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HYDROLOGIC STUDY OF THE ACUSHNET RIVER WATERSHED NEW BEDFORD, MASSACHUSETTS

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HYDROLOGIC STUDY OF THE ACUSHNET RIVER WATERSHED NEW BEDFORD, MASSACHUSETTS

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1.0 INTRODUCTION

The HEC-1 Flood Hydrograph Model (HEC-1) (Army Corps of Engineers, 1981) was used in this study by Balsam Environmental Consultants, Inc. (Balsam) to simulate the hydrologic processes occurring in the Acushnet River watershed shown in Figure 1. Synthetic hydrographs were generated for the 10-, 25- and 50-year storm events (recurrence interval in years, i.e. the 10-year storm event will occur, on the average, every 10 years) at the outlet to Saw Mill Dam located hydraulically upgradient of New Bedford Harbor. The objectives of this model study were as follows:

1) To provide a mechanism for evaluating the hydrologic conditions of the Acushnet River watershed, which is the primary contributor of runoff to the estuary and to evaluate whether additional data is needed for the accurate simulation of runoff entering the estuary.

2) To provide estimates of peak flows and maximum flow volumes for specific storm events to be used in contaminant transport models for the estuary and New Bedford Harbor.

3) To provide estimates of peak flows and volumes for specific storm events for the purpose of evaluating remediation programs.

4) To provide a baseline model to assist in evaluating the effects of watershed storage modifications on peak flows entering the harbor.

Existing hydrograph studies along with general information on watershed characteristics were used as input to the model. In order to assess model accuracy sensitivity analysis was conducted on model parameters whose values were either unknown or undefined. Model output consisted of outflow hydrographs at Saw Mill Dam (Figure 2) for the 10-, 25-, 50-year storm events.

2.0 EXISTING FLOOD HYDROGRAPH STUDIES

Existing analyses of peak flow hydrographs for the Acushnet River watershed were reviewed by Balsam to: 1) evaluate procedures used in each study; 2) help locate available data and information on the watershed; 3) assess the need for a more comprehensive watershed study; and 4) to aid in the calibration of the HEC-1 model. The intent of this literature review was not to compare study results to HEC-1 results since the procedures and data used in deriving the results in most instances vary. The specific studies conducted on the watershed consisted of: Preliminary Flood Hydrograph Assessment Investigation New Bedford Harbor (United States Army Corps of Engineers, 1987); and Flood Insurance Study Town of Acushnet, Massachusetts (Federal Emergency Management Agency, 1982). The following subsections discuss the procedures and information used and the results of each study.

2.1 PRELIMINARY FLOOD HYDROGRAPH ASSESSMENT INVESTIGATION (USACE, 1987)

A preliminary hydrologic study of the various in-harbor containment areas and their effect on flooding within the New Bedford Harbor area was conducted by the United States Army Corps of Engineers (USACE) to aid in the planning and design of remediation for the upper New Bedford Harbor. The streamflow portion of this study consisted of evaluating peak discharge frequencies based on gaged flows from a similar nearby watershed: the Wading River watershed near Norton, Massachusetts, located approximately 12 miles northwest of New Bedford, with a drainage area of 42.4 square miles. Peak discharge frequencies for the Acushnet River watershed were calculated by the USACE as proportional to the Wading River data by ratio of respective drainage areas to the 0.7 power. A Standard Project Flood (SPF) for the Acushnet River was computed by applying SPF rainfall to an adopted unit hydrograph; it should be noted that it was unclear from the report what rainfall magnitude was associated with the SPF. Using this methodology the peak flows at Saw Mill Dam for the 10-, 50-, and 100-year storm events and the SPF were estimated by the USACE to be 475, 740, 880, and 1350 cubic feet per second (cfs), respectively.

2.2 FLOOD INSURANCE STUDY (FEMA, 1982)

A flood insurance study was conducted by the Federal Emergency Management Agency (FEMA) for the town of Acushnet to evaluate potential flood hazards in the town and to assist planners in their efforts to implement sound flood plain management programs. Hydrologic analyses were carried out to establish peak discharge relationships for floods of selected recurrence intervals. The peak discharges were estimated by first determining an inflow hydrograph (time distribution of discharge) into the New Bedford Reservoir using a method derived by the Soil Conservation Service (1964) and routing the hydrograph through the reservoir. Secondly, regional discharge-frequency equations by Wandle (1977) were used to determine peak discharges from the remaining portion of the watershed. The final discharges from the Acushnet River were estimated by summing the discharges from the routing of the reservoir and the regional frequency-discharge equations. The peak discharges estimated at Saw Mill Dam for the 10-, 50-, 100-and 500-year storm events were 280, 475, 630 and 935 cfs, respectively.

3.0 WATERSHED CHARACTERISTICS

3.1 CLIMATOLOGY

The New Bedford area averages approximately 50 inches of rainfall per year. The mean annual temperature is approximately 50 degrees Fahrenheit. The temperature ranges from the lower 90's in July to the lower 30's in January.

There are two general types of storms that produce precipitation over the area: frontal and cyclonic. Frontal storms are more commonly called thunderstorms and are associated with cold fronts passing through the region in an easterly and southeasterly direction. They occur mostly in the summer months and usually produce locally intense rain of short duration. Cyclonic storms are associated with low pressure centers and generally move in an easterly and northeasterly direction over the region. These types of storms are not limited to any season or month but follow each other at irregular intervals with varying intensities throughout the year. These types of storms produce more uniform precipitation of longer duration. Tropical hurricanes are cyclonic storms and may yield flood-producing precipitation mostly during the late summer and early autumn months. A duration of 24-hours or more and an excess of four inches of precipitation is not uncommon for cyclonic storms in this geographical region. These types of storms usually produce the highest peak flows and volumes, and were used in the model to synthesize the outflow hydrographs.

3.2 DRAINAGE FEATURES

The watershed for the Acushnet River at Saw Mill Dam is delineated in Figure 2. The drainage area was estimated to be approximately 17.5 square miles. The upper portion of the watershed rises to over 150 feet above mean sea level. The outlet at Saw Mill Dam is approximately 20 feet above mean sea level, producing an elevation change of about 130 feet.

There are two main tributaries that join the Acushnet River at the New Bedford Reservoir: Squam Brook and Keene River. The reservoir receives runoff from these tributaries which together control approximately seven square miles of drainage area. Between the New Bedford Reservoir and Saw Mill Dam, the Acushnet River meanders in a southerly direction through large, flat swamplands and bogs ultimately discharging to New Bedford Harbor. A small dam was built upstream from Hamlin Road for irrigation purposes, and a second dam was built adjacent to Saw Mill as shown in Figure 2. Outflow from the Saw Mill Dam discharges to the harbor through a small channel that passes under Tarkiln Hill Road.

These types of basin characteristics (i.e. long, relatively flat with artificial storage) will produce hydrographs with attenuated peaks and wide bases. In addition, due to the large time of concentration of the basin, the lag to peak (difference between the time-center of rain and the time-center of runoff) will be relatively long. The lag to peak for this watershed is estimated to be approximately 20 hours, using a relationship between basin area and lag (Dunne and Leopold, 1978).

3.3 ARTIFICIAL STORAGE

As previously mentioned, there are three dams that impound runoff in the Acushnet River watershed: New Bedford Reservoir; Hamlin Street Dam; and Saw Mill Dam. Of the three, New Bedford Reservoir has the largest storage capacity. Based on information from the town of Acushnet, the dam is owned by the town of Acushnet and was built in 1875 for the purpose of creating a water supply for neighboring towns. The structural height of the dam is 24 feet with a normal hydraulic height of 20 feet (47 feet above mean sea level). A spillway with a 54 foot wide weir is located on the eastern end of the dam and is used as a discharge point into the Acushnet River. The normal impounding capacity (volume of impounded water at normal water depth) of the dam is approximately 1450 acre-feet with a flooding capacity (volume of water with water level elevation at four feet above weir crest) of close to 4,000 acre-feet.

The Hamlin Street Dam was built circa 1920 for irrigation purposes. The structural height of the dam is 12 feet with a normal hydraulic height (height of spillway crest) of approximately seven feet. The outflow is controlled by four separate box culverts passing under Hamlin Street.

The normal impounding capacity of the dam is approximately 105 acre-feet with a flooding capacity estimated to be 1000 acre-feet (hydraulic head at five feet above culvert crest).

The Saw Mill Dam was built in circa 1900 for use by the adjacent saw mill. The structural height of the dam is approximately six feet with a normal hydraulic height of six feet. The dam is an overflow type structure with a crest length of 100 feet. The normal impounding capacity of the dam is approximately 28 acre-feet with a flooding capacity estimated to be approximately 280 acre-feet (hydraulic head elevation of four feet above the dam).

The storage capacities of these structures will ultimately affect the flood wave as it moves down the Acushnet River. The effects are usually an attenuation of the hydrograph peak and a lengthening of the hydrograph base caused by flood retention within the impoundments. Due to the relatively large storage capacity of the New Bedford Reservoir, this impoundment will have a significant effect on the outflow hydrograph. The relatively small storage capacities of the Hamlin Street and Saw Mill Dams will have less effect on the outflow hydrographs due to the shorter retention times associated with the flood wave moving through the impounded water.

3.4 SOILS

In general, the soils in the Acushnet River watershed have low infiltration and percolation rates when thoroughly wetted due to the firm substratum that impedes the downward movement of water or the fine texture of the surface horizon which restricts infiltration. As a result, these types of soils most likely have a high runoff potential when thoroughly wetted and cause significant overland flow to the river during intense rainfall. A description of these soils are described below.

Soils of the Acushnet River watershed are comprised of three associations and a localized urban area as shown in Figure 3. The northern (generally north and west of New Bedford Reservoir) part of the basin consists mostly of the Hinckley-Freetown-Scarboro association. These types of soils are characterized by the Soil Conservation Service (1981) as being sandy and gravelly soils, and organic soils with slopes ranging from 0 to 25 percent. The association is about 35 percent Hinckley soils, 10 percent Freetown soils, 5 percent Scarboro soils and 50 percent minor soils.

The Hinckley soils are excessively drained and very permeable, and have a gravelly and sandy substratum. The soils are nearly level to steep and are dry most of the year. The Freetown soils are very poorly drained and consist of organic deposits more than 51 inches thick. These soils are nearly level or depressional. The Scarboro soils are very poorly drained and have a sandy substratum. The minor soils include the Merrimac, Windsor and Deerfield soils. These soils are moderately to excessively drained and have a loose, gravelly substratum.

The lower part of the basin (generally south of Hamlin Street) consists of the Paxton-Woodbridge-Ridgebury association. These soils, as described by the Soil Conservation Service (1981), are found on upland hills and ridges dissected by many small drainage ways. The association is about 15 percent Paxton soils, 15 percent Woodbridge soils, 10 percent Ridgebury soils and 60 percent minor soils.

The Paxton soils are nearly level to moderately steep, are well drained and have a very firm substratum at a depth of about 22 inches. In most instances, this substratum restricts the movement of water and the development of roots. The Woodbridge soils are nearly level to gently sloping. They are moderately well drained and have a very firm substratum at a depth of about 27 inches which also restricts the movement of water and the development of roots. The Ridgebury soils are nearly level to gently sloping and are adjacent to waterways. These soils are poorly drained and have a very firm substratum at a depth of about 29 inches. This substratum restricts the downward movement of water, and root development is restricted by a seasonally high water table and the very firm substratum. The minor soils in this unit include the Pawcatuck, Ipswich and Whitman soils. The Pawcatuck and Ipswich soils are very poorly drained and are associated with marshes. The Whitman soils are very poorly drained and have a very firm substratum.

