Palisades Interstate Park Commision – Queensboro Lake

Queensboro Lake, some 5 miles from Indian Point,k serves as the year-round water supply for Bear Mountain Inn. The inn facilities include the offices of the Palisades Interstate Park commission as well as a hotel and restaurant. Three other lakes feed into Queesboro Lake through stream flow or by pip connection. Only Queensboro Lake is connected directly to the water system and no bypass is available to route water around the lake from a more distant location.

In case of contamination of Queensboro Lake, Bear Mountain Inn would be deprived of its water supply. A neighboring community, Fort Montgomery, is served entirely by individual private wells. This would seem to indicate that installation of an emergency well supply for Bear Mountain Inn would be feasible.

Stony Point Water System – Utilities and Industries

The Stony Point supply of Utilities and Industries, and investor-owned water company, serves the towns of Stony Point and Haverstraw as well as the villages of Haverstraw and West Haverstraw. Total average consumption is about 1.l8 mgd with 1.0 mgd from a surface supply and 0.8 mgd from wells.

The impounding reservoir of the surface supply of 4.5 million gallon capacity os located some 3.5 miles from Indian Point 3 Point. With contamination of this supply, the system would be left with only the wells which furnish about 45 percent of total consumptions.

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Negotiations are now under way for purchase of the Stony Point supply by the Spring Valley Water Company, an investor-owned utility serving most of the remaining areas of Rockland County. This company derives water from a well system of 13 to 15 mgd capacity and up to 7 mgd from De Forest Lake outflow some 10.8 miles from Indian Point. Plans have been completed for construction this fall of a connection between the Spring Valley Water Company system and the Stony Point system. This connection will furnish well water form the Spring Valley supply to the Stony Point network.

As far as can be ascertained from public records, the above three systems comprise the only surface water usage within a 5-mile radius of the Indian Point power plant except for industrial cooling water usage of the Hudson River. All other supplies are reported as originating in wells or from surface storage outside the 5-mile limit.

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TABLE 1

WATER STORAGE RESERVOIRS

WITHIN 15 MILE RADIUS OF INDIAN PONT

WESTCHESTER COUNTY

Code	Reservoir	User	Capacity Million Gallons	Distance Miles	Surface Acres
W-8	Indian Brook	Ossining WB.	101	6.5	17
W-18	Pocantico Lake	New Rochelle Wat. Co.	200	11.9	63
W-14	Ferguson Lake	Pocantico Hills Est.	40 *	13.5	28
W-13	Tarrytown Res.	Tarrytown	313	14.0	85
W-13	Open Res. – 2	Tarrytown	1.75 & 1.10	14.0	1
W-1	Croton Res.	New York City (See List)	65,300 (Inside 15 mi.)		4059
W-10	Whippoorwill La.	New Castle Wat. Co.	25 *	13.3	8
W-11	Byram Lake	Mt. Kisco	950	15.0	133
W-11	Open Res.	Mt. Kisco	10 *	14.0	2
W-5	Lake Shenorock	Amawalk-Shenorock WD.	90 *	11.1	16
W-6	Open Res.	Lincoln Hall School	25 *	11.9	6
W-1A	Amawalk	NYC (See List)	10,000 (Included in	11.6	588

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W-1)

W-4	Camp Field Res.	Peekskill	54	2.9	11

* Estimated

TABLE 2

WATER STORAGE RESERVOIRS

WITHIN 15 MILE RADIUS OF INDIAN PONT

PUTNAM COUNTY

Code	Reservoir	User	Capacity Million Gallons	Distance Miles	Surface Acres
P-20	Lake Mahopac	See List	5,000 *	12.7	577
P-10	Oscawanna Lake	See List	3,500 *	9.5	362
P-21	Pelton Pond	N.Y.S. Fahnestock Park	125 *	14.0	11
P-6	Cold Spring	Cold Spring	150 *	13.0	25
B-3	Cargill Res.	Beacon	160	15.0	22
B-2	Mt. Beacon Res.	Beacon	180	14.5	17
B-1	Melzingah Res.	Beacon	60	13.3	8
W-4	Wicopee	Peekskill	1,200	11.7	166

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P-5	Lake Secor	Carmel WD #5	350 *	10.8	50

* Estimated

TABLE 3

WATER STORAGE RESERVOIRS

WITHIN 15 MILE RADIUS OF INDIAN PONT

ORANGE COUNTY

Code	Reservoir	User	Capacity Million Gallons	Distance Miles	Surface Acres
0-11	Lusk Res.	U.S. M.A.	50 *	7.5	16
0-4	Intake Res.	Highland Falls	2.5	6.5	
	Bog Meadow	" "	80	8.3	43
	Little Bog	" "	4.5	7.5	2
	Jims Pond	" "	40	8.4	16
0-12	Turkey Hill La.	Palisades Int. Park	150	5.9	58
	Nawahunta La.	" "	22	6.7	16
0-20	Silvermine La.	Palisades Int. Park	456	6.0	84
	Queensboro La.	" "	56	5.0	37
0-16	Lake Stahahe	Palisades Int. Park	230	11.1	90
0-16	Summit Lake	Palisades Int. Park	110	8.3	34
	Barnes La.	Pal. Int. Pk. & U.S.M.A.	24	8.0	18
	Te'ata La.	Pal. Int. Pk.	77	7.7	32

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	Upper Twin La.	" " "	105	7.7	24
	Lower Twin La.	" " "	88	7.6	26
	Massawiepa La.	" " "	104	7.7	29
0-17	Lake Tiorati	Pal. Int. Pk., Tiorati	1,500	6.7	296
0-10	Crowmwell Lake	Woodbury	80	11.2	55
0-2	Walton Lake	Chester	300	14.6	129
0-5	Lake Mombasha	Monroe	1,750	13.0	324
0-1	Echo Lake	Arden Farms	40 *	9.5	30
0-7	Op Res.	Sterling Forest	60 *	13.7	42

ORANGE COUNTY (CONT'D.) TABLE 3 (CONT'D)

Code	Reservoir	User	Capacity Million Gallons	Distance Miles	Surface Acres
0-8&9	Tuxedo Lake	Tuxedo & Tuxedo Pk.	2,500	14.5	294
0-3	Alex Meadow	Cornwall	23	9.2	9
	Arthur's Pond	Cornwall	115	9.2	20

* Estimated

TABLE 4

WATER STORAGE RESERVOIRS

WITHIN 15 MILE RADIUS OF INDIAN PONT

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ROCKLAND COUNTY

Code	Reservoir	User	Capacity Million Gallons	Distance Miles	Surface Acres
R-14	Lake Sebago	Sebago Lake, Pal. Int. Pk	1,100	10.8	300
R-18	Lake Welch	Welch Lake	1,000	7.2	209
R-13	Breakneck Pond	Breakneck Lake, Pal. Int. Pk.	100	9.2	63
R-3	Sec. & Third Res.	Letchworth Vill.	100	8.5	40
R-1	Open Res.	Utilities & Ind.	4.5	.3.5	5
R-7	Hillburn Res.	Hillburn	1.0	14.7	4
R-6	Deforest Lake	Hackensack Wat. Co. Spring Val. Wat. Co.	5,500	10.8	960

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TABLE 5

MULTIPLE USERS OF WATER SUPPLY SYSTEMS
WITHIN 15 MILE RADIUS OF INDIAN POINT
WESTCHESTER COUNTY

New Croton Aqueduct (New York City)
- Ossining Water Board
- Sing Sing Prison
- Village of North Tarrytown
- New Rochelle Water Company
  - Village of Bronxville
  - Town of Eastchester
  - Village of North Pelham
  - Village of Pelham
  - Village of Pelham Manor
  - Village of Tuckahoe
- Village of Irvington
- Village of Briarcliff Manor
- New Castle Water District #1
- Village of Tarrytown

Old Croton Aqueduct (New York City)
- Ossining Water Board
  - Village of Ossining
  - Town of Ossining
  - Sing Sing Prison

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TABLE 5 (CONT'D)

Kensico Reservoir (New York City)

- City of White Plains
- North Castle District #1
- Westchester Joint Water Works No. 1
  - Village of Mamaroneck
  - Town of Harrison
  - Town of Mamaroneck
  - City of Rye
  - City of New Rochelle
  - Village of Larchmont
  - Village of Scarsdale
  - Village of Pelham Manor
- Harrison District #1

Catskill Aqueduct (New York City)

- Grasslands (Westchester Co.)
- Hawthorne Improvement District
  - Hawthorne
  - Town of Mt. Pleasant
- Valhalla W D
  - Valhalla
  - Town of Mt. Pleasant
- City of Yonkers
- Village of Scarsdale
- New Rochelle Wat. Co (same as Pocantico Lake)

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TABLE 5 (CONT'D.)

Amawalk Reservoir (New York City)

Yorktown W S D D

Amawalk Heights W D

Town of Somers

Town of Yorktown (13 Water Districts)

Peekskill System (City of Peekskill)

City of Peekskill

Village of Buchanan

Town of Cortlandt

Indian Brook Reservoir (Ossining Water Board)

Village of Ossining

Town of Ossining

Sing Sing Prison

Whippoorwill Lake (New Castle Water Co.)

Town of New Castle (Part)

Town of North Castle (Part)

Pocantico Lake (New Rochelle Water Co.)

Village of Ardsley

Village of Dobbs Ferry

Town of Greenburgh

Village of Hastings

Village of Scarsdale

Village of Eastchester

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TABLE 5 (CONT'D.)

Tarrytown Reservoir

Village of Tarrytown

Glenville W D

Town of Greenburgh

Eastview

Town of Mount Pleasant

Village of North Tarrytown

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TABLE 6

MULTIPLE USERS OF WATER SUPPLY SYSTEMS

WITHIN 15 MILE RADIUS OF INDIAN POINT

PUTNAM COUNTY

Lake Oscawanna

Hiawatha Improvement Co.

Hilltop W D

Wildwood Knolls W D

Oscawanna Lake (Private Homes)

Lake Mahopac

Lake Gardens

Lake Mahopac Woods

Mahopac Hills

Mahopac Old Village

Lake Mahopac (Private Homes)

Lake Mahopac Ridge

Lake View Park

Mahopac School

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IPEC00034822

TABLE 7

MULTIPLE USERS OF WATER SUPPLY SYSTEMS
WITHIN 15 MILE RADIUS OF INDIAN POINT
ROCKLAND COUNTY

De Forest Lake

- Hackensack Water Co.
- Spring Valley Water Co.
  - Town of Clarkstown (Part)
  - Town or Ramapo
  - Town of Orangetown
- Nyack
  - Village of Nyack
  - Village of South Nyack
  - Upper Nyack
  - Town of Clarkstown (Part)

Stony Point Supply (Utilities and Industries)

- Town of Stony Point
- Town of Haverstraw
- Village of West Haverstraw

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IPEC00034823

TRANSPORT OF CONTAMINANTS

IN THE HUDSON RIVER

ABOVE INDIAN POINT STATION

QUIRK, LAWLER AND MATUSKY ENGINEERS
Environmental Science and Engineering Consultants
New York, New York

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IPEC00034824

BASIS FOR ANALYSIS

Transport of any substance in a tidal estuary is governed by the Law of Conservation of Mass. Figure 1 illustrates the application of this law in an estuary. After discharge to the estuary, waste particles are carried downstream, in the movement of upland runoff toward the ocean. This phenomenon is known as convection. The rate of convective mass transport across any river section is equal to the product of fresh water runoff, Q, and contaminant concentration, c.

