMP

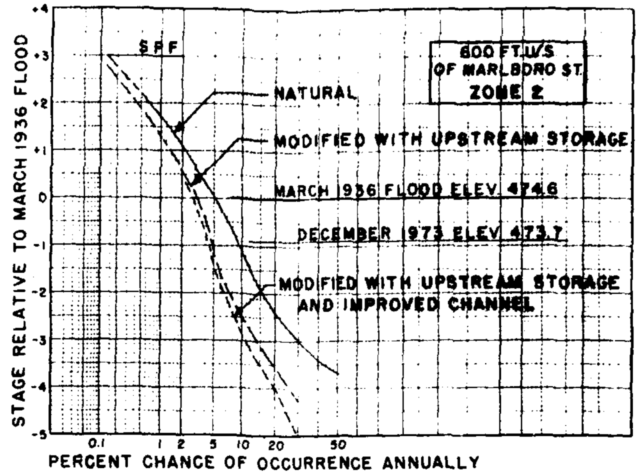
PLAN & PROFILE, OUTLET RATING CURVES AND AREA CAPACITY CURVES

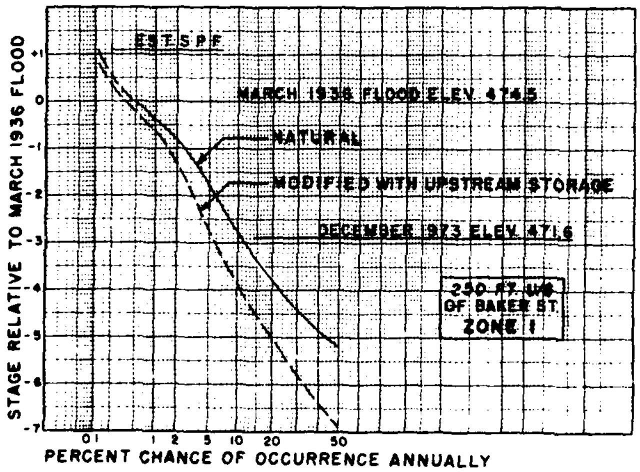
HYDRO ENG SECT FEB 1983

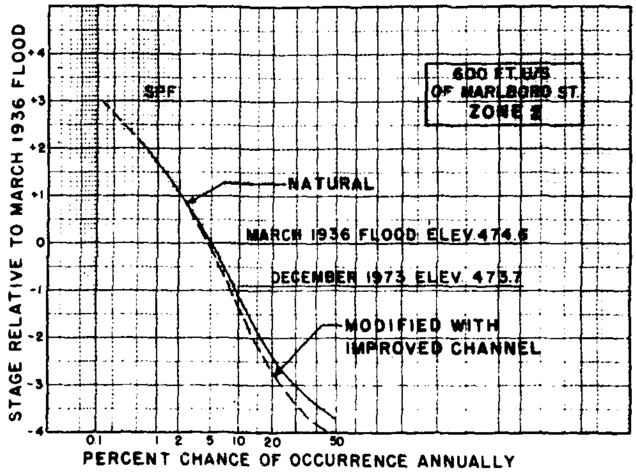
PLATE A-8

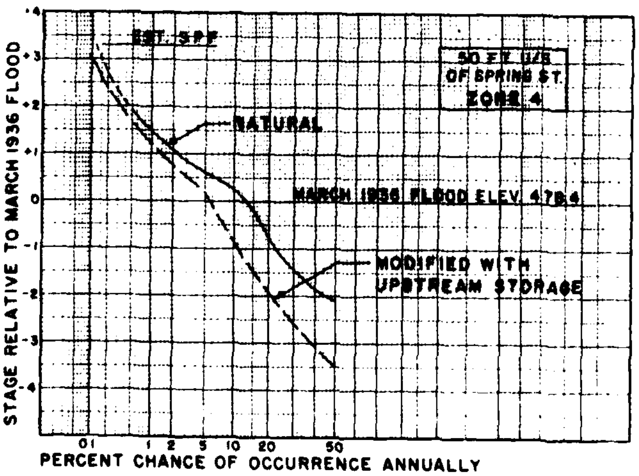
2

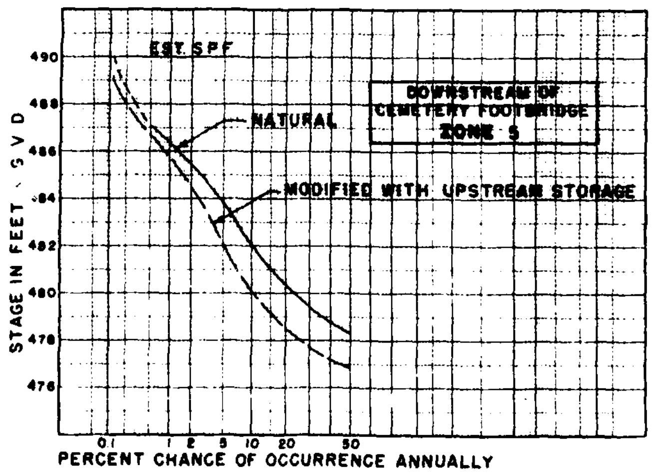

EST. SPF
MARCH 1936 FLOOD ELEV. 474.3
BETWEEN MAIN ST. & STATE ROAD 9, 10, 12
ZONE 0
NATURAL
MODIFIED WITH UPSTREAM STORAGE
DEC. 1973 ELEV. 468.7
STAGE RELATIVE TO MARCH 1936 FLOOD
PERCENT CHANCE OF OCCURRENCE ANNUALLY

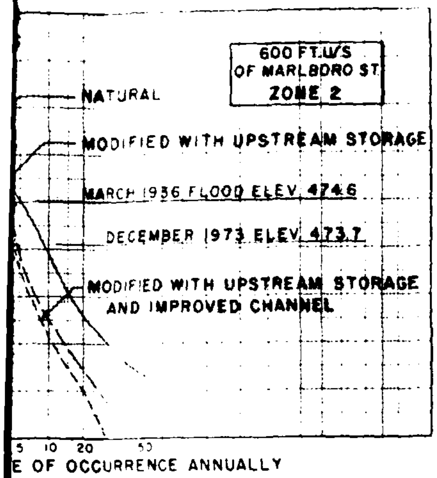

SPF
600 FT. U/S OF MARLBORO ST.
ZONE 2
NATURAL
MODIFIED WITH UPSTREAM STORAGE
MARCH 1936 FLOOD ELEV. 474.6
DECEMBER 1973 ELEV. 473.7
MODIFIED WITH UPSTREAM STORAGE AND IMPROVED CHANNEL
STAGE RELATIVE TO MARCH 1936 FLOOD
PERCENT CHANCE OF OCCURRENCE ANNUALLY

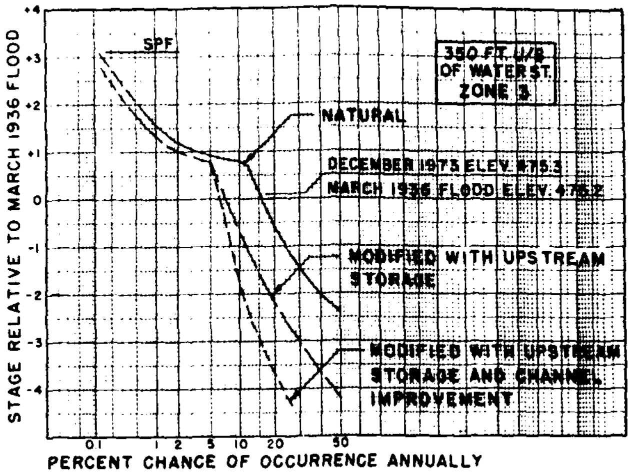

EST. SPF
MARCH 1936 FLOOD ELEV. 474.5
NATURAL
MODIFIED WITH UPSTREAM STORAGE
DECEMBER 1973 ELEV. 471.6
250 FT. U/S OF BAKER ST.
ZONE 1
STAGE RELATIVE TO MARCH 1936 FLOOD
PERCENT CHANCE OF OCCURRENCE ANNUALLY

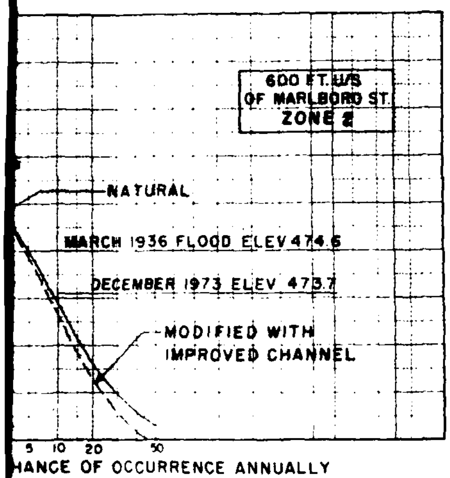

SPF
600 FT. U/S OF MARLBORO ST.
ZONE 2
NATURAL
MARCH 1936 FLOOD ELEV. 474.6
DECEMBER 1973 ELEV. 473.7
MODIFIED WITH IMPROVED CHANNEL
STAGE RELATIVE TO MARCH 1936 FLOOD
PERCENT CHANCE OF OCCURRENCE ANNUALLY

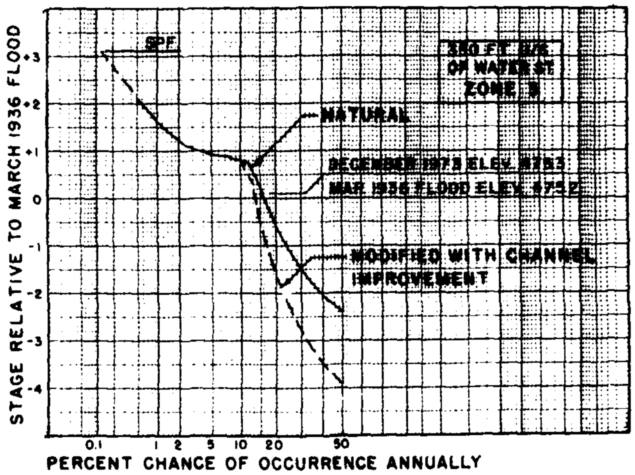

EST. SPF
50 FT. U/S OF SPRING ST.
ZONE 4
NATURAL
MARCH 1936 FLOOD ELEV. 478.4
MODIFIED WITH UPSTREAM STORAGE
STAGE RELATIVE TO MARCH 1936 FLOOD
PERCENT CHANCE OF OCCURRENCE ANNUALLY

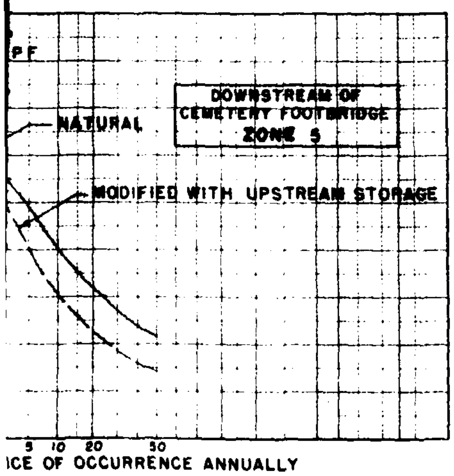

EST. SPF
DOWNSTREAM OF CEMETERY FOOTBRIDGE
ZONE 5
NATURAL
MODIFIED WITH UPSTREAM STORAGE
PERCENT CHANCE OF OCCURRENCE ANNUALLY











DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

ASHUELOT RIVER BASIN

BEAVER BROOK KEENE, N.H.

STAGE FREQUENCY CURVES

NATURAL AND MODIFIED

HYDRO ENG. SECT FEB. 1983

PLATE A-9

2

SECTION B

GEOTECHNICAL AND DESIGN CONSIDERATIONS

BEAVER BROOK LOCAL PROTECTION
CONNECTICUT RIVER BASIN
KEENE, NEW HAMPSHIRE
GEOLOGY, EMBANKMENTS AND FOUNDATIONS

TABLE OF CONTENTS

H. DESIGN OF DIKES AND DAM

LIST OF PLATES

A. PERTINENT DATA

1. Purpose. Local Flood Protection
2. Location.
   - State - New Hampshire
   - County - Cheshire
   - City - Keene

3. Design Storm
   a. Dam & Dikes

Frequency -	SPF
Freeboard -	2 ft.

   b. Channel

Frequency -	approx. 10 year flood
Freeboard -	none

4. Concrete Gravity Dam

Top Elevation Spillway -	varies 787 to 794 NGVD
Top Elevation Abutments -	799 NGVD
Top Width	5 feet
Slopes	
Upstream	Vertical
Downstream	4V on 3H
Total length -	251 feet
Overflow length -	200 feet

5. Dike Embankments

	Dike A	Dike B
Type	Earthfill with Stone Protection	Earthfill-topsoiled and seeded
Top elevation	799 NGVD	799 NGVD
Maximum height above streambed	17 feet	9 feet
Max. height above landside toe	8 feet	9 feet
Slopes: Riverside	1V on 2H	IV on 2.5H
Landside	1V on 2H	IV on 2.5H
Total Length	1150 feet	180 feet
Top width	12 feet	12 feet

6. Channel Improvement

Length	1750 feet
Bottom width	17 feet
Side slopes	1V on 2H

B. INTRODUCTION

7. Location and Description of Project

The Beaver Brook Flood Damage Reduction Project in the City of Keene, N.H. is located on Beaver Brook, a tributary of the Ashuelot River which in turn is a tributary of the Connecticut River. The flood control project consists of an upstream concrete gravity dam with adjacent earthfill dikes along the Route 10 highway located about 2.6 miles above the center of the city. In addition, a channel improvement consists of the widening of about 1750 feet of the existing channel between Marlboro and Water Streets in the center of the city. The locations, arrangements, and pertinent details of the structures are shown on Plates 4 through 10.

8. General Notes

Programs of subsurface investigations and soils engineering studies were undertaken for the design of the Beaver Brook Project. The subsurface investigations included geological studies, subsurface explorations and laboratory test programs carried out to determine the distribution and characteristics of foundation materials and to determine soil and bedrock conditions relevant to excavation operations and the design and construction of the embankments and concrete structures. Soil and rock engineering studies, based on the data obtained from the subsurface investigations, were conducted to develop safe and economical earthwork and foundation designs and construction methods.

9. Elevations

All elevations mentioned in this report are in reference to National Geodetic Vertical Datum (NGVD), which is the mean sea level of 1929.

C. TOPOGRAPHY, GEOLOGY & SEISMICITY

10. Topography.

The Beaver Brook project area is located within the New England Upland section of the New England physiographic province. The upland is generally characterized by a hilly terrain with occasional monadnocks that is dissected by narrow valleys. With elevations ranging from 500 feet NGVD to over 2,500 feet NGVD, a landscape with steep slopes is common. Glaciation has extensively modified the pre-glacial bedrock topography by erosion and deposition of glacial materials. Elevations near the project site range between 780 feet NGVD and 830 feet NGVD.

11. Geology.

a. Surficial. Pleistocene age, glacial deposits dominate the surficial geology of the area. Glacial till, a heterogeneous product of direct deposition, generally blankets the bedrock surface. In the area, the till has occasionally been molded into low hill features known as drumlins. The east-west valley of the Ashuelot River to the north was dammed by glacial till masses creating a temporary glacial lake, in which fine grained sediments were deposited. This lake may have spilled over the present divide into the north-south valley of Beaver Brook. The till in the lower sides of the Beaver Brook valley is overlain by remnants of gravelly terraces which were built by melt water streams. Recent alluvial deposits are found in valleys and floodplains.

b. Bedrock. The bedrock in the region is principally Devonian in age and consists primarily of granite and gneiss. Mica schist of the Littleton Formation fingers between these rocks along the valley of Beaver Brook. This zone of rock contacts probably accounts for a largely structural origin of the valley. Bodies of pegmatite frequently occur within the local bedrock formations. The strike of the foliation is generally N 20°E while the dip varies between 25 and 60° SE.

12. Seismicity.

The Beaver Brook area is located entirely within Zone 2 of the seismic zone map of the United States. This is a modification of the seismic risk map developed by the Environmental Science Administration and the Coast and Geodetic Survey and it is contained in Engineering Regulation 1110-2-1806 dated 30 April 1977. In accordance with this directive, a coefficient of 0.05g is recommended for use in any preliminary evaluation of the seismic stability of the dam. A detailed seismic evaluation will be performed during final design efforts.

Detailed remote sensing and fault compilation did not reveal the presence of a major or capable fault within a 75 mile radius of the project area. The nearest event with an epicenter based on non-instrumental data occurred approximately five miles from the site in 1854 with an intensity of IV MM (Modified Mercalli). The nearest event with an epicenter based on instrumental data occurred approximately 20 miles from the site in 1966 and had an intensity of V MM.

D. SUBSURFACE INVESTIGATIONS

13. Subsurface Explorations

a. Dam and Dikes. In March 1982, a preliminary subsurface investigation program, consisting of four borings, was performed. The borings were continuously driven into overburden to recover either 2 or 2 1/2 inch diameter samples. Upon refusal, the borings were advanced into bedrock to obtain up to 15 feet of NX size (2 1/8 inch I.D.) cores. Locations of the borings are shown on Plates 4 and 5.

In 1964, 35 test borings were drilled at the original Beaver Brook Dam and reservoir location. Although the original site is approximately 1,000 feet downstream from the present site, the results from these borings are useful in determining the local subsurface conditions. Records of these borings are contained in the Beaver Brook survey report dated July 1965.

b. Channel. No subsurface investigations were performed along the areas of the proposed channel improvements. Foundation conditions were inferred from the borings at the dam site, observations made from field reconnaissance trips, and from the well records and geologic logs of nine wells in the area. The well records and logs are contained in the 1973 U.S.G.S. report "Hydrologic Data of the Ashuelot River Basin, New Hampshire"

c. Future Explorations

(1) Dam and Dikes. At least ten more borings and several test pits will be required in the foundation areas of the concrete dam and dike embankments in order to determine the depth to an adequate bearing stratum for the concrete dam and training walls and seepage and shear strength characteristics for the dike foundation soils. Pressure testing of the bedrock will be required in borings along the centerline of dam. The borings in the dam foundation area will also be used to locate old channel fillings, gorges in bedrock surface, depth of weathered and decomposed bedrock and to determine the extent and characteristics of materials from required excavations. Machine probes will also be utilized along the centerline of the dam to assist in defining a sound and firm bedrock surface. About 20 foundation probes and a few shallow borings will be required along the riverside toe of the Dike A embankment to determine the depth of soft organic materials. About 10 probes and one boring will be required in the foundation area of the Dike B embankment to determine the depth of organics and other unsuitable materials.

(2) Channel. At least three test pits will be required alonq various reaches of the channel to determine the character of materials for design and construction purposes. A few shallow borings may also be required where utility lines are to be relocated.

14. Laboratory Tests.

All laboratory tests were performed in accordance with the procedures described in Corps of Engineers Manual EM 1110-2-1906 "Laboratory Soils Testing". All soil samples were visually classified in accordance with the Unified Soil Classification System. Grain size analyses, Atterberg Limit determinations, specific gravity and water contents were performed on selected samples to confirm the visual classifications and to provide more precise data where required. Tests on the rock samples included limited petrographic analyses and the determinations of specific gravity, percent absorption, and unconfined compressive strengths.

15. Presentation of Data.

The geological sections along the centerline of the proposed dam and dike are shown on Plate No. B-1. The results of laboratory soil and rock tests are shown on Tables 1 and 2 (Plates B-8, B-9), respectively.

E. CHARACTERISTICS OF DAM, DIKES AND CHANNEL FOUNDATION MATERIALS

16. Distribution and Description of Materials

a. General. The concrete dam will be built on the site of an existing dam (called Three Mile Swamp Dam). The existing dam consists of an earth embankment about six feet high with a supporting downstream rubble wall. The existing dam forms a long shallow impoundment about three to four feet deep behind the dam. The newly reconstructed State Route 10 embankment is located adjacent to the dam and runs the length of the valley. Dike A will be constructed adjacent and parallel to the highway embankment while Dike B will close a low swampy area north of the highway. Portions of the channel improvement will be built on areas of man-made land formed by filling during the last 100 years or so. There is no exact record of the timing and type of filling that has taken place. The filling was generally uncontrolled and the type of fill encountered in these areas is very heterogeneous.

b. Subsurface Water. Subsurface water levels in the proposed dam and dike foundation areas fluctuate with the level of the adjacent brook and pond, but generally will be found within five feet of the ground surface. Subsurface water levels in the channel areas fluctuate with the level of the adjacent brook.

c. Dam and Dike Foundations. As indicated by the Geologic Log Profile on Plate B-1, the overburden varies from about five feet in thickness on the left abutment to only one foot in the vicinity of the existing brook to about 15 feet adjacent to the highway embankment and under the proposed Dike A embankment. The overburden materials consist of a capping of about 6 to 12 inches of topsoil and forest litter overlying primarily glacial materials which in turn overlie schist and granitic bedrock.

Weathered to decomposed bedrock varying from 2 to 7 feet in thickness was encountered in the right abutment and under the existing brook.

The materials in the left abutment glacial till deposits consist of brown to gray-brown, loose to moderately compact, gravelly silty medium to fine sands and silty medium to fine sands (SM). Gravel contents range from 5 to 30 percent while silt contents range from 25 to 45 percent.

Underlying the topsoil in the right abutment and in the valley to the north of the brook, there is a 4-foot layer of gray brown moderately compact, gravelly silty medium to fine sand (SM, till). These sands overlie a 4 foot layer of gray, moderately compact, medium to fine sandy clay (CL, till). Underlying these deposits is a 5 foot zone of a more pervious gray brown, moderately compact gravely silty sand (SM) and silty sandy gravel (GM) containing cobbles and boulders.

Further upstream (a distance of about 200 feet) the overburden in the dike foundation area consists of gray, moderately compact to compact sandy clay (CL, glacial till). Sand contents of the clays range from 35 to 45 percent. Liquid limits run from 26 to 32 percent while plastic limits range from 16 to 18 percent.

No explorations were made in the foundation areas for the upper reaches of Dike A or for Dike B across the swampy area. Surficial evidence of the soils along Route 10 indicates that foundation materials grade into outwash sands and gravels in the upper reaches of Dike A. The swampy area, high water table and information from highway construction drawings indicate a shallow layer of organics over glacial till for the foundation area of Dike B.

d. Channel. In general, foundation conditions along Beaver Brook are defined by a relatively thick overburden cover, primarily glacial material, overlying bedrock.

Based on the geologic logs of nine wells in the general area, an idealized stratigraphic column would consist of four members. The upper twenty feet is characterized by sand of alluvial and glacial origin. The underlying ten feet consists of glacially-derived silt and sand. Underlying this layer is a variably-thick (40 to 70 feet) unit of silt and clay. The basal member is a compact till, consisting mainly of silt, gravel, and sand. Bedrock in the area is primarily granite with some schist. The bedrock surface was encountered generally at depths greater than 50 feet.

e. Characteristics of Foundation Bedrock

(1) General. The underlying bedrock at the dam consists primarily of interbedded granite and biotite schist. The bedrock in the left abutment is a white to dark gray sequence of interbedded granite and biotite schist. In general, the rock in this area is hard, fresh to slightly weathered, fine to medium-grained, and massive (granite) to well-foliated (schist). Bedrock in the right abutment is a white to dark gray, hard, fresh to weathered, medium to coarse grained well-foliated, biotite schist. The upper foot of rock is very weathered. Interbedded granite and biotite schist comprise the bedrock underlying the proposed dike. Weathered to decomposed bedrock up to seven feet thick was encountered adjacent to the existing brook.

(2) Bedrock Foundation for Dam. The severe weathering of portions of the bedrock at foundation grades may require more than the usual foundation treatment to remove loose blocks and slabs and to clean out weathered zones and seams. The strike of the bedrock is at approximately right angles to the centerline of the dam while the bedrock dips steeply in a southerly direction, which will result in an irregular surface when excavated.

Rock removal and filling of depressions with concrete may be required to establish a uniform base surface for the dam. Rock removal will be accomplished by line drilling and light blasting with jack hammers, and by barring and wedging with hand tools. The bedrock surface will be cleaned with jets of air or water or a combination thereof prior to placing concrete. With adequate treatment the rock will be satisfactory for support of heavy structural loads. Grouting of the bedrock under the centerline of dam will be considered during the final design phase in order to control seepage and maintain a permanent pool, and to control the development of seepage pressures under the toe.

17. Shear Strength.

No samples of foundation materials for this project were tested for shear strength. The shear strength parameters tabulated below have been estimated on the basis of visual examination of the samples, their grain-size distribution curves, data from exploration logs and experience with similar materials.

	ESTIMATED SHEAR STRENGTH PARAMETERS	
FOUNDATION SOIL	Ø, DEGREES	C, PSF
Dam & Dikes		
Gravelly silty sand (SM)	30	0
Sandy clay (CL) (Till)	30	0
Organic soils	0	100 to 200
Weathered and decomposed bedrock	25 to 30	0
Channel		
Alluvial sands	30	0
Existing fills	25	0

18. Permeability.

Permeability tests were not performed on samples of the foundation soils since the relative permeability characteristics can be judged with sufficient accuracy by visual examination of the samples, their grain size distribution curves and experience with similar materials. It is estimated that the vertical coefficients of permeability will be within the following ranges:

Material	Coefficient of Vertical Permeability (Kv)
Dam & Dikes	
Gravelly silty sands (SM)	0.5 to 5 x 10^{-4} cm/sec
Sandy clay (CL)	0.1 to 1.0 x 10^{-4} cm/sec
Weathered and decomposed bedrock	0.1 to 10 x 10^{-4} cm/sec
Channel	
Alluvial sands	50 to 150 x 10^{-4} cm/sec
Existing Fills	10 to 100 x 10^{-4} cm/sec

19. Consolidation.

Consolidation tests were not performed on samples of foundation soils. All soft and compressible surficial deposits will be removed prior to the construction of the dike embankments. The consolidation characteristics and natural densities of the foundation soils are such that no significant post-construction foundation settlement is anticipated under the proposed embankment loadings.

F. DISTRIBUTION AND DESCRIPTION OF MATERIALS FROM REQUIRED EXCAVATIONS

20. General.

The major excavations for this project are those for the concrete dam, stilling basin and discharge channel and for the downstream channel improvement. All suitable materials from the excavations for the dam features will be used to the extent practical in the permanent work. These materials will be used in random fill zones in the dike embankments, in backfills for the concrete structures, and in temporary cofferdams. All suitable materials from the excavations for the channel improvement will be used to the extent practicable to fill low areas adjacent to the channel. Excess and unsuitable materials shall be removed from the sites and spoiled in waste areas furnished by the city.