The central-western area consists of the Paxton-Woodbridge-Whitman association. The Soil Conservation Service (1981) describes the morphology of these soils as upland hills and ridges dissected by many small drainageways. The unit is about 25 percent Paxton soils, 15 percent Woodbridge soils, 10 percent Whitman soils and 50 percent minor soils. Paxton and Woodbridge soils were described above. Whitman soils are nearly level and are found in depressions and low-lying areas adjacent to waterways. The soils are very poorly drained and have a very firm substratum at a depth of about 15 inches that restricts the movement of water. The minor soils in this unit include the Freetown, Merrimac and Ridgebury soils. These soils were described previously and are moderately to poorly drained.

It should be noted that the southwestern part of the watershed has been urbanized and consists of areas that are so altered or obscured by urban development that soil identification and characterization are not practical. Onsite investigations would be needed to determine these soil characteristics. However, this area is less than 3 percent of the total watershed area.

4.0 MODEL DEVELOPMENT AND FORMULATION

4.1 APPROACH

The approach taken in this study was to utilize the HEC-1 model to combine appropriate precipitation-runoff processes to produce an outflow hydrograph at Saw Mill Dam, and thus, entering New Bedford Harbor. The HEC-1 model was chosen because it has the capability of simulating the surface water response of a river basin to precipitation by representing the basin as an interconnected system of hydrologic and hydraulic components. The precipitation-runoff processes include: precipitation; infiltration/ interception; unit hydrograph conversion of runoff to streamflow; baseflow; and reservoir and channel routing. Each model component simulates a precipitation-runoff process within a portion of the basin. A component may represent a runoff area, a reservoir or a channel. The modeling result is a combination of these processes through mathematical relationships, ultimately computing a streamflow hydrograph at desired locations in the river basin.

Figure 4 shows the HEC-1 flow chart used for the simulation of the Acushnet River watershed. The watershed was divided into three subbasins (Figure 2) for the purpose of calculating runoff generated for the 10-, 25-, and 50-year storm events. The resulting hydrograph computed for subbasin 1 was routed through the New Bedford Reservoir producing an outflow hydrograph into subbasin 2. This outflow hydrograph was then routed through the channel of subbasin 2, creating an inflow hydrograph into the Hamlin Street Dam. The Hamlin Street Dam inflow hydrograph was combined with the runoff hydrograph generated from the watershed of subbasin 2, and routed through the Hamlin Street Dam producing an outflow hydrograph into subbasin 3. The outflow hydrograph was ultimately routed through the channel in subbasin 3 and combined with the runoff hydrograph generated from subbasin 3. The combined hydrograph was then routed through Saw Mill Dam producing an outflow hydrograph into New Bedford Harbor.

The following sections describe the precipitation-runoff processes required in the simulation of the Acushnet River watershed response specifying the characteristics and mathematical formulations which describe each physical process.

4.2 RAINFALL

The HEC-1 model requires a precipitation hyetograph (time distribution of rainfall) as input for all runoff computations. The program has the capability of utilizing any hyetograph shape specified by the user including historical storms, or using subbasin averages from weighted precipitation gages.

Since temporal distributions of rainfall for particular cyclonic storm events were not available for each subbasin, a synthetic storm pattern based on a statistical analysis of long term precipitation data collected by the town of New Bedford was developed. Based on 24-hour precipitation amounts associated with the 10-, 25- and 50-year return periods, the maximum storm amounts for each year were selected and ranked for the purpose of plotting a cumulative frequency curve of daily peak rainfall events (Table 1). Balsam chose a commonly used ranking formula which is described by Linsley et.al. (1982) as Weibull's formula:

$$p = \frac{m}{n + 1} , \qquad 4\text{-}1$$

where p equals the probability of occurrence (probability of that storm's magnitude being equaled or exceeded for a given year), n is the number of years of record and m is the rank of the event in order of magnitude. The results were then plotted on probability paper as shown in Figure 5, and the 10-, 25- and 50-year storm magnitudes were estimated from the graph by matching the inverse of the return period with the rainfall amount. Using this methodology the storm magnitudes for the 10-, 25- and 50-year storm events were estimated to be 4.7, 6.3 and 7.9 inches, respectively.

TABLE 1

RECURRENCE INTERVALS FOR MAXIMUM STORM EVENTS

Year	Annual Maximum 24 hr. Rainfall (inches)	Rank (m)	Recurrence Interval (T)
1987	2.63	19	1.63
1986	2.88	16	1.94
1985	5.70	2	15.50
1984	3.51	11	2.82
1983	2.92	14	2.21
1982	2.74	17	1.82
1981	2.25	21	1.48
1980	1.99	27	1.15
1979	2.17	25	1.24
1978	4.20	5	6.20
1977	3.51	10	3.10
1976	5.74	1	31.00
1975	3.49	12	2.58
1974	1.98	28	1.11
1973	5.31	3	10.33
1972	4.57	4	7.75
1971	2.24	22	1.41
1970	3.76	7	4.43
1969	3.44	13	2.38
1968	2.68	18	1.72
1967	3.55	8	3.88
1966	2.24	23	1.35
1965	2.20	24	1.29
1964	2.03	26	1.19
1963	1.77	30	1.03
1962	3.52	9	3.44
1961	2.90	15	2.07
1960	3.88	6	5.17
1959	1.72	29	1.07
1958	2.31	20	1.55

Notes: $T = \frac{n + 1}{m}$

T = Recurrence Interval (Years)

m = Rank

n = Number of Years of Record

These precipitation amounts were ultimately used for each subbasin rainfall. The distribution of each rainfall amount for a 24-hour period was assumed to be uniform over the watershed area. The 24-hour storm duration was chosen in this study since it produces the highest outflow peaks and volumes entering the basin in comparison to other duration storm events (i.e. 6-hour or 12-hour). Since the lag to peak for this watershed is approximately 20 hours, this results in continuous rainfall throughout the movement of the flood wave as it migrates through the watershed, maximizing the peak flow for a given storm.

It should also be noted that the 10-, 25- and 50-year storm events were selected, because these events produce hydrographs that could be used in future model applications and during remediation design. The 100-year flood was not simulated due to the possible inaccuracies associated with selecting the 100-year rainfall event. As can be observed in Figure 5, the 90 percent confidence interval was calculated using existing data contained in Table 1. Due to the lack of rainfall related to the higher return periods, the error associated with selecting a return period magnitude increases with increasing time. For example, the 90 percent confidence interval for the 50-year return period rainfall has a margin of plus or minus 0.5 inches. If the 100-year return period was illustrated in Figure 5, the confidence interval margin would be much greater than plus or minus 0.5 inches making it undesirable for use in HEC-1.

4.3 INFILTRATION AND INITIAL STORAGE

Initial storage is the amount of water retained by vegetation interception and land surface depression storage. Infiltration represents the movement of water to areas beneath the land surface. The combination of initial storage and infiltration is known as precipitation loss, and the positive difference between precipitation loss and precipitation is equal to the amount of runoff produced.

The HEC-1 model assumes water that is initially stored or infiltrated is lost from the system. In addition, the model does not account for soil moisture and surface storage recovery. Therefore, it should be noted that the HEC-1 model is a single-eventoriented model.

The infiltration computations are used with the unit hydrograph procedure (Section 4.4) to produce a hydrograph for a certain rainfall event, and are considered to be uniformly distributed over each entire subbasin. The model also incorporates a technique for estimating the percentage of the subbasin where infiltration is negligible such as large surface water bodies (direct precipitation) or heavily urbanized areas.

An initial and uniform infiltration rate method was chosen for the purpose of simulating infiltration effects. An initial loss (units of depth) and a constant loss rate (units of depth per time) were needed as input for this method. It was assumed that all rainfall was lost from runoff until the volume of initial loss was satisfied. After the initial loss was satisfied, rainfall was lost at the assigned constant rate.

The initial loss rate was estimated using the Soil Conservation Service (SCS) curve number method (SCS, 1986), which is an empirically derived equation that relates initial losses to the SCS curve number (CN). This in turn is related to the soil and cover conditions of the subbasin. The empirical equations are as follows:

$$I_a = 0.2S \quad \text{and,} \qquad 4\text{-}2$$

$$S = \frac{1000}{CN} - 10 \qquad 4\text{-}3$$

Where I_a is the initial loss, S is the potential maximum retention after runoff begins, and CN is the SCS curve number for that particular subbasin.

The major factors that determine the CN are the hydrologic soil group and the cover type. The hydrologic soil groups divide soil types into four groups based on their minimum infiltration rate which is governed by subsurface permeability and surface intake rates. The minimum infiltration rate is estimated by the SCS for bare soil after prolonged wetting. Based on the information presented in a soil survey of the watershed area (SCS, 1981) most of the soils in the Acushnet River watershed are classified as group C. Group C soils have characteristically low infiltration rates when wetted and consist chiefly of soils with sublayers which impede the downward movement of water, and soils with fine textures that reduce infiltration. According to the SCS, these soils have a range of infiltration rates of 0.05 to 0.15 inches per hour.

Using a table of curve numbers presented by the SCS (1986), the curve number for the type of soil cover (woods-grass combination) and soil group (Group C) was estimated to be 79. This produced an initial loss of 0.53 inches for each subbasin in the Acushnet River watershed.

As was previously discussed, the most abundant soils present in the study area had group C characteristics with respect to infiltration rates. However, the range of infiltration rates for group C soils (0.05 to 0.15 inches/hour) was large and therefore resulted in significantly different outflows when applying these infiltration rates to the HEC-1 model. Because of this an additional calibration target was used to further define the infiltration rates of each subbasin soil type. This method consisted of calculating the 10-year peak flows for each subbasin utilizing regression equations derived by S. William Wandle (1983) incorporating gaged streamflow data for southeastern Massachusetts rivers. The 10-year peak discharges using the regression equation were calculated to be 256, 294 and 121 cfs for subbasins 1, 2 and 3, respectively.

Using these subbasin discharges and the 10-year precipitation amount of 4.72 inches, the HEC-1 model was used to simulate each subbasin runoff hydrograph. The infiltration rates were slightly modified until each subbasins peak discharge simulated by the model closely matched the peak discharges computed by the regression equation. The resulting infiltration rates were modified to 0.135 inches/hour for subbasin 1, 0.140 inches/hour for subbasin 2 and 0.120 inches/hour for subbasin 3, which are all within the infiltration rates derived for group C soils.

The initial loss rate of 0.53 inches and the calibrated infiltration rates for each subbasin were used in all subsequent simulations.

4.4 RUNOFF - UNIT HYDROGRAPH

A unit hydrograph technique is needed for each subbasin runoff component to transform rainfall runoff to subbasin outflow. A unit hydrograph is a typical hydrograph for a basin that produces one inch(or one centimeter) of runoff. Since the physical characteristics of a basin (shape, size, slope, etc.) are constant, the unit hydrograph shape will be similar to the shape of hydrographs from storms of similar duration. Once a unit hydrograph is derived for a basin, an outflow hydrograph can be derived for a rainfall event with similar characteristics by multiplying the ordinates of the unit hydrograph by the depth of runoff (inches) generated from that particular storm event. Therefore, the unit hydrograph is characteristic of the subbasin being modeled and is not storm dependent.