Besides convection, particles are transported in an estuary by longitudinal mixing. Longitudinal mixing, or dispersion, is a complex function of reversing tidal currents and salinity-induced circulation. Dispersive transport occurs only in the presence of a concentration gradient of the material being transported. The rate of dispersive transport is equal to the product of the dispersion coefficient, E, and the negative of the longitudinal concentration gradient, dc/dx. The dispersion coefficient, E, is a measure of the estuary's ability to transport material in the presence of a concentration gradient, and is a quantitative function of tidal current and salinity-induced circulation.

The concentration profile in Figure 1 indicates how convection and dispersion distribute estuarine contaminants. Since only contaminants that decay or, at best, are conserved, are being considered, the maximum containment concentration must occur at the point of introduction of the contaminant to the estuary. In the case of saline contamination, the salt is introduced at the mouth of the estuary so that the maximum salinity occurs here; in the case of discharge of radioactive contaminants at Indian Point, the maximum concentration of radioactivity will exist in this vicinity, as shown on Figure 1.

The concentration in the region downstream of the point of discharge, (x = 0), decays less rapidly than does its counterpart in the upstream region. This is so because in the downstream region, dispersion, in moving material in the direction of decreasing concentration, aids convection. More material is transported downstream than upstream, so, at the same absolute distance from the point of discharge, the upstream concentration is lower than the downstream concentration.

A mass balance over the incremental volume, AΔx, in Figures 1 is written:

Inflow – Outflow + Production = Accumulation . . . . . . (1)

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S-38 Figure 1

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Algebraic summation of the individual contributions, shown in Figure 1 to Equation 1 gives:

$$\left[Qc - EA\frac{dc}{dx}\right]_x - \left[Qc - EA\frac{dc}{dx}\right]_{x+\Delta x} - KCA\Delta x = \frac{d}{dt}\left[CA\Delta x\right] \quad \ldots\ldots (2)$$

in which:

c = contaminant concentration, ML^{-3}
x = distance along longitudinal axis of the estuary, L
t = time, T
A = cross-sectional area of the estuary, L^2
Q = fresh water flow (upland runoff), L^3T^{-1}
E = longitudinal dispersion coefficient, L^2T^{-1}
K = first order decay constant, T^{-1}

The production, or in this case, decay, term is the rate at which material is produced or consumed by physical, chemical, biochemical or nuclear reaction within the volume element.

For decay according to first order kinetics, the usual kinetics of radio-active decay, this rate of consumption of contaminant is equal to the product of the unit rate, Kc, times the volume, $A\Delta x$, within which the reaction is taking place.

The accumulation term completes the inventory by accounting for the net rate of increase or decrease of material upon summation of the rate of inflow, outflow and production. This is equal to the time rate of change of total con-tamination mass within the reactor volume, $A\Delta x$.

The parameters, Q, A, E and K, in most estuaries are functions of space and time. To avoid tenuous mathematical complexity, these parameters are often considered to be constants. This approach, justification of which appears in a later section of the report, has been selected for the analysis used in this report. For the case of constant Q, E, A and K, Equation 2 rearranges to:

$$E\left[\frac{\left.\frac{dc}{dx}\right|_{x+\Delta x} - \left.\frac{dc}{dx}\right|_x}{\Delta x}\right] - \frac{Q}{A}\left[\frac{c|x+\Delta x - c|}{\Delta x}x\right] - Kc = \frac{dc}{dt} \quad \ldots\ldots 3$$

The bracketed terms are average rates of change with respect to x. The

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limit of Equation 3, as Δx approaches zero, is as follows:

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$$E\frac{d^2c}{dx^2} - U\frac{dc}{dx} - Kc = \frac{dc}{dt} \quad \ldots\ldots (4)$$

U is equal to Q/A and is the average fresh water velocity. Equation 4 is a linear partial differential equation in x an t and is often referred to as the convection-diffusion equation for non-conservative substances. It has been selected as the defining equation for all subsequent analyses presented in this report.

At this juncture, it is important to note that the concentration, c, is actually a tidal smoothed, area averaged concentration. This means that rather than attempt to define local behavior at any point within a cross-section and during a tidal cycle, the analyst looks at the average concentration over an entire cross-section over a full tidal cycle. Justification of this procedure is given by Kent (1), Harleman and Holley (2), and Lawler (3).

This justification proceeds by starting with the equation of continuity of a single chemical specie (4), in which contaminant concentration is a function of three space dimensions and real time. Dependence on the lateral and vertical space coordinates is replaced by dependence on total cross-sectional area by integrating over the total width and depth. The resulting equation is then integrated over a tidal cycle and change with respect to real time replaced by change with respect to tidal cycle units of time.

In the course of these integrations, several new terms are generated, all of which contribute to the dispersion phenomenon. These are eventually replaced by the overall dispersion flux, $E\frac{dc}{dx}$.

Once contaminants are dispersed over the river channel, the various concentrations at specific points within the cross-section and tidal cycle can be expected to be less than 20% of the tidal smoothed, area averaged value. Figures 2 through 7 illustrate this for salt. The actual variation of salinity across various cross-sections within the reach between Indian Point and Chelsea is Show on Figures, 2, 4 and 6. Figures 3, 5 and 7 show the sinusoidal variation of the area averaged salinity at these sections over a tidal cycle, as well as a linearized plot of this variation.

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S-41 Figure 2

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S-42 Figure 3

S-43 Figure 4

S-44 Figure 5

S-45 Figure 6

S-46 Figure 7

II. SELECTION OF NUMERICAL VALUES OF PARAMETERS

Numerical values of the parameters E, U and K, which appear in the defining differential equation and therefore control the distribution of any contamination in the estuary, must be chosen for the Hudson River.

1. FRESH WATER DISCHARGE

Fresh water velocity, U, is obtained by dividing fresh water discharge by the river cross-sectional area, A. Fresh water flow into the Hudson is measured at Green Island, at mile point 152, where the tributary drainage area totals 8090 square miles. The drainage area of the Hudson Basin, tributary to the entire River, is approximately 13,370 square miles. Over 95% of this area is located north of Indian Point. Because of the inability to measure directly fresh water flow in tidal waters, the Green Island gage issued to establish lower River discharges. The ratio of tributary drainage areas between Indian Point and the gage is 1.57. Analysis of data developed by the United States Geological Survey (USGS) indicates a most probable value for yield factor of 1.22. All values of lower River flow referred to in this report were established using this ratio, i.e., lower River flow is equal to Green Island gaged flow times 1.22.

The pattern of the long-term monthly flows, shown in Figure 8, is indicative of the general variation of River discharge. During the months of March through May, the flow averaged 29,000 cfs or almost 3.5 times the average discharge during the months from June through October. This is equivalent to the statement that the volume of fresh water discharged during the spring months is in exces of twice the volume discharge during the subsequent five-month period.

Figure 1 and Equation 4 indicate that as fresh water velocity decreases, given a fixed value of the longitudinal dispersion coefficient, the dispersion effect increases. Therefore, contaminant concentration values in the region above Indian Point can be expected to increase as flow decreses. Furthermore, due to increased salinity intrusion during periods of low fresh water flow, the longitudinal dispersion coefficient, which is strongly dependent on salinity-induced circulation, can be expected to increase in the upper region of the River. For these reasons, analysis of the effect of pollutants on the River require that drought flows by selected in assigning values of U.

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S-48 Figure 8

IPEC00034828

Figure 9 shows a statistical analysis of Hudson River drought flows for the years 1918 through 1964. For drought durations of one week (seven consecutive days), and one month, a plot of flow versus the percent of the time such flow can be expected to occur is given. For example, Figure 9 indicates, for a duration of one week, a flow of 2630 cfs can be expected to occur 5% of the time or once in 20 years.

It should be noted that the response of the Hudson to area-wide droughts is significantly different from that of individual, smaller-sized basings in the region. The difference can be attributed to the size and number of sub-drainage areas within the overall basin and the degree of regulation obtained from up-River storage facilities, such as the Sacandaga and Indian Lake reservoirs.

2. CROSS-SECTIONAL AREA

Figure 10 shows 10 shows the variation of cross-sectional area with distance above the Battery. Variation is erratic and as such is not amenable to simple mathematical description; i.e., as an elementary function of distance. Between Indian Point and Chelsea, the area varies from a minimum of 120,000 square feet just north of Bear Mountain Bridge to a maximum of 175,000 square feet at the mouth of Newburgh Bay. The average area over this 22 mile river reach is 140,000 square feet; this number has been selected as the value of the constant parameter, A, in Equation 3.

3. LONGITUDINAL DISPERSION. COEFFICIENT

The value of the longitudinal dispersion coefficient at any point within the salt-intruded reach of the River can be conveniently obtained by analysis of salinity profiles. The limiting form of Equation 2 for the case of a conservative substance such as salt, and non-constant values of Q, A and E, is:

$$\frac{1}{A}\frac{d}{dx}\left[EA\frac{dc}{dx}-Qc\right]=\frac{dc}{dt} \qquad \ldots\ldots (5)$$

If the variation of salinity with x and t is know, the derivatives $\frac{\partial c}{\partial x}$ and $\frac{\partial c}{\partial t}$ may be obtained graphically or numerically. Equation 5 can then be used to compute the value of E at any point within the saline reach of the River.

This procedure requires that a number of profiles be available so that the time derivative, $\frac{\partial c}{\partial t}$, can be computed and also requires that the value of Q, now a time and distance dependent function, controlling the intrusion, be known. This

S-49 5

S-50 Figure 9

IPEC00034829

S-51 Figure 10

latter requirement poses some difficult in evaluating Hudson River dispersion. Fresh water flow can only be measured at Green Island, above the tidal region, and the attenuating effect of tidal mixing on time-variable flows is not known.

These difficulties have been avoided by recognizing that drought flows in the Hudson remain relatively constant for extended periods of time; Q, and therefore U, are known and the steady Q gives rise to steady salinity profiles during these periods. Under these conditions, the net flux of salt in the River must be zero since there is no sink or source of salt within the estuary. Equation 5 then reduces to:

$$E \frac{dc}{dx} - Uc = 0 \qquad \ldots\ldots (6)$$

Rearrangement of Equation 6 yields a solution for the dispersion coefficient.

$$E = U \left[2.303 \quad \frac{d \log c}{dx} \right]^{-1} \qquad \ldots\ldots (7)$$

Numerical values of $\frac{d \log c}{dx}$ may be obtained by graphical differentiation of a semi-logarithmic plot of salinity versus distance. U(x) is equal to the flow associated with that profile, divided by the area, A(x), at the point in question. Typical steady state salinity profiles are shown in Figure 11. Values of E, computed as described above, are shown in Figure 12 for these and several other drought profiles.

Figure 12 indicates that the dispersion coefficient at some points may increase as flow decreases whereas, at other points, the reverse may occur. For example, at mile point 20, the value of E, during the 1964 drought flow of 4100 cfs, was 12,000 sf/sec and, during the 1959 drought flow of 8700 cfs, was 6000 sf/sec. On the other hand, at mile point 50, E in 1964 was 4200 sf/sec and, in 1959, was 5000 sf/sec.