21. Dam, Stilling Basin and Discharge Channel Excavations.

Excavations for the dam and spillway will consist principally of gravelly silty medium to fine sands and silty medium to fine sands (SM), medium to fine sandy clays (CL) and silty sandy gravel (GM) with cobbles, glacial till. The random fill material from these excavations will be variable with silt contents of the sands ranging from 15 to 45 percent and gravel contents ranging from 5 to 30 percent. Silt contents of the gravels range from 5 to 15 percent while sand contents of the clay range from 35 to 45 percent. The depth and characteristics of the soils in the discharge channel are not known, but probably are similar to those in the dam foundation area. Since some of the materials will be below the water table, it is expected that drying back of these materials will be necessary.

22. Channel Improvement.

Excavations for the channel improvement will consist principally of sands, silty sands and silty gravelly sand (SP, SP-SM, SM) with some trash fill. Suitable material will be used in random fill zones along the channel. Excess and unsuitable materials shall be removed from the site and spoiled.

G. CHARACTERISTICS OF EMBANKMENT MATERIALS

23. General.

The major portions of the materials from the required excavations will not be suitable for use in construction of the compacted impervious fill and gravel fill portions of the dike embankment. The suitable materials from these excavations will be used to the extent practicable as random fill material. In view of the relatively high cost of developing government furnished borrow areas and the implications involved in acquiring land for borrow areas, it has been decided to have the contractor furnish all embankment materials other than those available from the required excavations.

24. Filter Design.

The gradation requirements for gravel fill, gravel bedding and filter stone materials have been established in accordance with the filter design criteria set forth in Engineering Manual for Civil Works Construction, EM 1110-2-1913, Design and Construction of Levees. Typical filter design studies (gradation ranges) are shown on Plate Nos. B-4 through B-5.

25. Random Fill.

The suitable embankment material from the required excavations of the dam, stilling basin, and discharge channel will include a wide range of soils from weathered bedrock to gravelly silty sands and sandy clays. In view of the variability of the materials and the impracticability of separating the various soil types during construction, it is planned to use the material in random fill zones in the dike embankments and for certain backfill zones for the dam and other concrete structures. Random fill material for the channel improvement will not contain significant quantities of cinders, ashes, topsoil and similar material and will be free of stumps, trash and large pieces of debris. For design purposes, the densities, permeability coefficients and shear strength parameters selected for impervious fill material have been used for the random fill material.

26. Impervious Fill.

Impervious fill will be furnished by the contractor and will consist of approved, natural, reasonably well graded, gravelly silty sand and sandy clay. It is estimated that compacted impervious fill material will have an average coefficient of permeability of less than 1×10^{-4} cm/sec and will develop shear strength parameters in excess of $\phi = 30$ degrees within the anticipated applied stress range for all conditions. Experience with similar materials on other projects indicates that placement moisture contents can be maintained within two percentage points of optimum with moderate moisture control and that in place compacted dry densities will be in the order of 120 pcf. The material will be required to meet the following gradation limits:

Sieve Size (U.S. Std)	Percent Passing by Dry Weight
6 inch	100
3 inch	85-100
No. 4	70-95
No. 40	35-70
No. 200	20-45

27. Gravel Fill.

Gravel fill shall consist of approved contractor furnished well graded sandy gravel composed of tough, durable particles of natural sand and gravel. The material shall meet the following gradation limits:

Sieve Size (U,S, Std)	Percent Passing by Dry Weight
6 inch	100
1 inch	45-90
No. 4	15-70
No. 16	7-50
No. 200	0-5

(In addition, not more than 10 percent, by dry weight, of the component passing the No. 4 sieve shall pass the No. 200 sieve.)

28. Class I Gravel Bedding.

Gravel bedding shall consist of bank-run reasonably well graded, cobbly, sandy gravel composed of tough, durable particles of natural sand and gravel. The material shall meet the following gradation limits:

Sieve Size (U.S. Std)	Percent Passing by Dry Weight
6 inch	100
1 inch	50-90
No. 4	25-70
No. 16	15-50
No. 200	0-5

(In addition, not more than 10 percent, by dry weight, of the component passing the No. 4 sieve shall pass the No. 200 sieve.)

29. Class II - Gravel Bedding.

The material shall meet the following gradation requirements.

Sieve Size (U.S. Std)	Percent Passing by Dry Weight
9 inch	100
2 inch	60-90
No. 4	15-60
No. 16	7-40
No. 200	0-5

(In addition, not more than 10 percent, by dry weight, of the component passing the No. 4 sieve shall pass the No. 200 sieve.)

30. Filter Stone.

Filter stone shall be furnished by the contractor and shall consist of hard, durable and sound quarried rock fragments. The material shall be well graded with fragments varying in size from 3 inches to 5 inches. The material may contain a maximum of 20 percent, by dry weight, of particles retained on a 5 inch grizzly and a maximum of 15 percent, by dry weight, of particles that pass the 3-inch sieve. The filter stone material shall not contain particles with a maximum dimension greater than 8 inches. No stone shall have a minimum dimension less than 2 inches.

31. Shear Strength and Permeability Characteristics.

On the basis of the above gradation and specifications it is estimated that gravel fill, gravel bedding and filter stone materials will develop angles of internal friction of at least 30 degrees and will have coefficients of permeability between 20×10^{-4} and 100×10^{-4} cm/sec.

32. Rock and Concrete Materials.

More than 3,900 cubic yards of Class I Stone Protection and 3,700 cubic yards of Class II Stone Protection are needed for this project. There are two known commercial sources for stone protection materials located north of the city of Keene, within 20 miles. Concrete aggregate is available from commercial suppliers in the tri-state area within a 35 mile radius of the project site.

H. DESIGN OF DIKES AND DAMS

33. Design Criteria.

Current design criteria as set forth in the pertinent sections of the Engineering Manual for Civil Works Construction EM 1110-2-1913 "Design and Construction of Levees" have been followed in the design of dikes for this project.

34. Materials for Embankment Construction.

a. Materials from Required Excavations. It is estimated that there will be about 29,000 cubic yards of required earth excavations for this project. About 17,000 cy will be excavated from the dam and dike areas and about 11,000 cy from the channel improvement. Of the materials from these excavations, about 4,000 cubic yards will be suitable for use in the random fill zone of the dikes and certain backfills around concrete structures. About 1,000 cubic yards of material will be utilized in temporary cofferdams. About 25,000 cubic yards of material shall be removed from the site and spoiled. About 600 cubic yards of required rock excavations will be utilized as Stone Protection in the discharge channel. Future subsurface investigations will determine more accurately the characteristics of the foundation materials and materials from required excavations. Once these characteristics are more accurately defined, the design may be modified to utilize more materials from required excavations.

b. Materials Furnished by Contractor. All dam and dike materials other than random fill materials and Stone Protection for the dam discharge channel will be furnished by the contractor.

c. Materials Usage. A chart showing the proposed utilization of materials from the required excavations and of materials furnished by the contractor is shown on Plate B-7. The quantities shown are subject to change as more detailed quantity estimates are developed during the preparation of plans and specifications for the project.

35. Selection of Dike Sections.

The sections for various reaches of the dikes developed as a result of design studies are shown on Plates 5 and 6. The selection of the sections was influenced by the foundation conditions, the availability and characteristics of earth and gravel materials from required excavations and other sources, the proximity of Route 10, seepage control requirements, stream erosion, ice action and construction considerations. In general, the Dike A embankment will consist of a large reservoir side toe of gravel fill, a large central zone of compacted impervious fill, a smaller zone of compacted random fill above elevation 795 and a shallow landside pervious toe drain of compacted gravel fill. The embankment will be protected by layers of stone protection and gravel bedding on the reservoir side slopes and by seeded topsoil on the landside slopes. Dike B will consist of a large section of dumped and compacted random fill. The embankment will be protected by seeded topsoil.

36. Seepage Control

a. General. The design hydrostatic head for the dike has been taken as that produced by the SPF (El 797) on the waterside and a water level at the ground surface on the landside. On this basis, the design hydraulic head will range from 2 to about 6 feet.

b. Seepage Through the Dike. Seepage through Dike A will be controlled through the arrangement, sizes and differences in permeabilities between the impervious fill zone and those of the landside gravel fill zone and the relatively long seepage path through the random zone above elevation 795. Seepage through Dike B will be controlled by the relatively long seepage path through the random fill zone.

c. Foundation Seepage. Seepage and uplift pressure at the landside toe above elevation 795 will be controlled by the length of the seepage path. In the reach from Sta 0+00 to Sta 10+00 for Dike A where the hydraulic head is between 2 and 6 feet, a shallow toe drain has been provided to intercept seepage and prevent softening of the landside toe.

37. Dike Sections

a. Dike Stations 0+00 to 4+00. Dike A in this reach will have a gravel fill (dumped and compacted) waterside toe. Its height on the landside will average about 8 feet while its height on the waterside will average about 17 feet. All organic materials in the pond bottom within the dike foundation area will be excavated to insure embankment stability against shear failure. The section will consist of dumped impervious and gravel fills placed in water below elevation 787 and compacted impervious and gravel fills above that elevation. The bottom of the pervious foundation drain will extend into the foundation materials by 2 feet.

In order to utilize materials from required excavations, compacted random fill will be placed above elevation 795 where hydraulic heads are low. Stone protection will be placed on the riverside as the dike will be subjected to water and ice action and to increase stability during drawdown.

b. Dike Stations 4+00 to 11+80. The dike in this reach is also constructed parallel and continuous to the highway. Its height on the landside will average about 5 feet while its height on the waterside will average 11 feet. The dike section will be similar to the reach from stations 0+00 to 4+00 except that the height of the embankment is less. The dike will have a small waterside gravel fill toe. Below elevation 795 the section will consist of compacted impervious fill with compacted random fill above Elev. 795. No toe drain is provided in this reach because of low differential heads and long seepage path. Stone protection will be placed on the riverside slopes as the dike will be subjected to ice action, and to increase stability during drawdown.

c. Dike B. Dike B will close off a low swampy area on the north side of Route 10 and will divert overland flow through an existing 36 inch culvert under the highway. The dike will reach a maximum height of about 9 feet. All organic material within the foundation area will be excavated to insure embankment stability and control seepage. The section will consist of dumped random fill below elevation 793 and compacted random fill above that elevation. All slopes will be topsoiled and seeded.

38. Embankment Stability.

a. General. A section at Sta 1+00 of the riverside portion of the dike, where the height of slope is about 18 feet was selected for stability analysis as being a section combining maximum embankment height with average foundation strengths. This section was analyzed for stability against shear failure by the circle method for the End of Construction, the Sudden Drawdown Condition and Intermediate Flood Stage Condition.

b. Selection of Design Values. The design unit weights and shear strength parameters have been selected on the basis of experience with similar materials on other projects and are tabulated below:

	Unit Weight (pcf)				Shear Strength					
Material	Sat	Moist	Dry	Sub.	Ø(degrees)	C(psf)	Ø	C	Ø	C
1. Stone Protection	135	118	116	73	40	0	40	0	40	0
2. Gravel Bedding & Comp. Gravel Fill	145	135	130	83	35	0	37	0	37	0
3. Dumped Gravel Fill	135	120	115	73	30	0	33	0	33	0
4. Comp. Impervious & Random Fills	140	135	120	78	30	0	30	0	32	0
5. Dumped Impervious & Random Fills	135	125	110	73	25	0	25	0	25	0
6. Foundation Soil	140	135	120	78	30	0	30	0	30	0

c. Results of Stability Analyses. The minimum factors of safety for a section of maximum height and average foundation strength are tabulated below. On the basis of the results of these analyses, it is considered that the selected embankment sections are safe against shear failure.

Condition Analyzed	Minimum Factor of Safety	Criteria
1. End of Construction	1.6	1.3
2. Sudden Drawdown from Max Pool (El. 797.0)	1.1	1.0
3. Intermediate Flood Stages	1.6(El. 791)	1.4

39. Settlements.

Except for the organic soils the foundation soils and fills are of low compressibility and no significant settlements are anticipated from these soils. All organic soils under portions of the riverside slopes will be removed prior to construction of the dikes. It is expected that settlement of the foundation soils and embankments will occur entirely during construction.

40. Construction Considerations.

a. Dewatering Construction Areas. All areas in which compacted fills are to be constructed will be dewatered. Along portions of the riverside toe of the dike A, there will be a gravel fill zone which will serve as a cofferdam. The lower portion of this zone will be dumped into 2 to 5 feet of water while the upper portion will be compacted. It is anticipated that the dewatering of the construction areas, in general, will be possible by the usual methods of construction drainage including open pumping. In the excavation for the dam, special methods such as well pointing may be necessary. Portions of the gravel bedding and stone protection for the channel realignment will be placed below water level without diversion or dewatering of the construction area. Above water level, these materials will be placed in the dry.

b. Rate of Embankment Construction. In general, the dikes will be constructed to their full width in reaches long enough to permit proper operation of compaction equipment. An exception to this requirement will be made to allow construction of the dumped gravel fill cofferdam in advance of the adjacent fill zones. It will be required that prior to certain flood seasons, all partially completed dike reaches will be completed to their full width including stone protection.

c. Control and Diversion of River During Construction of Concrete Dam

(1) General. Plate 13 schematically indicates a means by which Beaver Brook can be controlled and diverted during construction of the concrete dam including stilling basin and discharge channel.

(2) Stage I - The construction plan provides for excavation of a trapezoidal diversion channel on the left side capable of conveying a flow of at least 700 cfs (10 yr. freq. flow) with an upstream cofferdam to approximately Elev. 792. This plan will allow construction of the right side of the dam, including the stop log structure and the low flow weir (El. 787) during Stage II. The diversion channel will be excavated to final excavation grade. Bedrock will be removed at this stage so as to minimize damage (from blasting operations) to future concrete structures. The brook will be diverted to this side after completion of all earth and rock excavation in this area.

(3) Stage II - The right side of the dam including the stop-log structure and portions of the stilling basin and downstream channel will be constructed during this phase. The upstream and south side cofferdams will be constructed to Elev. 792.

(4) Stage III. During stage III, flow through the stop-log structure and over the low flow weir will produce a capacity in excess of 700 cfs with the upstream cofferdam to approximately El. 794 NGVD. This plan provides for construction of the left side of the dam with only a minimum of rock blasting required. Special rock excavation may be required adjacent to the previously constructed monolith to minimize damage from vibration effects.

41. Environmental Considerations.

The duck nesting season takes place in the existing pond during May and June. A relatively stable pool must be maintained at about elev. 787 during construction to insure a successful nesting season. A low earth and rockfill overflow weir will be constructed about 120 feet upstream of the existing dam (where reservoir narrows) to maintain a wildlife conservation pool at elev. 787 NGVD. All earthwork shall be planned and conducted to minimize the duration of exposure of unprotected soils. Such methods as necessary shall be utilized to effectively prevent erosion and control sedimentation.

42. Methods of Placement and Compaction

a. Spreading. Materials for fills shall be spread with bulldozers or other approved equipment or by hand to form uniform loose layers of the following thicknesses:

	Maximum Loose Layer Thickness (Inches)	
Material	General	Restricted Areas
Comp. Impervious Fill	6	4
Comp.. Random Fill	6	4
Comp. Gravel Fill	8	4
Uncompacted Random (channel improvement)	12	—

b. Compaction. Materials for compacted fills shall be compacted as follows according to its fill type:

Fill Type	Compaction
Comp. Impervious	At least 6 coverages of the tread of the heavy tractor.
Comp. Random	At least 4 coverages of the tread of the heavy tractor or at least 8 coverages of the tread of the light tractor.
Comp. Gravel	At least 6 passes of the large vibratory roller.

c. Compaction in Restricted Areas. In restricted areas, impervious and random (dike embankment) fills shall be compacted by at least four coverages of the tamping foot of the power tamper. Random (channel improvement) and gravel fill in restricted areas shall be compacted by a plate vibrator or the small vibratory roller.

d. Equipment. Compaction equipment shall conform to the following requirements and shall be used as prescribed in subsequent paragraphs.

(1) Heavy Tractor. A "heavy tractor" to be used for compacting fill material shall be a standard commercial make crawler type tractor weighing not less than 35,000 pounds and exerting a tread pressure of not less than 9 pounds per square inch. The tractor shall be equipped with standard width treads.

(2) Light Tractor. A "light tractor" to be used for compacting fill material shall be a standard commercial make crawler type tractor weighing between 7,500 and 12,000 pounds and having a width of 5 1/2 feet or less, measured between the outside edges of the crawler tracks.

(3) Small Vibratory Roller. A vibratory roller shall be a unit designed for the compaction of soil or rock by vibration and shall be the product of a manufacturer nationally recognized for the design and production of such equipment. The roller shall have a double drum having a width of 25 inches or more. The roller shall weigh more than 1700 pounds and shall be self-propelled with forward and reverse speeds.

(4) Large Vibratory Roller. The vibratory roller shall be equipped with a smooth steel compaction drum and shall be operated at a frequency of vibration during compaction operations between 1100 and 1500 vpm. Vibratory rollers may be either towed or self-propelled and shall have an unsprung drum weight that is a minimum of 60 percent of the roller static weight. Rollers for compacting fill shall have a minimum static weight of 8000 pounds, a minimum dynamic force of 16,000 pounds when operating at 1400 vpm, and an applied force not less than 5000 pounds nor greater than 9000 pounds per foot of compaction drum length.

(5) Power Tamper. A power tamper shall be an approved pneumatic or mechanical tamper designed for the compaction of soils and the product of a manufacturer nationally recognized as a specialist in the design and manufacture of such equipment. Jackhammers or similar equipment not specifically designed for the compaction of earth material will not be approved. The tamper shall deliver no more than 1000 blows per minute and in addition shall have a circular tamping foot not more than 6 inches in diameter and shall be of a type that can operate satisfactorily at angles of up to 45 degrees to the vertical.

(6) Plate Vibrator. A plate vibrator shall be a plate surface vibrator designed for the compaction of soils by vibration and the product of a manufacturer nationally recognized as a specialist in the design and manufacture of such equipment. The surface contact plate shall be between 18 and 21 inches in width.

43. Slope and Channel Protection.

a. Stilling Basin. Layers of Class I Stone Protection, filter stone and Class I gravel bedding will be constructed downstream from the stilling basin for the dam to provide protection against stilling basin exit velocities of approximately 9 feet/sec. and high turbulence levels in the flows. These layers have been designed in accordance with the criteria for stone sizes for riprap and channel bottom and side slopes downstream from stilling basins as set forth in Hydraulic Design Chart 712-1.

b. Dike. The riverside slopes of dike A will be protected from stream erosion and ice action by layers of slope protection and gravel bedding. The landside slopes of dikes will be topsoiled and seeded while the top will have a gravel maintenance road. The same stone sizes as used in the stilling basin (Class I) will be specified for the Dike A slopes between stations 0+00 and 11+00.

c. Channel Improvement. Hydraulic analysis for erosion control of channel improvement indicates that vandal-proof stone sizes are adequate for streambank protection between Marlboro and Water Streets. Velocities along the channel at the toe of slopes are 3 to 4 fps and about 5 fps at bridges. Due to the low energy-gradient on this brook, it is feasible to terminate stone protection on the slope at about the 2-year flood level (water depth at 4 feet above channel invert). The slopes above the stone protection will be topsoiled and seeded with flood tolerant grasses. Full bank protection will be provided within 50 feet upstream and downstream of bridges to provide protection against accelerated flow velocities at these locations.

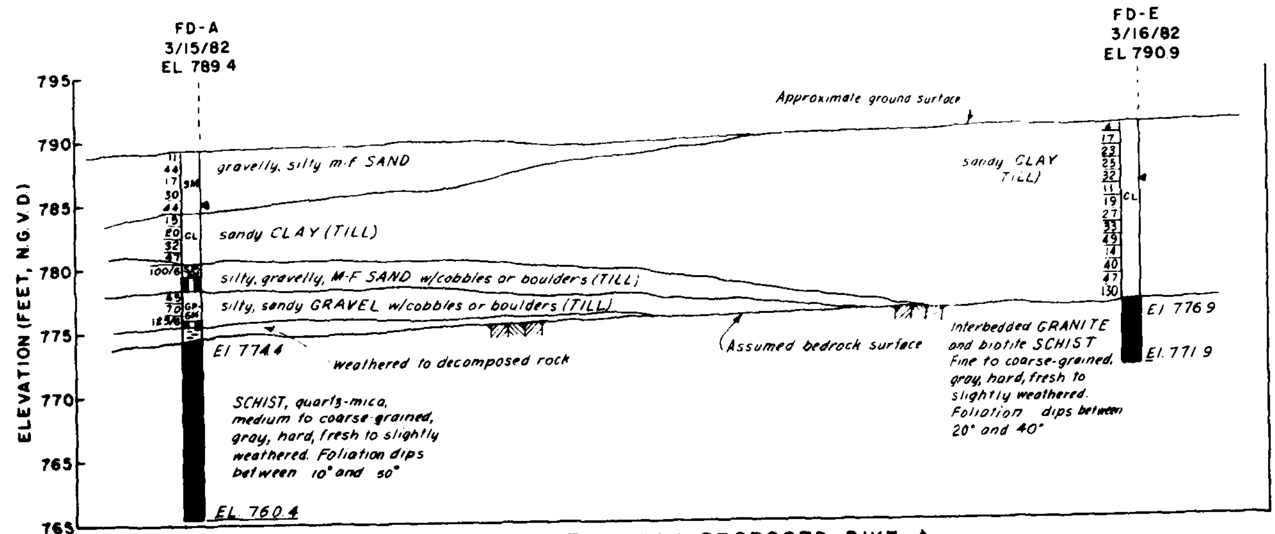
d. Layer Thickness and Gradation Range. The layer thickness, gradation range (assuming unit weight of stone as 165 pcf) and other pertinent data are shown in the following table:

Class	Basic Layer Thickness (inches)	Percent Lighter By Weight (SSD)	Limits of Stone Weight (lbs)
I	36	100	Bet. 250 &.1000
(max)			
(Stilling Basin)		50	Bet. 120 & 300
		15	Less than 150
		0	2 (min)
I	24	100	Bet. 250 & 1000
(Dike A)		50	Bet. 120 & 300
		15	Less than 150
		0	2 (min)
II	18	100	Bet. 150 & 400
(Channel)		50	Bet. 60 & 90
		15	Less than 40
		0	2 (min)

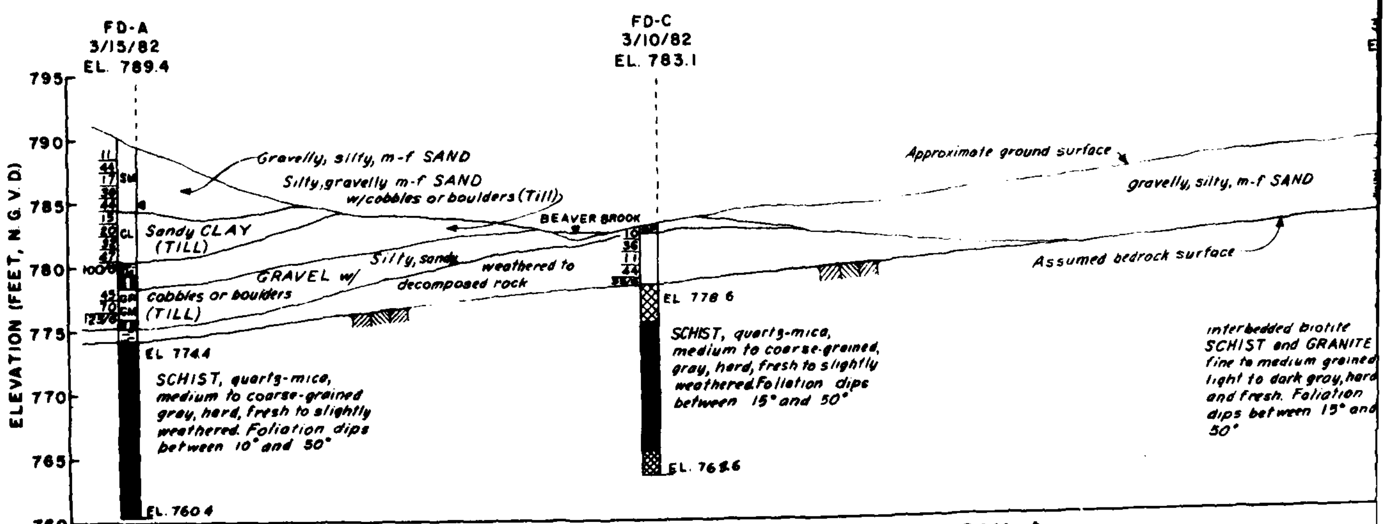
44. Concrete Dam.

The dam will consist of a centrally located 200 foot wide overflow spillway with weir crests at three different elevations (Elevs. 787, 792 and 794) and non-overflow concrete abutments. The height of the spillway will vary from 8 to 15 feet above the top of stilling basin (El. 779) and about 10 to 18 feet above the bedrock surface. The entire concrete dam will be founded on a firm and sound bedrock surface.. Rock excavation will be required for the left side of the stilling basin. A horizontal concrete slab will be required on the right side to construct the stilling basin at Elev. 779. The concrete slab will also be founded on bedrock as the elevation of the bedrock surface in the area of the stilling basin slab only varies from about Elev. 778 to 775. As described in paragraph 16e.(2), the severe weathering of portions of the schist and granite bedrock at foundation grades may require more than the usual foundation treatment to produce a sound and firm bedrock surface.

The left side training wall will be partially excavated in bedrock while the right training wall will be founded on bedrock or glacial till. Future explorations will determine the depth to bedrock or a suitable earth bearing stratum. A grout curtain will be considered in final design in order to maintain the permanent pool during low flows.