The HEC-1 model allows for the direct input of a unit hydrograph created from streamflow data for the subbasin being modeled, or a synthetic unit hydrograph can be computed from user-supplied parameters. Since streamflow data on the Acushnet River was unavailable, the synthetic unit hydrograph approach was chosen. The technique chosen was the Snyder Unit Hydrograph method (Snyder, 1938), which consists of empirical relationships derived from watershed studies conducted in the northeast.

Snyder found the basin lag, Tp, to be a function of basin size and shape:

$$Tp = Ct\ (L\ Lc)^{0.3} \qquad 4\text{-}4$$

where L is the main stream distance from the outlet to the drainage divide, Lc is the distance along the stream from the stream outlet to a point opposite the basin centroid, and Ct is a coefficient that takes into account differences in basin slope and storage. Snyder also found that the unit hydrograph peak, Qp could be estimated from:

$$Qp = \frac{CpA}{Tp} \qquad 4\text{-}5$$

where A is the drainage area and Cp is a peaking coefficient found to range from 0.58 for gently sloping basins to 0.69 for steeper basins, with an average of 0.63.

The Snyder method determines the unit hydrograph's peak discharge and time to peak. The model then computes the remaining graph using a technique derived by Clark (1945). The Snyder parameters, Tp (time to peak) and Cp (peaking coefficient) were needed to construct the unit hydrograph. Tp was estimated using a regression formula discussed by Dunne and Leopold (1978) relating basin area to Tp. The relationship was derived using existing streamflow data for watersheds in the northeastern United States. The Tp values for subbasins 1, 2 and 3 were estimated to be 3.0, 3.2 and 1.8 hours, respectively. A Cp value of 0.58 was chosen using guidelines discussed in Dunne and Leopold (1978).

Using the Snyder parameters estimated for each subbasin and the rainfall discussed in sections 4.2 and 4.3, an outflow hydrograph could then be computed for each subbasin for use in reservoir and channel routing.

4.5 RESERVOIR ROUTING

In order to account for artificial storage affects on a passing flood wave, reservoir routing was used to simulate flood wave movement through each storage impoundment in the watershed. The reservoir routing method used in HEC-1 was the Modified Puls Technique which is based on the continuity equation and a relationship between storage and outflow (Linsley et.al., 1982). The continuity equation may be expressed as:

$$I - O = \frac{dS}{dt} \qquad 4\text{-}6$$

or

$$S = S_2 - S_1 = \int_{t_1}^{t_2} I\,dt - \int_{t_1}^{t_2} O\,dt \qquad 4\text{-}7$$

where I is the inflow rate, O is the outflow rate, S is the storage and t is the time interval. To provide a form more convenient for reservoir routing, this equation can be transformed into:

$$I_1 + I_2 + \frac{2S_1}{t} - O_1 = \frac{2S_1}{t} + O_2 \qquad 4\text{-}8$$

During the routing interval, I_1, I_2, O_1 and S_1 were known, and O_2 and S_2 were determined using a storage-outflow relationship, as described below.

To utilize this technique, a storage outflow relationship was developed for each dam in the Acushnet River watershed. This was conducted using information from the town of Acushnet on normal storage requirements (Section 3.4), planimitering flood storage areas for certain stage elevations, and using the weir equation to calculate outflow. The weir equation can be written as:

$$Q = 3.33LH^{3/2} \qquad 4\text{-}9$$

where Q is outflow, L is the weir length and H is the stage above the weir crest taken at a point upstream where the velocity head is negligible.

Outflow was calculated for stage intervals of approximately one foot and related to the volume of water impounded for that particular stage. Figures 6, 7 and 8 show the storage-outflow relationships calculated for New Bedford Reservoir, Hamlin Street Dam and Saw Mill Dam, respectively. I_1 and I_2 were estimated from the inflowing hydrograph generated using the unit hydrograph technique discussed in Section 4.4. Since HEC-1 is a single-event-oriented model, O_1 was assumed to be zero during the first time step and S_1 was obtained from the storage-outflow relationship. The corresponding values of S_2 and O_2 were computed by solving the routing equation and the storage outflow relationship, simultaneously. During the second time step, I_2 became I_1, the new I_2 was obtained from the inflow hydrograph, O_2 became O_1, and S_2 became S_1 and the process was then repeated until the flood wave was routed through the impoundment creating an outflow hydrograph leaving the dam.

4.6 CHANNEL ROUTING

In order to account for river storage and lagging affects of the Acushnet River on the flood wave movement, a channel routing technique was used in the HEC-1 model. The technique chosen was the Muskingum method as this method can be used with a minimum amount of information on channel dimensions (Linsley et.al., 1982). The method assumes that in the previously discussed continuity equation (4-8), S_1 and S_2 are replaced by:

$$S = K[XI + (1-X)O] \qquad 4\text{-}10$$

where K is the storage constant, X is the Muskingum constant, I is inflow and O is outflow. The constant X expresses the relative importance of inflow and outflow in determining storage. As discussed in Linsley (1982), the value of X for most streams is between 0 and 0.3 with a mean value near 0.2. The constant K is the ratio of storage to discharge and is approximately equal to the travel time through the channel reach.

The Muskingum constant, X was assumed to be 0.2 for the two channel reaches simulated (subbasin 2 and 3). The storage constant K was calculated using estimated channel velocities obtained during a site visit on November 18, 1987. The estimated K values were 2.59 and 0.88 hours for subbasins 2 and 3, respectively.

A similar time stepping approach was taken as described in the reservoir routing procedure except the storage outflow relationship was replaced with equation 4-9, thus producing an outflow hydrograph for the end of the river reach simulated.

4.7 BASE FLOW

Two distinguishable contributions to a streamflow hydrograph are direct runoff (Section 4.4) and base flow. Base flow results from the movement of groundwater into the stream. In the northeastern United States, base flow is what sustains a river's discharge during dry periods. The HEC-1 model provides a means to include the effects of base flow on the streamflow hydrograph as a function of the initial flow in the river and the antecedent moisture conditions. The parameters needed to implement this feature of HEC-1 are commonly estimated using the receding part of an existing hydrograph from the watershed being modeled.

Base flow estimates were not included as a streamflow source for the following reasons:

1) there were no hydrographs available for analysis of the parameters needed to simulate this component.

2) the objective of this study was to simulate flood events for the 10-, 25- and 50-year storm occurrences, and base flow would only be a small percentage of the streamflow hydrograph since runoff dominates during intense rainfall.

In addition, base flow may be even less significant during flooding because of the relatively high river stage (most likely higher than the groundwater elevation) possibly resulting in ground water recharge from the stream during the flooding event.

4.8 SNOWFALL AND SNOWMELT

Snowfall and snowmelt during the spring may generate runoff and thus may be a source of water to a streamflow hydrograph during the spring months. Due to the insufficient data on snowfall, snow pack density and rate of snowmelt, snowfall and snowmelt was not included as part of the hydrologic processes used in computing outflow hydrographs. However, because of the geographic location of the project site, significant snow pack is not expected to occur, making this precipitation class unlikely to create large runoff events.

5.0 SENSITIVITY ANALYSIS

Sensitivity analyses were conducted on the model parameters associated with highest uncertainty with respect to their values. This analysis was used to evaluate the effects of variation in the values of these parameters on the modeling results and to establish confidence in their use to accurately estimate peak surface water flows and volumes entering New Bedford Harbor from the Acushnet River watershed. The three parameters chosen for sensitivity analysis were: 1) the Muskingum X used to represent the affects of channel storage during channel routing; 2) Snyder's Cp used to simulate the peak flow of the unit hydrograph; and 3) infiltration rates used to compute the runoff depth during rainfall. The results of the sensitivity analysis are presented below.

The sensitivity analysis consisted of running the HEC-1 model a number of times, changing the value of each parameter while holding all other parameters constant, and evaluating the effects of parameter variation on the peak discharge computed by the model. Table 2 displays the results of all sensitivity runs. As can be observed in Table 2, varying the Muskingum X from zero (the minimum value for this parameter) to 0.5 (the maximum value for this parameter) did not significantly change the results. In fact, only the 10-year event peak discharge was affected while the 25- and 50-year peak discharge events remained unchanged.

With respect to the Snyder Cp, this value was modified from 0.58 (the minimum value used for coastal-flat watersheds) to 0.63 (the average value used for watersheds found in the northeast). The maximum value for this parameter was not evaluated because it is associated with watersheds found in mountainous areas of the northeast. The results in Table 2 show that an increase in the Cp value caused a slight increase in the peak discharge, with the relative difference in discharges increasing with an increase in the return period. However, the maximum difference was only 17 cfs occurring during the 50-year simulation.

TABLE 2

SENSITIVITY ANALYSIS RESULTS

		Peak Outflows (CFS)	
Parameter	10-Year Event	25-Year Event	50-Year Event
Muskingum X	392	867	1397
X = 0	392	867	1397
X = .2	394	867	1397
Snyder Cp			
Cp = .58	392	867	1397
Cp = .63	400	879	1415
Infiltration Rate			
I = .05	1003	NC	NC
I = .10	605	NC	NC
I = .15	272	NC	NC

Notes: 1) Peak outflow is the outflow from Saw Mill Dam

2) NC - sensitivity run not conducted

The final parameter evaluated was the infiltration rate. The infiltration rate was modified from 0.05 inches/hour (minimum infiltration rate for hydrologic group C soils) to 0.15 inches/hour (maximum infiltration rate for group C soils). As is illustrated in Table 2, a change in the infiltration rate, even within the hydrologic group range, produced significant changes in the peak outflow with, as expected, the lower infiltration rate producing the highest peak outflows. Since it was clear that additional calibration was needed for this parameter, sensitivity runs were not conducted for the 25- and 50-year storm events.

Due to the magnitude of the peak outflow variation, additional calibration of the infiltration rate was conducted as previously discussed in Section 4.3, resulting in more accurate estimates of the infiltration rates for each subbasin. An X of 0.2; a C_p of 0.58; and infiltration rates of 0.135, 0.140 and 0.120 inches/hour for subbasins 1, 2 and 3, respectively, were used in all subsequent simulations.

6.0 RESULTS

Three simulations were conducted using the HEC-1 model to produce an outflow into the upper estuary during the 10-, 25- and 50-year storm events. Each operation (runoff, reservoir routing and channel routing) was simulated for each subbasin and then combined, ultimately producing an outflow for the Acushnet River watershed. Table 3 summarizes the HEC-1 results and Figures 9, 10 and 11 show the respective hydrographs for the each storm event. These simulations of outflow hydrographs should be regarded as estimates based upon available data and assumptions discussed previously. However, where only limited data were available for model development and calibration, conservative values were generally assigned.