These phenomena can be explained in terms of the mechanisms contributing to longitudinal dispersion. In the lower part of the saline region, under drought conditions (less than 12,000 cfs), salinity-induced circulation, which depends strongly on the salt concentration, is the predominating mechanism, whereas, toward the end of the intrusion, this saline effect is less predominant and also less variable. The relative contribution of fresh water flow to the dispersion characteristics of the River increase as the absolute contribution of the salinity

S-52 6

S-53 Figure 11

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IPEC00034830

S-54 Figure 12

decreases. Thus, increases in fresh water flow can, under some conditions, outweigh the corresponding decrease in salinity, the net effect being an increase in the dispersion coefficient.

Under other conditions, the reverse is true and a decrease in the dispersion coefficient in the presence of an increased flow will be observed. Details for these phenomena and a quantitative method for the prediction of E(x) in the Hudson River as a function of flow are more fully discussed in previous reports (5), (6).

The determination of E as a function of x has been presented to justify the use and selection of constant values of E in this report. A choice of E equal to the maximum value of E(x) within the reach between Indian Point and Chelsea will result in a conservative analysis for the following reasons:

(1) As Chelsea is approached, the true value of E will fall below this maximum, causing the actual contaminant concentration to be lower than that predicted by constant parameter analysis.

(2) The predicted downstream flux will be less than the actual downstream flux because the true E values, in this region, are larger than the constant E. Thus, the predicted value of the fraction of total contamination discharge moving upstream will be greater than the actual value of this fraction.

These qualitative statements can be seen more clearly by reference to Figure 1.

Figure 12 indicates that maximum E in the reach between Indian Point and Chelsea occurs between mile points 45 and 50. Accordingly, the values of E for this analysis have been selected by obtaining the average E between mile points 45 and 50 for any given flow. A second choice of E has been made by obtaining the average between mile point 43 and 65 (Indian Point and Chelsea).

The average value of E over a finite length of River is obtained by application of the mean value theorem for derivatives to Equation 7. This yields:

$$\left[\frac{E}{U}\right]_{AVG} = \left[2.303\,\frac{\Delta \log c}{\Delta k}\right]^{-1} \qquad \ldots\ldots (8)$$

$$E_{AVG}\ 8\ U_{AVG}\left[2.303\,\frac{\Delta \log c}{\Delta k}\right]^{-1} \qquad \ldots\ldots (9)$$

A correlation of all available Hudson River salinity and flow data is shown on Figure 13. Values of E used in this report have been computed by application

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S-56 Figure 13

of Equation 9 to these data. For example, at a flow of 4000 cfs, the computa-

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tion for average E between mile points 43 and 65 is:

$$E = \left[\frac{4000}{141,300}\right]\left[\frac{2.303\,(\log 7000 - \log 2200)}{[-43(-65)]5280}\right]^{-1}$$

= 2830 sq. ft/sec
= 8.74 sq. mile/day

Correspondingly, for the same flow, the average E between mile points 45 and 50 is:

$$E = \left[\frac{4000}{123,500}\right]\left[\frac{2.303\,(\log 6500 - \log 5400)}{-45-(-50)}\right]^{-1}$$

= 4640 sq. ft/sec
= 14.3 sq. mile/day

Figure 14 shows the variation, with flow, of average E, computed by Equation 9 as shown above.

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S-58 Figure 14

III. EFFECT OF CONTINUOUS DISCHARGE ON CHELSEA INTAKE

This section analyzes the effect of a continuous discharge from Indian Point on water drawoff at Chelsea and is subdivided as follows:

1. A steady state of equilibrium analysis
2. A transient analysis or approach to steady state

1. ANALYTICAL DEVELOPMENT FOR STEADY STATE CONDITION

Figure 15 depicts the problem. The defining differential equation is given by Equation 4. Since this equation does not include discharge at Indian Point or drawoff at Chelsea, it will not define behavior across these two planes. For these reasons, the Hudson is divided into three regions, one above Chelsea, one between Chelsea and Indian Point and one below Indian Point. A solution for each region is obtained by application of proper boundary conditions to the general solution of Equation 4.

The steady state form of Equation 4 is:

$$\frac{Ed^2c}{dx^2} - \frac{Udc}{dx} - Kc = 0 \qquad \ldots\ldots (10)$$

The general solution of this second order, linear, ordinary differential equation is:

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in whic h

$$c = C_1e^{jx} + C_2e^{kx} \quad \ldots\ldots (11)$$

$$j = \frac{U + \sqrt{U^2 + 4KE}}{2E}$$

$$k = \frac{U - \sqrt{U^2 + 4KE}}{2E}$$

C_1, C_2 = arbitrary constants

Equation 11 is the form of the general solution for each of the three regions. Designating River velocity above Chelsea as U_1 and below Chelsea as U_2, the general solution in each of the three reaches is written:

$$c_I = C_1e^{j_1x} + C_2e^{k_1x} \quad \ldots\ldots (11a)$$

$$c_{II} = C_3e^{j_2x} + C_4e^{k_2x} \quad \ldots\ldots (11b)$$

$$c_{III} = C_5e^{j_2x} + C_6e^{k_2x} \quad \ldots\ldots (11c)$$

S-59 9

S-60 Figure 15

in which

$$\left.\begin{matrix} j_1 \\ k_1 \end{matrix}\right\} = \frac{U_1 \pm \sqrt{U_1^2 + 4KE}}{2E}$$

$$\left.\begin{matrix} j_2 \\ k_2 \end{matrix}\right\} = \frac{U_2 \pm \sqrt{U_2^2 + 4KE}}{2E}$$

To evaluate the six arbitrary constants, six boundary conditions are necessary. These are developed as follows:

1. The contaminant can be expected to reach negligible concentrations before passing out of the estuary into the ocean. This is not due to any diluting effect of the ocean, but rather because the distance between Indian Point and New York Harbor is sufficiently long to permit virtually complete disappearance of contaminant originating at Indian Point by the time this contaminant reaches the Harbor. This means that the downstream end of the estuary has no influence on contaminant distribution in the estuary. The estuary may therefore be considered to be

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infinitely long and the first boundary condition may be written:

$$C_{III}\Big|_{x=\infty} = 0 \qquad \text{BC \#1}$$

2. In the upsteam region, convection opposes dispersion and the distance from Indian Point to the upper end of the estuary is even greater than the distance from Indian Point to the lower end. For these reasons, the statements concerning BC #1 are even more applicable here and the second boundary condition is written:

$$C_I\Big|_{x=-\infty} = 0 \qquad \text{BC \#2}$$

3. Although Equation 10 does not define behavior across sections at Indian Point and Chelsea, and discontinuity in some derivative will occur at these points, the contaminant concentration itself is continuous and therefore single-valued at all points. This fact gives rise to the third and forth boundary conditions:

$$C_I\Big|_{x=a} = C_{II}\Big|_{x=a} \qquad \text{BC \#3}$$

$$C_{II}\Big|_{x=0} = C_{III}\Big|_{x=0} \qquad \text{BC \#4}$$

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4. To describe the behavior at the boundary between regions II and III, a material balance about the plane of discharge is constructed as shown on Figure 15. The steady state material balance is written:

$$\left[Q_2 c_{II} - EA\frac{dc_{II}}{dx}\right]_{x=\frac{\Delta x}{2}} + {}^{qr}c_{II} + {}^{q}IP\ {}^{c}IP - {}^{qr}c_{II} - \left[Q_3 c_{III} - EA\frac{dc_{III}}{dx}\right]_{x=\frac{\Delta x}{2}} = K\bar{c}A\Delta x \qquad \ldots\ldots (12)$$

in which Q_2 = River flow above Indian Point

${}^{q}IP$ = volumetric discharge from plant

Q_3 = $Q_2 + {}^{q}IP$ = net River flow below Indian Point

${}^{c}IP$ = concentration of plant contaminants prior to introduction to recirculating flow

qr = recirculating River flow through plant

Simplifying Equation 12 and taking the limit as $\Delta x \rightarrow 0$ yields:

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$$ {}^{q}IP\left[{}^{c}IP - {}^{c}II\right]_{x=0} - EA\left[\frac{{}^{dc}II}{dx} - \frac{{}^{dc}III}{dx}\right]_{x=0} \quad \text{. . . . . . (13)} $$

In reality, virtually all of the flow from Indian Point is recirculated from the River. Therefore ${}^{q}IP <<< Q_2$, and for all practical purposes $Q_2 = Q_3$. Call (${}^{q}IP \cdot {}^{c}IP$), W, the continuous load on the River, take the limit of Equation 12 and obtain for the fifth boundary condition:

$$ W = EA\left[\frac{{}^{dc}II}{dx} - \frac{{}^{dc}III}{dx}\right] \quad \text{BC \#5} $$

Notice that the first derivatives of the contaminant concentration are discontinuous at the point of discharge. This behavior is shown clearly by the contaminant profile in Figure 1.

5. The behavior at the boundary between regions I and II is developed similarly. A material balance about the plane of drawoff is constructed in Figure 15 and is written:

$$ \left[Q_1 c_I - EA\frac{dc_I}{dx}\right]_{a-\frac{\Delta x}{2}} - q_c c_a - \left[Q_2 c_{II} - EA\frac{{}^{dc}II}{dx}\right]_{a+\frac{\Delta x}{2}} - K\bar{c}A\Delta x = 0 \quad \text{. . . . . . (14)} $$

in which Q_1 = River flow above Chelsea

q_c = drawoff at Chelsea

Q_2 = Q_1 - qc River flow below Chelsea

c_a = contaminant concentration at Chelsea

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As Δx approaches zero, ${}^{c}I = {}^{c}II = {}^{c}a$ and Equation 14 becomes:

$$ \left.\frac{dc_I}{d_x}\right|_{x=a} = \left.\frac{dc_{II}}{dx}\right|_{x=a} \quad \text{BC \#6} $$

Notice, in the case of drawoff from the River, the concentration of contaminant in the withdrawn flow is identical to the concentration of contaminant in the River at the point of drawoff. In the case of discharge to the River, the contaminant concentration in the discharged flow is much larger than in the River at this point. Thus, in the case of drawoff, the defining differential equation does not hold across the point of drawoff because the River flow is changed, while in the case of discharge, it does not hold because of the imposition of a net load on the River.

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Substitution of Equations 11a, b, c into these six boundary conditions yields values for the six arbitrary constants. The explicit solutions for contaminant concentration becomes:

$$c_I = \frac{W e(j_2 - j_1)}{AE(j_1 - k_2)} a + j_1 x \quad \text{. . . . . . (15)}$$

$$c_{II} = \frac{W}{AE}\left[\frac{e^{j_2 x}}{j_2 - k_2} + \frac{(j_2 - j_1) e^{(j_2 - k_2)\ a + k_2 x}}{(j_2 - k_2)\ (j_1 - k_2)}\right] \quad \text{. . . . . . (16)}$$

$$c_{III} = \frac{W}{AE}\left[\frac{(j_1 - k_2) + (j_2 - j_1) e^{(j_2 - k_2) a}}{(j_2 - k_2)\ (j_1 - k_2)}\right] e^{k_2 x} \quad \text{. . . . . . (17)}$$

For the case of no decay, K = 0, and:

$$j_1 = \frac{U_1}{E}$$

$$j_2 = \frac{U_2}{E}$$

$$k_2 = 0$$

For this case, the concentrations at x = 0 (Indian Point) and at x = a (Chelsea) are, respectively:

$$C_O = \frac{W}{Q_1}\left[\frac{U_1}{U_2}(1 - e\frac{U_2.a}{E}) + e\frac{U_2.a}{E}\right] \quad \text{. . . . . . . (18)}$$

$$C_a = \frac{W}{Q_1} e\frac{U_2.a}{E} \quad \text{. . . . . . . (19)}$$

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For no drawoff at Chelsea, Equation 18 and 19 reduce to the simple case of discharge of a conservative contaminant at x = 0; i.e., $U_1 = U_2$, $Q_1 = Q_2 = Q$ and:

$$C_o = \frac{W}{Q} \qquad \text{. . . . . . . (20)}$$

$$C_a = \frac{W}{Q} e^{\frac{U}{E}a} \qquad \text{. . . . . . . (21)}$$

The ratio of concentration at Chelsea to concentration at Indian Point is:

$$\frac{C_{\text{Chelsea}}}{C_{\text{Indian Point}}} = \frac{C_a}{C_o} = \frac{1}{1 + \frac{U_1}{U_2}\left(e^{\frac{U_2 a}{E}} - 1\right)} \qquad \text{. . . . . . . (22)}$$

For the case of no drawoff at Chelsea, this reduces to $e^{\frac{U_a}{E}}$.