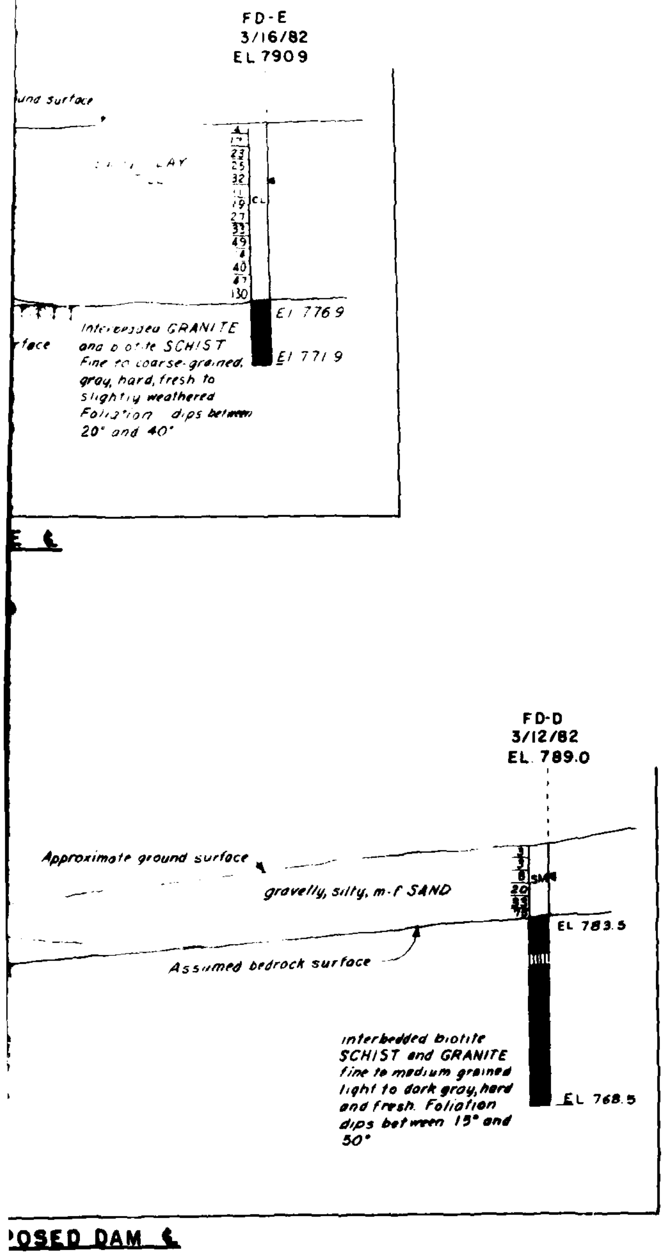


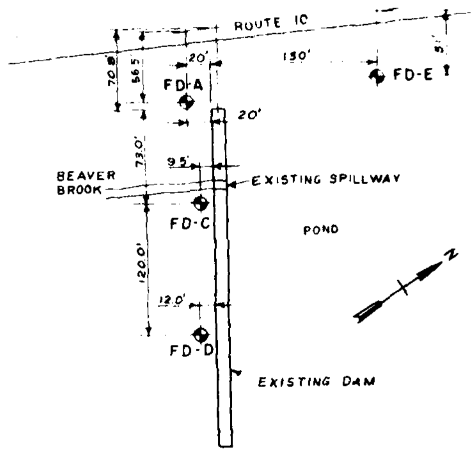
GEOLOGIC PROFILE THROUGH PROPOSED DIKE ℄
(LOOKING WEST - NORTHWEST)



GEOLOGIC PROFILE THROUGH PROPOSED DAM ℄
(LOOKING UPSTREAM)

NOTES
1. Boundaries between materials are approximate.
2. Topsoil cover, up to one foot thick, not shown





LOCATION PLAN OF EXPLORATION

SCALE: 1" = 50'

LEGEND FOR GRAPHIC LOGS

FD-610 *Type and number of exploration.*
1/7-15/54 *Observation Well installed & period of observation*
1/15/54 *Month/Day/Year exploration completed.*
N15,000 W9,878 *Location by coordinate.*
EL 789.4 *Elevation of ground surface at time of exploration*

Maximum artesian head.
Subsurface water level.
Artesian flow encountered.
Range of subsurface water level during observation period
Bottom of observation well.
SM *Group letter symbol according to Unified Soil Classification System.*
NR *No Recovery or unsatisfactory soil samples recovered*
NS *Not Sampled (Core drilled, blasted and/or wash bored)*
24 *Number of blows per foot of penetration using a 300 pound hammer, falling freely an average drop of 18 inches to drive a sample spoon of 2" O.D. or 2½" I.D. in size, equipped with bevelled and sharpened drive shoe.*
* *Blow count not recorded, or not considered representative.*
Cobble or boulder (Core drilled)
Coarse gravel or nested cobbles and boulders. (Core-drilled and/or blasted and chopped)
EL 4881 *Elevation of bedrock surface.*
Rock core recovery 0-25 %
Rock core recovery 25-50 %
Rock core recovery 50-75 %
Rock core recovery 75-90 %
Rock core recovery 90-100 %
EL 4614 *Elevation at bottom of exploration*

...daries between materials are approximate.
...il cover, up to one foot thick, not shown

GRAPHIC SCALES

VERT 1" = 5'

HORIZ 1" = 10'

DES. BY	DR. BY	CK. BY
EB	EB	C.K.

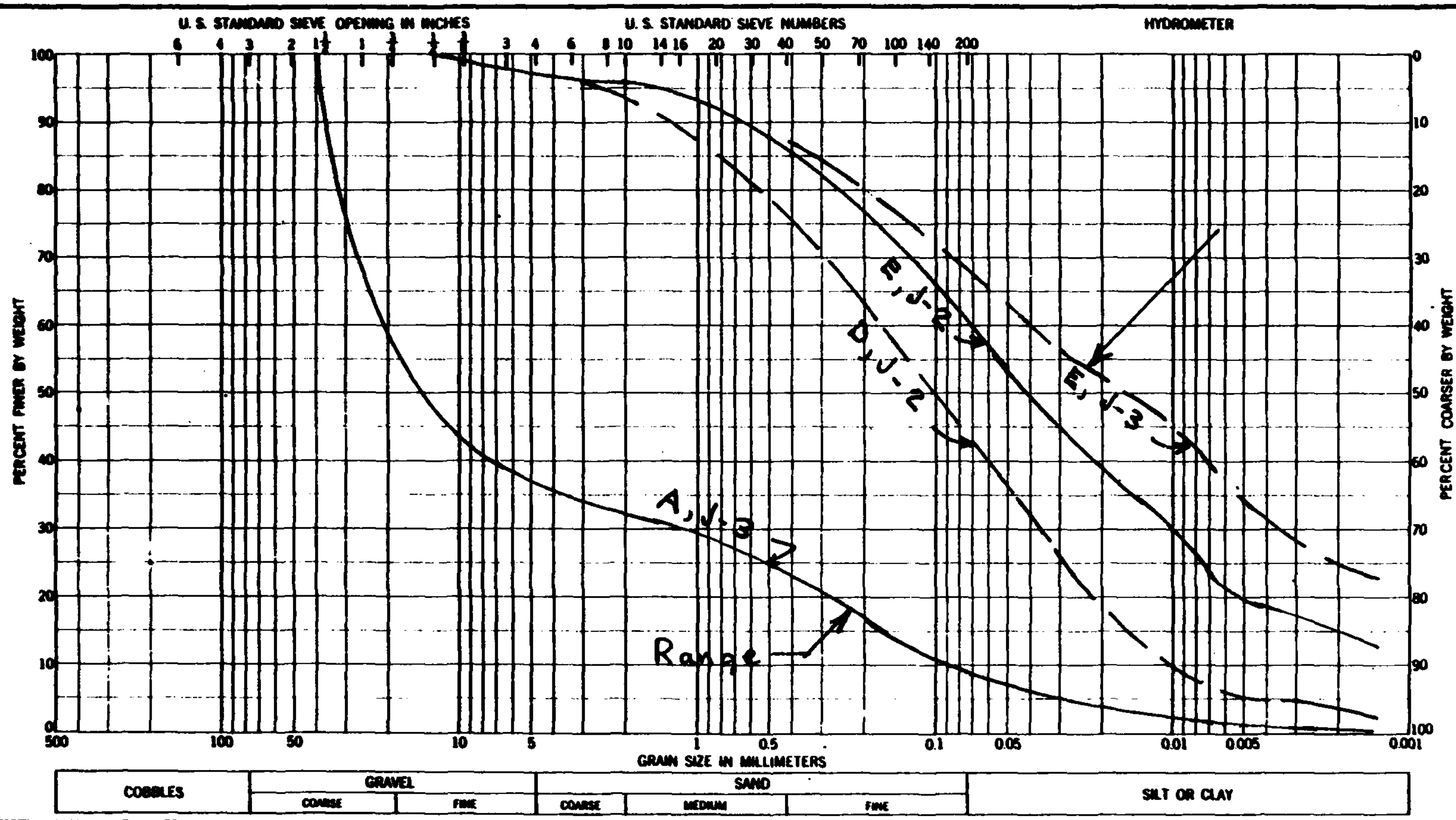
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

PLAN OF EXPLORATIONS AND GEOLOGIC LOG PROFILE

BEAVER BROOK, KEENE, N.H.

Plate No. B-1

2



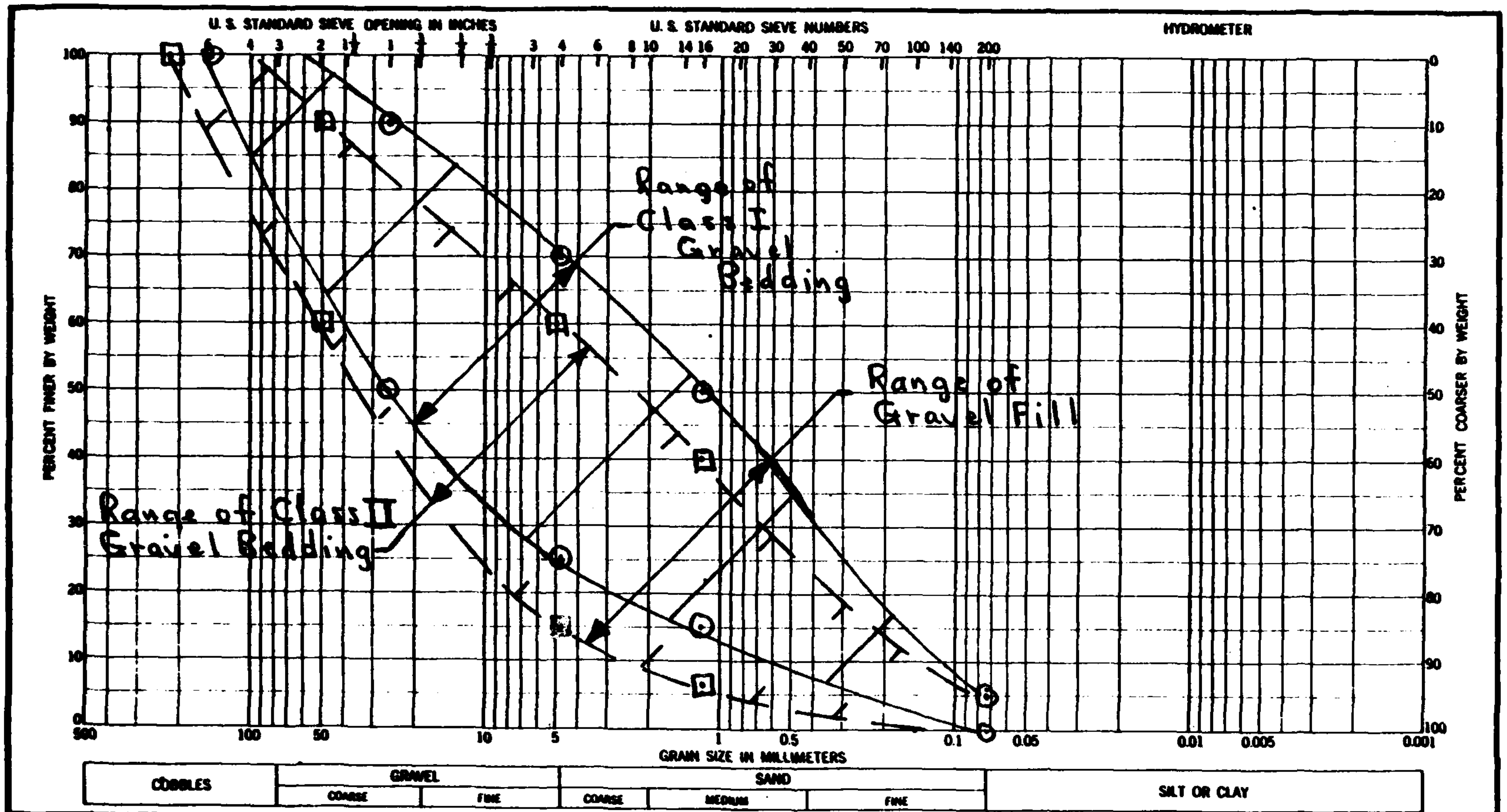
Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI

GRADATION CURVES

Project Beaver Brook L.P.P.
Selected Test Data
Area Foundation Materials -
Boring No. Dam & Dike A
Date Feb. 1983

ENG FORM 1 MAY 63 2087

Plate No. 8-2



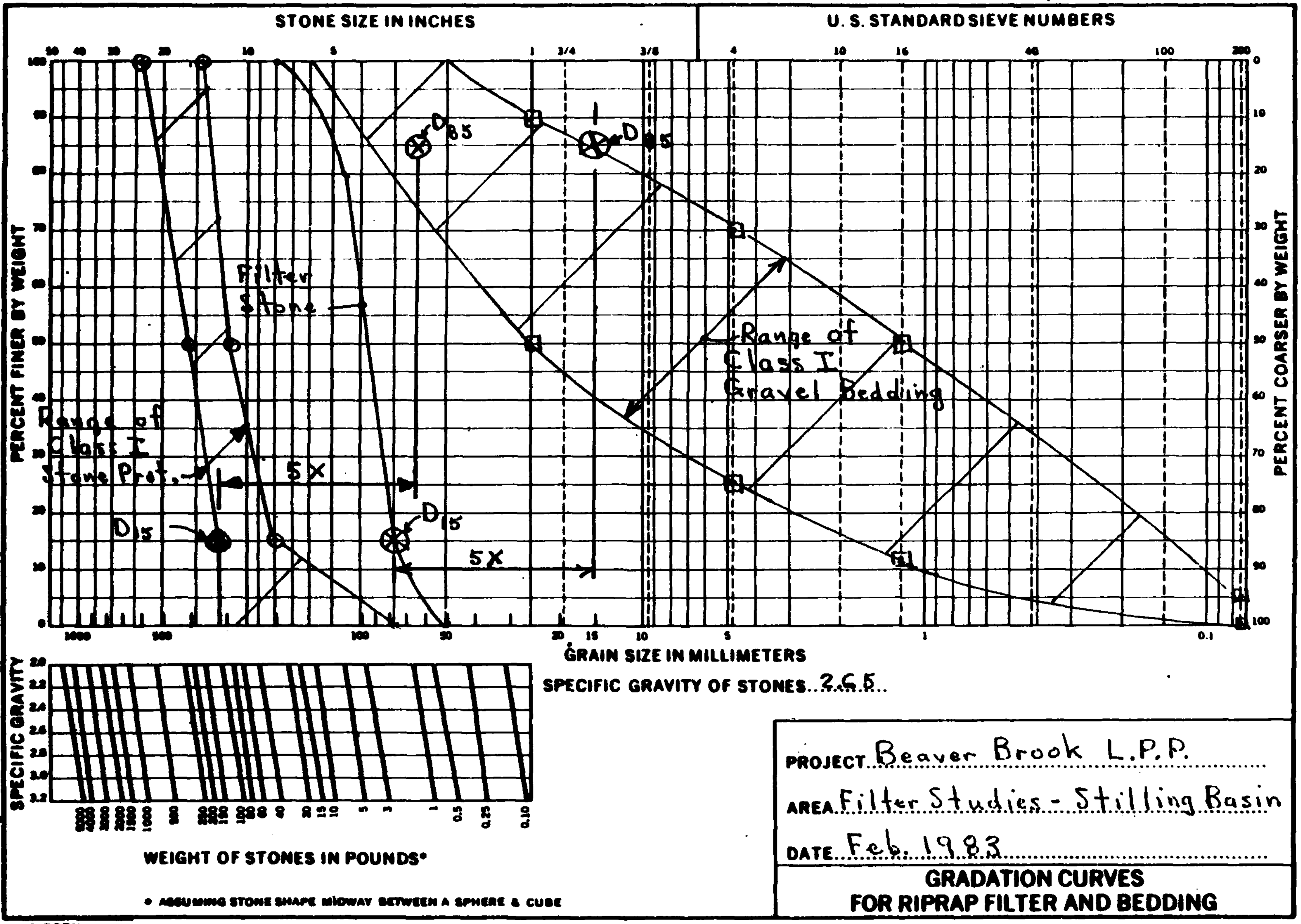
Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI

Project Beaver Brook L.P.P.
Gradation Specifications
Area Gravel Bedding &
Boring No. Gravel Fill
Date Feb. 1983

GRADATION CURVES

ENG FORM 1 MAY 63 2087

Plate [illegible] III-3



PROJECT Beaver Brook L.P.P.

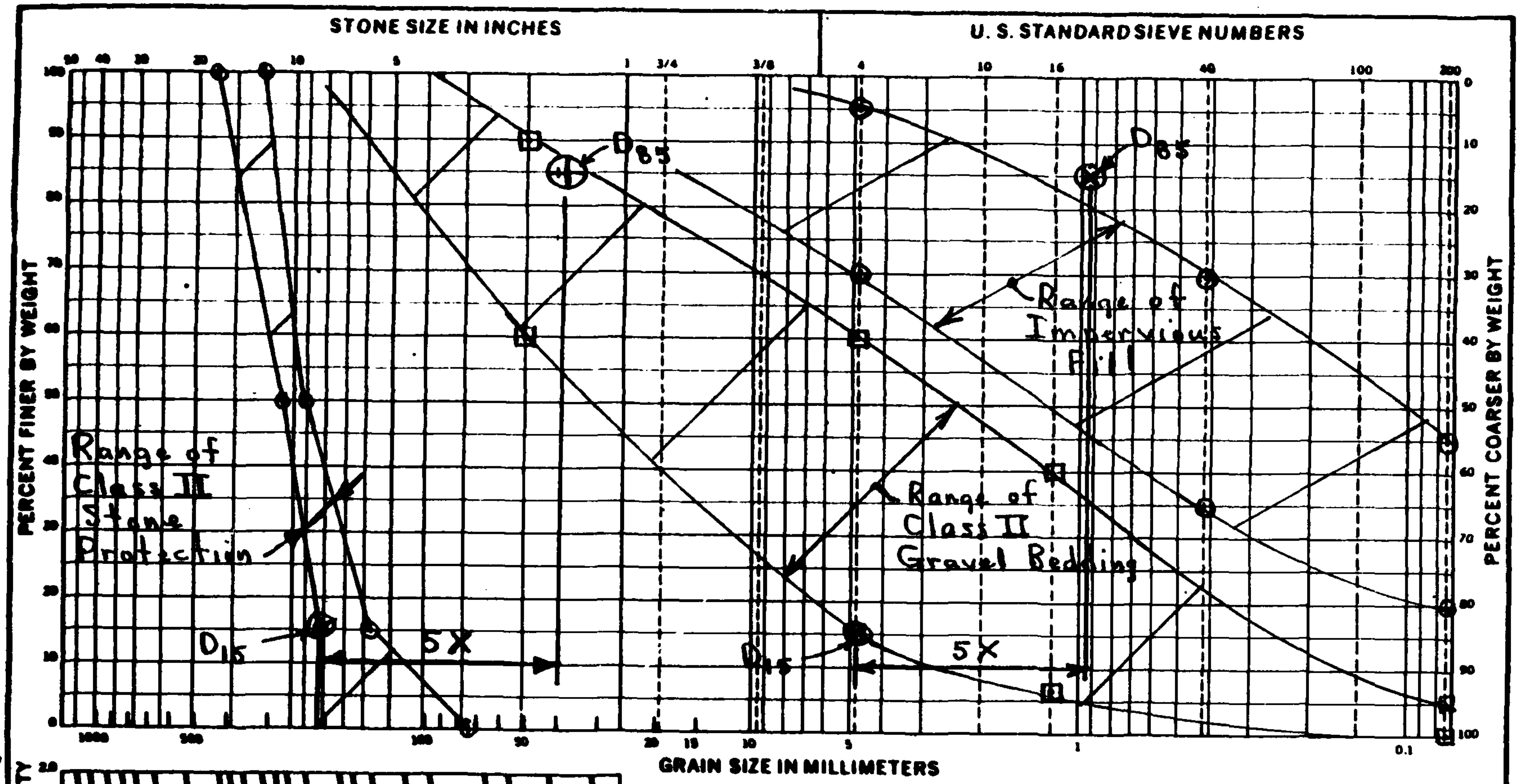
AREA Filter Studies - Stilling Basin

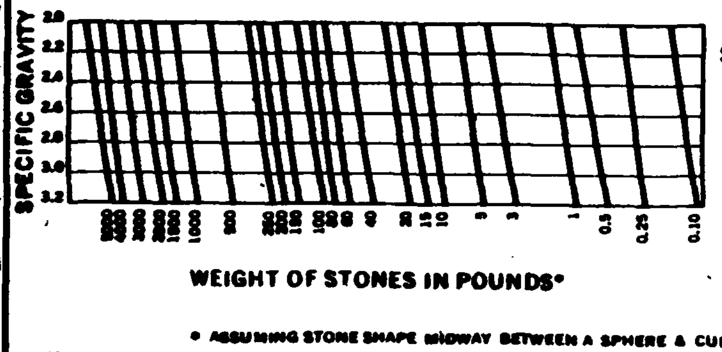
DATE Feb. 1983

GRADATION CURVES FOR RIPRAP FILTER AND BEDDING

ENG FORM 4056
APR 67

Plate No. B-4





SPECIFIC GRAVITY OF STONES 2.65

PROJECT Beaver Brook L.P.P.

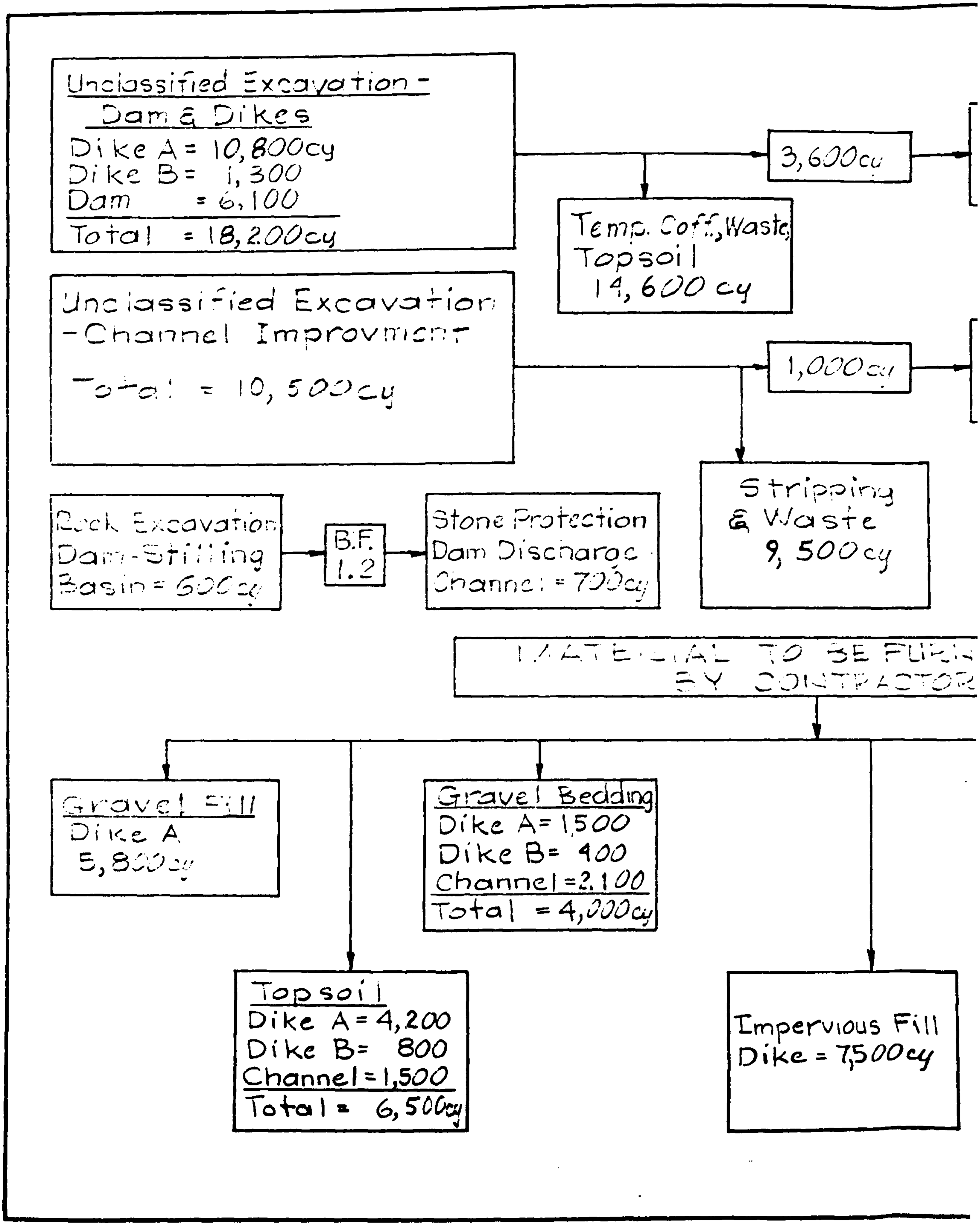
AREA Filter Studies

DATE Feb. 1983

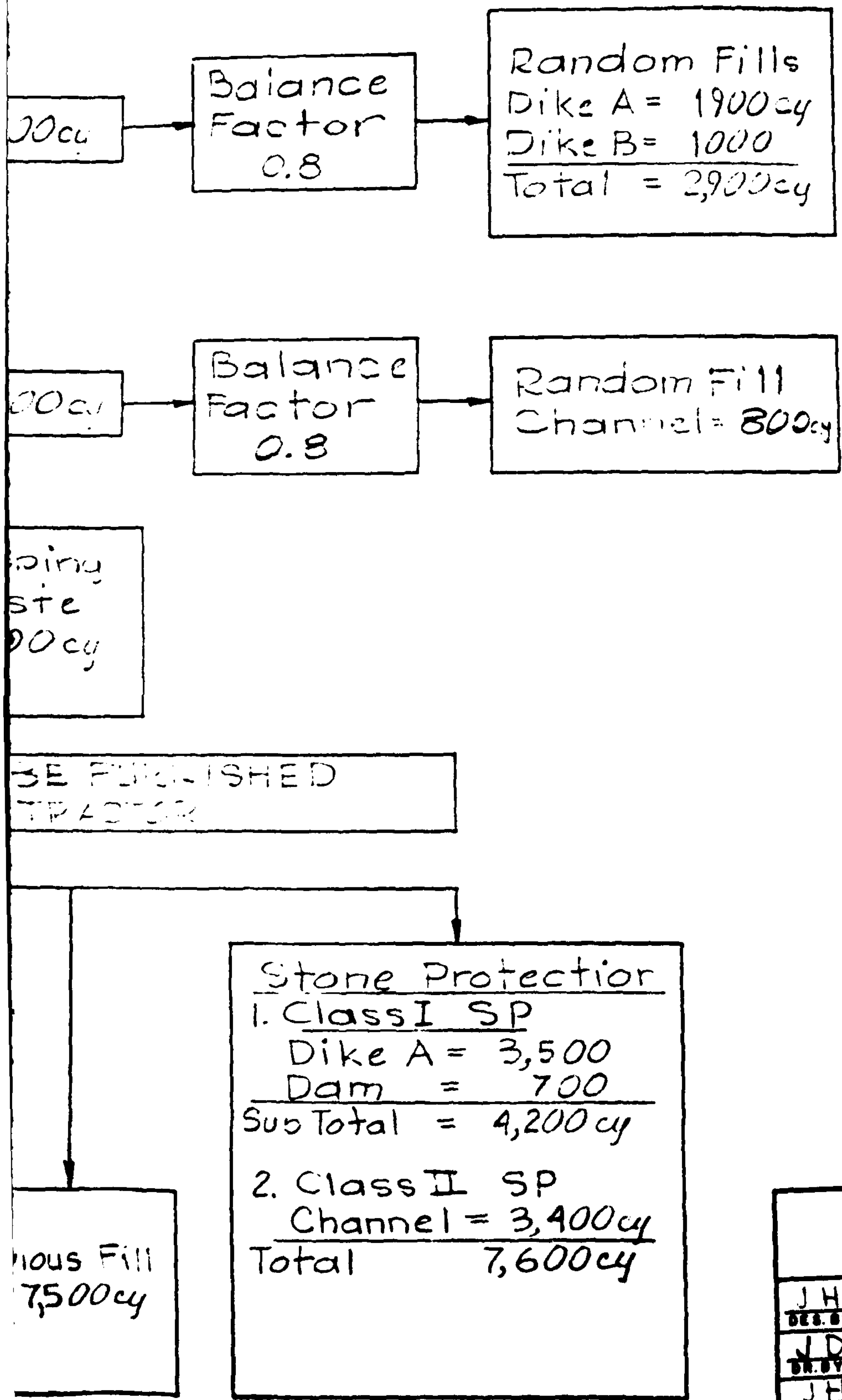
GRADATION CURVES FOR RIPRAP FILTER AND BEDDING

ENG FORM 4056
APR 67

Plate No. B-5



Unclassified Excavation - Dam & Dikes
Dike A = 10,800cy
Dike B = 1,300
Dam = 6,100
Total = 18,200cy
3,600cy
Temp. Coff., Waste, Topsoil 14,600 cy
Unclassified Excavation - Channel Improvment
Total = 10,500cy
1,000cy
Stripping & Waste 9,500 cy
Rock Excavation Dam-Stilling Basin = 600cy
B.F. 1.2
Stone Protection Dam Discharge Channel = 700cy
MATERIAL TO BE FUR
BY CONTRACTOR
Gravel Fill
Dike A
5,800cy
Gravel Bedding
Dike A = 1,500
Dike B = 400
Channel = 2,100
Total = 4,000cy
Topsoil
Dike A = 4,200
Dike B = 800
Channel = 1,500
Total = 6,500cy
Impervious Fill
Dike = 7,500cy



DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS

J H DES. BY J D DR. BY J H CK. BY	BEAVER BROOK L.P.P. MATERIALS USAGE CHART (PRELIMINARY)
GEOTECH. ENG. BR. SK. NO.	SCALE: DATE: 2/16/83

SOIL TESTS RESULTS

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS GRAVEL %	SAND %	FINES %	D_{10} mm.	ATT. LIMITS LL	PL	SPECIFIC GRAVITY	NAT. WATER CONTENT % DRY WT TOTAL	-NO4	COMPACTION DATA STND. AASHO OPT. WATER % DRY WT	MAX. DRY DENS. LBS/CU FT	* PVD LBS/CU FT	NAT. DRY DENSITY LBS/CU FT TOTAL	-NO4	OTHER TESTS SHEAR	CONSOL	PERM.	
A	789.4	J-2	5.0-9.0	CL	4	38	58	<0.001	26	16	2.75	17.3										BEAVER BROOK, KEENE, NH
		J-3	11.0-13.5	GP-GM	63	29	8	0.09														
D	789.0	J-2	4.5-5.5	SM	4	51	45	0.01	NP	NP		14.7										
E	790.9	J-2	5.0-10.0	CL	3	38	59	<0.001	26	16		16.5										
		J-3	10.0-14.0	CL	4	28	68	0.001	32	18	2.78	15.3										

NED FORM JUL 63 510 * PROVIDENCE VIBRATED DENSITY TEST

TABLE 1

Plate No. B-7

ROCK TEST RESULTS

Results of Absorption, Specific Gravity, and Unit Weight (ASTM C-97) are summarized below:

Sample	Schist FD-A(17')	Schist FD-C(8')	Granite FD-D-7.5')
Bulk Specific Gravity	2.766	2.832	2.636
Bulk Specific Gravity (SSD)	2.776	2.842	2.644
Apparent Specific Gravity	2.792	2.860	2.657
Dry Unit Weight (SSD)	173.22	177.34	164.99
Percent Absorption	0.33	0.34	0.31

Results of Compressive Strength of Natural Building Stone (ASTM C-170) are summarized below:

Sample	FD-A(17')	FD-C(8')	FD-D(7.5')
Unconfined Compressive Strength (psi)	6,700	10,600	24,800

Table 2

Plate No. B-8

U. S. ARMY
CORPS OF ENGINEERS
NEW ENGLAND DIVISION

FIELD LOG OF TEST BORING

Site Beaver Brook — Page 1 of 3 Pages
Boring No. A — Desig. FD-623 — Diam. (Casing) 4
Co-ordinates. N ____ E ____

Elevation Top of Boring ____ M.S.L.		Hammer Wt. 300	Boring Started 3-12-82
Total Overburden Drilled 14 Feet		Hammer Drop 18	Boring Completed 3-15-82
Elevation Top of Rock ____ M.S.L.		Casing Left 0	
Total Rock Drilled 15 Feet		Subsurface Water Data ____ Page ____	
Elevation Bottom of Boring ____ M.S.L.		Obs. Well ____ 4.5'	
Total Depth of Boring 29 Feet		Drilled By Bryan Engineering	
Core Recovered 86 % No. Boxes 1		Mfg. Des. Drill Acker - Bombadier	
Core Recovered 14 Ft NX Diam. 2⅛ in.		Inspected By: NA Lanney	
Soil Samples 2 in. Diam. 7 No.		Classification By: NA Lanney	
Soil Samples ____ in. Diam. ____ No.		Classification By: [illegible]	

DEPTH	CORE/SAMPLE NO.	SIZE	DEPTH RANGE	BLOWS PER FT. / CORE RECVY	SAMPLING AND CORING OPERATIONS	CLASSIFICATION OF MATERIALS
0–1	1	2	0–1	11	Drove 2" solid spoon sampler from 0' to 5' and recovered 18"	TOPSOIL
1–5	1A 2 jars	2"	1–5	44, 17, 30, 44	Drove 4" casing to 5' and washed out casing with 3" roller bit	~~SANDY SILT slightly plastic, 90% fine sand~~ gray, stiff, damp ~~(ML)~~ gravelly silty f sand (SM)
5–9	2 2 jars	2"	5–9	15, 20, 32, 47	Drove 2" solid spoon sampler from 5 to 9.5' and recovered, 2'-4" Bouncing refusal at 9.5' Drove 4" casing from 5 to 10' and washed casing with 4" roller bit. Could not roller bit past 10.0'	Gray SANDY CLAY ~~slightly plastic, 10-20% coarse to fine sand~~ stiff, ~~gray~~, wet (CL) w/ traces of gravel.
9–10	2A	2'	9–9.6	100%		Cobs - silty, gravelly, m/f sand (cobbles)

1130 HRS
1203

GENERAL REMARKS:

Boring No. A

B-9

Site: Beaver Brook
Keene NH

Boring No. A

DEPTH		CORE/SAMPLE			BLOWS PER FT. CORE RECOV	SAMPLING AND CORING OPERATIONS	CLASSIFICATION OF MATERIALS
WATER ON CASING	Ft. 2'	NO.	SIZE	DEPTH			
	10		NX	10 / 11	6"	Cored from 10' to 11' with NX double tube barrel ~~and recovered 6"~~	
	12	3	2"	11 / 13.5	45 / 70	Cored cobble. Drove 2" solid barrel sampler from 11.0 to 13.5'. Bouncing refusal at 13.5. Recovered 20". Cored through boulder with roller bit & drove 4" casing to 14.0	Gr.br. Sandy SILTY GRAVEL (GP-GM), coarse to fine subangular gravel ~~to 2" max, ... coarse to fine ... sand, ... nonplastic fines, gray (GM)~~
		1 year		13.5 / 14	6"	Refusal at 14'	TOP OF ROCK AT 14'
	14	1	NX	14 / 15	0	~~Cored from 14' to 15' with NX double tube barrel and recovered nothing~~	
	16	2	NX	15 / 20		Cored from 15 to 20' with NX double tube barrel and recovered 60"	Biotite Schist, foliated, fresh to slightly weathered, hard, medium to coarse grained. 15.8 Foliation plane jt. 16.7-17 qtz vein. 17.5 foliation plane jt. 18.0 Jt w/ qtz coating. 19.0 Foliation pl. joint. 19.4 Jt w/ qtz coating
	18						
	20	3	NX	20 / 25		Cored from 20 to 25' with NX double tube barrel and recovered 60"	21.0 Foliation pl joint. 22.75 Foliation plane jt
	22						
	24						

3-5-86

ENG FORM 1836 (Test)

Boring No. A

Site: Beaver Brook
Keene, NH

Boring No. A

Page 3 of 3

DEPTH		CORE/SAMPLE			BLOWS PER FT.	SAMPLING AND CORING OPERATIONS	CLASSIFICATION OF MATERIALS
		NO.	SIZE	DEPTH			
	26	4	NX	25 29		Cored from 25 to 29 with NX double tube barrel and recovered 48"	
	28						
						End of Boring at 29.0'	
	30						

Boring No. A

U. S. ARMY
CORPS OF ENGINEERS
NEW ENGLAND DIVISION

FIELD LOG OF TEST BORING

Site BEAVER BROOK, KEENE, NH Page 1 of 3 Pages

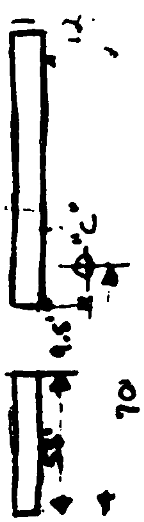
Boring No. C Desig. ED-82-1 Diam. (Casing) 4.0"

Co-ordinates. N ______ E ______

Elevation Top of Boring ______ M.S.L.	Hammer Wt. 300 lb	Boring Started 3-9-82	
Total Overburden Drilled 4.5 Feet	Hammer Drop 18 in.	Boring Completed 3-10-82	
Elevation Top of Rock ______ M.S.L.	Casing Left 0		
Total Rock Drilled 15.0 Feet	Subsurface Water Data ______ Page ______		
Elevation Bottom of Boring ______ M.S.L.	Obs. Well —		
Total Depth of Boring 19.5 Feet	Drilled By Baker Engineering		
Core Recovered 91 % No. Boxes 1	Mfg. Des. Drill Acker - Track Mounted		
Core Recovered 13.7 Ft: NX Diam. 2 1/8 in.	Inspected By: Ronald F. Bukoski		
Soil Samples 2" In. Diam. 3 No.	Classification By: Ronald F. Bukoski (N. Lessing)		
Soil Samples ______ In. Diam. ______ No.	Classification By: [illegible] NED Mat'ls Lab		

DEPTH: Blows on Casing	DEPTH: Ft.	CORE/SAMPLE: No.	CORE/SAMPLE: Size	CORE/SAMPLE: Depth Range	Blows per ft. / Core Recv'y	SAMPLING AND CORING OPERATIONS	CLASSIFICATION OF MATERIALS
15	0	S-1 (1 of 1)	2.0"	0.0 to 0.5'	10	DROVE 2.0" ID × 5.0' SOLID SPOON SAMPLER FROM GROUND SURFACE TO 4.5' USING 300 lb HAMMER DROPPED 18". BOUNCING REFUSAL AT 4.5'. RECOVERED APPROXIMATELY 13".	SURFACE: SURFACE FEATURES OBSCURED BY 30" OF SNOW. DEPTH OF WATER AT BORING LOCATION WAS 1.2' BENEATH ICE AND SNOW. SEVERAL LARGE COBBLES WERE VISIBLE IN WATER AT BORING LOCATION.
46	1	S-1A (1 of 1)	2.0"	0.5' to 3.0'	36	DROVE 4.0" CASING FROM GROUND SURFACE TO 4.5' AT WHICH POINT BOUNCING REFUSAL WAS ENCOUNTERED.	S-1: SILTY SANDY GRAVEL (GP-GM) ~~Brown~~, COARSE TO FINE SUBANGULAR GRAVEL, ~~[illegible]~~ SATURATED, ~~Brown, (GP-GM)~~.
49	2				11	PROBE ATTEMPTED 2' FROM CASING TOWARD DAM CENTER LINE USING "AN" DRILL RODS. DROVE RODS TO REFUSAL	S-1A: DECOMPOSED SHIST, HIGHLY MICACEOUS, COLONEM BROWN.
48	3	S-1B (1 of 1)	2.0"	3.0' to 4.5'	44	AT 5.5'. ASSUMED TOP OF ROCK AND CORING OPERATIONS WERE STARTED FOLLOWING THE WASHING OUT OF THE CASING USING A ROLLER ROCK BIT.	S-1B: HIGHLY WEATHERED SHIST, 2" SEAM OF GRAVELLY SILT OVERLYING, LOW PLASTICITY, ~~[illegible]~~ FINE SUBROUNDED GRAVEL. GRAY, (ML), MICACEOUS SHIST, THIN LAYERS OF QUARTZ TO 1/2" THICK, gray.
52	4				35/0.5'		
	5						

GENERAL REMARKS: BLOWS ON CASING FOR 300 lb HAMMER DROPPED 24 in.



NED [illegible] (Test) Boring No. C

Site: Beaver Brook Dam
Keene, NH

Boring No. C

Depth (Ft.)	Core/Sample No.	Size	Depth	Blows per ft. / Core Recov.	Sampling and Coring Operations	Classification of Materials
5–7.5	1	NX	4.5 to 7.5	30"	Cored with NX double tube barrel from 4.5 to 7.5' and recoverd 30"	SCHIST, gray, micoceous, fresh to slightly weathered, hard, medium to coarse grained 5.6-5.75 fractured zone 6' foliation plane jt 6.1-6.3 fractured zone 6.5 foliation plane jt
7.5–12.5	Run 2	NX	7.5 to 12.5	60"	Cored with NX double tube barrel from 7.5' to 12.5' and recovered 60"	9' foliation plane jt 9.3 " " " 9.8 " " " 10.2 " " " 12.3 foliation plane jt 45 12.5-13 highly fractured zone
12.5–					Cored with NX double tube barrel from 12.5' to 17.5' and recovered 56"	

Site: Beaver Brook Dam Keene NH	Boring No. C	Page 3 of 3

Depth		Core/Sample No.	Size	Depth	Blows per ft. / Core Recov	Sampling and Coring Operations	Classification of Materials
	13						13.25 Foliation plane jt
	14	Run 3	NX	12.5 to 17.5	60"		14.25 Foliation plane jt
							14.75 " " "
	15						
	16						16 Foliation plane jt
							16.25 " " "
	17						
	18	Run 4	NX	17.5 to 19.5	18"	Cored with NX double tube barrel from 17.5' to 19.5' and recovered 18"	17.5-19.5 Fractured zone
	19						
	19.5					End of Boring at	19.5'
	20						

Boring No. C

U. S. ARMY
CORPS OF ENGINEERS
NEW ENGLAND DIVISION

FIELD LOG OF TEST BORING

Site Beaver Brook Dam Page 1 of 2 Pages

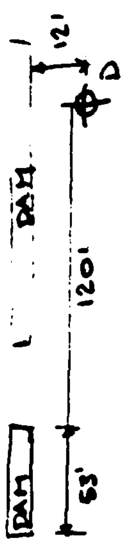
Boring No. D Desig. FD-82-2 Diam. (Casing) 4"

Co-ordinates. N ______ E ______

Elevation Top of Boring ______ M.S.L.	Hammer Wt. 300	Boring Started 3-11-82	
Total Overburden Drilled 5.5 Feet	Hammer Drop 18"	Boring Completed 3-12-82	
Elevation Top of Rock ______ M.S.L.	Casing Left 0		
Total Rock Drilled 15 Feet	Subsurface Water Data 2.75' Page ______		
Elevation Bottom of Boring ______ M.S.L.	Obs. Well ______		
Total Depth of Boring 20.5 Feet	Drilled By Briggs Engineering		
Core Recovered 94 % No. Boxes 1	Mfg. Des. Drill Acker Bombadier		
Core Recovered 14 Ft: NX Diam. 2 1/8 In.	Inspected By: NA Lanney		
Soil Samples 2" In. Diam. 3 No.	Classification By: NA Lanney		
Soil Samples ______ In. Diam. ______ No.	Classification By: [illegible]		

Depth	Ft.	Core/Sample No.	Size	Depth Range	Blows per ft. / Core Recvy	Sampling and Coring Operations	Classification of Materials
0.5		1	2"	0 – 0.5	3	Drove 2" solid spoon sampler from 0 to 3.5'. Refusal at 3.5' Recovered 18".	TOPSOIL
	2	1A	2	0.5 – 3.5	3, 8, 20	Moved hole 6" and drove 4" casing to 4.5' and washed out casing with 3" roller bit	~~SANDY SILT~~, ~~slightly plastic, coarse to sand~~, few roots, brown, saturated, gray, silty m/f sand (SM)
4.5	4	2	2"	4.5 – 5.5	33, 75	Drove 2" solid spoon sampler from 4.5 to 5.5'. Refusal at 5.5'. Recovered 6"	Silty m/f sand (SM) ~~slightly plastic~~ ~~few fine sand~~ gray-brown saturated ~~sand~~ with gravel
5.5	6	Run 1	NX	5.5 – 8.5		TOP OF ROCK AT 5.5' Cored from 5.5 to 8.5' with NX double tube barrel and rec. 36"	GRANITE, light gray, sound, very hard, fine-grained; 6.6 30° joint
	8	Run 2	NX	8.5 – 9.5		Cored from 8.5 to 9.5 with NX double tube barrel and recovered [illegible]	9.0 Rust stn joint
	10						

GENERAL REMARKS:



55(Test) Boring No. D

Site: Beaver Brook Keene NH	Boring No. D	Page 2 of 2

DEPTH		CORE/SAMPLE			BLOWS PER FT. CORE RECVY	SAMPLING AND CORING OPERATIONS	CLASSIFICATION OF MATERIALS
	F.-2'	NO.	SIZE	DEPTH			
	11.0	Run		9.5		Cored from 9.5 to 14.5 with double	
	12	3	NX	14.5		tube NX Core barrel and recovered 60"	Interbedded GRANITE G[illegible] and BIOTITE SCHIST, foliated, fresh, hard medium to fine grained. 11.0 joint 11.7 foliation plane jt 12.0 clay coated jt 12.5 " " " 12.7 rust std jt. 13.0 clay coated jt 14.3 jt 15.25-15.5 clay seam 16.5-17 Highly fractured zone 19.25 Clay coated jt 19.75 " " "
	14						
		4	NX	14.5 15.5		Cored from 14.5 to 15.5 with double tube NX core barrel and recovered 11"	
	16						
	18	5	NX	15.5 20.5		Cored from 15.5 to 20.5 with NX double tube barrel Recovered 58	
	20						
	22					End of Boring at 20.5'	

Boring No. D

U. S. ARMY
CORPS OF ENGINEERS
NEW ENGLAND DIVISION

FIELD LOG OF TEST BORING

Site Beaver Brook　Page 1 of 2 Pages
Boring No. E　Desig. FD-12-4　Diam. (Casing) 4"
Co-ordinates. N ______ E ______

Elevation Top of Boring ______ M.S.L.
Total Overburden Drilled 14 Feet
Elevation Top of Rock ______ M.S.L.
Total Rock Drilled 5 Feet
Elevation Bottom of Boring ______ M.S.L.
Total Depth of Boring 19 Feet
Core Recovered 100 %　No. Boxes 1
Core Recovered 5 Ft : NX　Diam. 2 1/8 in.
Soil Samples 2½ In. Diam. 3 No.
Soil Samples 2 In. Diam. 3 No.

Hammer Wt. 300　Boring Started 3-15-82
Hammer Drop 18"　Boring Completed 3-16-82
Casing Left 0
Subsurface Water Data ______ Page ______
Obs. Well ______
Drilled By Briggs Engineering
Mfg. Des. Drill Acker - Bombadier
Inspected By: N A Lanney
Classification By: NA Lanney
Classification By: [illegible] NED Soils Lab

DEPTH	CORE/SAMPLE NO.	SIZE	DEPTH RANGE	BLOWS PER FT. / CORE RECV'Y	SAMPLING AND CORING OPERATIONS	CLASSIFICATION OF MATERIALS
0–1	1	2½	0–1	4	Drove 2½" solid barrel sampler from 0 to 5' and recovered 30"	TOPSOIL
1–5	1A, 2 jars	2½	1–5	17, 23, 25, 32		SANDY CLAY, ~~slightly plastic, 20-30% fine sand~~ olive-gray, stiff (CL) GLACIAL TILL
5–10	2, 3 jars	2	5–10	11, 19, 27, 33, 49	Drove 2" solid barrel sampler from 5' to 10' and recovered 30". Drove 4" casing to 10' and washed out casing with 3½" chopping bit	SANDY CLAY ~~slightly plastic 30-40% coarse to fine sand~~, traces of gravel, (CL) olive-gray, stiff GLACIAL TILL

GENERAL REMARKS:

NED 35(Test)　Boring No. E

B-12

Site: Beaver Brook Keene NH	Boring No. E	Page 2 of 2

DEPTH		CORE/SAMPLE			BLOWS PER FT. CORE RECVY	SAMPLING AND CORING OPERATIONS	CLASSIFICATION OF MATERIALS
[illegible]	f=2'	NO.	SIZE	DEPTH RANGE			
	10–14	5 2 jars	2"	10 14	14 40 47 130	Drove 2" solid barrel sampler from 10 to 14' and took sample Recovered 24" Drove 4" casing to 13.5' and washed out casing with chopping bit	~~CLAYEY SILT, slightly to non plastic, 19-99% sand = 3% gravel~~ stiff, dark gray ~~(?)~~ GLACIAL TILL Sandy, clay (CL) w/traces of gravel. TOP OF ROCK AT 14'
	14–18	Run 1	NX	14 19		Cored from 14' to 19' with NX double tube barrel and recovered 60".	GRANITE, slightly weathered, hard, finegrained, clay filled joints 14-14.5 fractured zone 15.5-16, 16.5-17 fractured zones 17.75 joint 18.0 joint 18.5, 19 joints
	18–19						BIOTITE: fresh, hard, foliated
	20					END OF BORING AT 19'	

ENG FORM 1836 (Test) Boring No. E

SECTION C

ECONOMIC ANALYSIS

Economic Analysis

Beaver Brook

Keene, NH

Table of Contents

ECONOMICS OF THE SELECTED PLAN

This section presents the economic analysis of the flood control plans selected for detailed evaluation. The methodology of the benefit-cost analysis is explained; the annual costs and benefits are presented; and, the benefit-cost ratio is calculated. The contents of this section focus on those facets of the proposed improvements which can be quantified in dollar values.

METHODOLOGY

The economic analysis of the proposed improvements provides an essential function in the evaluation of a comprehensive flood control program for the project area. Comparing the average annual costs (i.e. interest, amortization and operation and maintenance) to an estimate of the average annual benefits, which are anticipated to occur over the 100-year life of the project, provides a measure of the economic viability of project alternatives. The benefits should equal or exceed the costs for the Federal government to participate in the project.

The development of costs and benefits in this report follows standard Corps of Engineers practice. The value of all goods and services used in the project is estimated on the cost side, while on the benefit side, damages prevented are estimated. The assessment of damages prevented is based on flood damage surveys which provide dollar estimates of both physical and nonphysical losses related to various stages or elevations of flooding. These losses are then related to stage-frequency data which result in expected losses on an annual basis. Annual benefits are calculated by determining the reduction in annual losses that would occur with the project.

The value of future benefits and costs are made comparable by conversion to an equivalent time basis using the Federal interest rate of 7-7/8 percent which is mandated by law for water resource projects. The net effect of converting benefits and costs in this manner is the development of comparable average annual values. All costs and benefits are expressed at February 1983 price levels.

FLOOD LOSSES

The potential flood damages for Keene were determined by damage survey specialists who estimated the physical and nonphysical losses to properties in the flood plain based on various elevations of flooding. The flood plain area of Keene was divided into six zones for purposes of analysis. The divisions were based upon hydrologic considerations.

Character of the Flood Area

The floodprone area along Beaver Brook extends from its confluence with the Branch River upstream about 6.8 miles to the north corporate limit of Keene. The flood damage zones have been numbered 0 through 5. The zone farthest downstream has been numbered Zone 0, with Zones 1 through 5 proceeding upstream.

Zone 0 starts at the confluence of Beaver Brook and the Branch River and extends a distance of 800 feet upstream to the Rt. 101 Bypass. Losses to both residential and commercial properties have been experienced in this zone. Zone 1 extends 2350 feet from the Rt. 101 Bypass to Marlboro Street and also encompasses residential and commercial losses. Zone 2 is the area from Marlboro Street to the B&M Railroad Bridge, a distance of 1400 feet. Damages to residential, commercial and industrial properties in this zone are approximately 21 percent of the total average annual losses. Zone 3 extends for a distance of 2350 feet between the B&M Railroad Bridge and Roxbury Street and like Zone 2 involves damages to residential, commercial, and industrial properties. Losses in Zone 3 make up 50 percent of the total average annual losses in the study area. Zone 4 extends 1200 feet from Roxbury Street to Beaver Street and sustains flood damages to residential properties. Zone 5 is the area upstream of Beaver Street to the corporate limit, a distance of 27,700 feet. Damage to residential, commercial, public and industrial properties in this zone account for about 15 percent of the total average annual losses.