6.1 10-YEAR STORM EVENT

The 10-year event consisted of simulating the effects of the 10-year, 24-hour rainfall event over the Acushnet River watershed. As can be observed in Table 3, the peak flow entering the reservoir from rainfall excess was 269 cfs which occurred 22 hours after rainfall had started. The attenuation effects of the reservoir were substantial, reducing the peak to 91 cfs as the flood wave moved through the impoundment. As the flood wave moved down the river to the Hamlin Street dam, little further attenuation of the flood peak occurred. The peak flow generated from subbasin 2 occurred at 23.25 hours, and when combined with the routed hydrograph, produced a peak flow of 375 cfs entering the Hamlin Street dam. The storage effects of Hamlin Street dam were also significant reducing the peak flow to 301 cfs as the flood wave moved through the impoundment. Again, the attenuation effects of the channel on the peak flow were negligible as the flood wave migrated down subbasin 3 to Saw Mill Dam. The peak flow generated from subbasin 3 occurred at 14.25 hours, and when combined with the routed hydrograph, produced a peak flow of 407 cfs. When routed through Saw Mill Dam, the peak flow was attenuated to 392 cfs. This flow was the estimated flow entering the upper estuary and occurred at 25.50 hours from the start of rainfall.

Figure 9 shows the complete outflow hydrograph for the 10-year storm event. As can be observed from the Figure 9, although the peak flow occurred at 25.50 hours, the hydrograph responded to the storm at approximately 3 hours when the hydrograph started to rise. The hydrograph appeared to end at approximately 70 hours, producing an estimated 10,000 cubic feet (area under hydrograph) of water entering the upper estuary.

6.2 25-YEAR STORM EVENT

The 25-year event consisted of simulating the effects of the 25-year, 24-hour rainfall event over the Acushnet River watershed. Table 3 summarizes the HEC-1 results with the sequence of operations being the same as the 10-year event. However, as expected, the increase in rainfall magnitude augmented the peak flows associated with each subbasin. The peak flows were computed to be 548, 650 and 232 cfs for subbasins 1, 2 and 3, respectively. Note that the times to peak are approximately 0.5 hours sooner than the 10-year event. This was caused by the increase in rainfall intensity and the constant initial storage amounts and infiltration rates. Because initial precipitation losses were satisfied sooner for higher intensity storms, initial runoff occurred sooner, resulting in a quicker peak flow. This simulation ultimately produced a peak outflow at the Saw Mill Dam of 867 cfs, occurring at 25.25 hours.

Figure 10 shows the complete outflow hydrograph for the 25-year storm event. As is shown in Figure 10, the hydrograph first responded to rainfall at about 3 hours, with the peak flow occurring after 25 hours. The hydrograph receded after about 79 hours with a hydrograph volume estimated to be approximately 21,000 cubic feet.

TABLE 3

SUMMARY RESULTS OF HEC-1 OUTPUT

Operation	Station	10 YEAR Peak Flow	10 YEAR Time of Peak	25 YEAR Peak Flow	25 YEAR Time of Peak	50 YEAR Peak Flow	50 YEAR Time of Peak
Hydrograph at	Subbasin 1	269	22.00	548	21.50	839	21.00
Routed Through	New Bedford Reservoir	91	29.25	204	29.00	375	28.25
Routed Through	Subbasin 2	91	32.00	204	31.50	374	31.00
Hydrograph at	Subbasin 2	306	23.25	650	22.75	1009	22.25
Combined at	Hamlin St. Dam	375	24.25	795	24.25	1285	24.50
Routed Through	Hamlin St. Dam	301	27.00	696	26.50	1151	26.25
Routed Through	Subbasin 3	301	27.75	696	27.25	1151	27.25
Hydrograph at	Subbasin 3	142	14.25	232	13.75	327	13.25
Combined at	Saw Mill Dam	407	24.50	884	24.50	1415	24.50
Routed Through	Saw Mill Dam	392	25.50	867	25.25	1397	25.25

Notes:
1) Peak flows in cfs (cubic feet per second)
2) Time of peak in hours
3) Results of Saw Mill Dam routing is outflow into harbor

6.3 50-YEAR STORM EVENT

The 50-year storm event consisted of simulating the effects of the 50-year, 24-hour rainfall event over the Acushnet River watershed. Again, the sequence of operations were the same as the 10- and 25-year events. As observed before, New Bedford Reservoir and Hamlin Street Dam had significant effects on attenuating the peak flow. The peak outflow simulated for this event was computed to be 1397 cfs, and it occurred at 25.25 hours.

Figure 11 shows the outflow hydrograph for this simulation. Again, the hydrograph initially responded to rainfall after about 3 hours, and subsided after about 70 hours, producing a volume of runoff approximately equal to 37,000 cubic feet.

7.0 SUMMARY AND CONCLUSION

The HEC-1 model was utilized to simulate the combined hydrologic processes of rainfall, infiltration, runoff, reservoir routing and channel routing occurring in the Acushnet River watershed. The surface runoff response in the watershed to the 10-, 25-, and 50-year 24-hour rainfall events was modeled to produce estimates of peak outflows and volumes entering the upper estuary.

This study was conducted to assist engineers and scientists in their evaluation of the hydraulic processes that affect New Bedford Harbor. Two previous flood studies on the Acushnet River were reviewed. However, these studies were empirically based, did not account for storage affects of Hamlin Street and Saw Mill Dams, and were unable to incorporate local data on the Acushnet River watershed as it became available. A significant advantage to the study described here compared to the previous studies is the ability to evaluate the effects of potential future modifications of the Hamlin Street Dam on peak outflows entering the harbor for the purpose of controlling peak outflows during possible harbor remediation.

It should be noted that the HEC-1 results are based upon the assumptions inherent in the model and the limited data available for model development and calibration. During model development, the values for three parameters were initially unknown, and thus estimated for model use. Sensitivity analyses were conducted on these parameters to determine their effect on model results. These parameters included infiltration, Muskingum's X and Snyder's Cp. It was observed during the sensitivity analysis that the model was very insensitive to X and Cp; however, slight changes in the infiltration rates resulted in significant variations in the computed outflow. As a result, the infiltration rates for each subbasin were calibrated to an empirical relationship producing a more accurate estimate of the infiltration rates used in the model.

In addition, temporal and spacial distributions of rainfall were not used in the model. Instead, a uniform rainfall distribution was assumed for each 24-hour event. It is recognized that the intensity of rainfall may vary during a storm as well as over geographical regions. However, due to the relatively small watershed being considered and the nature of cyclonic storms (cyclonic storms produce more uniform rainfall in comparison to frontal or thunder storms), temporal and spacial changes in rainfall intensity should not produce significant inaccuracies in the modeling results.

The outflow results produced by the HEC-1 model, for the 10-year event are fairly similar to the results computed by the Flood Insurance Study conducted by the Federal Emergency Management Agency and the Preliminary Flood Plain Management Investigation conducted by the U.S. Army Corps of Engineers. A summary of outflows from these three studies is provided below:

Outflow Results (CFS)

Storm Event	HEC-1 Study	FEMA Study	USACE Flood Plain Study
10-Year	392	280	475
25-Year	867	Not Computed	740
50-Year	1397	475	980
100-Year	Inadequate Data	630	1350

However, the 50-year result computed by HEC-1 is significantly larger than the 50-year results produced by the other two studies. One possible reason for the discrepancy in peak flows may be due to the large rainfall magnitude used in the HEC-1 model for the 50-year rainfall event (approximately 7.9 inches). As can be observed in Figure 5 and Section 4.2 the error associated with the technique used to estimate rainfall magnitudes from the existing data increases with magnitude due the the insufficient amount of data for the higher storm magnitudes. The U.S. Weather Bureau (TR-40, 1961) estimates the 50 year, 24-hour rainfall to be approximately 6.2 inches in comparison to 7.9 inches used in the HEC-1 model. However, the U.S. Weather Bureau estimate is based on regional data taken prior to 1961, and is not site specific. Therefore, the outflow associated with 7.9 inches of rainfall appears to produce the most accurate and conservative results.

In summary, the HEC-1 model simulated hydrographs entering New Bedford Harbor for the 10-, 25- and 50-year storm events. These hydrographs were produced using a limited amount of data on the Acushnet River watershed, and do not take the place of gaged streamflow data. However, they do produce reliable estimates of peak flows and discharge storm volumes using the most current available data and techniques.

8.0 REFERENCES

Army Corps of Engineers; Hydrology of Floods New Bedford Harbor Massachusetts, Hydrologic Engineering Section, Water Control Branch, Waltham, MA, 1987

Clark, C.O.; Storage and the Unit Hydrograph, Transactions of the American Society of Civil Engineers 110, pp. 1419-1446, 1945.

Dunne, T., L.B. Leopold; Water in Environmental Planning, W.H. Freeman and Company, San Francisco, CA, 1978.

Federal Emergency Management Agency; Flood Insurance Study, Acushnet, MA, Bristol County, 1982.

Linsley, R.K., M.A. Kohler, J.L.H. Paulhus; Hydrology for Engineers, Third Edition, McGraw Hill Book Company, New York, 1982.

Snyder, F.F.; Synthetic Unit Hydrographs, Transactions of the American Geophysical Union, Vol. 19, Part 1, pp. 447-454, 1938.

Soil Conservation Service; National Engineering Handbook, U.S. Department of Agriculture, Washington, D.C., 1964

Soil Conservation Service; Urban Hydrology for Small Watersheds, TR-55, U.S. Department of Agriculture, Engineering Division. Printed by Rockingham County Conservation District, Exeter, NH, 1986.

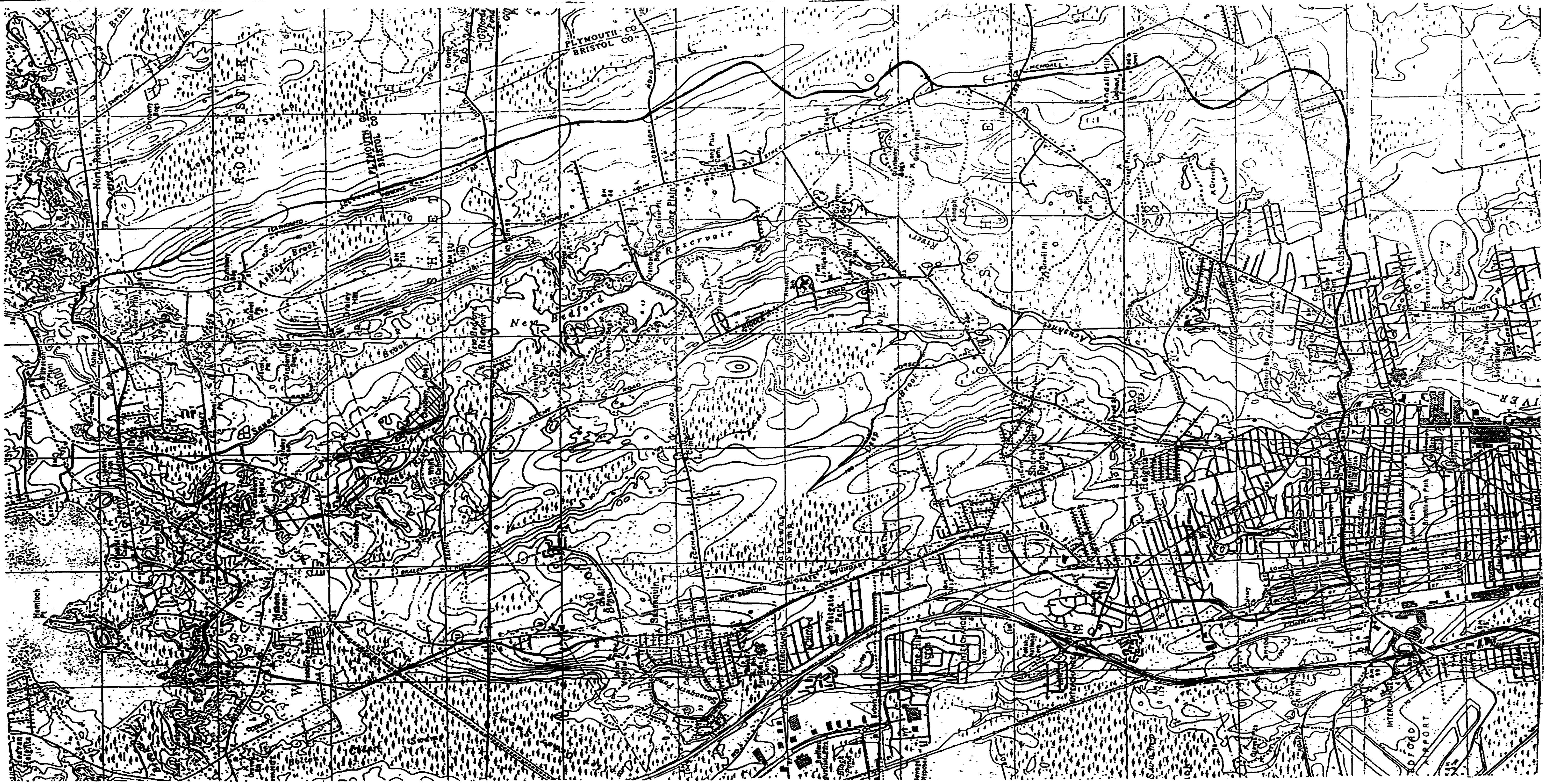
Soil Conservation Service; Soil Survey of Bristol County Massachusetts Southern Part, U.S. Department of Agriculture, 1981.