2. TRANSIENT CONDITION

Subsequent to commencement of a steady, continuous discharge, a time lag occurs before steady state profiles, described by Equation 15 through 21, are established. To determine concentration build-up as a function of time as well as of space, an unsteady state analysis of Equation 4 must be made. Such an analysis has been judged necessary in this study, not only to establish the rapidity of approach to steady state, but also to serve as a basis for a computer solution of the maximum permissible continuous release when radioactive decay is taken into account (7).

Analysis shows that the 100 mgd Chelsea draw has only a slight effect on equilibrium concentration at Chelsea. The same can be expected during the approach to equilibrium so that transient analysis without consideration of drawoff was used. This has been developed previously in considerable detail (8). The final equation for the distribution of contaminant upstream of the point of waste discharge is:

$$C_I(x,t) = \frac{W}{2Q\sqrt{1+\frac{4KE}{U^2}}} \left[E\,x\,P\left[\frac{U}{2E}\left(1+\sqrt{1+\frac{4KE}{U^2}}\right)x\right] \cdot ERFC\left(\frac{-x}{\sqrt{4Et}} - \sqrt{\frac{U^2+4KE}{4E}t}\right) - EXP\left[\frac{U}{2E}\left(1-\sqrt{1+\frac{4KE}{U^2}}\right)X\right] \cdot ERFC\left(\frac{-x}{\sqrt{4Et}} + \sqrt{\frac{U^2+4KE}{4E}t}\right) t \right] \qquad \text{. . . . . (23)}$$

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The corresponding steady state solution, given by Equation 16 when $i_1 = i_2$ (no drawoff, $U_1 = U_2$), is:

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$$ {}^{c}I\,(x) \;=\; \frac{W}{Q\sqrt{1+\frac{4KE}{U^2}}}\,e^{\frac{U}{2E}\left[1+1\sqrt{+\frac{4KE}{U^2}}\right]x} \qquad \ldots\ldots(24) $$

The ratio of the transient response to the equilibrium response is:

$$ \frac{{}^{c}I(x,t)}{{}^{c}I(x,\infty)} \;=\; \frac{1}{2}\left[ERFC\left[\frac{-x}{\sqrt{4Et}}-\sqrt{\frac{U^2+4KE}{4e}t}\right] - EXP\left[-\frac{U}{E}\sqrt{1+\quad\frac{4KE}{U^2}x}\right].\,ERFC\left[\frac{-x}{\sqrt{4Et}}+\sqrt{\frac{U^2+4KE}{4E}t}\right]l\right] \qquad \ldots\ldots(25) $$

For the case of no decay, Equation 25 reduces to:

$$ \frac{{}^{c}I(x,t)}{{}^{c}I(x,\infty)} = \frac{1}{2}\left[ERFC\left[\frac{-x}{\sqrt{4Et}}-\sqrt{\frac{U^2t}{4E}}\right] - EXP\left[-\frac{U}{E}x\right]\;.\;ERFC\left[\frac{-x}{\sqrt{4Et}}+\sqrt{\frac{U^2t}{4E}}\right]\right] \qquad \ldots\ldots(26) $$

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IV. INSTANTANEOUS RELEASE

This case represents the condition of an accidental spill of radioactive contaminant to the River. A slug of material is released over a short time interval, which for practical purposes can be assumed to be instantaneous. The object is to determine the time of appearance of and the value of maximum concentration at Chelsea.

1. PREVIOUS STUDIES

Studies of the effect of instantaneous release of conservative substances at Indian Point were conducted on the Hudson River Model at the Waterways Experiment Station, Vicksburg, Mississippi, circa 1962 (9). Figure 16 is a reproduction of Plate 30, reference 9, and shows the distribution of conservative dye, released over a single tidal cycle at Indian Point, for a River flow of 12,000 cfs. Notice that the spread is asymmetrical, favoring the downstream direction. This documents the variable nature of the dispersion coefficient and the fact that it increases in the downstream direction, as shown previously in Figure 12. A more detailed analysis of these data, in terms of the mechanisms which cause E to vary, may be found in references 5 and 6.

The occurrence of maximum upstream E values between mile points 45 and 50 is demonstrated by Figure 16. Within this reach a decreasing slope, particularly for tidal cycles 15 through 30, can be seen, indicative of greater spreading or longitudinal dispersion. For a flow of 12,000 cfs, salinity is well below mile point 55, the approximate location of the mouth of Newburgh Bay, and therefore not available to induce circulation, i.e., increase E. Below this

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point the channel narrows, the velocity is higher, and the downstream-directed convection strong. However, the rate of tidal energy dissipation, besides salinity-induced circulation, the other major cause of dispersion, is relatively high and dispersion is enhanced and the dye moves up this far.

Tidal energy dissipation in the larger expanse of the bay is relatively low; without salinity-induced circulation present, dispersion becomes negligible and is overpowered by downstream-directed convection. Thus, at the flow of 12,000 cfs, dye does not appear above mile point 55.

At drought flows, of course, salinity is present for north of this point; significant dispersion, at these times, can be expected in the vicinity of Chelsea.

S-66 15

S-67 Figure 16

2. <u>CONSTANT PARAMETER ANALYSIS</u>

Draw off at Chelsea is not considered; the results of the continuous analysis indicate this is not a serious omission. Detailed analysis of the instantaneous release for constant River characteristics has been developed previously (8); a brief outline of the development is given here.

The defining differential equation is Equation 4. The initial and boundary conditions are developed as shown on Page 9 and are:

Initial Condition: $C\big|_{t=0} = 0, \ -\infty \le x \le \infty$

Boundary Conditions #1, #2: $C\big|_{x=\pm\infty} = 0$, all t

Boundary Conditions #3 $C\big|_{x_- \to 0} = C\big|_{x_+ \to 0}$ all t

Boundary Conditions #4 $AE\left[\dfrac{dc}{dx}\bigg|_{x_- \to 0} - \dfrac{dc}{dx}\bigg|_{x_+ \to 0}\right] = f(t), \ t>0$

f(t) in B. C. #4 is the delta function and is written:

$$f(t) = \begin{cases} \dfrac{M}{\Delta t}, & 0 < t < \Delta t \\ 0, & \Delta t < t \le \infty \end{cases}$$

in which M = Mass of contaminant released

The Laplace Transform Solution of Equation 4, subject to the above conditions, yields:

$$C(x,t) = \frac{M}{2A\sqrt{\pi E t}} e^{-\frac{(x-Ut)^2}{4Et} - Kt} \quad \ldots\ldots \ (27)$$

To compute the dilution effect only, set K – 0. Equation 27 becomes:

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$$C(x,t)=\frac{M}{2A\sqrt{\pi Et}}e-\frac{(x-Ut)^2}{4Et} \qquad \ldots\ldots (28)$$

The maximum value of C(x,t) at a given x is desired. Differentiate Equation 28 with respect to t and equate the results to zero to determine the time at which the maximum concentration occurs. This procedure yields:

$$t_{critical}=\frac{E}{U^2}\left[-1+\sqrt{1+\frac{U^2x^2}{E^2}}\right] \qquad \ldots\ldots (29)$$

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VIII. REFERENCES

1. Kent, R. "Diffusion in a Sectionally Homogeneous Estuary." Paper No. 3392, Transactions, American Society of Civil Engineers, Vol. 128, Part III (1963).

2. Holley, E. R., Jr. and Harleman, D. R. F. "Dispersion of Pollutants in Estuary Type Flows." Report No 74, Hydrodynamics Laboratory, Massachusetts Institute of Technology, Cambridge, Mass. (January 1965).

3. Lawler, J. P. "Mathematical Analysis of Estuarine Pollution," in "Engineering Aspects of Marine Waste Disposal." Environmental Health Sciences and Engineering Training Course Manual, R. A. Taft Sanitary Engineering Center, Cincinnati, Ohio (1965).

4. Bird, R. B., Stewart, W. E. and Lightfoot, E. N. "Transport Phenomena." John Wiley and Sons, New York, N. Y. (1960).

5. "Evaluation of Waste Diffusion in the Hudson River at Peekskill." Report to Standard Brands, Incorporated by Quirk, Lawler and Matusky, Water Resources Engineers, New York, N. Y. (December 1964).

6. "Hudson River Dispersion Characteristics." Progress Report to Consolidated Edison Company of New York, Incorporated by Quirk, Lawer and Matusky, Water Resources Engineers, New York, N. Y. (October 1965)

7. "Indian Point Station – River Diffusion Study." Consolidated Edison Company of New York, Incorporated – Memorandum from C. A. Larson to W. J. Cahill, Jr. (April 26, 1966).

8. Lawler, J. P. "Differential Equations" in "Stream and Estuarine Analysis." Training Manual, Tenth Summer Institute in Water Pollution Control, Manhattan College, New York, N. Y. (June 1965).

9. "Joint Pollution Studies in the New York Harbor Model." Miscellaneous Paper No. 2-558, U. S. Army Engineer Waterways Experiment Station,

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Vicksburg, Miss. (February 1963).

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KARL R. KENNISON

CIVIL AND HYDRAULIC ENGINEER
361 CLINTON AVE., BROOKLYN, N.Y.

Mr. G. R. Milne Nov. 18, 1955
Mechanical Engineer
Cons. Edison Co. of N.Y.
4 Irving Place
New York 3, N. Y.

Dear Sir :

You have described to me the general features of the atomic-energy power plant which you are planning to construct on the east bank of the Hudson River below Peekskill. I understand that you wish me to report on such hydrologic features of the site as may effect your plans.

From the information that you have made available to me I conclude that the most useful information I can give you is that which relates to the amount and character of the flow in the river. At the proposed site the river has a width of about 4500 to 5000 feet, a maximum depth of 55 to 75 feet at less than 1000 feet off shore, and a cross-sectional area of about 165,000 to 170,000 square feet. Sheet 1 shows a number of cross sections of the river, plotted from the U.S.C. &G.S. charts, at intervals of 1500 feet, from 3750 feet upstream to 5250 feet downstream from the proposed plant.

At this site the effect of the tides is all important and so far outweighs any other consideration that, at least for present purposes, the information already available on the day-by-day variation of the runoff from the tributary watershed is adequate.

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On Sheet 2 I have plotted an approximate flow-duration curve from data I had already calculated covering a period of

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- 2 -

17 years.