Table 1 summarizes property types by zones within the SPF flood plain.

Table 1
The SPF Flood Plain
Property Type per Zone
Beaver Brook, Keene, N.H.
(numbers of structures)

Zones	Residential	Commercial	Industrial	Public	Total
Zone 0	8	8	-	-	16
Zone 1	139	5	-	-	144
Zone 2	77	11	3	-	91
Zone 3	95	17	9	-	121
Zone 4	51	-	-	-	51
Zone 5	14	4	1	1	20
Total	384	45	13	1	443

Recurring Losses

Losses by stages referenced to the record flood level were tabulated for the floodprone area. When related to frequencies, recurring losses for various storms are obtained. Losses for each zone for 20, 50 and 100 year events are provided below. (Table 2)

Table 2
Recurring Losses
Beaver Brook
($000's, Feb. 1983 price level)

Zones	20 year	50 year	100 year
	Event		
0	$51.0	$148.0	$180.6
1	16.9	63.1	99.9
2	301.9	1,134.3	1,798.0
3	1,244.6	1,435.9	1,664.1
4	134.2	205.4	281.2
5A	37.0	64.5	77.5
5B	158.6	209.5	260.7
TOTAL	$1,944.2	$3,198.2	$4,362.0

Losses are fairly evenly distributed among the three major damage categories: residential, commercial, industrial. A small proportion of losses are experienced in the public category. Percent losses for these categories for each storm are presented below.

Recurring Losses by category
(percent of total per event)

	20 year	50 year	100 year
Residential	32.3%	30.6%	30.0%
Commercial	27.7	27.6	29.4
Industrial	38.4	40.4	39.3
Public	1.6	1.4	1.2

Growth and Development

Population projections have been provided by the City of Keene. These projections indicate modest growth of about 10 percent over the 40 year period from 1980 to 2020 and are shown below.

1990	23,500
2000	25,600
2010	-
2020	29,900

Although significant future development is likely throughout Keene, development in the Beaver Brook flood plain would be limited. This limitation is mostly a result of the lack of available land. Anticipated development near the flood plain would be in commercial uses, resulting from redevelopment activities in the central business district.

COSTS

First costs for each plan are presented in the main report. Investment costs were also included by adding the cost of interest during construction. Total investment costs for each plan are as follows: Plan A, $1,704,300; Plan B, $30,600; and Plan C, $1,734,900. These costs are proportioned between the Federal government and non-Federal interests. This proportionment is also displayed in the main report.

Estimates of annual costs are based on a 100-year period of analysis at a 7-7/8 percent interest rate. Annual costs include expenditures for annual maintenance and funding for periodic replacement of equipment. Annual costs for Plan A are $135,800; Plan B, $5,300; and Plan C, $141,100.

BENEFITS

Flood Damage Reduction Benefits

The main benefit accruing to the proposed project is flood damage reduction. The flood damage reduction benefits are determined by taking the difference between annual losses under the without project conditions and residual annual losses anticipated with the alternative plans. This includes both physical and nonphysical losses prevented for activities located within the flood plain. Table 3 shows the average annual losses expected without a project (the natural conditions), the average annual losses remaining with project alternatives (residual losses), and the average annual benefits attributed to the project. Residual losses and benefits have been categorized by project features; namely, flood storage, channel improvements and floodwarning. Average annual losses are the average losses estimated over a period of time based on the probability of occurrence of floods of different magnitudes. Sample stage-damage and damage-frequency relationships for Zone 1 are graphically displayed on Plates C-1 and C-2 at the end of this section.

Floodwarning Benefits

Benefits to a floodwarning plan are the reduction of flood damages by the movement of damageable goods and property when a floodwarning has been issued.

The actual dollar value of the benefit is measured as the difference between damages with and without the floodwarning plan. Individual damage survey sheets for the flood plain properties were consulted to determine the amount of recurring damage to potentially moveable items. Conditions along Beaver Brook would allow for 3 hours of lead time to prepare for the flood.

For residential properties the following assumptions were made: 1) 75 percent of total recurring residential losses are physical, 2) 10 percent of physical losses are to moveable contents, 3) 50 percent of these losses could be eliminated with floodwarning. These resulted in an annual benefit for residential properties in each zone as follows:

Reach	0	$ 347
	1	183
	2	357
	3	2,539
	4	899
	5a	35
	5b	1,706
	Total	$6,066 - say $6,100

For commercial and industrial properties, a similar set of assumptions were developed as follows: 1) 50 percent of physical damages are to contents and equipment, 2) 10 percent of contents could or would be moved once a floodwarning has been disseminated, 3) reduction in losses is halved by costs incurred with activating a preparedness plan. Annual benefits (losses prevented) for each zone are as follows:

Reach	0	$40
	1	-
	2	1,036
	3	2,714
	4	-
	5a	26
	5b	112
	Total	$3,928 - say $3,900

The total annual benefit for the floodwarning plan alone would approximate $10,000. When acting in conjunction with structural elements as in Plan C, the floodwarning feature adds about $5,430 to total project benefits.

Affluence Benefits

Affluence benefits are future benefits based on the rate of increase in the real value of flood-susceptible household contents. The OBERS regional growth rate for per capita income is used as the basis for increasing the real value of residential contents. OBERS information shows that per capita income in Cheshire County (non-SMSA part of BEA Economic Area 006) is expected to increase from $4,800 in 1978 to $14,000 in the year 2030. This amounts approximately to a 2-1/8 percent average annual compound growth rate.

Using WRC regulations as a guide, the value of contents may not exceed 75 percent of the structural value of the residence. Homes in the Beaver Brook flood plain include about half single family and half 2 or more family dwelling units. Some single family homes are quite large; many of the 2 family homes were once single homes. Data provided by the city assessor places an average value of $45,000 for the homes in the Beaver Brook area.

From an examination of damage survey sheets for residential properties, an average contents value is estimated at 40 percent of total structure value. The following equation therefore holds true: .40X (growth factor) = .75X where X = residential structure value. Solving the equation for the growth factor reveals that the contents value can be expected to increase by a factor of 1.875. At an annual compound growth rate of 2-1/8 percent it will take approximately 30 years for the contents value to grow to 75 percent of the structural value of a residence. Since it will take 30 years from the 1978 survey date for content value to reach 75 percent of structural value, and project completion is anticipated in 1985, the affluence growth reaches the 75 percent level during the 23rd year of the project life.

Although potential contents losses can be expected to increase in the future, other losses to residential structures will remain relatively constant. As contents value reaches 75 percent of the structural value, the overall growth factor for total physical residential losses will amount to 1.25. This is shown below:

	STRUCTURAL VALUE		CONTENT VALUE	TOTAL
Present:	Y	+	.4Y	= 1
.4Y				
Future:	Y	+	.75Y	= 1
.75Y				

1.4Y (growth factor) = 1.75Y
growth factor = 1.75Y + 1.4Y
growth factor = 1.25

This factor was applied to physical residential losses and annualized for both without and with project conditions. The increase in losses due to the application of the affluence factor in each zone is exhibited in the following table.

Table 4
Increased Losses and Benefits Due to Affluence
February 1983, price level
(no discounting)

Zone	Increase in Annual res. losses without project	Increase in Annual res. losses with Storage & Channel	Increase in Benefits
0	$ 1,310	$ 680	$ 630
1	690	400	290
2	1,460	800	660
3	9,570	3,710	5,860
4	3,400	1,740	1,660
5	6,250	6,120	130
Total	22,680	13,450	9,230

These benefits are realized after the contents value reaches 75 percent of structural value during the 23rd year of project life. Therefore, the benefits are discounted at an interest rate of 7-7/8 percent and a 100-year project life.

These benefits are summarized below at the following points of reference: P_N - Existing Condition, P_1 - first year of project life, and P_{100} - last year of project life.

Total discounted affluence benefits	
$P_N - P_1$	$2,155
$P_1 - P_{100}$	3,475
TOTAL	$5,630

Summary of Benefits

Flood damage reduction and affluence benefits are totaled for each plan and presented in Table 5 below.

Table 5
Summary of Estimated Annual Benefits
(February 1983 price level)

Category	Plan A Storage & Channel	Plan B Floodwarning	Plan C Storage, Channel & Floodwarning
Flood Damage Reduction	$228,640	$10,000	$234,070
Affluence	5,600	-	5,600
TOTAL	$234,240	$10,000	$239,670

JUSTIFICATION

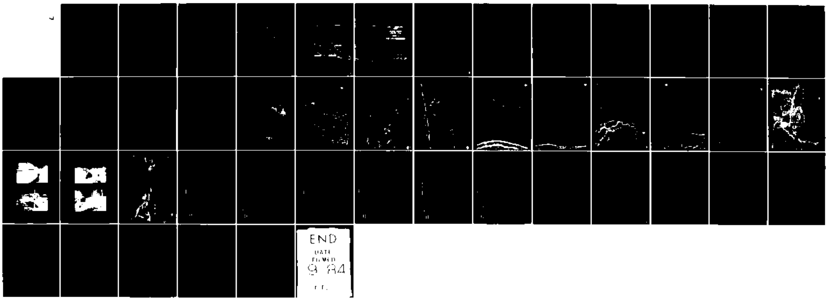
The estimated annual costs, annual benefits and the ratio of benefits and costs for each plan is summarized in Table 6. This analysis indicates that each plan is economically justified. Plan C yields the highest net benefits.

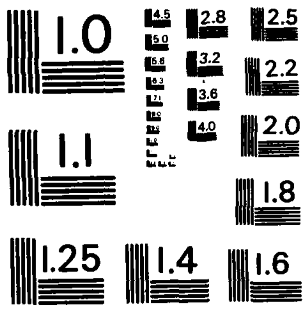
Table 6
Summary of Economic Analysis

	Plan A	Plan B	Plan C
Average Annual Benefit	$234,200	$10,000	$239,700
Average Annual Costs	135,800	5,300	141,100
Benefit-to-Cost Ratio	1.72	1.89	1.70
Net Benefits	$98,400	$ 4,700	$ 98,600

AD-A143 375 BEAVER BROOK KEENE NEW HAMPSHIRE FLOOD DAMAGE REDUCTION PROJECT SUPPORTIN..(U) CORPS OF ENGINEERS WALTHAM MA NEW ENGLAND DIV FEB 84 2/2

UNCLASSIFIED F/G 13/2 NL



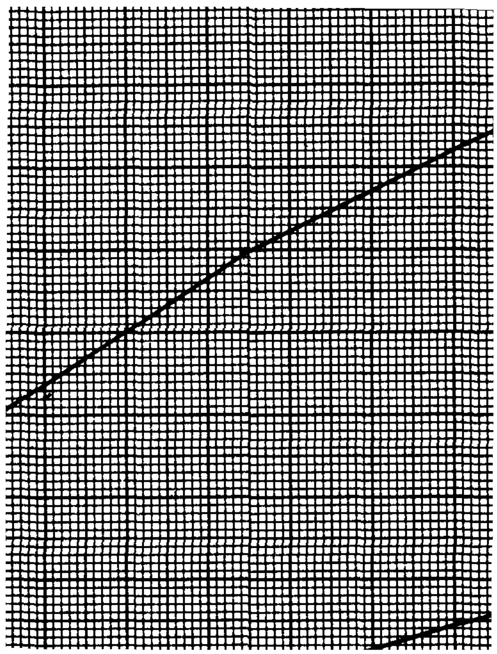


MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

Table 3

Average Annual Losses, Residuals, and Benefits
Beaver Brook, Keene, NH
($000's February 1983)

Zones	Natural Losses	Storage		Channel		(Plan B) Floodwarning		(Plan A) Storage & Channel		(Plan C) Storage, Channel and Floodwarning		Floodwarning (last added)	Channel (last added)
		Residual Losses	Benefits	Residual Losses	Benefits	Residual Losses	Benefits	Residual Losses	Benefits	Residual Losses	Benefits	Benefits	Benefits
0	11.87	6.24	5.63	11.87	0	11.48	.39	6.24	5.63	6.04	5.83	.20	0
1	4.91	2.85	2.06	4.91	0	4.72	.19	2.85	2.06	2.75	2.16	.10	0
2	88.06	46.12	41.94	72.55	15.51	86.67	1.39	42.48	45.58	41.75	46.31	.73	3.64
3	249.42	125.94	123.48	204.69	44.73	244.17	5.25	101.33	148.09	99.19	150.23	2.14	24.61
4	24.08	12.31	11.77	24.08	0	23.18	.90	12.31	11.77	11.79	12.29	.52	0
5A	5.91	2.97	2.94	5.91	0	5.85	.06	2.97	2.94	2.95	2.96	.02	0
5B	62.86	50.29	12.57	62.86	0	61.04	1.82	50.29	12.57	48.57	14.29	1.72	0
TOTAL	447.11	246.72	200.39	386.87	60.24	437.11	10.00	218.47	228.64	213.04	234.07	5.43	28.25



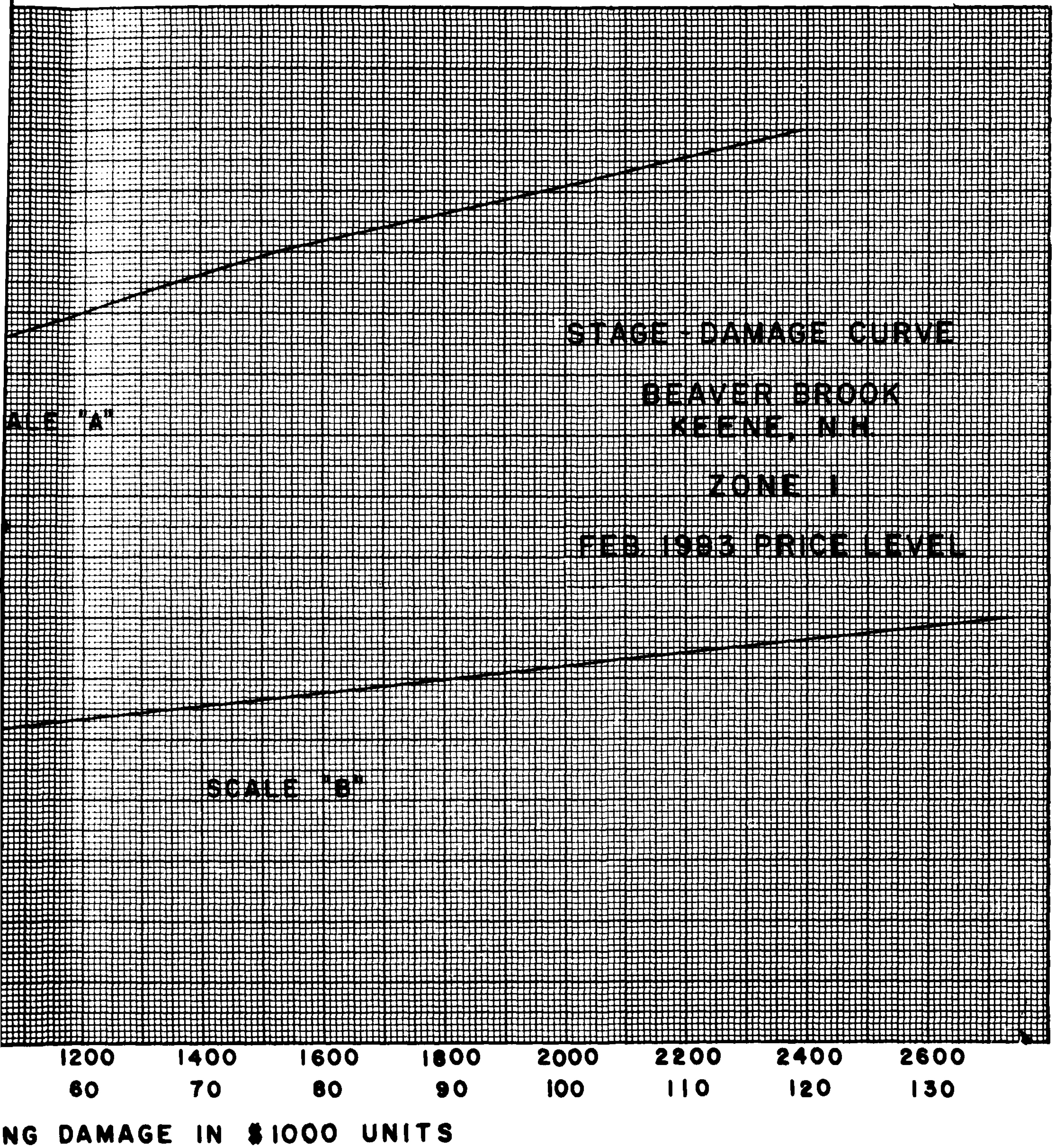
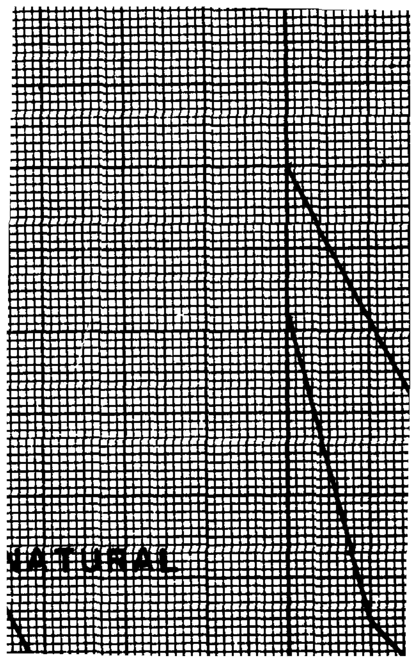

STAGE - DAMAGE CURVE
BEAVER BROOK
KEENE, N.H.
ZONE 1
FEB. 1983 PRICE LEVEL
ALE "A"
SCALE "B"
1200 1400 1600 1800 2000 2200 2400 2600
60 70 80 90 100 110 120 130
NG DAMAGE IN $1000 UNITS

PLATE C-1

2


NATURAL

FEB. 1

SECTION D

ARCHAEOLOGICAL RECONNAISSANCE REPORT

AN ARCHAEOLOGICAL RECONNAISSANCE
FOR THE BEAVER BROOK LOCAL FLOOD PROTECTION PROJECT
KEENE, NEW HAMPSHIRE

PREPARED FOR: U.S. Army Corps of Engineers
New England Division
Waltham, Massachusetts

BY: John S. Wilson, Division Archaeologist
Richard S. Kanaski, Co-Investigator

AUGUST 1982

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LIST OF FIGURES

INTRODUCTION

The following report has been prepared in conjunction with a Feasibility Report and Environmental Assessment for the Beaver Brook Local Protection Project. The project consists of channel improvements between Marlboro and Water Streets in downtown Keene, New Hampshire (Figures 1 & 2), and construction of a small dam and dike at Three Mile Swamp near the northeast corner of the city (Figures 1 & 3). The latter area is currently occupied by a dry laid fieldstone and earth dam. Normal water level in the swamp following project construction would be as at present, but during flood stages the new dam could hold a maximum of 10 feet additional depth. Therefore, the maximum includes all areas north of the proposed dam and below el. 797 (Figures 3-9) as well as construction areas at the dam and within the downtown channel improvement area.

RESEARCH DESIGN

Environmental Background

Marlboro Street to Water Street: The present environment of this vicinity is urban in character, with primarily industrial property. Soils within this area are classified as Ondawa fine sandy loam and Podunk fine sandy loam, both of which are moderately well drained and subject to occasional flooding (USDA Soil Conservation Service 1969) (Figure 10). However, field inspection revealed that this area is actually higher filled ground. The west bank from Marlboro Street up to the vicinity of Myrtle Street is a brushy bank behind a residential neighborhood, while the east bank is partially concrete lined and composed of fill adjacent to a modern factory (Figure 11). Both banks north of Myrtle Street to the railroad are fill comprising a parking lot (Figure 12) and mill yard, except two concrete-lined portions where a modern factory ell and access bridge cross the brook. The railroad bridge is a simple deck structure atop cut stone abutments. A portion of the east bank north of the railroad bridge is also of cut stone (Figure 13) but most resembles the area to the south, and is now used as mill yard. The Water Street bridge and a mill access bridge immediately to the south are typical small early 20th century concrete structures. A concrete factory foundation wall adjoins Water Street, along the west bank.

Three Mile Swamp: This area consists primarily of wetlands, with steep slopes to either side and scattered knolls of glacial outwash. The wetland consists primarily of Rummey and Scarboro fine sandy loams and muck and peat deposits (Figure 15). A thin strip of seasonally wet Ondawa fine sandy loam and Acton very stony fine sandy loam borders the eastern slopes, which consist of Canton very stony fine sandy loam with slopes exceeding 15%. Outwash deposits of Hinkley loamy sand occupy a narrow terrace west of Route 10 and small knolls within project area. All non-wetlands are wooded with considerable evidence of logging.

Prehistoric Background and Research Strategy: Studies in the Ashuelot Valley (Curran, 1980; Wilson, 1979) indicate considerable prehistoric occupation on glacial terraces near the river and the mouths of tributaries, but no professional archaeological work has previously been undertaken in the surrounding uplands. Recorded sites within the lower Ashuelot span the Paleo-Indian period to the mid-18th century (ca. 9000 BC - AD 1755) and appear to generally represent short-term occupations with a riverine orientation related to a mixture of aquatic and non-aquatic resources. Use of the valley as a transportation route may also have affected site location, given the steep uplands to either side. A contact period trail system ran up the main valley of the Ashuelot.

While riverine loctions along the Ashuelot probably provided better locations for taking anadromous fish and may have produced more diverse and productive habitats for terrestrial flora and fauna (Curran, 1980;

17-22), areas such as Three Mile Swamp could have provided locations for hunting camps occupied as satellites of possible riverine (or earlier lakeshore) base camps or as units comprising part of a seasonal settlement round.

Conversations with Ms. Curran, who conducted the only major archaeological survey of the Ashuelot basin, and with Mr. Arthur Whipple, Sr., a noted collector in the area, revealed no knowledge of recorded sites within impact areas of the project. The nearest finds reported by Mr. Whipple were located at Woodland Cemetery, about 22 kilometers north of Water Street. Examination of the soils map and comparison with surface conditions during an initial walkover of the impact area at Three Mile Swamp confirmed the wetland character of the Rumney and Scarboro soil areas, and also revealed that the area of stony Acton soils extends further south than anticipated. Both the Acton and Ondawa soils are wet and forested with numerous tree throws. Due to these conditions, presence of prehistoric sites appears unlikely between the hills and wetlands. Similarly, steepness and stoniness removed the lower slopes of the hills from consideration for subsurface testing. The area of the proposed dike along Route 10 is disturbed ground and fill deposited during relocation of the highway in the 1970s.

This analysis leaves only one type of landform at Three Mile Swamp having high potential for presence of prehistoric sites, the isolated knolls of Hinkley loamy sand. It is these areas which were subjected to subsurface testing. Two knolls were tested in this manner.

Subsurface testing employed excavation of 50 cm. sq. shovel test pits at 10 m. intervals along a series of transects. On the larger of these two knolls, three transect, were excavated approximately 15 m. apart. Test pits on these were staggered to minimize distance between them. Where possible cultural material was located, additional test pits were excavated at 4 m. intervals nearby. Length of transects was limited by wetland margin and areas of extensive logging slash piles. Location of individual test pits was adjusted to avoid trees and slash piles. All soil was screened through 1/4 in. hardware cloth.

Due to the intensive disturbance of the project area between Marlboro and Water Streets no subsurface testing was undertaken in that area.

Historic Background

Examination at 19th century maps revealed evidence of industrial activity in the area between Marlboro and Water Streets (Figure 2). In 1877, the Taft & Co. pottery stood on the intersection of Foster and Myrtle Street, west of the project area (Rochwood, 1877). This firm may have operated as late as 1892 (Hurd), under tenants. At that time, the Wilkins toy factory stood south of the railroad on the opposite bank. Portions of this mill are still occupied, and the present owners have expanded west to the brook. These later additions have already been noted in the section on Environmental Background (See Figure 11). The area northeast of the railroad and south of Water Street was occupied in 1892 by the New Hampshire Moulded Granite Co. (Hunt, 1892). This company may have built the retaining wall in this area (Figure 13) but none of its buildings bordered the brook.

As noted above the only structures indicated on 19th century maps which bordered Beaver Brook are the abutments for the small railroad bridge and retaining wall north of the east abutment. These are typical cut stone structures of little historic interest or archaeological potential.

Due to this limited historic period land use and evidence of considerable modern disturbance no subsurface testing for historic period sites was performed in the Marlboro to Water Streets portion of this project area.

Examination of 1858, 1877, and 1892 maps failed to reveal any indications of the earth fill and dry laid fieldstone faced dam at Three Mile Swamp (Figure 14). (Fagan, 1858; Rockwood, 1877; Hunt, 1892). Field work in this area concentrated on surface examination in hopes of better dating this feature and determining its purpose.

RESULTS OF THE FIELD SURVEY

Prehistoric Survey at Three Mile Swamp

A total of 19 test pits were excavated along 10 m. interval transects, 17 on three transects occupying a large knoll a short distance north of the dam, and the remaining two on a smaller knoll to the northeast. Most of the latter location was covered with dense timber slash, as were smaller areas on the larger knoll. Lower and smaller knolls to the northeast were even more disturbed by logging activity or construction and maintenance of a high tension line, and were not tested.

Horizonation generally consisted of approximately 15-20 cm of dark brown sandy loam over 10-15 cm of orange brown sandy loam, above a horizon of tan sand. Pebbles, cobbles and number roots were preserved throughout all profiles with occasional larger rocks of up to about 20 cm. dia. Most stone was a very friable mica-schist, with smaller pebbles of quartz and quartzite. Pits were excavated until gravel and large rocks precluded deeper testing, generally 40-50 cm below surface. Some variations in profiles occurred due to lenses of weathered schist which were frequently encountered (purple-orange in color) decayed roots (light gray in color), and occasional thin lenses of fairly recent charcoal, probably resulting from burned and buried timber slash (Figures 16-21).

Small amounts of possibly modified quartz and quartzite were encountered in 14 test pits. Analysis by the cultural resource staff indicates that they could either be result of natural splitting or of tool manufacture (Figure 22). Therefore, test pits at 4 m. intervals were excavated along the cardinal directions from 3 of the test pits. Test pit T-3:TP-0 had no south or east satellite pits, due to proximity of the badly eroded surface of a logging road in these directions and T-4:TP-0 had dense timber slash all round. None of the satellite pits produced any potential cultural material or features. Therefore, the materials found in T-1:TP-40, T-2:TP-22, and T-3:TP-0 appear to be either naturally modified or to represent isolated activity associated with prehistoric hunting stands or similar.