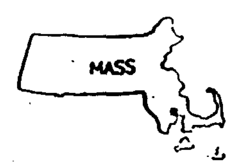
U.S. Army Corps of Engineers; HEC-1 Flood Hydrograph Package Users Manual, The Hydrologic Engineering Center, Davis, CA, 1981.

U.S. Weather Bureau; Rainfall Frequency Atlas of the United States, Technical Paper No. 40, May, 1961.

Wandle, S.W., U.S. Department of Interior, Geological Survey, Water Resources Investigations 77-39; Estimating the Magnitude and Frequency of Floods on Natural-Flowing Streams in Massachusetts, Washington D.C. 1977.

Wandle S.W., U.S. Department of Interior, Geological Survey, Water Supply Paper 2214; Estimating Peak Discharges of Small, Rural Streams in Massachusetts, Alexandria, Virginia, 1983.



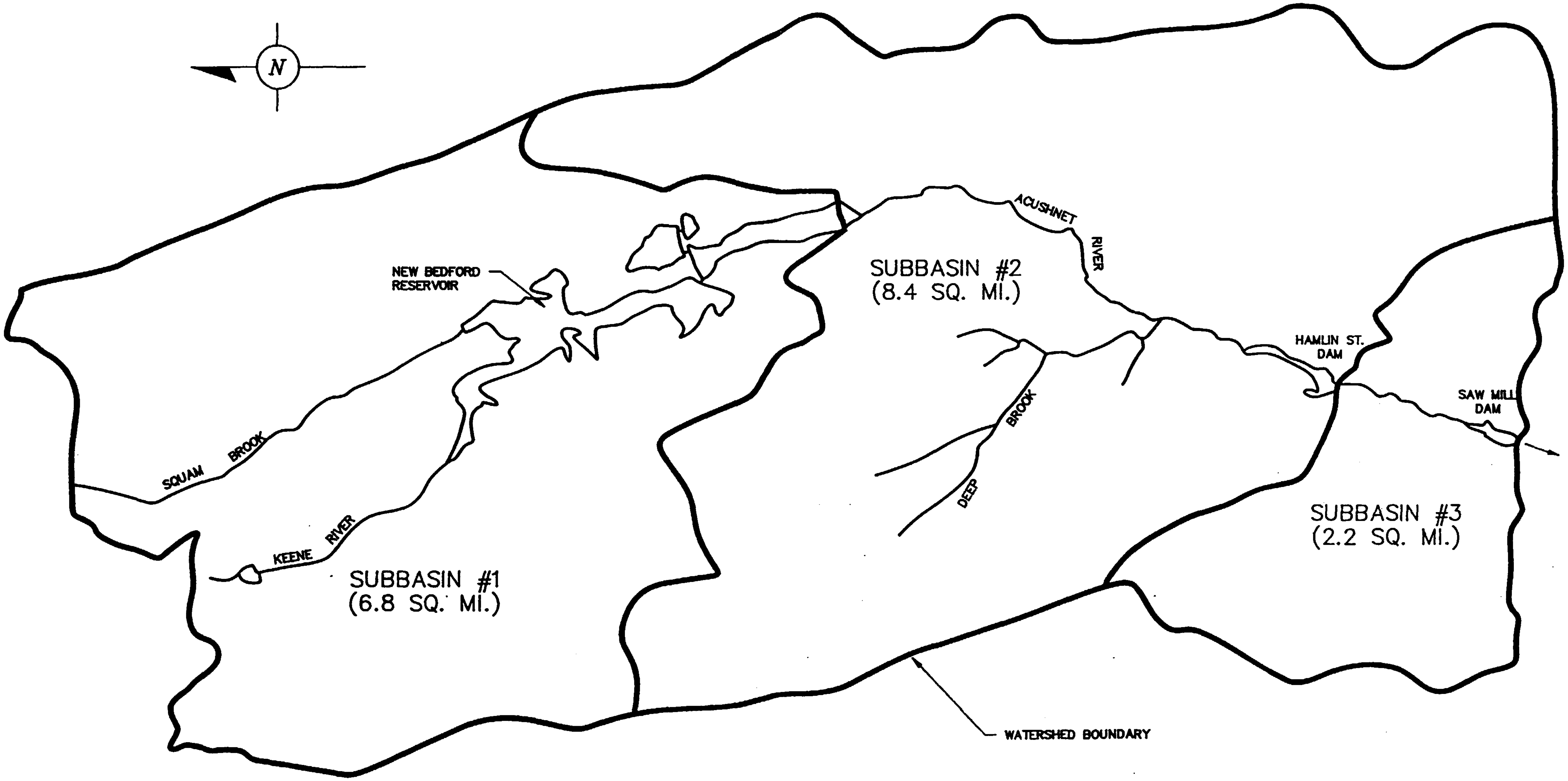


QUADRANGLE LOCATION

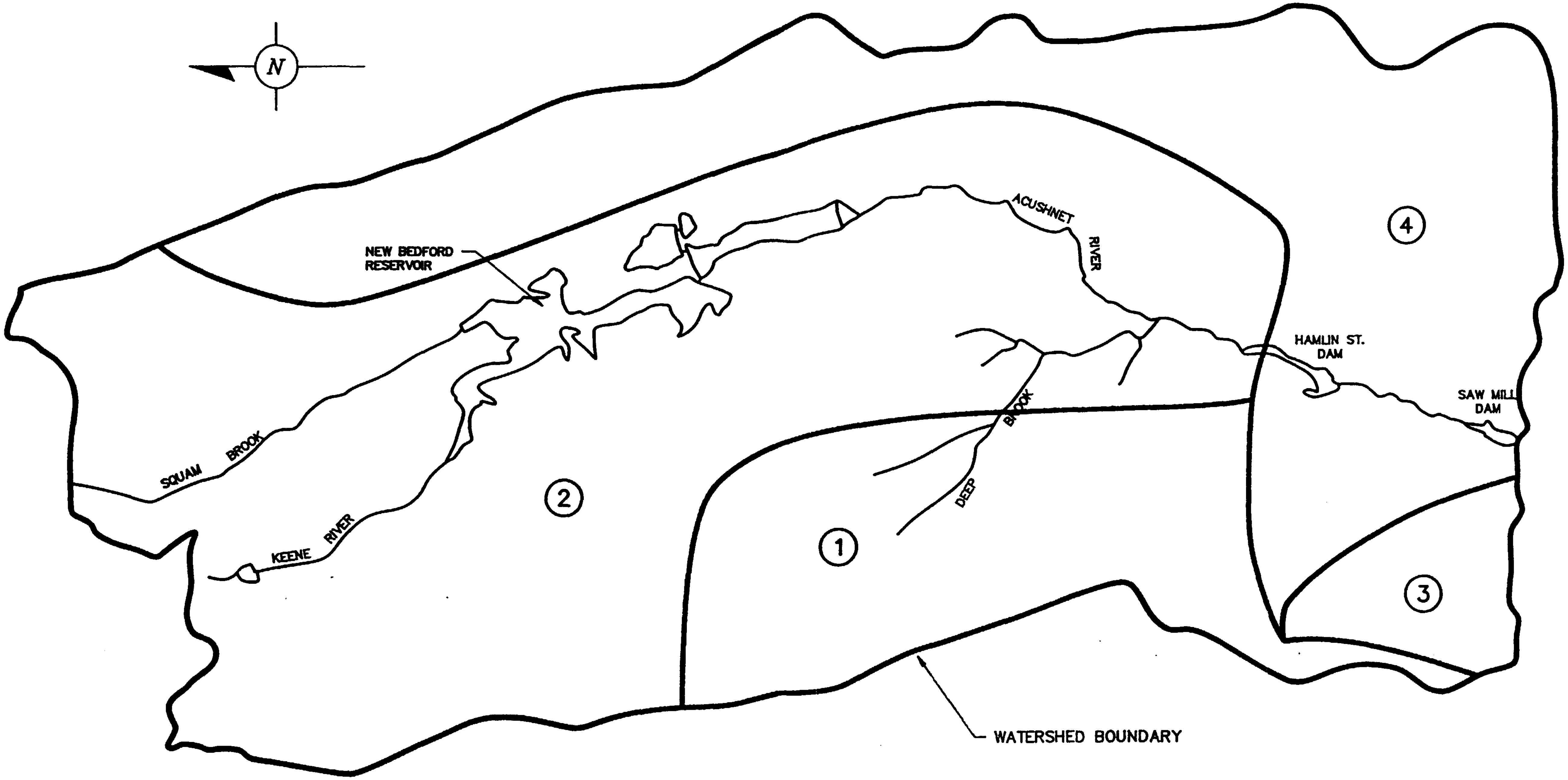
SOURCE:
USGS NEW BEDFORD NORTH QUADRANGLE
MASSACHUSETTS, US GEOLOGICAL SURVEY
7.5 MINUTE SERIES (TOPOGRAPHIC), 1981

NOTE: ACUSHNET RIVER WATERSHED HAS BEEN DELINEATED AS SHOWN ON THIS FIGURE.