An average rate of about 26000 cfs may be expected to be exceeded 20% of the time
" " " " " 15250 " " " 40% " " "
" " " " " 10500 " " " 60% " " "
" " " " " 7000 " " " 80% " " "
for say 2 % of the time the rate may be as low as 4000 cfs

However as above indicated the ebb and flow of the tide is the all important consideration. The river is tidal to as far upstream as Troy. Its hourly behavior in the tidal range varies throughout its length. The U. S. Coast & Geodetic Survey has tabulated a great deal of information from which a general picture of conditions off the shore at the proposed site can be obtained.

On Sheet 3 I have plotted the data, as they are applicable to this particular site. This indicates that the elevation of the water surface is so responsive to the tidal cycle that the average rate of flow, or runoff from the tributary watershed, has relatively little effect on the velocity past the site. I conclude that it is this velocity and the resulting volume of flow available for mixing and dilution in which you are primarily interested. In the limited time at my disposal I can only draw general conclusions. These may be adequate for present purposes. You could obtain better information by running a series of tests on surface and sub-surface floats, at varying distances off shore, throughout the tidal cycle.

The velocity recorded by the U.S.C.&G.S. is that in

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midstream at or near the surface. In order to be on the safe side in drawing conclusions, I have assumed that 80 % of this velocity represents the average vertically from surface to bottom, and that 80 % also represents the average horizontally from side to

side, hence that roughly 64 % represents the average over the entire cross section. I have also assumed that 15 % of the total cross section, or a stretch about five or six hundred feet wide off shore, is all that should be used in considering the initial mixing or diluting effect. In making this assumption I am governed to some extent by Hazen's studies relative to the off-shore distance of Poughkeepsie's water intake to avoid direct contamination by its sewage. I have further assumed that the velocity in this off-shore stretch is only 60% of the midstream velocity, hence that roughly 48% represents the average over the cross section of this off-shore stretch.

On Sheet 4 I have shown the result of these assumptions, which, as above stated, are believed to be on the safe side in considering the direct effect of mixing or dilution of your wastes. This emphasizes the all-important effect of the tides, the quantity available for dilution varying in about three hours from a maximum of eight or ten million gallons per minute to nothing.

Although you will have to put up with this variation as far as your continuous cooling water circulation is concerned, it does point to the desirability of incorporating in your design a method of controlling the time for the discharge into the cool-

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ing water outlet of any and all waste that is to any extent radioactive. I would say that this should be done in any event for the drainage from your routine and emergency demineralizers, and it might well be done also for drainage from all areas liable to accidental contamination.

From your estimate of the extent of dilution already

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accomplished in the demineralizer waste overflow, I trust you can get an approximate figure for the dilution that may result in the river off shore, and can compare this with what you may find necessary or desirable for adequate protection of fish life or of the fish eating public.

As far as the effect on public water supplies is concerned, the use of the Hudson River for water supply, other than condenser cooling, it ver limited. The nearest municipality involved is Poughkeepsie, 30 miles or more upstream, and even at that distance threatened at times with the problem of salinity. There is no likelihood that in the future any nearer municipality will take its domestic water supply from the Hudson. In fact the tendency is the other way, and the more remote muncipalities of Catskill and Hudson have abandoned earlier supplies taken from the river.

As far as the effect on ground water is concerned, you have acquired an ample area of surrounding land. I can see no possibility of any deletorious effect.

I trust that this information which I have assembled in the limited time available will be helpful to you. If from

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these approximate figures there appears to be any question as to the adequacy of the safety factor in dilution, you may, as above stated, require additional information from float test.

From what you have told me about your proposed designs and methods of operation, I suspect that there is no real question of safety but only one of public relations – avoidance of even the appearance of danger.

Very truly yours,

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[Historical Information]

Consolidated Edison Company of New York, Inc.

EVALUATION OF FLOODING CONDITIONS AT INDIAN POINT NUCLEAR GENERATING UNIT NO. 3

April, 1970
Revision of Report of February, 1969

Quirk, Lawler & Matusky Engineers
Environmental Science & Engineering Consultants
505 Fifth Avenue
New York, New York 10017

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ACKNOWLEDGMENTS

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We are particularly grateful to Mr. Dwight E. Nunn, of the AEC and Mr. P. Carpenter of the FWPCA for their many helpful suggestions and contributions.

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We are grateful also to Messr. Kenneth I. Darmer and Chintu Lai of the U. S. Geological Survey for the supply of several publications and discussions concerning the Hudson River activities of the Water Resources Division.

We are indebted to Messrs. Andrew Matuskey and Frank L. Panuzio of the Corps of Engineers for advise, consultation and assistance in providing helpful data regarding the probable maximum flood and hurricane.

the investigation was conducted by Quirk, Lawler & Matusky Engineers under the direction of Dr. John P. Lawler. Mr. Karim A. Abood of Q.L & M served in the capacity of project engineer and wrote this

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report. Dr. Karel Konrad of Q.L. & M. contributed materially to this study and performed any of the calculations reported herein.
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SUMMARY OF FINDINGS, CONCLUSIONS & RECOMMENDATIONS

1. Consolidated Edison Company of New York, Incorporated, (Con Ed) plans to build a third nuclear generating unit at its Indian Point site. The site is located in the town of Buchanan, Westchester County, New York and lies along the east bank of the Hudson River, some 43 river miles above New York City Harbor. The proposed facility will have a guaranteed output of 965 megawatts electric.

2. The Atomic Energy Commision (AEC), by virtue of the Atomic Energy Act, is empowered to review and issue licenses to construct and operate nuclear power plants. The AEC licensing regulations include submission of a Preliminary Safety Analysis Report (PSAR). site safety criteria for this report requires an analysis of the area's hydrology.

3. Quick, Lawler & Matislu Engineers (Q/L. & M.), Environmental Science & Engineering Consultants were retained by Con Ed to study the hydrology of the Indian Point site and to determine the maximum water-surface elevations that can occur as a result of possible flooding conditions at the site. The establishment of such flood levels is necessary to provide adequate protective measures during flood conditions.

Three previous studies had been performed prior to this investigation. The results of those studies were submitted to the AEC. The first appears in Supplement 10, Docket 50-286; the second is Q. L. & M's report of February, 1969 and the third is Q.L. & M's Summary of March, 1970.

The present study was conducted along the lines of a number of specific guidelines developed during several meetings with AEC staff personnel. The study is of a comprehensive and detailed nature involving careful analysis and accepted methodology and follows closely the procedures utilized by the U.S. Corps of Engineers for estimating the hydraulic events under consideration.

4. Several flooding conditions governing the maximum water elevation at the site, were investigated including:
   a. Flood resulting from runoff generated by a Hudson River Probable maximum Precipitation (PMP).
   b. Flooding caused by the occurrence of a dam failure

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concurrent with heavy runoff generated by a Hudson River

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Standard Project Flood (SPF). This condition was considered because the previous condition did not result in a dam failure.

c. Flooding due to the occurrence of a Probable maximum Hurricane (PMH) concurrent with spring high tide.

5. The determination of the most severe water surface elevations at the site was based upon the simultaneous occurrence of the above-delineated flooding conditions with several critical boundary conditions in the Hudson River including:

a. Mean Water Elevation
b. High tide Water Elevation
c. Low tide water Elevation
d. Standard Project Hurricane (SPH)
e. Probable maximum Hurricane (PMH)

6. The work required to achieve these objective included:

a. Determination of the Hudson River hydraulic and Hydrodynamic characteristics, which include channel Geometry, flow resistance, tidal dynamics and basin Hydrology.
b. Derivation of the Maximum Probable Flood (PMF) and Standard Project Flood (SPF) for the Hudson River Basin upstream of the site.
c. Structural and hydraulic analysis of the major dams in the basin to determine their stability and failure possibility under PMP conditions.
d. Determination of the Probable Maximum Hurricane (PMH) and Standard Project Hurricane (SPH) for the New York Harbor area and resulting peak storm surge heights in the Hudson River.

The specific details of these items are given in Chapters II, III, IV and Appendix A respectively. The results of the determination of the maximum water-surface elevations resulting from the flooding conditions of Item 4 and boundary conditions of Item 5 above are presented in Chapter V.

The procedures employed and major findings are summarized below in the following six items (#7 through 12).

7. A PMP of 14 inches in 72 hours and a probable maximum storm (PMS) pattern similar to the one derived by the U.S. Westher Bureau for the adjacent Susquehanna Basin at Danville were used in this study. These were adopted on the basis of similarity in topography,

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shape and size between the two neighboring basins.

The 6-hour incremental rainfall depths and related runoffs for the entire basin arranged in a critical time sequence documented in Figure S-1.

The Hudson Basin was divided into 28 subbasins and a flood hydrograph for each subbasin was established using the PMP values corresponding to the selected PMS together with the appropriate rainfall losses and unit hydrograph. To illustrate the procedures employed, a brief description of the development of a typical subbasin flood hydrograph is presented below.

Figure S-2 summarizes the basic flood data for the Catskill Creek Basin used in this study. An isohyetal map of the subbasin appears in the upper right hand corner. The isohyets, shown as broken lines, represent the subbasin portion of the total Hudson Basin PMS. this total PMS is shown in Figure III-8. The rainfall values are for the total storm duration of 72 hours. Twelve sets of such values corresponding to twelve 6-hour subdurations were established.

For each time increment, average rainfall depth over the subbasin was computed. These increments were then rearranged in accordance with a recommended critical time sequence. The appropriate initial and infiltration losses (1" and .05"/hr. respectively) were subtraced to obtain the subassin rainfall excesses (runoff) for each subduration. The resulting rainfall and runoff distributions are shown in the upper left hand corner.

The generaged runoff was applied to the 6-hour unit hydrograph to develop the subbasin flood hydrograph.

with base flows added the flood hydrographs of the 28 subbasins were combined in their proper time sequence and/or routed downstream to give the inflows into the Hudson River from its tributaries.

An example of the flood hydrograph combination and routing is depicted in Figure S-3. the example considers subbasins No. 15, 16 and 17 of the Mohawk River Basin. The flood hyrographs of subbasins No. 15 and 16 were combined to obtain the Mohawk River flood hydrograph at the mouth of Schoharie Creek. The resultant was routed through the river using the classical Muskingum coefficient method to Cohoes. This routed hydrograph was then combined with subbasin No. 17 flood

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Figure S-1

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Figure S-2

I-8 Figure S-3

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hydrograph to establish the total Mohawk River hydrograph at the confluence of the Mohawk River with the Hudson.

the main river inflows were then routed downstream in a similar manner until the flood hydrograph at Indian point was determined.

The routed hydrographs at several locations along the Hudson River are depicted in Figure S-4.

The above-listed steps were verified by predicting flows using actual rainfall-runoff records and then comparing the results with published values at Cohoes, Green Island and the Wallkill River.

The peak discharge thus obtained at Indian Point amounts to 1.1 million cubic feet per second and represents the Probable Maximum Flood (PMF) at the site. The corresponding Standard Project Flood was selected as equal to 50 per cent of this value.

In developing the PMF, every effort was made to make all the necessary assumptions as conservative as possible and the results were closely checked with the U.S. Army Corps of Engineers generalized curves.

The computed PMF agrees reasonably well with its Susquehanna, Delaware and Potomac Rivers counterparts. It is more than 60 per cent higher than the statistically evaluated 10,000 year flood at Indian Point.