Historic Survey at Three Mile Swamp

As study of historic period maps and surface examination of the project area revealed no other historic period features, field survey consisted of examination of the present dam at the south end of the swamp (Figure 14). The dam is of earth fill with dry laid field stone on both faces. Though typical of 18th or early 19th century dam construction, this method was employed into the latter part of the 19th century for small dams. Traces of a concrete sluice frame indicates use in the late 19th or early 20th century. Careful examination revealed no evidence of any mill foundation associated with this dam. If any mill was associated with this dam, it was either built entirely on pilings and/or obliterated by flooding. In view of the relatively intact nature of the dam another hypothesis seems more likely; that the dam was constructed to provide additional storage for mills downtown. As deforestation of uplands during the 19th century made watercourses more seasonally variable, many large firms or groups of firms built upland reservoirs to provide a reliable supply during seasons of low water. The apparent late 19th-early 20th century features of this dam somewhat support this hypothesis.

ANALYSIS OF IMPACTS

The only indications of possible prehistoric activity within the project impact area consist of a small amount of lithic material found in three non-adjacent test pits. Further testing within 4 m. of each of these find spots revealed no additional material. Therefore, it appears that this material is either the result of natural shattering of quartz and quartzite cobbles or representative of one or more isolated loci where artifact manufacture or retouching took place. Hunting stands would have been reasonable places for such activity areas. While such sites would theoretically be of interest; absence of any larger horizontal distribution, features, or diagnostic material severely limits any further archaeological potential of these loci. Therefore, it appears unlikely that any occasional flooding of the area behind the new dam would affect archaeological resources eligible for inclusion in the National Register of Historic Places.

The only historic period features potentially affected by project construction are the dam at Three Mile Swamp and railroad bridge abutments and retaining wall between Myrtle Street and Water Street. The dam does not have any associated mill foundation nearby, and may have been built to provide seasonal storage for factories downriver. It is of unexceptional design and has little archaeological potential. The bridge abutments and retaining wall also have little archaeological and potential or historic interest being typical examples of the period.

In conclusion no effect upon significant historic or archaeological resources is anticipated from construction or operation of the proposed Beaver Brook Local Protection Project.

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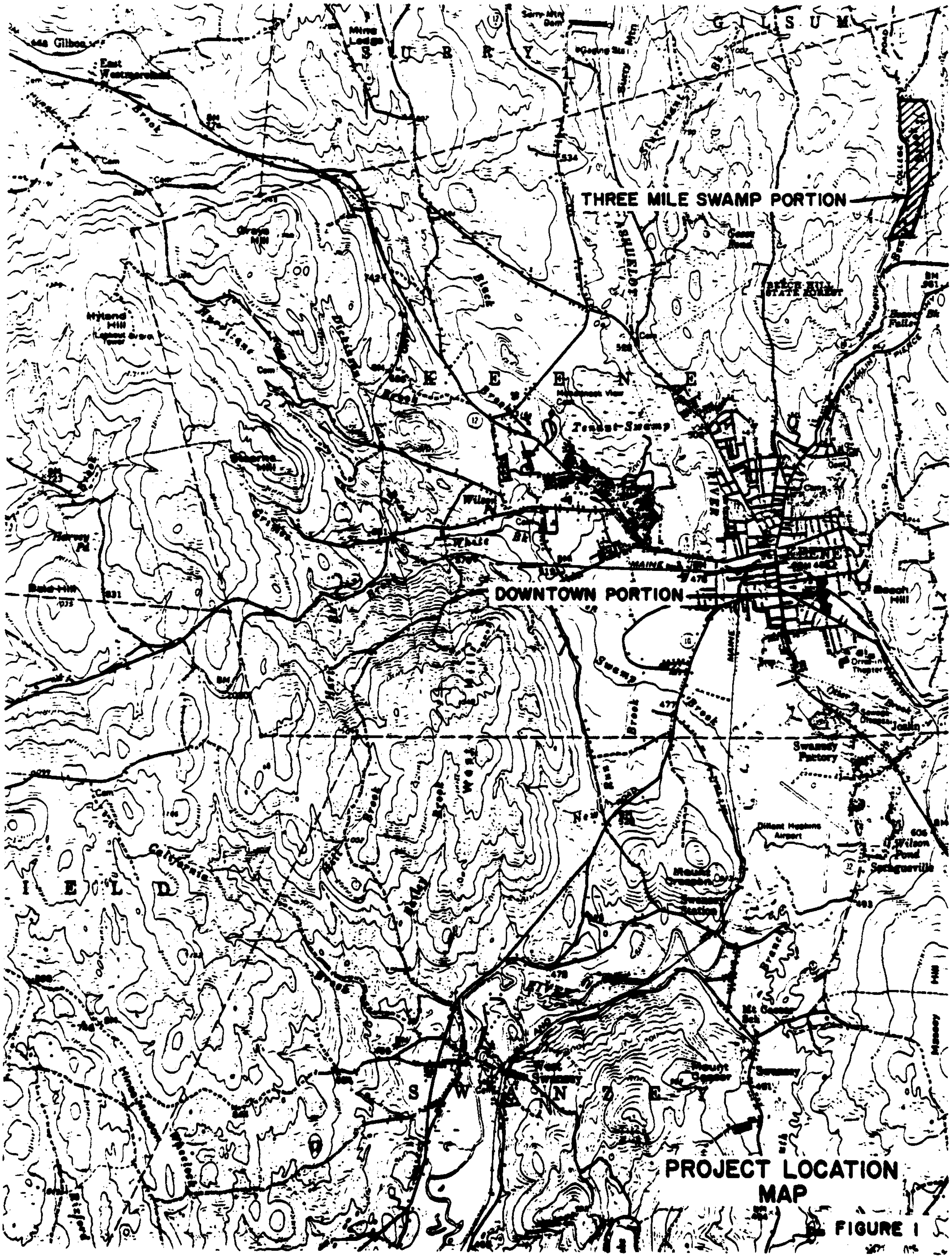
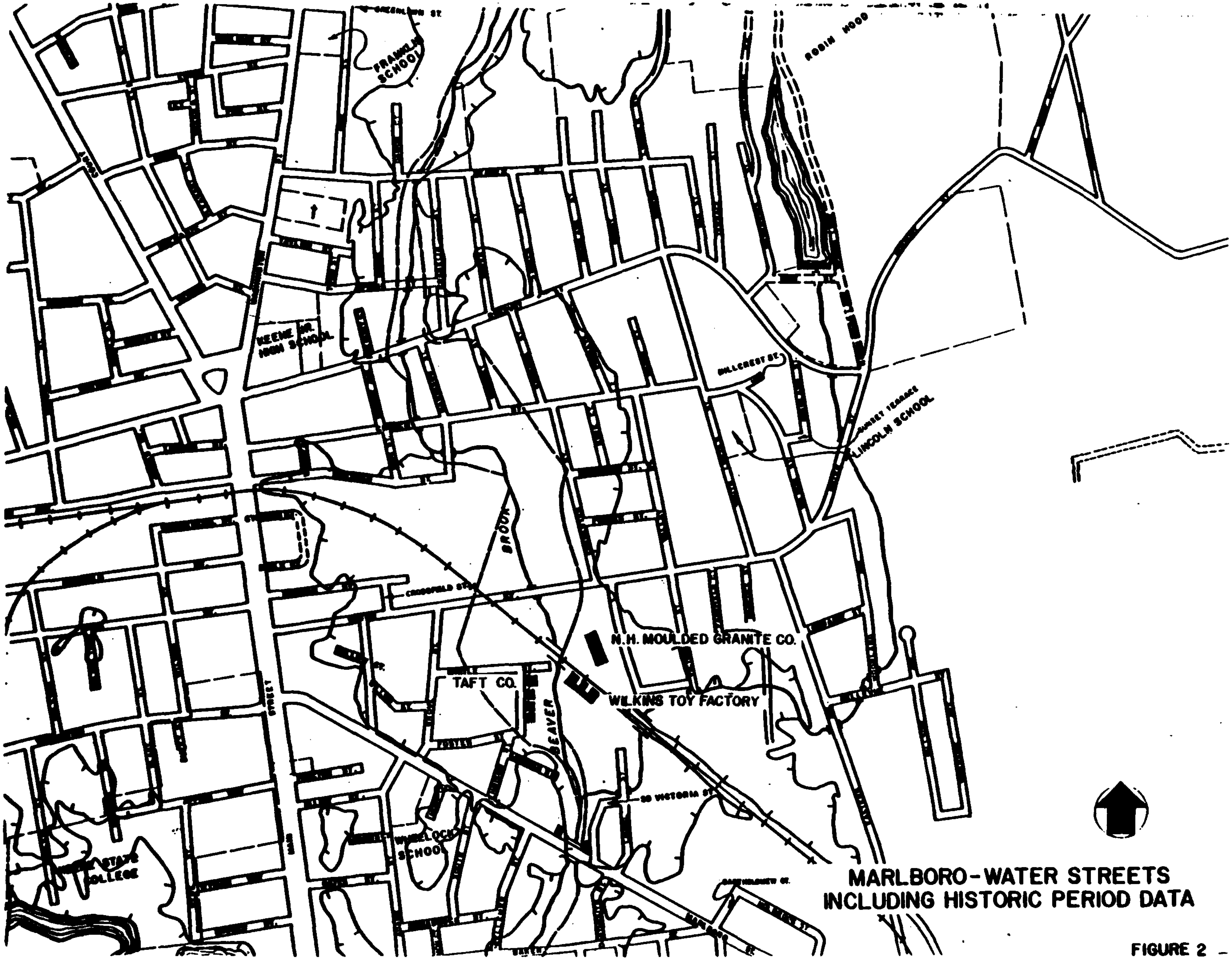


PROJECT LOCATION MAP

FIGURE 1



MARLBORO-WATER STREETS
INCLUDING HISTORIC PERIOD DATA

FIGURE 2 -

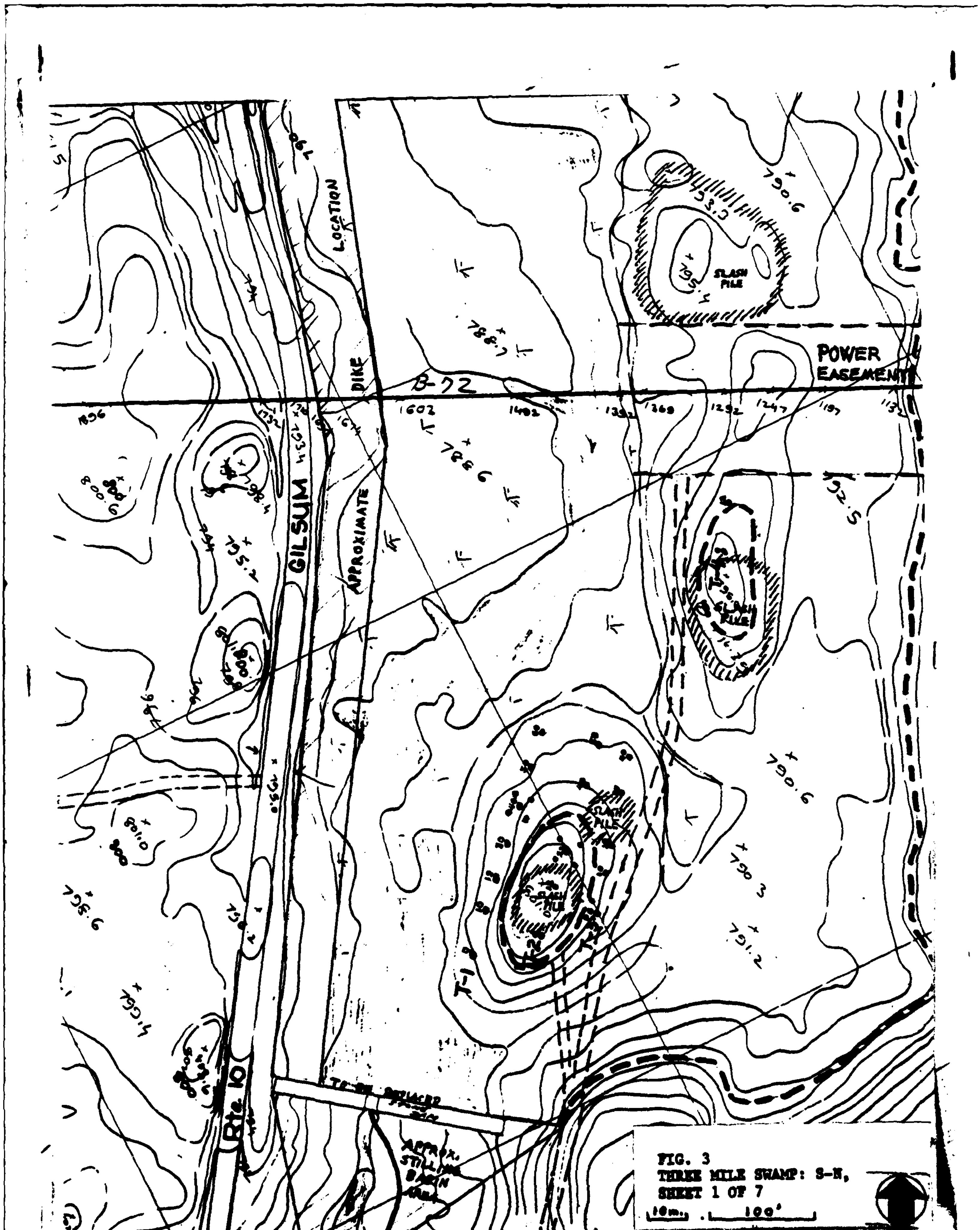


FIG. 3
THREE MILE SWAMP: S-N,
SHEET 1 OF 7
10m. 100'

FIG. 4
THREE MILE SWAMP: S-N,
SHEET 2 OF 7
100'