BALSAM ENVIRONMENTAL CONSULTANTS, INC. Salem, N.H.			CLIENT NUTTER McCLENNEN &FISH	
			TITLE WATERSHED LOCUS PLAN	
DATE 2/26/88	DRAWN BY D.J.P.	CHECKED R.J.W.	PROJECT NEW BEDFORD HARBOR	
SCALE 1"=2778'	DESIGNED R.J.W.	APPROVED L.C.S.	FIGURE NO. FIG. 1	PROJECT NO. 6002



BALSAM ENVIRONMENTAL CONSULTANTS, INC. Salem, N.H.			CLIENT NUTTER, McCLENNEN & FISH	
			TITLE ACUSHNET RIVER WATERSHED	
DATE 2/25/88	DRAWN BY D.J.P.	CHECKED R.J.W.	PROJECT NEW BEDFORD HARBOR	
SCALE 1"=2778'	DESIGNED R.J.W.	APPROVED L.C.S.	FIGURE NO. FIG. 2	PROJECT NO. 6002



LEGEND:

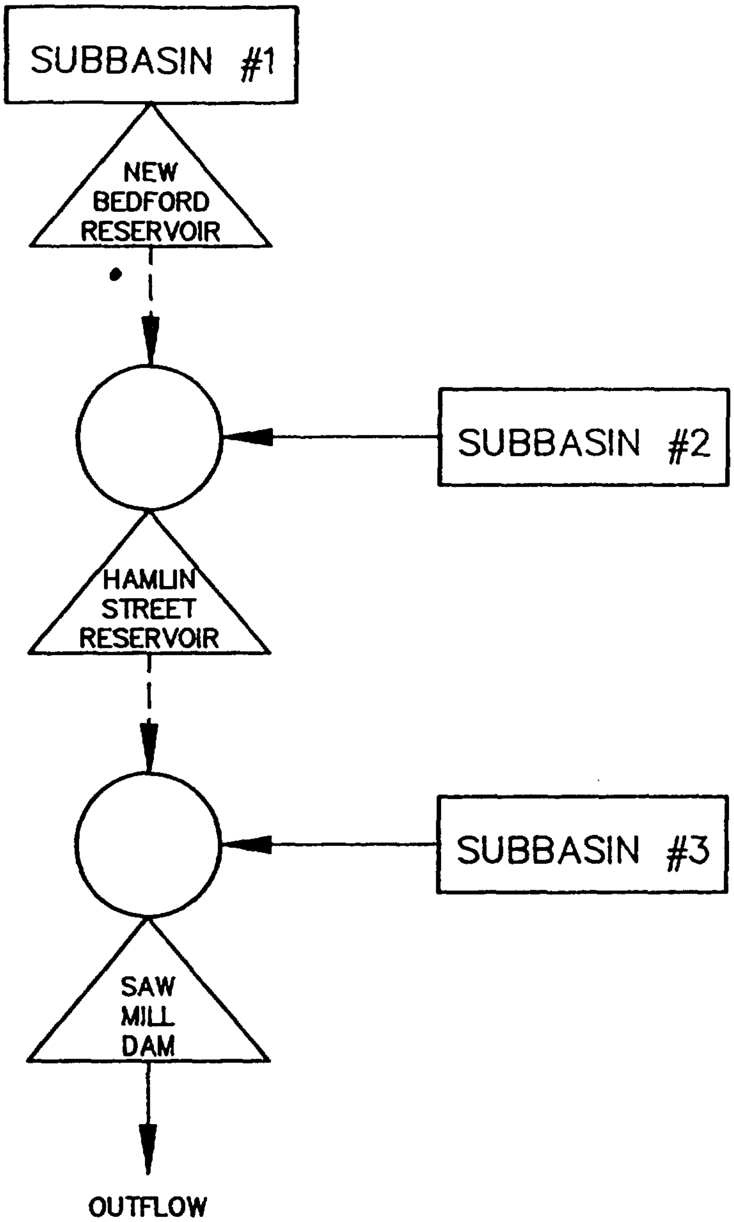
(1) = PAXTON–WOODBRIDGE–WHITMAN ASSOCIATION

(2) = HINCKLEY–FREETOWN–SCARBORO ASSOCIATION

(3) = URBAN LAND ASSOCIATION

(4) = PAXTON–WOODBRIDGE–RIDGEBURY ASSOCIATION

BALSAM ENVIRONMENTAL CONSULTANTS, INC. Salem, N.H.			CLIENT NUTTER, McCLENNEN & FISH	
			TITLE GENERAL SOILS MAP	
DATE 2/25/88	DRAWN BY D.J.P.	CHECKED R.J.W.	PROJECT NEW BEDFORD HARBOR	
SCALE 1"=2778'	DESIGNED R.J.W.	APPROVED L.C.S.	FIGURE NO. FIG. 3	PROJECT NO. 6002



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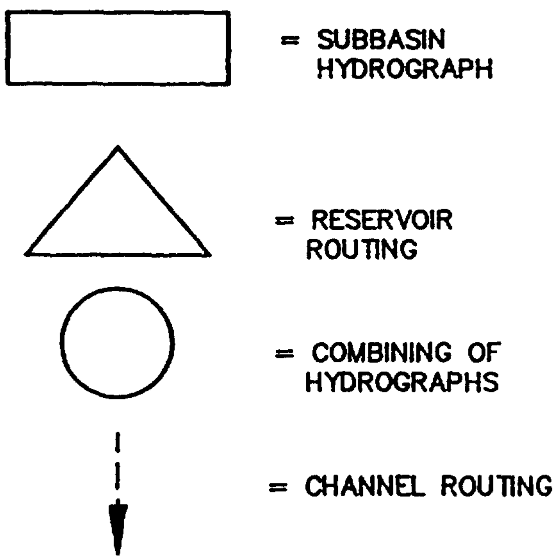
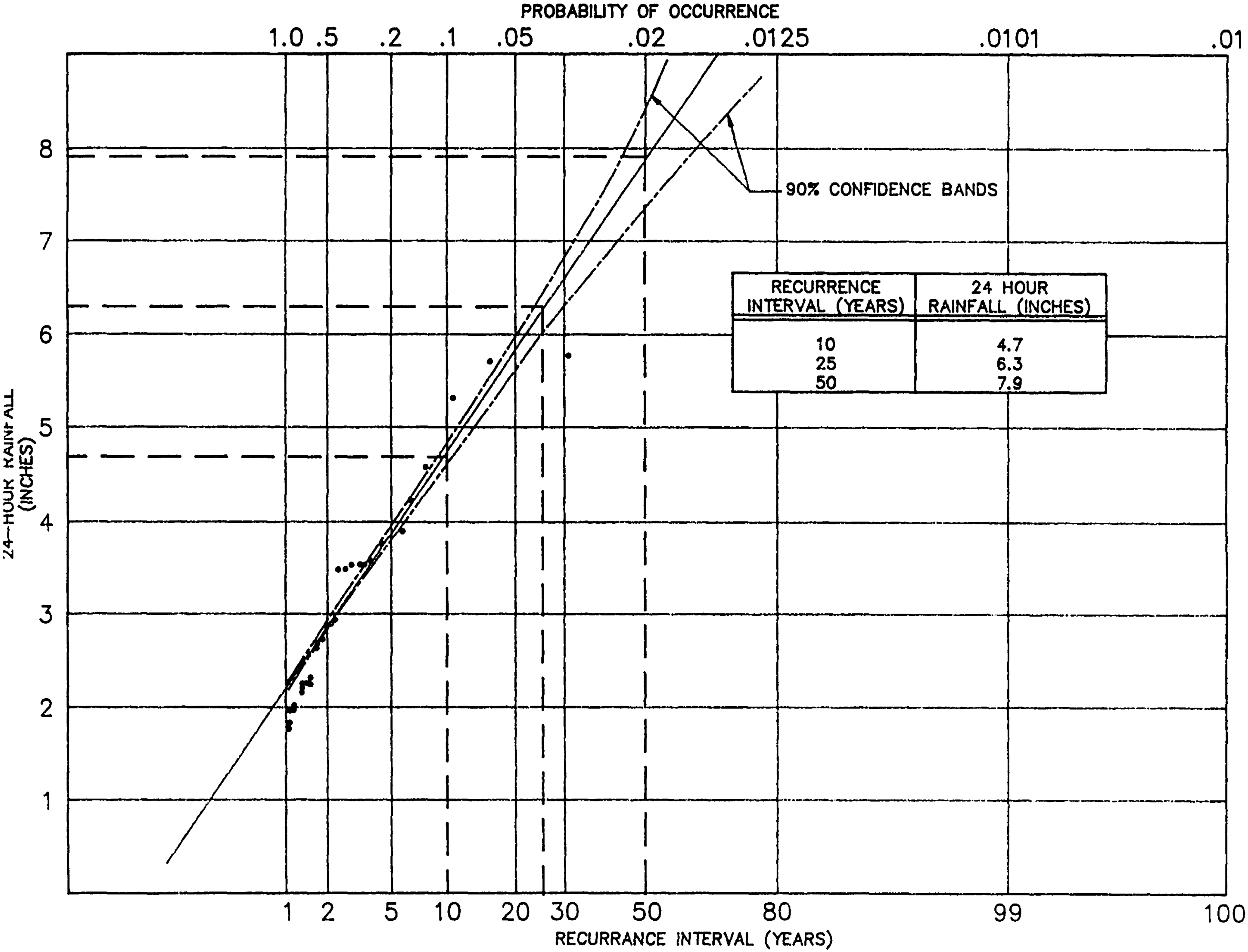
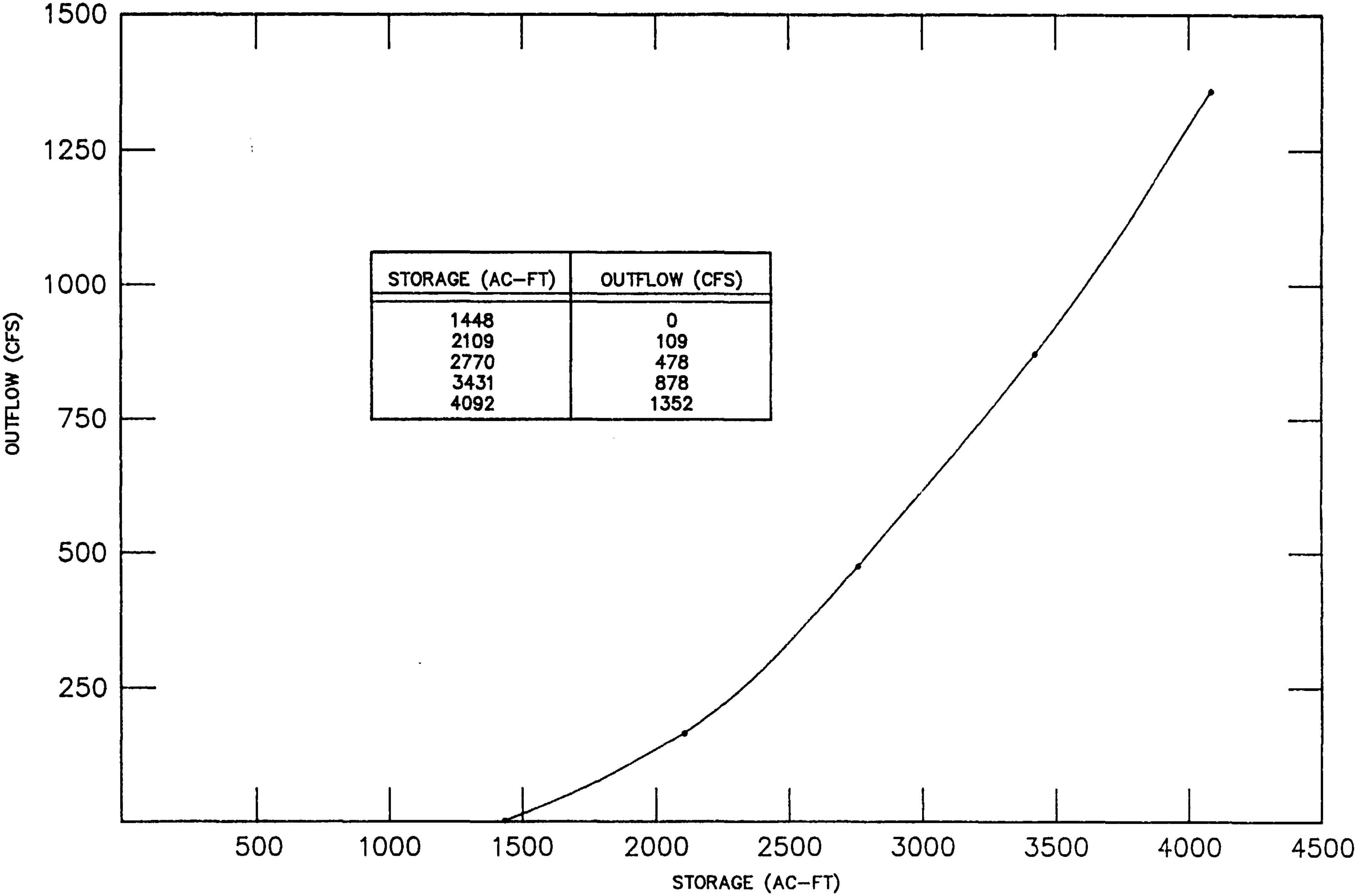