8. To obtain the most severe flooding conditions, the heavy rainfall centers of the selected PMS were deliberatedly placed over the basins of the largest two reservoirs in the Hudson River Basin. These reservoirs are the largest regulating feature in the basin (Sacandaga) and New York City's reservoir at Ashokan. These reservoirs are located some 180 and 75 miles upstream of the site respectively

Inflows to Sacandaga and Ashokan Reservoirs resulting from placing the heavy rainfall centers of the PMS over their basins were routed through the reservoirs. The results clearly indicate that both dams have sufficient storage to pass this flood without danger of overtopping or piping, i.e. errosion of the surface of the downstream

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face. The specific details follow.

9. The Sacandaga Reservoir Dam, know as the Conklingville Dam, is extremely stable and has proven its ability to withstand sever
I-10
Figure S-4

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earthquakes and floods. It has an ample cross-section with a very broad base and a substantial freeboard of 24 feet above the crest of spillway at elevation 771.0.

The dam is relatively impervious, made of excellent material consisting of glacial drift, has an impermeable earth core and is founded on rock. It has an unusually large toe, sizable cover of riprap and sufficient outlet works and spillway capacity.

when routing the probable maximum inflow through the reservoir, the maximum stage reached was elevation less than elevation 784.0, more than 11 feet below the crest of the dam.

Computation of the location of the free water surface within the dam showed that the seepage water caused by the maximum possible water elevation would collect in the rock toe and would be safely discharged without carrying soil grains with it. This was the case even when a highly conservative and hypothetical case consisting of the maximum possible stage upstream and a 20 foot water depth downstream of the dam.

These estimates are based upon several conservative assumptions including:

a. Outlet works would start operation when the reservoir level reaches elevation 771.0, the crest of spillway elevation.
b. High initial reservoir level at elevation 768.0 prior to the beginning of the PMS.
c. Inoperative power house pumps.
d. Homogeneous material, i.e. no advantage was taken of the deflection in seepage curve caused by the relatively impermeable core.
e. Presence of water downstream of the dam.

10. The study of the effect of routing the Esopus Creek flood hydrograph (upstream of the Ashokan Dam) resulting from the Hudson River PMS through the Ashokan Reservoir indicates that the dam would not fail from overtopping or seepage. The maximum possible reservoir elevation would be below the crest of the masonry core in the dikes and more than 11 to 15 feet below the crest of the east

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and west basin reservoirs respectively. The dam has proven its ability to withstand such high elevation on March 31, 1951 when the elevation in the east basin reached a record high of 592.23 feet, about 1.5 feet higher than the elevation resulting from the Hudson River PMP.

11. Since the results of the previous item indicated that the Ashokan Dam would not fail, a more critical condition resulting in a dam failure was investigated. The reason behind consideration of a more severe Ashokan Dam condition stems from the fact that the Ashokan Reservoir contains the largest volume of stored water in the basin within less than 100 miles from the Indian Point site.

This flooding condition consists of the simultaneous occurrence of a probable maximum flood over the Esopus Creek Basin upstream of the Ashokan Dam and a Standard Project Flood over the rest of the Hudson River Basin.

this condition resulted in a peak reservoir inflow of 200,000 cfs causing a dam failure as a result of a result of a reservoir elevation higher than the crest of the masonry wall in the west dike and earth part of the main dam.

However, the structural analysis of the masonry part of the main dam showed that this part would not fail from overturning and sliding with minimum factors of safety of about 1.1. and 1.9 respectively.

The failure of the Ashokan Dam would cause a maximum reservoir out-flow of 2.6 million cfs and create a seven billion cubic feet lake extending from the dam to Glenerie Falls some 5 miles west of the Hudson River. The lake would have a total length of 14 miles and a surface width ranging form 4,000 to 1,800 feet. It would also have a control section at Glenerie Falls. The outflow of this lake would be about 620,000 cf.s

This outflow was combined with the flood hydrographs resulting from Esopus Creek PMP and Hudson River PMP and routed downstream to Indian Point. Much of the discussion on the determination of the PMF at Indian Point summarized in Item 7 above applies here, mutatis mutandis.

the maximum discharge at Indian Point resulting from this condition would be 705,000 cfs.

12. The maximum water-surface elevations at the site resulting from the flooding conditions summarized in item 4 concurrent with the

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initial conditions of Item 5 above were determined. These conditions were conservatively grouped in seven different ways on the basis of a simultaneous occurrence of an appropriate set of flooding, hurricane and tidal conditions.

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-S-7-

As in the case of flooding from dam failure, the tidal variation in the Hudson Estuary was treated as an integral part of the system and its influence was simultaneously coupled with the other hydraulic occurrences. The results of this study suggest that the river above the Tappan Zee Bridge, some 27 miles above the Battery, would experience relatively small tidal variations under the PMP conditions. However, to examine the effect on Indian Point the upstream extent of tidal influence was extended to the site.

In the evaluation of the maxium water surface elevation at the site, the wind produced local oscillatory short period waves were also included. The computed stages were increased by one foot to account for this effect.

The standard Step Method was employed in this investigation to determine the flow profiles and stages at the site. The computations were initiated at the Battery, the mouth of the Hudson River. The results were checked with a more refined and newly developed method.

The seven groups of flooding, hurricane and tidal conditions and the corresponding stages at Indian Point are summarized in Table S-1

Cases No. 1 through 3 consider the Indian Point stage at peak discharge due to runoff generated by the PMP over the basin and mean, high and low tide stages at the Battery. The flow profiles corresponding to these cases are depicted in Figure S-5.

Case No. 4 considers the maximum river stage at the site resulting from the dam failure concurrent with the Hudson River SPF summarized in Item 11 above.

To carry the degree of severity a step further, a more critical boundary condition consisting of the peak storm surge at the Battery resulting from the SPH was imposed on the flooding condition of steps No. 5 and 6.

To maintain the selection of severe conditions and in accordance with the high degree of conservatism adopted in this report, the SPH based on the transposition of the September 1944 hurricane rather than the less critical SPH of the 1938 hurricane was used. The 1944 value is 75 per cent higher than the maximum storm surge during hurricane "Donna" which struck on September 1, 2960 and which is the storm having the greatest effect since 1821.

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Case No. 6 considers an even more critical set of occurrences con-

I-14.

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Figure S-5

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-S-8-

sisting of the simultaneous occurrence of the following three severe conditions:

a. PMP over the Ashokan Reservoir resulting in a dam failure
b. runoff generated by the SPP over the Hudson Basin
c. Peak storm surge resulting from the SPH for the New York Harbor area

The simultaneous occurrence of these three conditions is extremely remote. Each one of these three conditions are in themselves unlikely events. The combination of all three decreases, therefore, the likelihood of occurrence significantly.

The flow profiles corresponding to cases No. 4 through 6 are depicted in Figures S-6.

For convenience, the results of routing the PMH through the Hudson River to the site presented in our report are also listed in Table S-1.

Conclusion – These analyses show clearly that the maxiumu elevation at Indian Point due to flooding and wave runup is 15 feet or less.

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Figure S-6

I-18

I. <u>INTRODUCTION</u>

Consolidated Edison Company of New York, Incorporated, plans to build a third nuclear generating unit on the Hudson Rive at Indian Point. The new facility is called Indian Point No. 3 reactor since two other nuclear power units have been authorized for the same site.

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The Indian Point site is located in the town of Buchanan, Westchester County, New York. the property lies along the east bank of the Hudson River, some 43 river miles above New York City Harbor.

The proposed facility will have a guaranteed output of 965 megawatts electric. On completion of Unit No 3, the total manufacturer's guaranteed rating of Unit Nos. 2 & 3 will be 2123 megawatts electric.

the Atomic Energy Commission (AEC), by virtue of the Atomic Energy Act, is empowered to review and issue licenses to construct and operate nuclear power plants. The AEC licensing regulations include submission of a Preliminary Safety Analysis Report (PSAR). Site safety criteria for this report requires an analysis of the area's hydrology.
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Quirk, Lawler & Matusky Engineers (QL&M), Environmental Science and Engineering Consultants were retained by Consolidated Edison to study the hydrology of the Indian Point site and to determine the maximum water elevations that can occur as a result of possible flooding conditions at the site of the Indian Point Nuclear Generating Unit No. 3. The establishment of such a flood level at the site is necessary to provide the necessary protective measures during flood conditions.

Preliminary analysis based upon highly conservative estimates were conducted in 1968. The results of that study are delineated in Supplement 10, Indian Point Unit No. 3, AEC Docket 50-286.

The following two flooding conditions, which the AEC indicated would govern the maximum water elevation, were investigated:

1. Flooding from maximum rainfall concurrent with dam failure at Sacandaga and Ashokan Reservoirs and flow at ebb tide.

2. Flooding resulting from the probable maximum hurricane concurrent with spring high tide.

In the 1968 report it was noted that a more refined flooding analysis would give more realistic results, and that the hurricane surge analysis was based upon a theory applicable to conditons of open
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The 1968 preliminary values were then modified as a result of a new study which considered surrounding topography, land use, river geometry and frictional losses. A report documenting the results of the new study was submitted to the AEC in February, 1969.

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Because the 1969 results indicated that the hurricane surge analysis proved to be controlling, specific attention was paid to determining the water elevation resulting from the occurrence of probable maximum hurricane.

After a review of the February, 1969 report, including conferences with AL & M and Con Ed by the AEC, the commission requested that the applicant restudy in depth the runoff resulting from heavy rainfall over the basin drainage area and revise the water elevation computation at Indian Point.

In particular, the AEC objected to the use of the so-called "rational formula" for the runoff computations and the Manning Open Channel Flow Equation for the determination of the water elevation
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The AEC suggested a more refined approach including the use of the Corps of Engineers findings and procedures to determine the probable maximum flood and the backwater profile method to predict the water surface elevation resulting from flooding due to probable maximum precipitation over the basin.
A study along the lines of the suggested procedures was instituted in January of this year and involved a thorough evaluation of the maximum predictable stage of the Hudson River at the Indian Point site Major emphasis was placed on the determination of the probable maximum flood, dam failure analysis, influence of the tidal flow and the maximum water elevations for several flooding conditions. Two meetings were held on March 26 and April 21, 1970 between the AEC staff personnel, Con Ed and Q & M to discuss the proposed program and to delineate requirements for further refinements. At these meetings, as number of specific guidelines for the analyses of all pertinent parameters and accepted methodology were developed. The procedures and methods adopted for this study follow closely the developed guidelines and those utilized by the U.S. Corps of Engineers.

The purpose of this report is to present the results of this study. For convenience, the results of the revised probable maximum hurricane study of the February, 1969 report are also included in this report.
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the report is formulated as follows:

1. A general description of the Hudson River Basin and the

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channel characteristics is given in Chapter II.

2. The specific details and adopted methodology in computing the Hudson River probable maximum flood at Indian Point are presented in Chapter III. The results of the previous flood investigations are also documented in this chapter.

3. chapter IV presents the results of the flood routing and dam failure analysis of the Hudson River Basin Reservoirs.

4. A discussion of the procedures and results of the maximum water elevations for several flooding conditions resulting from a probable maximum flood, dam failure and tidal flow is given in Chapter V.

5. The results of the revised analysis of the water elevation at Indian Point resulting from the occurrence of a probable maximum hurricane concurrent with spring high tide are documented in Appendix A. The material included in this Appendix is essentially that of Chapter III of the February, 1969 report.

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II. DESCRIPTION OF HUDSON RIVER BASIN *

A. Description of the Basis

The Hudson River Basin is located in the eastern part of New York State, draining an area of 12,650 square miles. Most of this area lies in the east central part of New York State, with small portions in Vermont, Massachusetts, Connecticut and new Jersey. A plan view of the Basin is shown in Figure II-1. Several locations of interest to this study are underlined in Figure 11-2. The distance in river miles above the mouth of the river is also shown in Figure II-2.