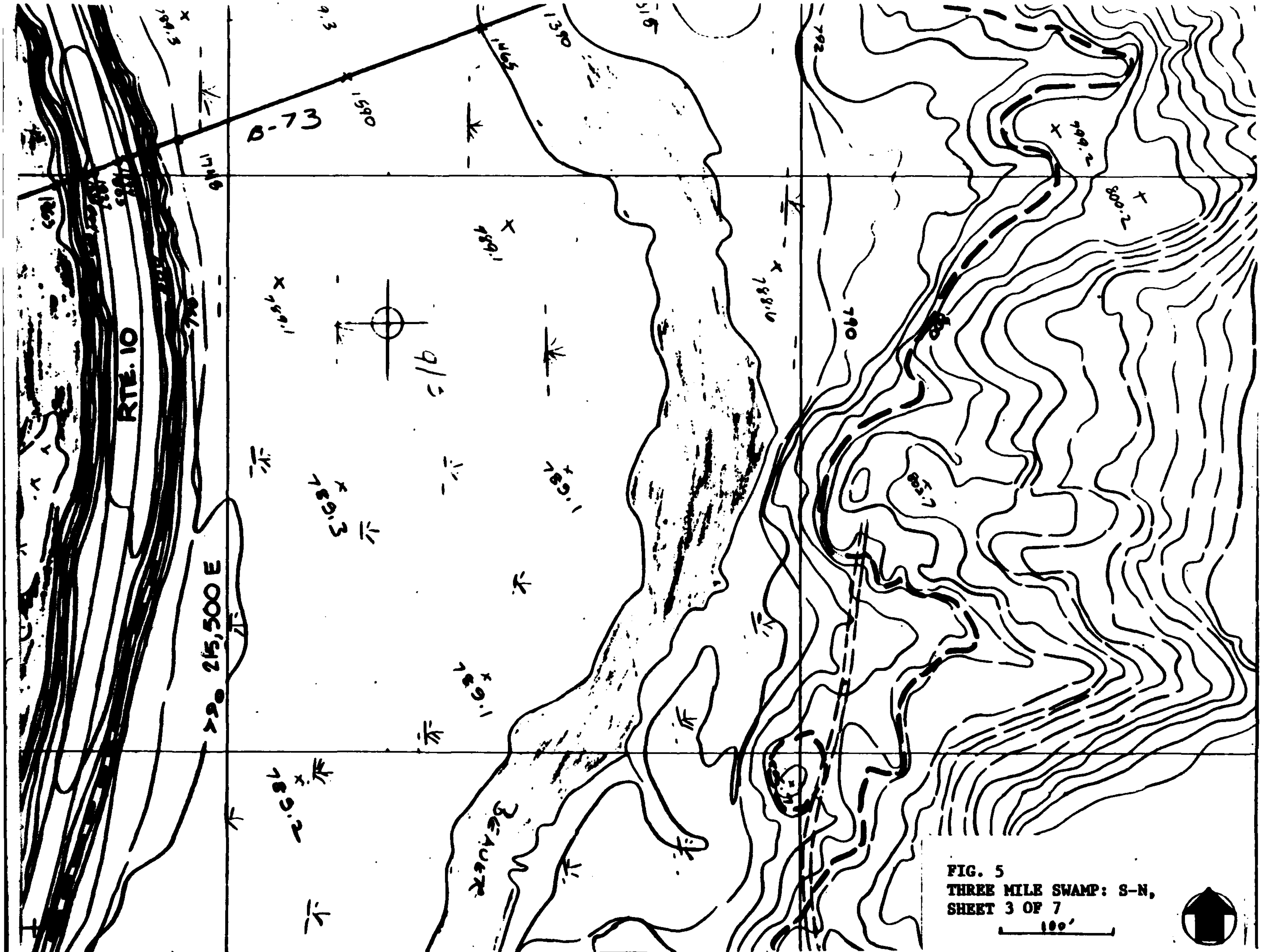


FIG. 5
THREE MILE SWAMP: S-N,
SHEET 3 OF 7

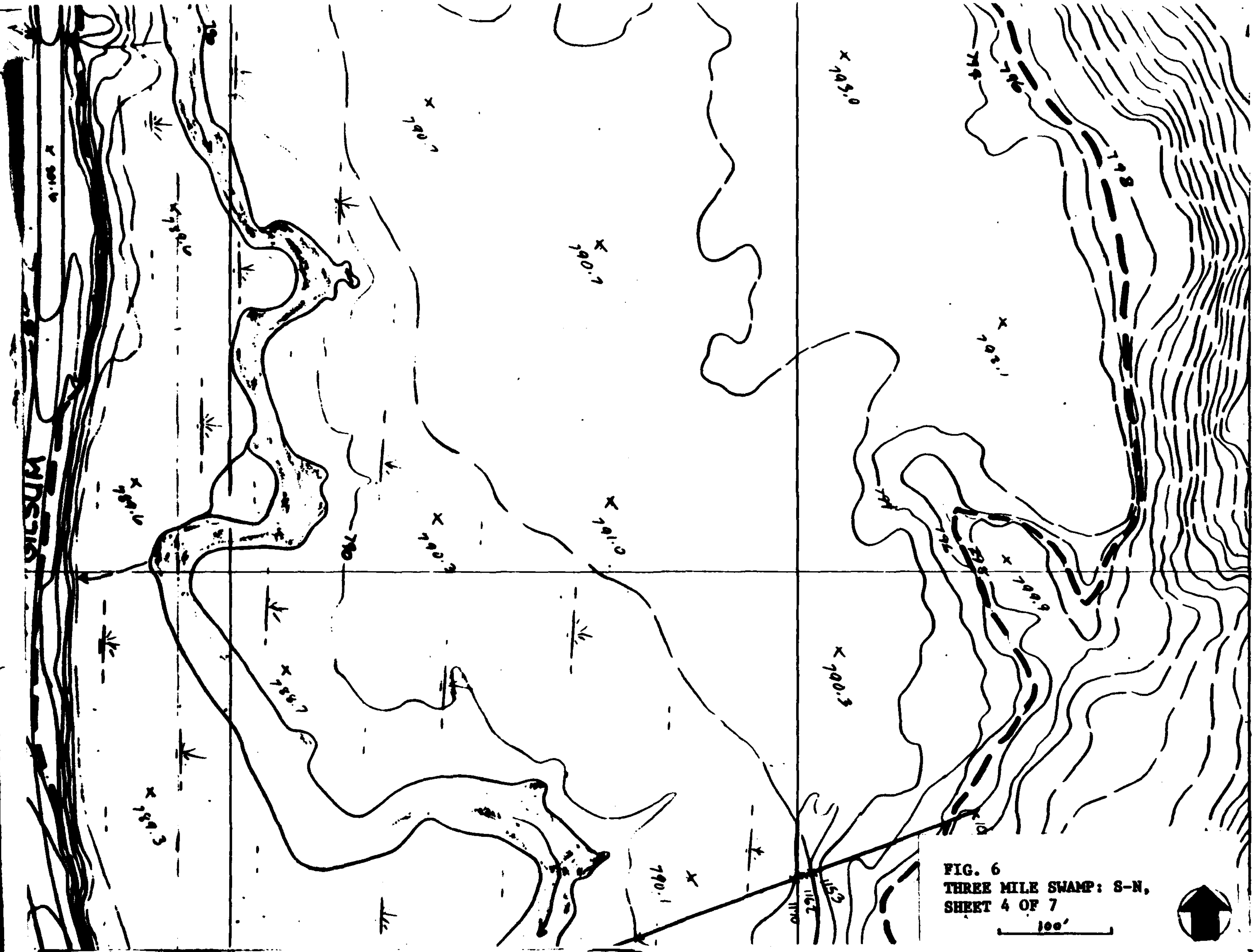


FIG. 6
THREE MILE SWAMP: S-N,
SHEET 4 OF 7

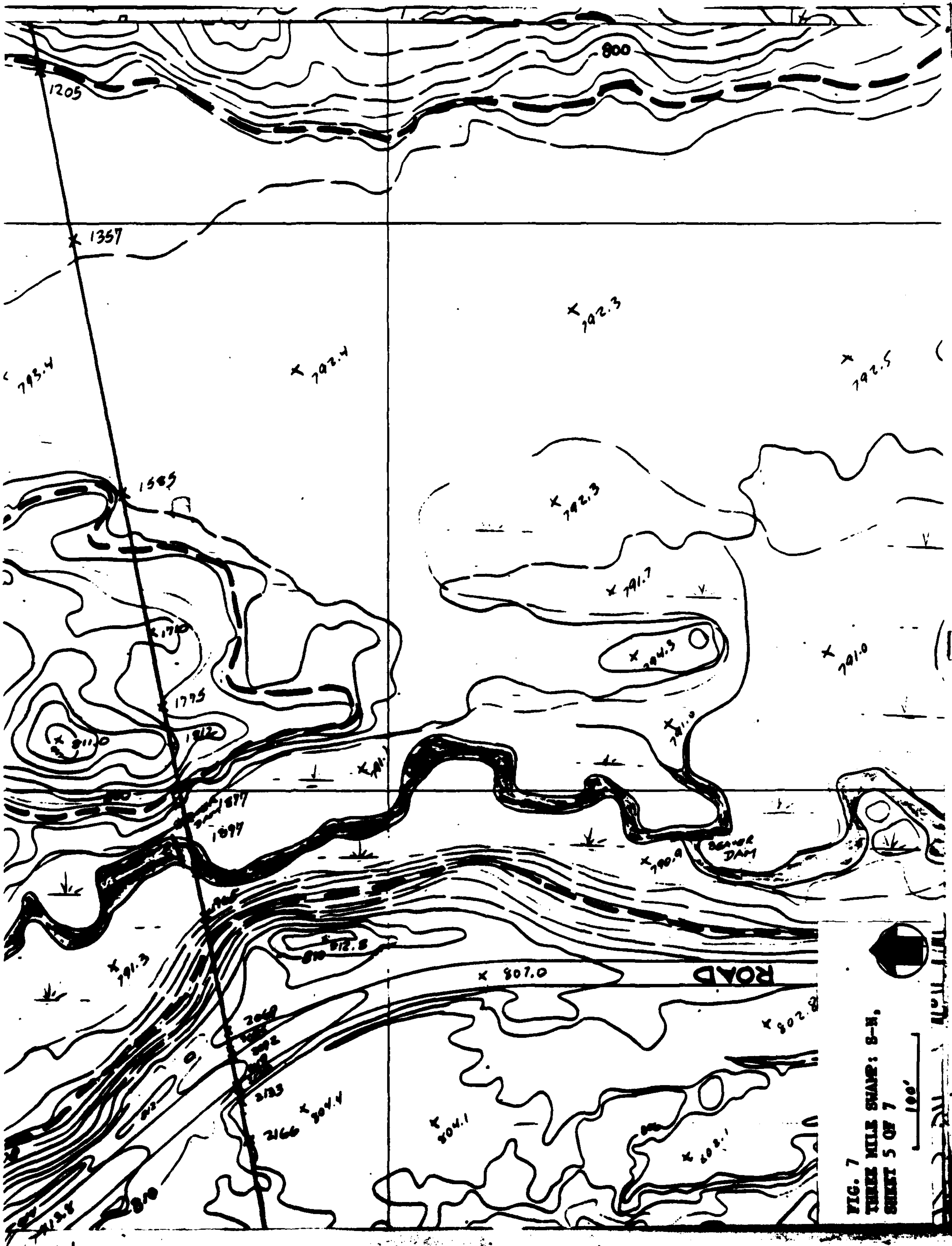


FIG. 7
THREE MILE SWAMP: S-N,
SHEET 5 OF 7

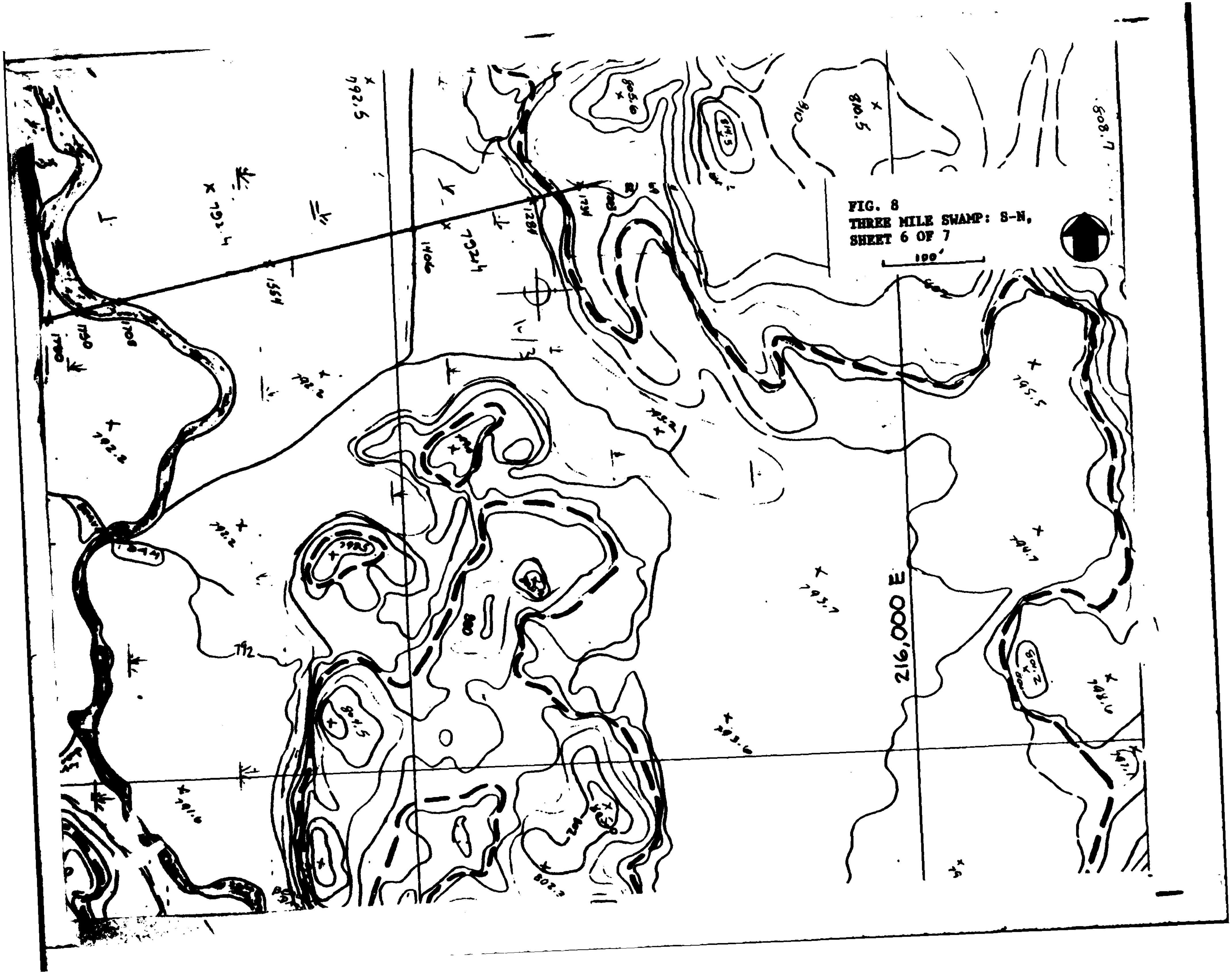


FIG. 8
THREE MILE SWAMP: S-N,
SHEET 6 OF 7

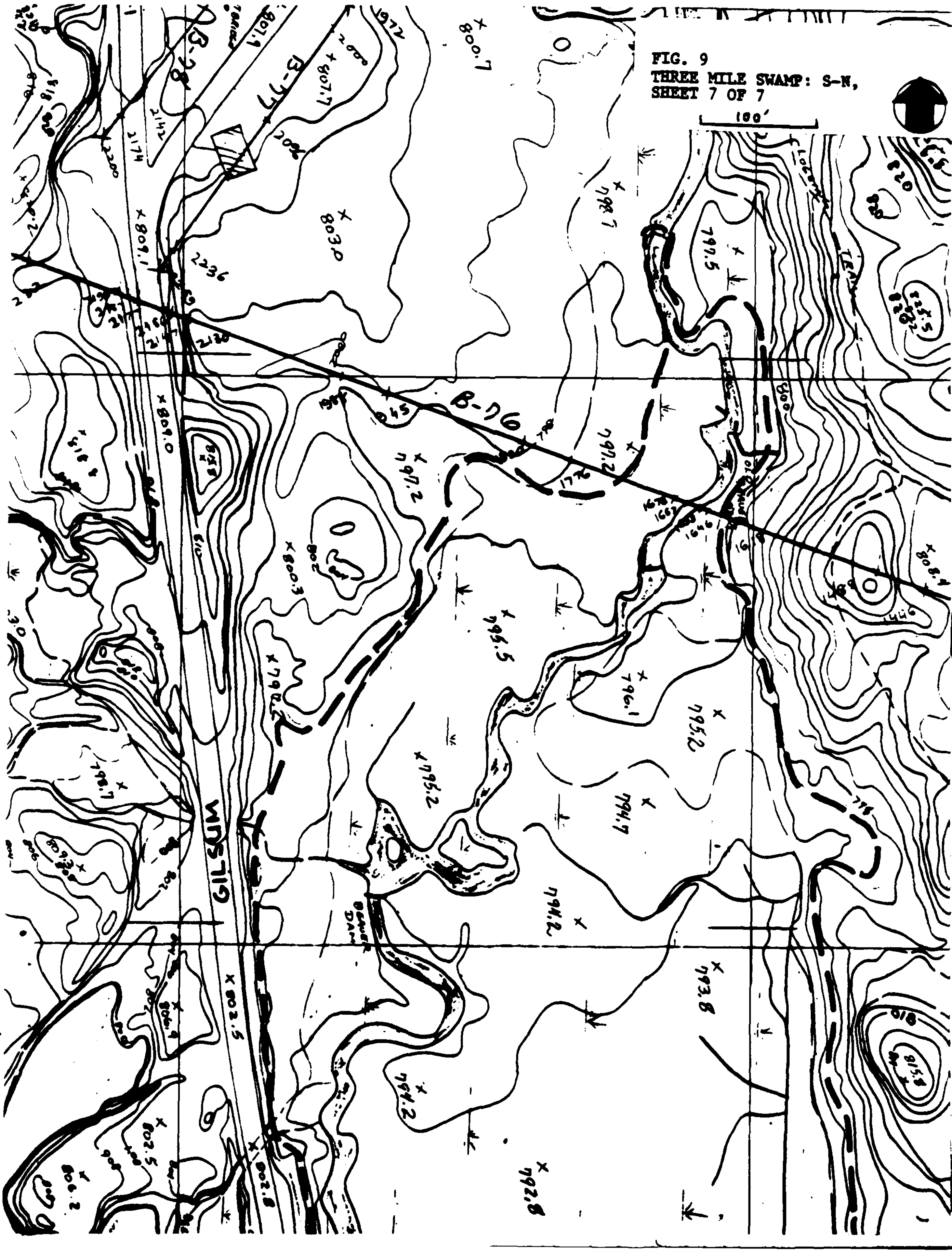
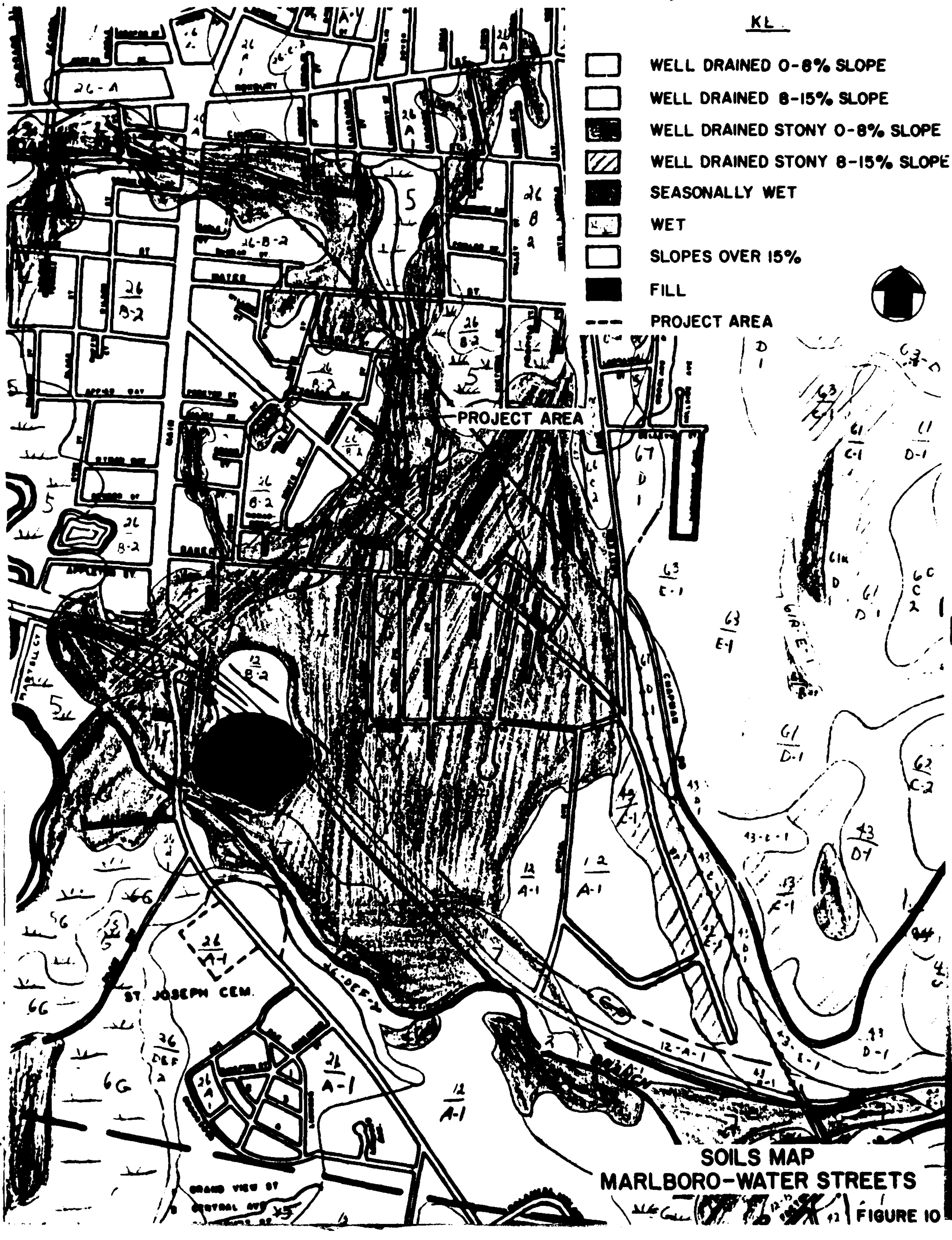


FIG. 9
THREE MILE SWAMP: S-N,
SHEET 7 OF 7


KEY
WELL DRAINED 0-8% SLOPE
WELL DRAINED 8-15% SLOPE
WELL DRAINED STONY 0-8% SLOPE
WELL DRAINED STONY 8-15% SLOPE
SEASONALLY WET
WET
SLOPES OVER 15%
FILL
PROJECT AREA
PROJECT AREA
ST JOSEPH CEM.
SOILS MAP
MARLBORO-WATER STREETS
FIGURE 10

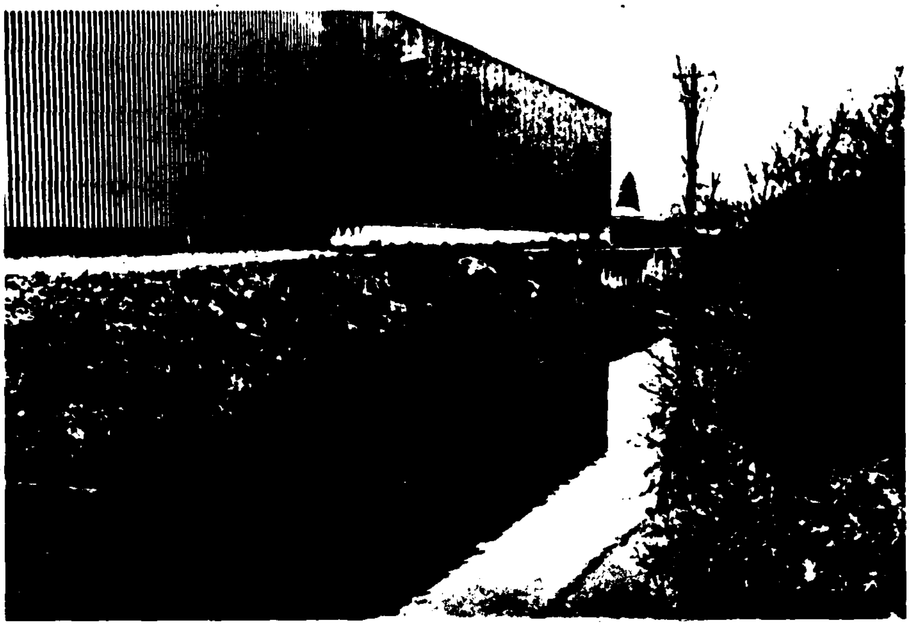
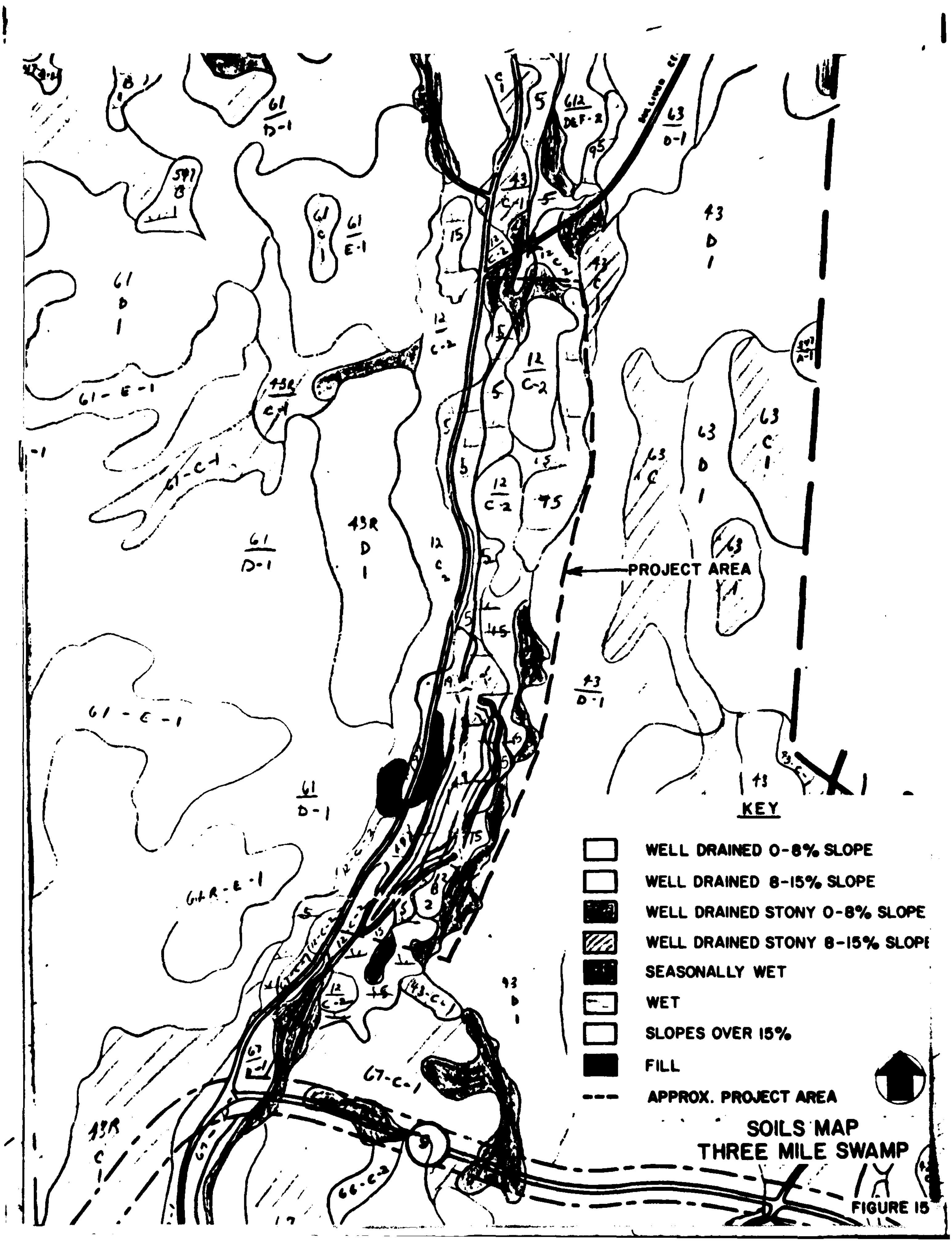


FIG. 11
VIEW S.W. FROM END OF GARDNER ST

FIG. 12
VIEW N. FROM MYRTLE STREET

FIG. 13
VIEW N. FROM RAILROAD BRIDGE

FIG. 14
DAM AT 3 MILE SWAMP: view to NW



SOILS MAP
THREE MILE SWAMP

FIGURE 15

TRANSECT RECORD

SITE/SURVEY UNIT Three Mile Swamp, Keene, NH

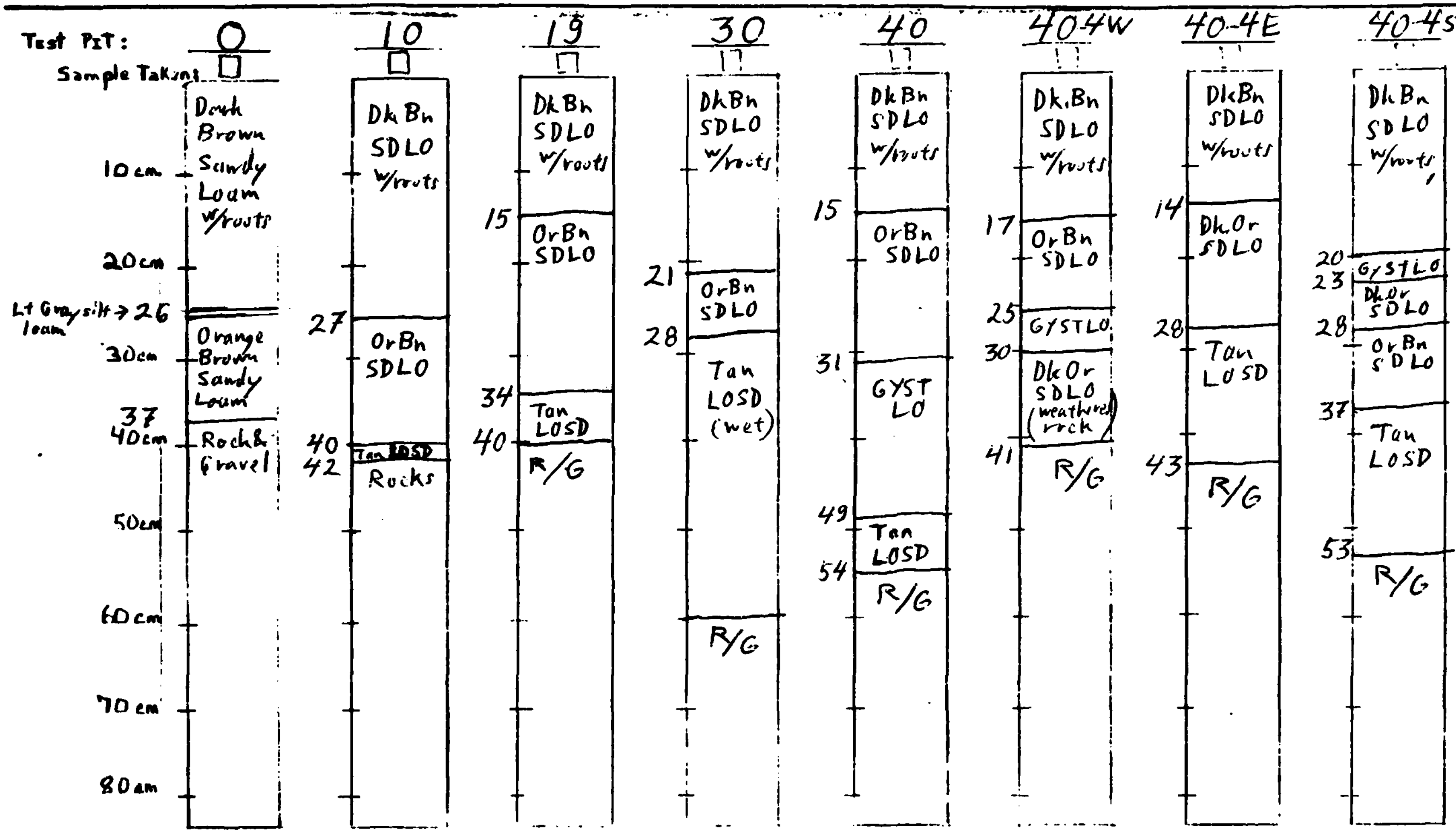
DATE 6/8/82

TRANSECT T-1

RECORDER J.S.W.

INTERVAL: PACES ____ METERS 20 FEET ____

FIG. 16



TRANSECT RECORD

SITE/SURVEY UNIT Three Mile Swamp, Keene, NH

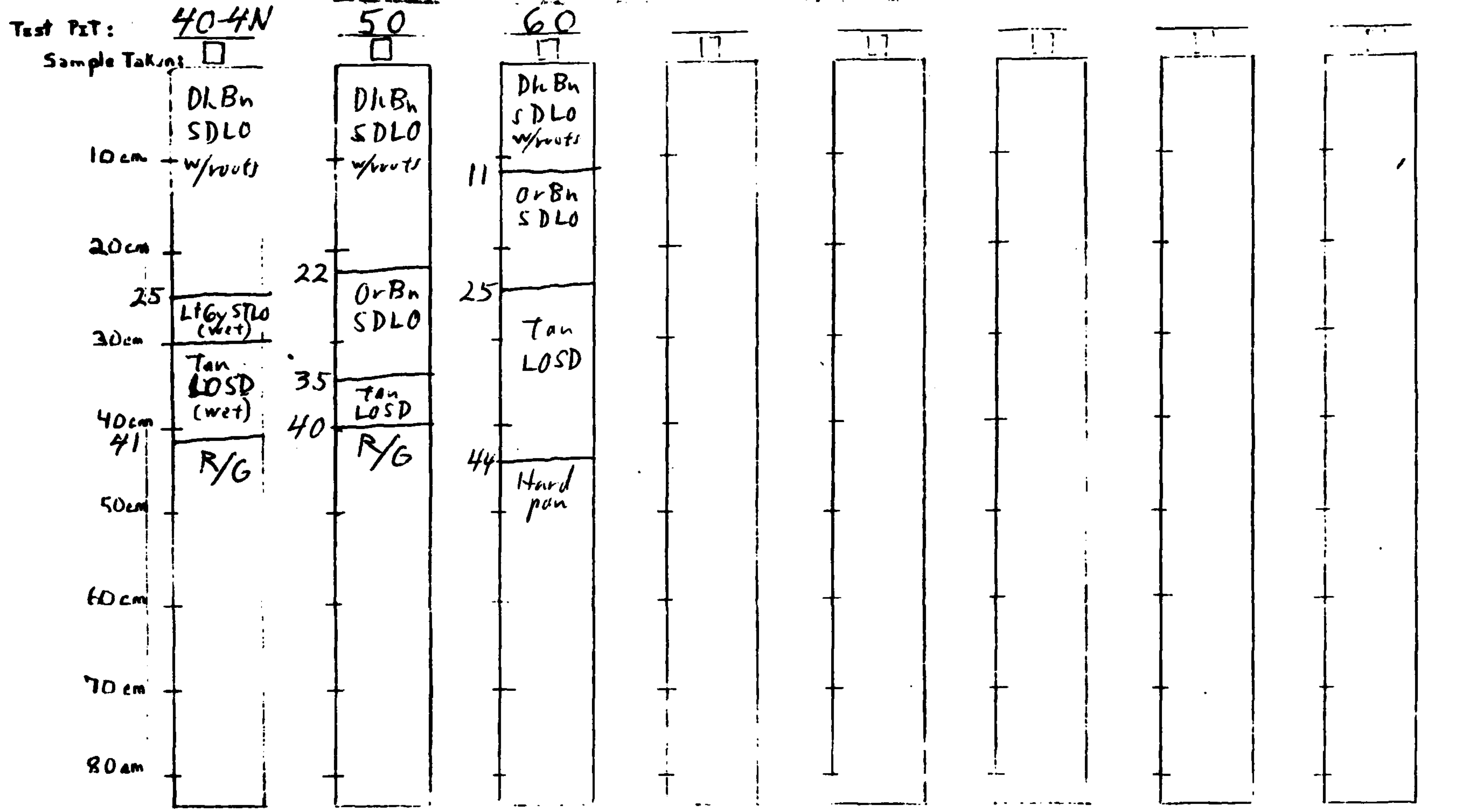
DATE 6/8/82

TRANSECT T-1 (continued)

RECORDER J. S. W.