BALSAM ENVIRONMENTAL CONSULTANTS, INC. Salem, N.H.			CLIENT NUTTER McCLENNEN & FISH	
			TITLE HEC-1 FLOW CHART	
DATE 4/5/88	DRAWN BY D.J.P.	CHECKED R.J.W.	PROJECT NEW BEDFORD HARBOR	
SCALE N/A	DESIGNED R.J.W.	APPROVED L.C.S.	FIGURE NO. FIG. 4	PROJECT NO. 6002

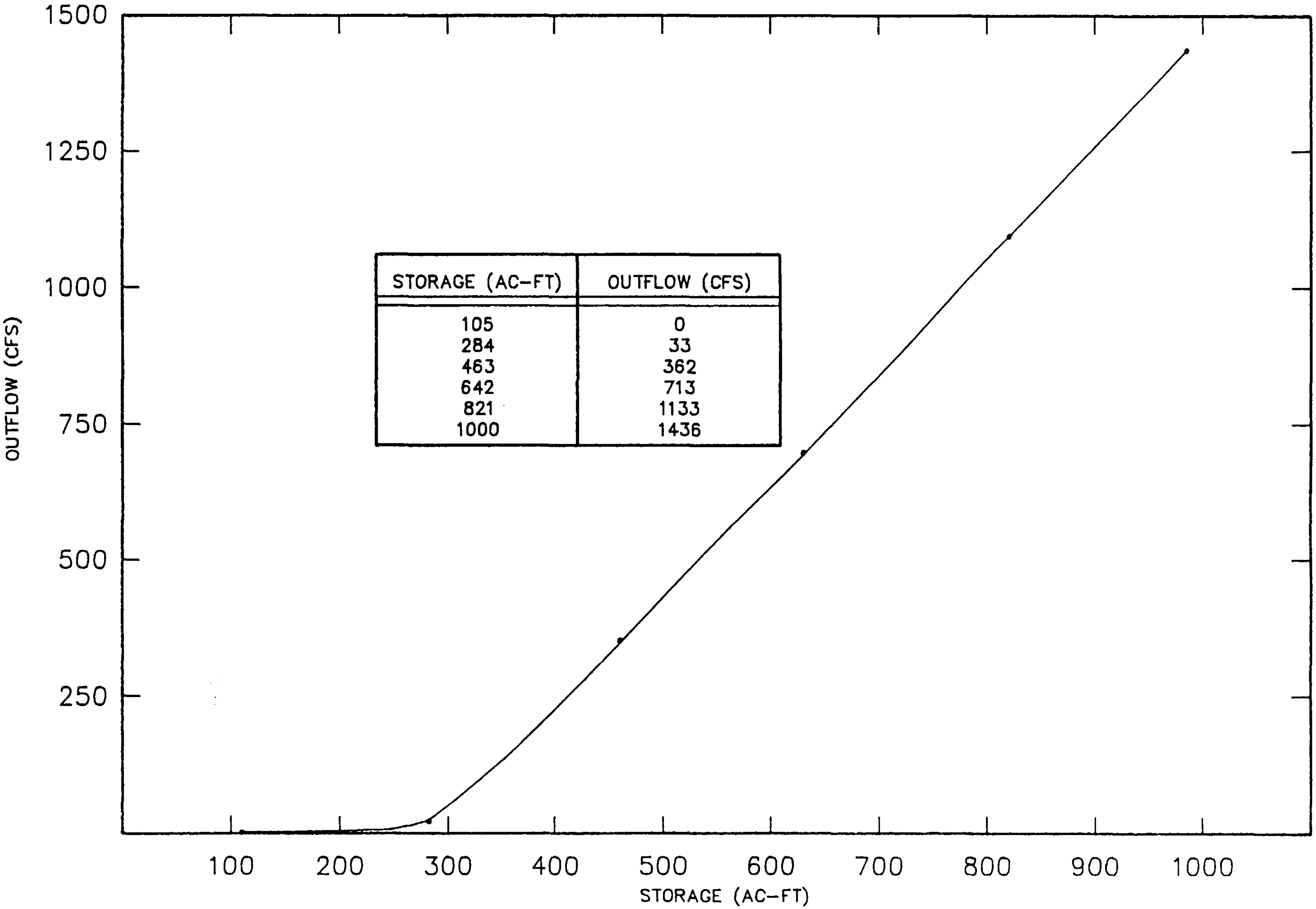


24-HOUR RAINFALL – FREQUENCY DISTRIBUTION

FIGURE 5

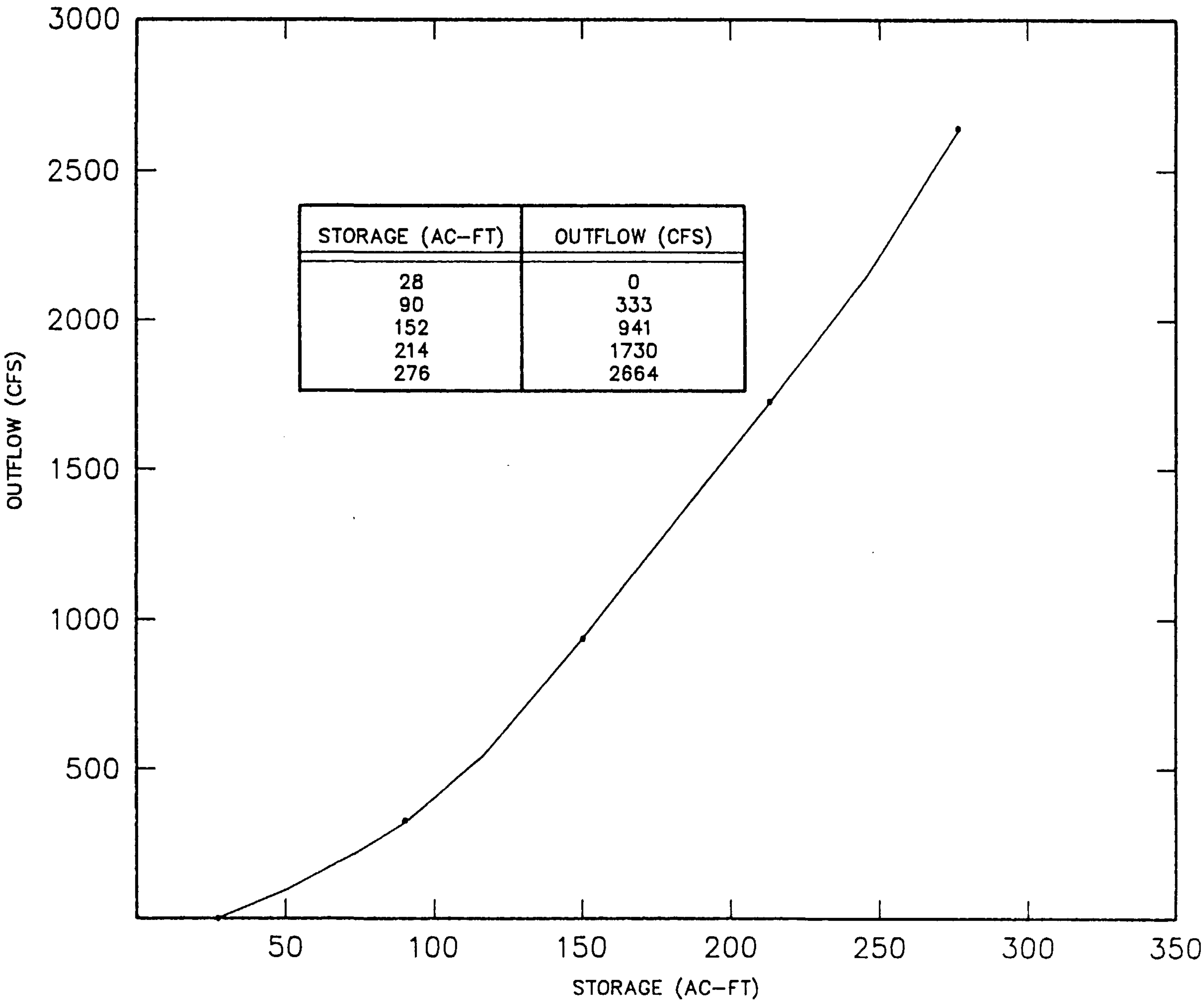


STORAGE–OUTFLOW RELATIONSHIP
NEW BEDFORD RESERVOIR
FIGURE 6



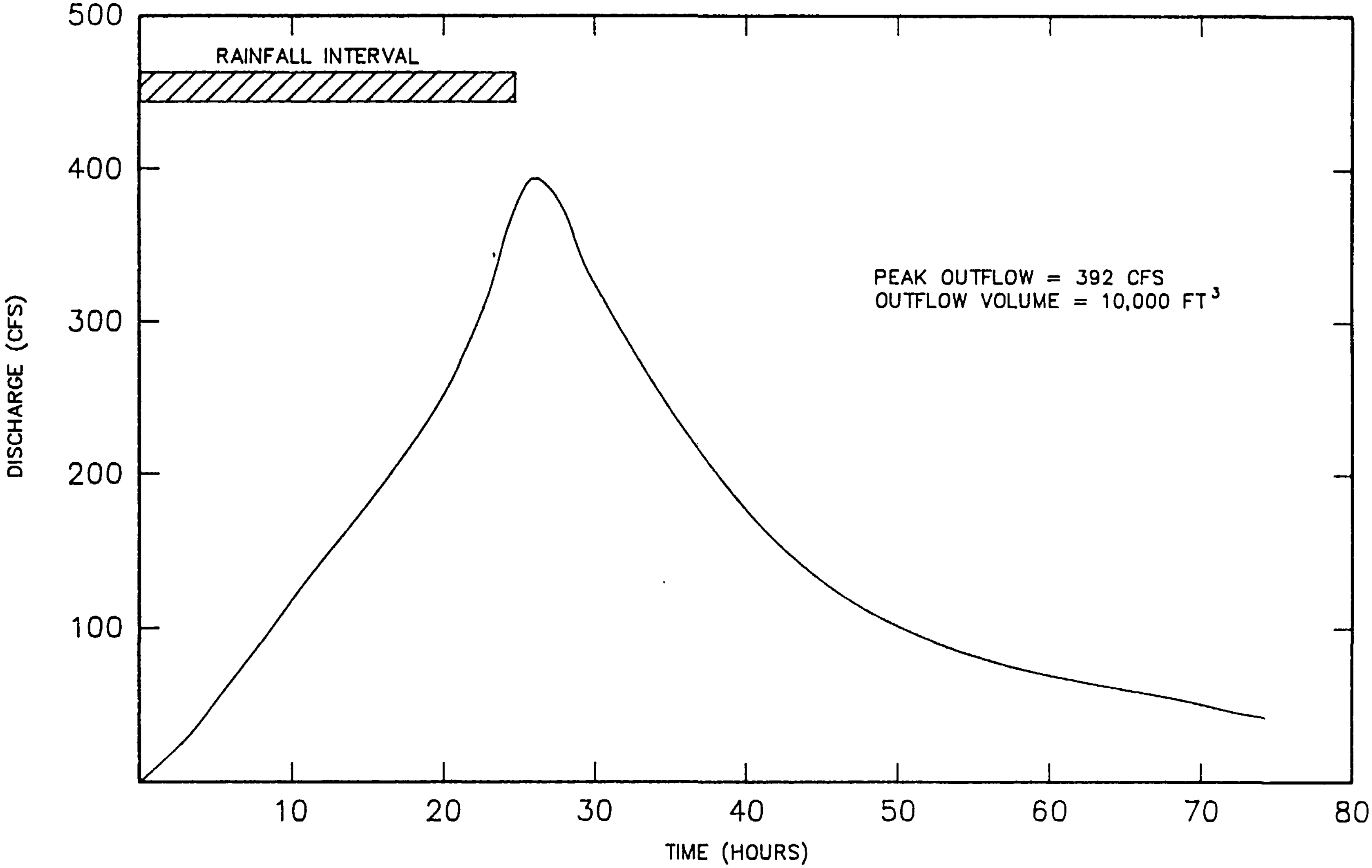
STORAGE (AC-FT)	OUTFLOW (CFS)