The Hudson River Basin is bounded on the north by the St. Lawrence

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and the Lake Champlain Drainage basins; on the east by the Connecticut River, the Housatonic River Basins and the Connecticut Coastal Area; on the west by the Delaware, Susquehanna, Oswego and Black River Basins; and on the south by the basins of small streams tributary to the Hudson River in New York Harbor. The Hudson River watershed extends 128 miles east-to-west and 238 miles north-to-south.

The Hudson River has its source in Henderson Lake in the Adirondack

Mountains in northern New York State and flows generally south for 315 river miles to its mouth at the Battery where it discharges into New

* Some of the material covered in ther first two sections of this chapter is based on several Federal and State publications on the Hudson River 8-14

II-1

II-2

FIGURE II-2

II-3

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York Upper Bay. Upstream of Henderson lake, the stream is known as the Opalescent River and its headwaters are in Lake Tear-of-the-Clouds on the Southwest slop of Mount Marcy in Essex County.

The major tributaries entering the main stream are mohawk River, Hoosic River, Kinderhook Creek, Indian River, Sacandaga River, Esopus Creek and Rondout Creek. The drainage areas of these and other principle tributaries are listed in Table II-1. For convenience, the entire Hudson River Basin has been separated into three principal drainage areas; the Upper Hudson, the Mohawk River, and the Lower Hudson subbasins.

The division between the Upper and Lower Hudson Basin is at the confluence of the Mohawk River with the Hudson at Green Island. The Federal Dam at Troy, some 154 river miles above the Battery, is the head of tidewater.

The Upper Hudson River flows generally south-southeast to the confluence with the Sacandaga River where it turns to the east. At Hudson Falls it turns again to the South. Its total length to Green Island is about 150 miles. The river drains an area of some 4627 square miles. From its source to Troy Dam, the Hudson River drops 1810 feet, resulting in an average bottom slope of about 12 feet per mile.

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TABLE II-1

Drainage Areas Of

Hudson River Basin

Stream	Drainage Area (square miles)
Upper Hudson River	
Cedar River	164
Indian River	201
Boreas River	92
Schroon River	568
Sacandaga River	1,058
Batten Kill	441
Kayaderosseras – Fish Creek	252
Hoosic River	713
Minor streams and direct drainage	1,138
Sub-total	4,627
Mohawk River	
Oriskany Creek	146
West Canada Creek	562
East Canada Creek	291
Schoharie Creek	926
Minor streams and direct drainage	1,537
Sub-total	3,462
(Hudson River at Green Island)	8,090)
Lower Hudson River	
Kinderhook Creek, including Stockport Creek	512
Catskill Creek	417
Roeliff-Jansen Kill	208
Esopus Creek	425
Rondout Creek, including Wallkill River	1,197
wappinger Creek	208
Minor streams and direct drainage	1,594
Sub-total	4,561
Total of Hudson River Basin at the Battery	12,650

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The Mohawk River has its source in the hills near the boundary between Lewis and Oneida Counties, New York. It flows in a southerly direction to Rome, thence it follows a general east-southeast course to its junction with the Hudson River at Cohoes, New York. The total length of the river is about 155 miles and it drains some 3462 square miles. The Mohawk River falls irregularly from its source at elevation 1800 feet above mean sea level to elevation 14.3 feet where it joins the Hudson River at Cohoes.

The Lower Hudson River commences at the junction of the Mohawk and Upper Hudson Rivers at Troy and discharges into the Upper New York Bay. All of this section of the river is tidal. Because of its special nature, detailed description of the estuary portion of the Hudson River is presented in another section of this chapter.

The total length of the Lower Hudson River is about 154 miles asnd it drains an area of some 4561 square miles.

The average slope in the Lower Hudson, as represented by the half-tide level, is about 2 feet in 150 miles. The slope is greatest in the section of the river from troy to Catskill and least between Catskill and Tarrytown.

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The major physiographic features of the Hudson River Basin are a mountainous terrain covering 48 per cent of the basis; cultivated lands covering 42 per cent of the basin; lakes and water bodies covering 2 per cent of the basin; and urban developments covering 8 per cent of the basin.

B. Basin Hydrology

The general climate of the Hudson River Basin may be considered as moist continental. The Upper Hudson Basin has comparatively long, cold and snowy winters sand short mild summers. In the Lower Hudson Basin, the climate is much milder due to the modifying influence exerted by the valley. For these areas there are usually longer summer periods and milder winters. The Mohawk River Basin has variable weather conditions with characteristics of both areas.

The average annual temperature within the basin ranges from 50°F in the southern portion to 40°F in the Adirondack Mountains. The corresponding average July temperature varies form 75°F to 65°F and the average January temperature is 30°F and 15°F respectively The maximum and minimum temperatures recorded in the basin are 106°F

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and -42°F respectively.
II-7 -10-

The average annual precipitation varies from 34 inches in the center of the basin to more than 50 inches in the Adirondacks. Most of the basin receives an average of about 40 inches. In general, the precipitation is distributed evenly through the year with a slight rise during the summer. Figure II-3 depics the normal annual precipitation for the entire Hudson River Basin.

The average annual snowfall for the basin ranges from about 30 inches in New York City to over 130 inches in the Adirondack mountains.

A summary of major basin characteristics including rainfall and runoff data for the three subbasins i.e. the Upper Hudson, Mohawk and Lower Hudson Rivers, is given in Table II-2.

The U.S. Geological Survey (U.S.G.S.) maintains stream gages at some 62 locations in the basin; 21 in the Upper Hudson, 11 in the Mohawk and 30 in the Lower Hudson Basin. Because of tidal oscillation, it is not possible to measure the fresh water flow in the Lower Hudson River directly. Flow histograms in the tidal portion of the river are usually constructed from a knowledge of the Green Island time-discharge relation (the most downstream gaging station above tidewater). The ration of the drainage areas tributary to Green Island and the Battery is 1.65. However, analysis of data developed by the U.S.G.S. indicates that a yield factor of 1.225

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II-9

TABLE II-2

HUDSON RIVER BASIN CHARACTERISTICS

	Drainage Area	Channel Length	Mean Annual	Mean Annual pre-	Mean Annual Evaporation
Mean Annual	Storage				
Subbasin	(sq.mi.)	(miles)	Temp °F	cipation	& Trans
Runoff	Usable Capacity				
				(inches)	Precipitation
ity acre – ft.					

cfs cfs in

sq.mi.

Upper Hudson

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River Basin	4627	150	40	40	16	7400	1.(
Mohawk River Basin	3462	155	45	46	24	5600	1.(
Lower Hudson River Basin	4561	154	48	42	20	8700*	1.(

* Estimated

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represents the statistically probably value of the ration of the Lower Hudson discharge to Green Island discharge.

A summary of runoff data at several stations representative of the three subbasins, i.e. the Upper Hudson, Mohawk and Lower Hudson Rivers, is given in Table II-3.

A comparison of long term Lower Hudson monthly average flows with the 1964 histogram is shown on Figure II-4. These histograms were prepared from the Upper Hudson flow measurements at the Mechanicville gage and the Mohawk River measurements at the Cohoes gage for the period from 1918 to 1947 and the Green Island gage for later years. The Lower Hudson River values referred to in this section were established using the yield factor of 1.225.

Flood records indicate that in the past several major floods in the Hudson River Basin have occurred during the spring as well as during the fall or early winter. General storms covering the entire watershed are usually of the transcontinental (cyclonic) or tropical types.

The greatest flood of record over most of the basin occurred in March, 1913 as the result of a period of rapid thaw followed by five days of heavy rainfall. The March, 1936 storm, which followed a sudden rise in temperature after a winter of heavy snowfall was the second greatest

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TABLE II-3

STREAMGAGE SUMMARY AT REPRESENTATIVE STATIONS IN THE HUDSON RIVER BASIN

Station	Area (sq.mi.)	Mean Flow (cfs)	Mean (csm)	Runoff (inches)	Years of Record
Mohawk River Basin					
Mohawk River					
near Rome	150	379	2.53	34.4	47
near Little Falls	1,348	2,708	2.01	27.1	41
at Cohoes	3,453	5,537	1.61	21.8	43
Upper Hudson River Basin					
Hudson River					
near Newcomb	192	387	2.02	27.4	43
at Gooley	419	830	1.98	27.0	52
at North Creek	792	1,534	1.94	26.3	61
at Hadley	1,664	2,849	1.71	23.3	47
at Mechanicville	4,500	7,431	1.65	22.42	70
Sacandaga River					
near Hadley	1,055	2,106	2.00	27.2	61
Hoosic River					
near Eagle Bridge	510	903	1.77	24.1	56
Schroon River					
at Riverbank	527	795	1.51	20.6	61
Lower Hudson River Basin					
Hudson River At Green Island	8,090	13,060	1.62	22.0	22

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Kinderhook Creek at Rossman	329	454	1.38	18.8	44

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TABLE II-3 (Continued)

STREAMGAGE SUMMARY AT REPRESENTATIVE
STATIONS IN THE HUDSON RIVER BASIN

Station	Area (sq.mi.)	Mean Flow (cfs)	Mean (csm)	Runoff (inches)	Years of Record
Catskill Creek at Oak Hill	98	126	1.29	17.5	58
Walkill River at Gardiner	711	1,045	1.47	20.0	44

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FIGURE II-4

-12-

Several tropical type storms occurred in October, 1869; November, 1927; and September, 1938.

Table II-4 summarizes the peak discharges and stages of the Hudson River at Troy and Albany during the eight largest floods since 1846.

Several peak discharge frequency curves based upon a number of statistical evaluation methods are shown in Figure II-5. Further discussion of these curves is presented in the next chapter.

Most of the major floods occurred prior to construction of the Sacandaga Reservoir in 1930 which greatly moderated downstream flows. This reservoir and several other reservoirs control about 20 per cent of the entire Hudson River watershed, they have been built to control floods, augment the natural river flows, and for municipal and industrial supply. The major seven reservoirs in the Basin are listed in Table II-5. Detailed description of these reservoirs appear in Chapter IV of this report.

C. Hudson River Channel Characteristics

The major Hudson River Channel characteristics used in flood routing of the probable maximum flood and backwater computations are summarized below.

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TABLE II-4

Flood Stages and Discharges, Hudson River

(Hudson River Basin)

Flood	Troy, N. Y. Drainage Area 8,090 sq.mi.		Albany, N. Y. Drainage Area 8,270 sq.mi.	
	Peak Stage (ft.m.s.1.)	Peak Discharge (c.f.s.)	Peak Stage (ft.m.s.1.)	Peak Discharge (c.f.s.)
October 5, 1869 1/ 2/			18.98	
April 10, 1895 1/ 2/			16.42	
March 28, 1913 1/ 2/	29.4	223,000	21.45	228,000
April 12, 1922 1/	25.7	154,000	15.98	158,000
November 4, 1927 1/	24.8	150,000	15.96	153,000
March 19, 1936	19.17	215,000	17.5	220,000
September 22, 1938	27.1	183,000	16.5	187,000
December 31, 1948	26.74	181,000	17.46 3/	185,000 3/
Bank full stage	24.0		11.0	

1/ Before completion of Sacandaga Reservoir and 27 ft. navigation channel to Albany
2/ Before construction of Federal lock and dam at Troy.
3/ January 1, 1949.