INTERVAL: PACES ____ METERS 20 FEET ____

FIG. 17



TRANSECT RECORD

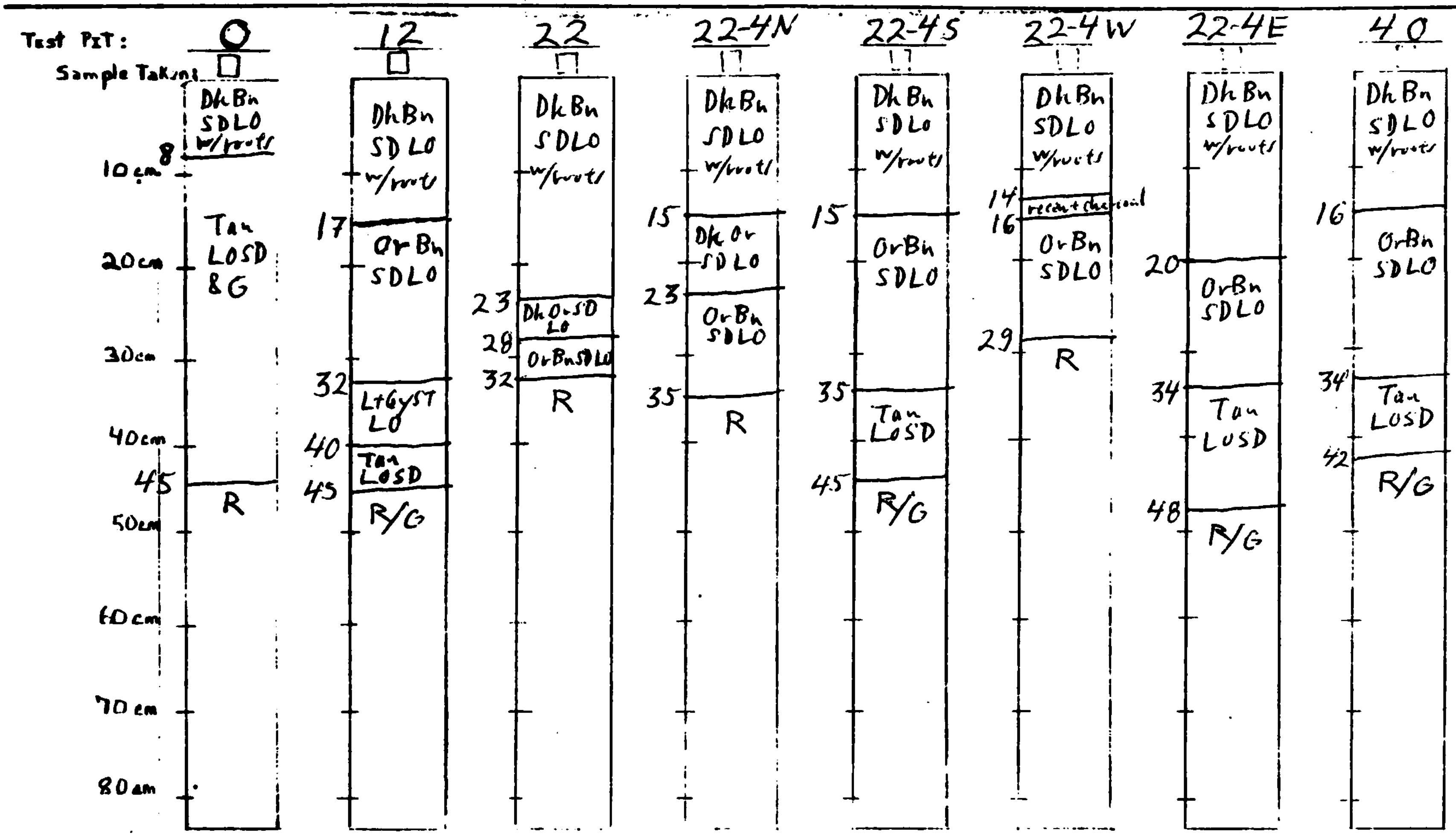
SITE/SURVEY UNIT Three Mile Swamp, Keene NH DATE 6/8/82

TRANSECT T-2 RECORDER J.S.W.

INTERVAL: PACES ____ METERS 20 FEET ____

FIG 18

(Note: M30 on steep slope)



TRANSECT RECORD

SITE/SURVEY UNIT Three Mile Swamp, Keene, NH

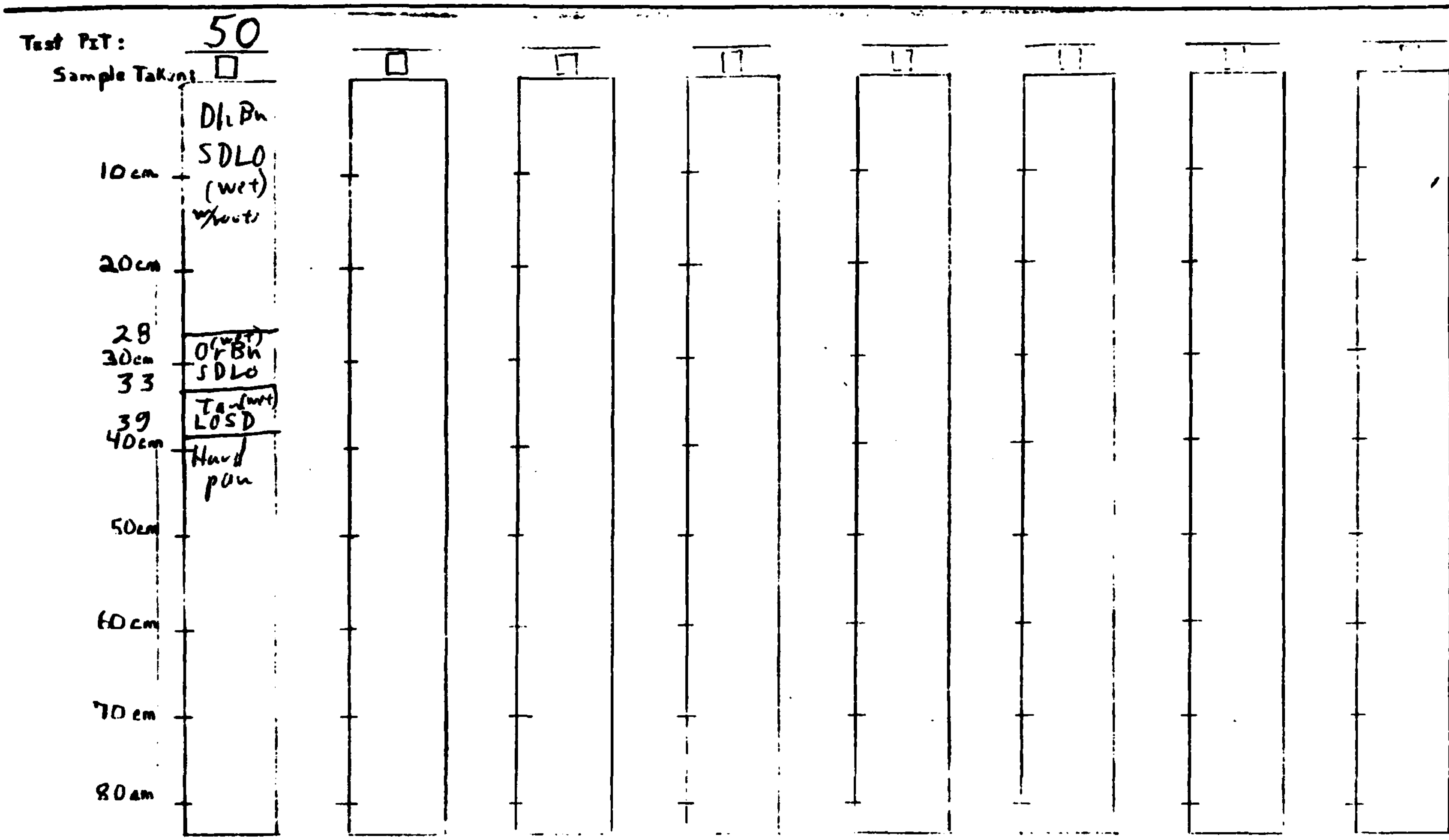
DATE 6/9/82

TRANSECT T-2 (continued)

RECORDER J. S. W.

INTERVAL: PACES ___ METERS 20 FEET ___

FIG 19



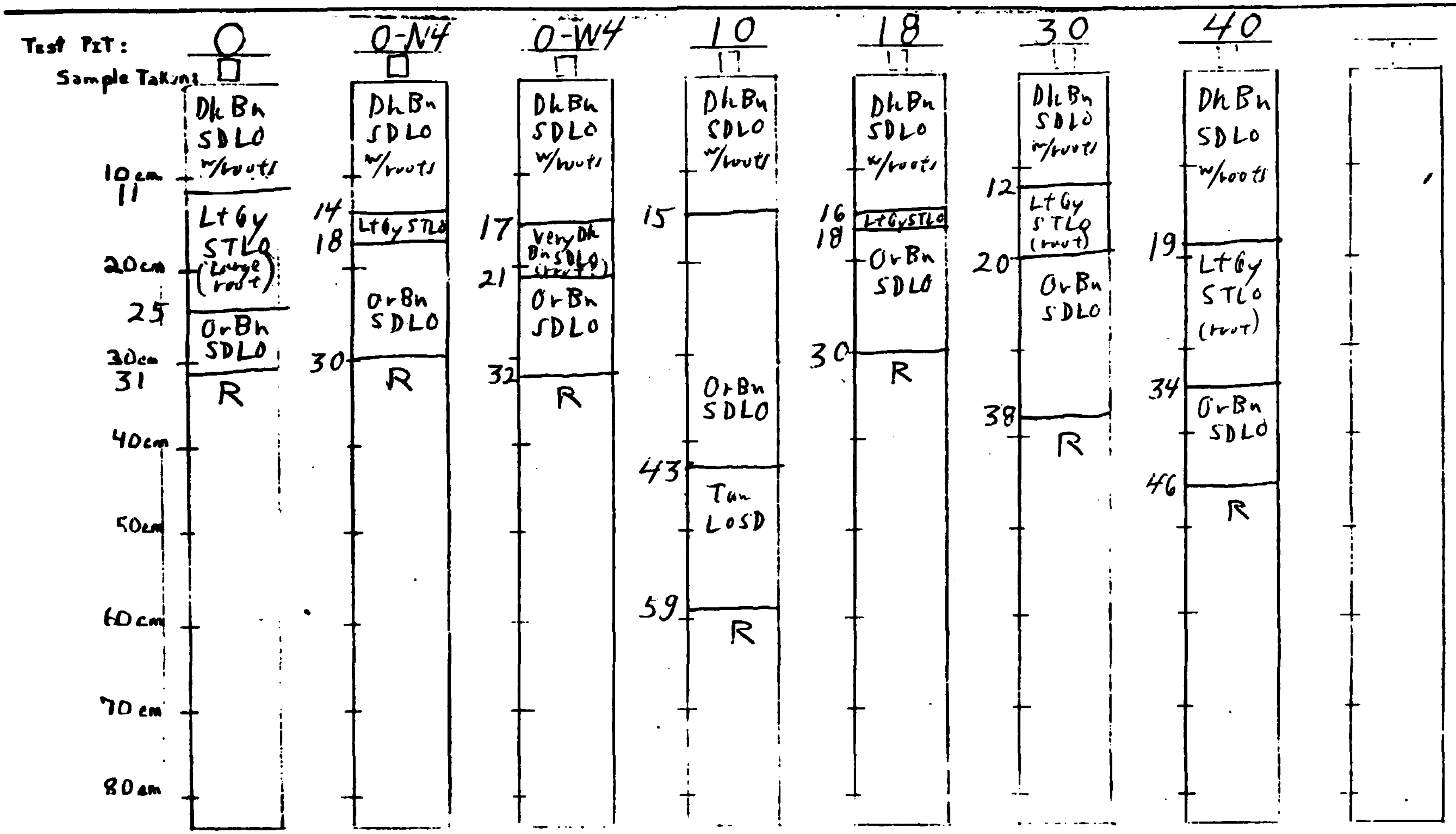
TRANSECT RECORD

SITE/SURVEY UNIT Three Mile Swamp, Keene, NH **DATE** 6/15/82

TRANSECT T- 3 **RECORDER** J. S. W.

INTERVAL: PACES ___ METERS 20 FEET ___

FIG. 20



TRANSECT RECORD

SITE/SURVEY UNIT Three Mile Swamp, Keene, NH

TRANSECT T-4

INTERVAL: PACES ____ METERS 20 FEET ____

DATE 6/15/82

RECORDER J.S.W.

FIG. 21

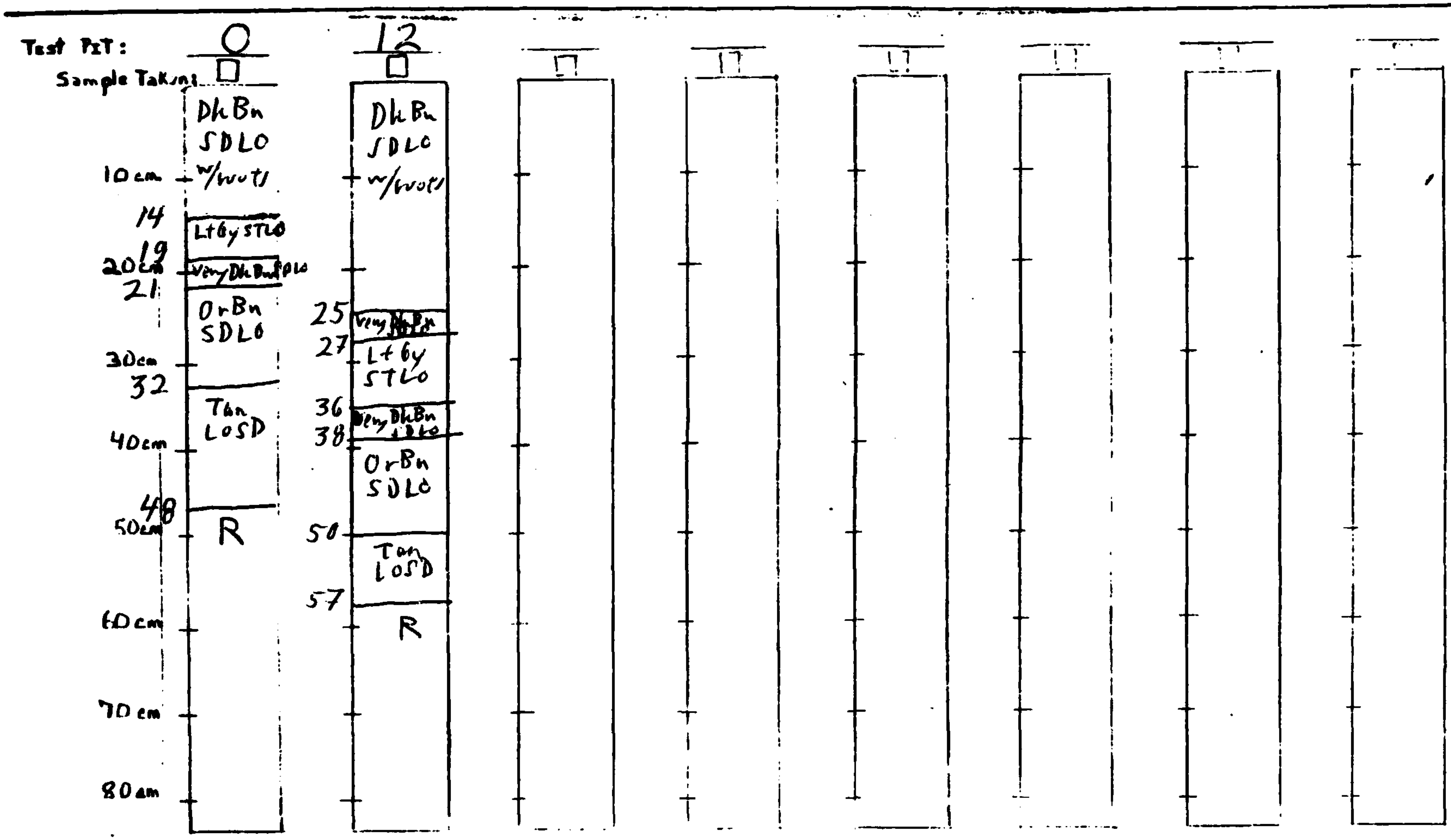


FIGURE 22

LIST OF MATERIALS RECOVERED

Test Pit	Material	Description
T-1:TP-40	Clear Quartz	Possible rectangular biface no distal portion, base incurvate-thinning flake scars, thumbnail size.
	Clear Quartz	Possible biface frag.
T-2:TP-22	Clear Quartz	Possible rectangular biface, no distal portion, base incurvate-thinning flake scars, convex cross-section, thumbnail size.
T-3:TP-0	Coarse Grained Quartzite	Cortex on dursal surface, possible flaking along edges, edges crushed, ventral surface-large broad flake scars, possible natural shattering.
T-4:TP-0	Smoky Quartz	Small triangular flake, no striking platform or bulb of percussion. Dorsal surface has central flake scars. Distal tip has step fracture, base shaped/ rounded on ventral surface, sharp on dorsal surface.

SECTION E

REAL ESTATE

DEPARTMENT OF THE ARMY

NEW ENGLAND DIVISION, CORPS OF ENGINEERS

424 TRAPELO ROAD

WALTHAM, MASSACHUSETTS 02154

PRELIMINARY ESTIMATE OF REAL ESTATE COSTS

THREE-MILE SWAMP

AND

CHANNEL IMPROVEMENTS

BEAVER BROOK LOCAL FLOOD PROTECTION

KEENE, NEW HAMPSHIRE

MARCH 1983

PREPARED BY Edward J. Fallon
EDWARD J. FALLON
Appraiser

APPROVED BY William D. Brown, Jr.
WILLIAM D. BROWN, JR.
Acting Chief, Appraisal Branch

PURPOSE

The purpose of this report is to estimate the preliminary real estate costs concerning the Beaver Brook Local Flood Protection Project in Keene, New Hampshire.

INSPECTION OF THE REAL ESTATE

The properties affected by the proposed project were viewed in the field during January and February 1983 by this writer.

LOCATION

The first part or upper portion of the proposed project is located in the northwesterly section of the city of Keene adjacent to Route 10 in an area known as Three-Mile Swamp.

The second part, Channel Improvement, or lower Beaver Brook basin lies along Beaver Brook between Water and Marlboro Streets. Both Water and Marlboro Streets commence in the downtown area of the city of Keene.

PROJECT DESCRIPTION   Three-Mile Swamp

The Three-Mile Swamp area consists of constructing a larger dam at the site of the existing rockwall dam which is located in the southerly section of Three-Mile Swamp. The construction would improve storage conditions during flood periods by creating a permanent pool of approximately 70 acres at elevation 790 feet NGVD during non-flood periods. During the standard project flood, the pool is expected to rise to elevation 797 feet NGVD thus, inundating an additional 50 acres of land.

The project would affect approximately 11 ownerships of which one is considered to be of public ownership, Route 10.

CHANNEL IMPROVEMENTS

The Channel Improvements would be constructed in the lower Beaver Brook basin between Water and Marlboro Streets. The improvements will consist of widening the channel in some areas and installing stone rip rap along the side slopes. The overall width of the channel would be increased by about 30 feet to an overall width of about 60 feet. The channel enlargement from its point of beginning at the southerly side of the bridge on Water Street would affect 8 private and 1 public ownership (B&M Railroad) and would extend downstream a distance of about 1700 feet following the brook's course and terminating on the northerly side of Marlboro Street. At this time no known improvements will be affected.

TAX LOSS

The anticipated tax loss for the Beaver Brook study based upon the 1982 Tax Assessment of $33.60 per thousand is estimated to be about $1000.00.

ACQUISITION COSTS

Acquisition costs will include costs for mapping, surveying, legal descriptions, title evidence, appraisals, negotiations, closing and administrative costs for possible condemnations. The acquisition costs are based upon this office's experience in similar civil works projects in the general area and are estimated at $3,000 per ownership. About 20 total ownerships will be affected of which 11 are attributable to the three-mile swamp area while the remainder is for the channel improvement area.

RELOCATION ASSISTANCE COSTS

Public Law 91-646, Uniform Relocations Assistance Act of 1979, provided for uniform and equitable treatment of persons displaced from their homes, businesses, or farms by a Federally Assisted Program. It also establishes uniform and equitable land acquisition policies for these projects. Included among the items under PL 91-646 are the following:

a. Moving Expenses

b. Replacement Housing (Homeowners)

c. Replacement Housing (Tenants)

d. Relocation Advisory Services

e. Recording Fees

f. Transfer Taxes

g. Mortgage Prepayment Costs

h. Real Estate Tax Refunds (Pro-rata)

Within a reasonable time prior to displacement, the taking authority must certify that there will be available, in areas generally not less desirable and at rents and prices within the financial means of the families and individuals displaced, decent, safe, and sanitary dwellings, equal in number to the number of, and available to, such displaced persons who require such dwellings and reasonably accessible to their places of employment.

There are no known ownerships in the area to be acquired in fee where the people would have to be relocated. Therefore, the following estimates are included for planning purposes to cover the implementation of this act.

Three Mile Swamp

10 Private Ownerships	@	$200	=	$2,000	
1 Public Ownership	@	$200	=	200	
					$2,200

Channel Improvement

8 Private Ownerships	@	$200	=	$1,600	
1 Public Ownership	@	$200	=	200	$1,800
		Total			$4,000

SEVERANCE DAMAGES

Severance damages usually occur when partial takings are acquired which restrict the remaining portion from full economic development. The severance damages are measured and estimated on the basis of "before" and "after" appraisal methods and will reflect actual value loss incurred to the remainder as a result of partial acquisition. Detail appraisals will reflect these losses. Preliminary investigation indicate that no severance damages is anticipated by the proposed project. However, for planning purposes costs for severance damages if any are provided for in the contingency factor.

PROTECTION AND ENHANCEMENT OF CULTURAL ENVIRONMENT

In accordance with insturctions set forth in teletype DA (DAEN) R 191306A, dated October 1971, Subject: "E011593, 13 May 1971, Protection and Enhancement of Cultural Environment"; a study has been made in the subject areas. The study revealed that no local, State, Federally owned nor Federally-controlled property of historical significance would fall within the provisions of EO 11593.

CONTINGENCIES

A contingency allowance of 20 percent is considered to be reasonable adequate to provide for possible appreciation of property values from the time of this estimate to acquisition date, for possible minor property line adjustments or for additional hidden ownerships which may be developed by refinement to taking lines, for adverse condemnation awards and to allow for practical and realistic negotiations.

GOVERNMENT-OWNED FACILITIES

Section III of the Act of Congress approved 8 July 1958, (PL 85-500) authorized the protection, realteration, reconstruction, relocation or replacement of municipally-owned facilities. A preliminary inspection of the property area indicated no Government-owned facilities are affected.

FEE REQUIREMENTS

Three-Mile Swamp

Preliminary investigations indicate that no improvements would be affected in the Three-Mile Swamp area. The Three-Mile Swamp area is comprised of undeveloped lands with most of the 70 acres being of wetlands, scrub growth and vegetation commonly found in wetland areas. There are some trees along the perimeter areas but nothing that would have a commercial value.

Therefore, the fee areas that are necessary for acquisition are estimated as follows:

5 Private Ownerships
70 acres @ \$375.00 per acres = \$26,250

EASEMENT AREAS

Permanent Easement Areas

Permanent easements for construction and maintenance purposes is necessary and are as follows:

Three-Mile Swamp

A permanent right-of-way access over the existing right-of-way that lies just south of the dam is necessary for construction and maintenance purposes. The right-of-way would commence on the easterly sideline of Route 10 and extend approximately 300 feet in an easterly direction and then turn and extend approximately 150 feet in a northerly direction. This access route would be 25 feet wide and contain 11,250 square feet.

Additional permanent easements for flowage are necessary and would extend out from the permanent pool at elevation 790 feet NGVD to elevation 797 feet NGVD. The easement areas vary in width due to the topographical layout of the area, and contain about 50 acres and would affect approximately 11 ownerships of which one is considered to be public in that of Route 10.

Channel Improvements

Channel Improvements would necessitate the widening of portions of the existing channel, of Beaver Brook, between Water and Marlboro Streets, Approximately 30 feet to a top width of about 60 feet. The easement areas vary in width throughout the project area and contain about 0.39 acres of private lands.

The realignment of portions of the river would affect approximately 8 private and one public ownership(s) (Boston & Maine Railroad).

Preliminary investigations indicate that after the imposition of the permanent easement interest, the highest and best use of the remainders of the properties affected will not be materially affected. The cost to acquire the permanent easement areas would be equivalent to the underlying fee value since those uses would be for project purposes. However, lands would remain in their private ownerships to maintain conformity of their existing lot areas.

The following costs are predicated on estimated market values as indicated below:

Three-Mile Swamp

Right-of-Way		
0.26± acres Private Land		
@ $375.00 per acre		$ 97.00
Flowage Easement		
50± acres Private Land		$18,750.00
50.26± Total Acres	Total	$18,847.00
	Call	$18,900.00

Channel Improvements

13,000 square feet (0.30± acres) Private Land (Commerical Use)		
@ $1.11 per square foot		$14,430.00
4,000 square feet (0.09± acres) Private Land (Residential Use)		
@ $0.60 per square foot		$ 2,400.00
17,000 square feet (0.39± acres) Total	Total	$16,830.00
	Call	$16,800.00

TEMPORARY CONSTRUCTION EASEMENTS

The temporary construction easements which are necessary for construction and staging areas are as follows:

Three-Mile Swamp

A temporary construction easement in the form of a staging area comprised of a one acre parcel located adjacent to the dam.

Channel Improvements

Approximately one acre for a staging area is necessary in the Marlboro Street area. Additional temporary construction easements 25 feet wide, containing about 1.51± acres, along each side of the channel widening is required.

All of the 1.51± acres are under private ownership with approximately 1.08± acres being for commercial/industrial use and the remaining .43± acres for residential use.

The land areas to be encumbered by temporary easements are predicated upon a fair return of invested capital, for use of the owner's land for a one year term. Therefore, based on the indicated market values the following estimates are given:

Three-Mile Swamp

Staging Area

1.0± acres Private Land	
@ $375.00 per acre	$375.00
Fair Return	
@ 15% per year (for one year)	$ 56.00
($375.00 X .15 = $56.25) Call	$100.00

Channel Improvements

Staging Area	
1.0± acres Private Land	
@ $1.11 per square foot	$ 48,352.00
Channel Area	
1.08 acres Private Land	
@ $1.11 per square foot	51,338.00
0.43± acres Private Land	
@ 0.60 per square foot	11,250.00
1.51± Total Acres Total	$110,940.00
Fair Return	
@ 15% per year (for one year)	$ 16,641.00
Call	$ 16,700.00

$$\begin{array}{r} \$110,940.00 \\ .15\% \\ \hline \$\ 16,641.00 \end{array}$$

CONCLUSIONS AND SUMMARY OF REAL ESTATE COSTS

The areas of study for the Beaver Brook Local Flood Protection Study are based upon lines super imposed upon assessors and topographic maps.

The contents of this report are subject to refinement prior to acquisition and the proposed construction.

The value of the lands within the project areas have been estimated by use of the market data or Sales Comparison approach. A search was conducted in the general area to obtain market data. Local officials, Real Estate Brokers and Appraisers were interviewed to obtain data and value estimates.

SUMMARY OF REAL ESTATE COSTS

There follows and estimate of the real estate costs for the interests proposed for acquisition for a one year period.

Three-Mile Swamp

Lands		
Fee Acquisition	$26,250	
Permanent Easements	18,900	
Temporary Easements	100	
	$45,250	
Contingency - 20% of Above	9,050	
Sub Total		$54,300
Acquisition Costs		33,000
Relocation Assistance Costs		2,200
Total Estimated Real Estate Costs		$89,500
Call		$90,000

Channel Improvements

Lands		
Permanent Easements	$16,800	
Temporary Easements	16,700	
	$33,500	
Contingency - 20% of Above	6,700	
Sub Total		$40,200
Acquisition Costs		27,000
Relocation Assistance Costs		1,800
Total Estimated Real Estate Costs		$69,000

Three-Mile Swamp	$ 90,000
Channel Improvements	69,000
Total Estimated Project Real Estate Costs	$159,000

DATE
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8 8