105	0
284	33
463	362
642	713
821	1133
1000	1436

STORAGE-OUTFLOW RELATIONSHIP
HAMLIN STREET DAM
<u>FIGURE 7</u>

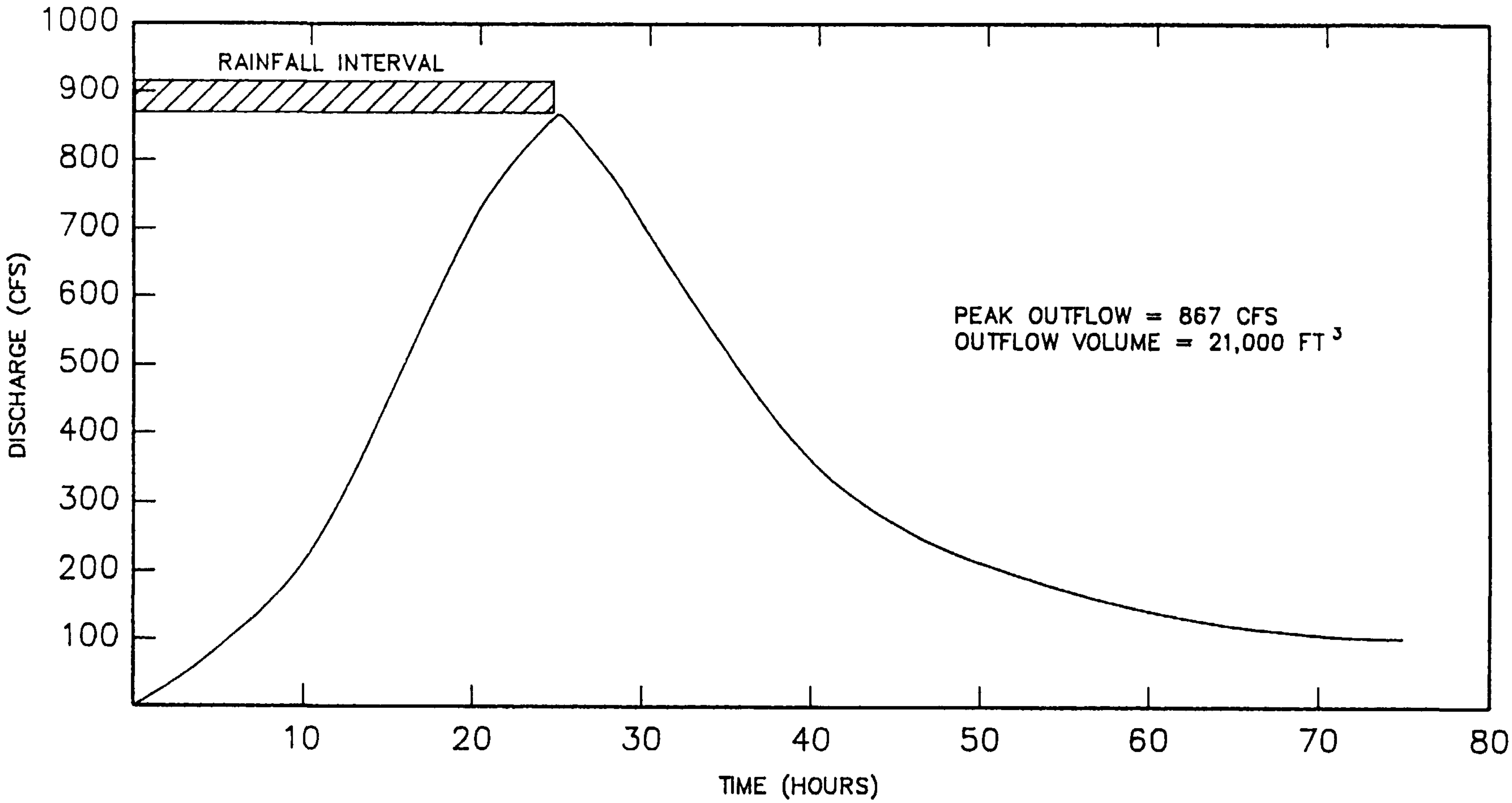


STORAGE (AC-FT)	OUTFLOW (CFS)
28	0
90	333
152	941
214	1730
276	2664

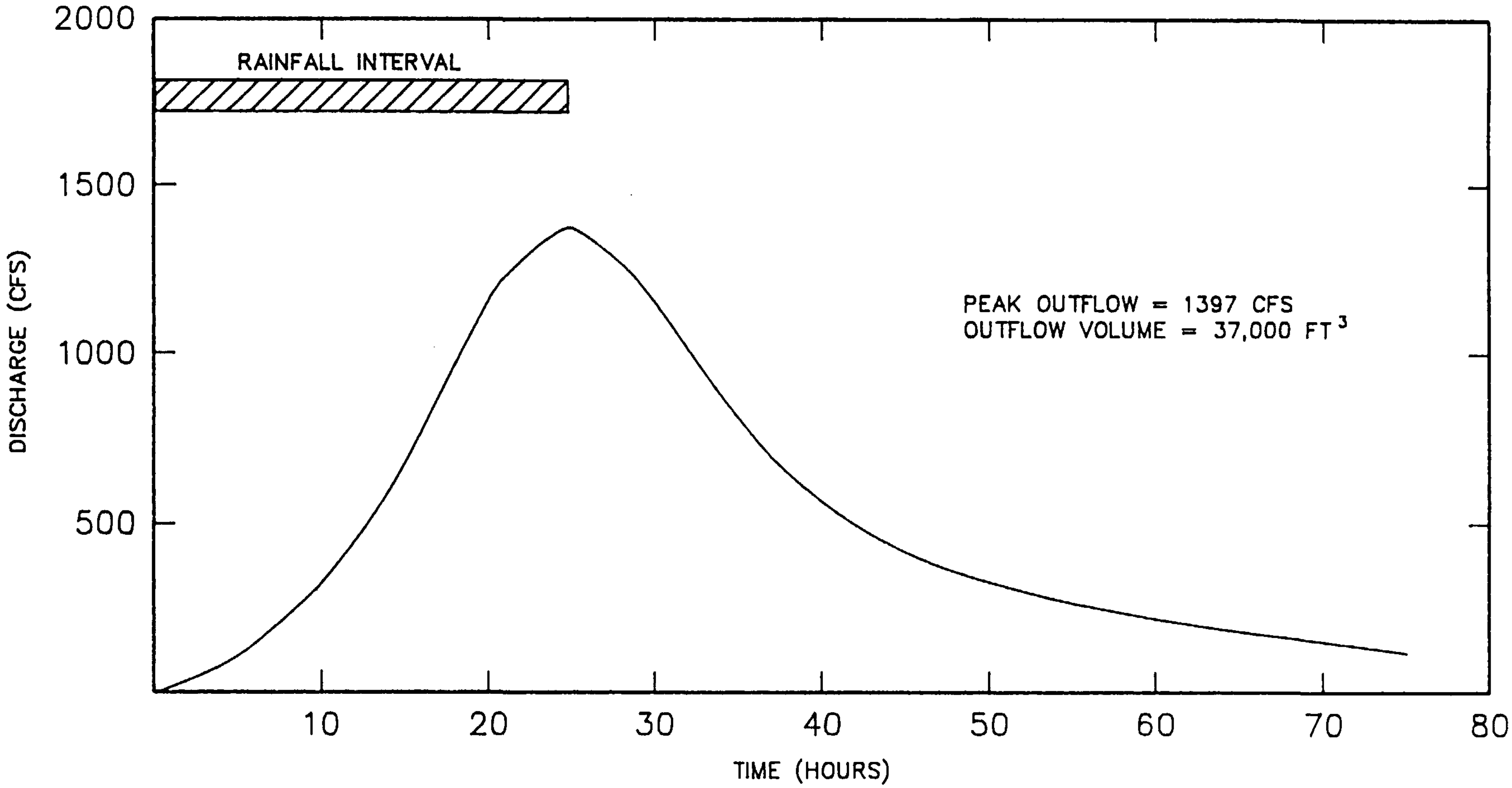
STORAGE-OUTFLOW RELATIONSHIP
SAW MILL DAM
<u>FIGURE 8</u>



OUTFLOW HYDROGRAPH
AT SAW MILL DAM
FOR 10-YEAR RAINFALL
FIGURE 9



OUTFLOW HYDROGRAPH
AT SAW MILL DAM
FOR 25-YEAR RAINFALL
FIGURE 10



OUTFLOW HYDROGRAPH
AT SAW MILL DAM
FOR 50-YEAR RAINFALL
FIGURE 11