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II-16
Figure II-5

II-17

TABLE II-5

MAJOR HUDSON RIVER BASIN RESERVOIR SUMMARY

No.	Reservoir	Year Placed in Distance Area	Stream Area	Usable Capacity Service	Drainage from Ind-	Purposes (ac-ft)	ian Point (miles)
	MOHAWK RIVER BASIN						
1	Delta Reservoir	1912	Mohawk River	64,500	140	Canal Regulation	225
2	Hinckley Reservoir	1914	West Canada Creek	78,000	374	Canal Regulation and Utica Water Supply	217
3	Schoharie Reservoir	1926	Schoharie Creek	61,400	314	New York City Water Supply	190
	UPPER HUDSON RIVER BASIN						
4	Indian Lake	1898	Indian River	103,000	131	River Regulations, Power	250

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5	Sacandaga Reservoir	1931	Sacandaga River	760,000	1,044	River Regulation, Flood Control, Power	180
	LOWER HUDSON RIVER BASIN						
	Ashokan Reservoir	1915	Esophs Creek	383,000	257	New York City Water Supply	75
	Rondout Reservoir		Rondout Creek	158,000	95	New York City Water Supply	75

The Lower Hudson River Channel is a relatively deep and straight channel. The aviation in the cross sectional area, at mean low water, between the ocean entrance and the head at the Federal Dam at Troy is shown in Figure II-6. The variation is significant and erratic and cannot be accurately described by a simple mathematical model. The total area in the Lower Hudson ranges between over 250,000 sq. ft. in Haverstraw Bay and less than 50,000 sq. ft. at Troy with an average of about 120,000 sq. ft.

The top width at mean low water is shown in Figure II-7. The variation in the surface width is even more erratic than the cross-sectional area. This is due to the presence of several bays and shoals in the Lower Hudson. Two major bays are located on Figure II-6. These are Haverstraw Bay and Newburgh Bay.

The mean depth of the river is shown in Figure II-8. The mean depth is defined as the cross-sectional area divided by the top width. From the Battery to the head of Haverstraw Bay, the mean depth generally decreases from some 38 feet to about 16 feet. Upstream of Haverstraw Bay the depth abruptly increases, in an erratic manner, reaching a maximum of approximately 100 feet in the vicinity of West Point. Point values approach some 200 feet. This abrupt increase in depth has a significant effect on salt water intrusion in the river.

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FIGURE II-6

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II-20

FIGURE II-7

II-21 FIGURE II-8

II-22

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As indicated before, the Lower Hudson River is a tidal estuary between the Federal Dam at Troy and the New York Harbor. It is also classified as a dampened, reflected tidal wave regimen.[15] Dampening occurs by dissipation of tidal energy via channel friction as the oceanic tidal wave progresses upstream. Reflection includes secondary wave propagations caused by channel obstructions. Complete reflection occurs at the Federal Dam at Troy. Additional wave reflections occur due to significant changes in channel width. As widths increase, wave amplitudes tend to decrease, whereas a decrease in channel width causes an increase in wave amplitudes. Tidal behavior at any section is the composite effect of ocean tide, channel friction and wave reflection. The primary ocean tides are also variable with maximum amplitudes occurring during spring tide and minimum amplitude during neap tide. Variations in fresh water discharge and the barometric conditions also contribute to changes in amplitude.

Figure II-9 illustrates the principal tidal characteristics along
The stretch of river between the ocean entrance and Poughkeepsie.

High and low water above mean low water at Sandy Hook, New Jersey are shown on the upper figure. The half-tide level indicates the average slope in the river. The total fall from Troy to the sea is about 2 feet.

II-23

FIGURE II-9

II-24

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The variation in the tidal range along the river is shown in the middle figure. It will be noted that, in moving upstream, the range of tide diminishes from about 4.4 feet at the Battery to a minimum of about 2.6 feet at a point near Storm King (mile point 56). The tidal range then reaches a maximum of 4.1 feet at a point near Catskill and then increases to 4.8 feet at Troy.

The high water and low water lunar intervals, as referred to the transit of the moon over the meridian of Greenwich, are shown on the lower figure.

Figure II-10 shows the variation of mean sectional, tidal velocity

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along the river. The raw data for the ebb and flood strengths were obtained from the 1929 U.S.G.S. study. Ebb and flood strengths were each averaged across the river cross section. Mean absolute velocity over a tidal cycle ws then obtained by averaging section averaged ebb and flood strengths.

The variation of the mean tidal "flow" along the river is also shown on Figure II-10. It will be noticed that the mean tidal flow decreases from a maximum of 425,000 cfs at the Battery to zero at the Federal Dam at Troy (about mile point 153).

II-25

FIGURE II-10

II-26

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Hudson River salt intrusions at equilibrium, or near equilibrium conditons, for selected flows are shown on Figure II-11 Data are shown for both the Hudson River model and the prototype conditions. Prototype profiles were obtained by chemical and electtrical measurements sand are representative of equilibrium conditions. Model profiles represent steady state equilibrium conditions and were obtained by chemical measurement on the Hudson River model at the Waterways Experiment Station, U.S. Army Corps of Engineers, Vicksbury, Mississippi. Salinities shown are tidal average values over a tidal cycle and a full channel cross-seciton. The Vicksbury model is operated for the mean tide.

Figure II-12 shows the relationship between the length of salt water intrusion at equilibrium in miles and fresh water flow in the lower Hudson. this length is defined as the point on the intrusion curve at which the salinity is 100 ppm.

These results indicate that the salinity intrusion is influenced by the oceanic tidal action which causes the ocean-derived salt to advance landward and the up river inflow which tends to flush the estuary.

II-27

FIGURE II-11

II-28

FIGURE 11-12

II-29

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III. HUDSON RIVER BASIN PROBABLE

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MAXIMUM FLOOD AT INDIAN POINT

A. Introduction

The probable maximum flood (PMF) has been defined as a hypothetical flood produced by the most severe, but reasonably possible, rainfall and related runoff, at a particular area. A rainfall producing a probable maximum flood is ofter called "Probalbe Maximum Precipitation" (PMP). The probable maximum flood has also been described as the boundary between possible floods and impossible floods, i.e. a flood having a probability of occurance approaching zero or a return period of infinity.

No detailed investigation had been performed prior to this study to determine the maximum probable flood for the entire Hudson River Basin.

A semi-empirical approach was followed by Q L & M in early 1970 to estimate such a flood at Indian Point. This approach was based upon several Corps of Engineers studies of a number of neighboring river basins as well as a detailed study of the Upper Hudson River Basin. The probable maximum flood for the Upper Hudson River Basin was conducted by Stone & Webster Engineering Corporation in 1969, The shaded area in Figure III-1 represents the area studied by Stone & Webster.
III-1

Figure III-1

III-2

-18-

A more comprehensive study of a formal or detailed nature involving careful analyses and accepted methodology was conducted in 1970 by Q L & M. The details of that study are given in this chapter. However, the procedures followed are delineated below.

1. A pattern and a time distribution of a probable maximum precipitation were developed for the Hudson River Drainage Basin at Indian Point.

2. The Hudson River Drainage Basin was divided into twenty-eight subbasins and a unit hydrograph was developed for each subbasin.

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The twenty-eight subbasins are located on Figure II-1.

3. The probable maximum precipitation of Item 1 was applied to the developed unit hydrographs of Item 2 with the appropriate initial and infiltration losses to establish the subbasin flood hydrographs.

4. The subbasin flood hydrographs of Item 3 were combined in their proper time sequence to give a main river hydrograph. The main river hydrograph was then routed downstream. This process started at the uppermost subbasin and continued in the downstream direction until the resultant probable maximum flood hydrograph at Indian Point was obtained.

The results of the sem-empirical study are presented first followed by the delineation of the detailed study. A copy of the Stone & Webster Upper Hudson River study is appended to this report.

B. Previous Investigations

As indicated in Chapter I, three flood studies had been performed prior to this investigation to determine the maximum flow for the Hudson River at Indian Point. A brief description of these studies follows.

The first study was of a preliminary nature and was based upon highly conservative estimates. The results of that study which was conducted in 1968 are delineated in Supplement 10, Indian Point unit No. 3, AEC Docket 50-286.

The 1968 preliminary estimates were then modified as a result of a new study in the early part of 1969. This study considered surrounding topography, land use, river geometry and frictional losses. The so-called "rational formula" was used for the runoff computations. The results of the 1969 study are summarized in Table III-1.

Another investigation based upon several Corps of Engineers studies of a number of adjacent river basins and on a detailed study of the

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Upper Hudson River Basin was conducted in the early part of 1970. A brief discussion of this study is presented below.

Six different methods were used to determine the probable maximum

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food for the Hudson River at Indian Point. These methods and the corresponding results are listed in Table III-2.

The PMF values at Indian Point were predicted on the basis of a thorough comparison of the Hudson River Basin with the following basins:

1. Susquehanna River at Harrisburg
2. Potomac River at Washington D.C.
3. Delaware River at Montague
4. Hudson River at Stillwater

The following four basins parameters were used for this purpose:

1. Size of the drainage area (A)
2. Available storage capacity (S)
3. Maximum observed peak discharge (Q)
4. Probable maximum flood (PMF)

The values corresponding to the above listed basins are compared with their Hudson River at Indian Point counterparts in Table III-3. Several interesting conclusions may be drawn from this comparison. On the basis of drainage area size, the Hudson River has the highest available storage capacity and the lowest maximum observed peak. These results indicate that the Hudson River is a highly regulated

basin. Therefore, prediction of its PMF on the basis of the drainage area size alone may be misleading.

The relationship between the drainage area and computed probable maximum flood for the adjacent basins is shown in Figure III-2. The Upper Hudson River value at Stillwater is also shown and is well below the majority of the points. For comparison purposes, the value corresponding to the present study is also shown.

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The fact that the Hudson River Basin is regulated appreciably is clearly seen in Figure III-3 which correlates the maximum observed peak flow with the drainage area size for the basins of interest. The Hudson River maximum observed peak is 60 per cent lower than that of an adjacent river of equivalent size.

A prediction of the Hudson River PMF on the basis of the maximum observed peaks was also attempted. The results are depicted in Figure III-4. The result of the formal PMF study of this report is also shown on this Figure.

A more realistic approach taking into consideration the combined influence of drainage area size (A), available storage capacity (S), and maximum observed peak flow (Q) on the probable maximum flood (PMF) was then investigated. The classical dimensional analysis

III-9

FIGURE III-2 PROBABLE MAXIMUM FLOOD – DRAINAGE AREA RELATIONSHIP

Figure III-3

II-11

Figure III-4

III-12

-22-

method was employed for this purpose. This tool recognizes the fact that the physical factors influencing a physical phenomenon should be related in a mathematical model which is dimensionally homogeneous. The parameters A, S and Q were treated as the independent variables representing the physical factors and the PMF as the dependent variable defining the physical phenomenon. The results of this approach are shown in Figure III-5. The value corresponding to the present study is also shown.

Another simple estimate equating the probable maximum flood to a statistically evaluated 1:10,000 year peak flood is also shown in Table III-2. A discharge-frequency curve based on a statistical evaluation of the available 50 U.S.G.S. data points (1918-1969) were prepared as shown in Figure II-5 and extended to 10,000 years. The data points corresponding to discharges prior to the completion of the Sacandaga Reservoir in 1930 were not adjusted for discharges from the Sacandaga river which would have been impounded by the Conklingville Dam.

III-13

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Figure III-5

III-14

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C. Probable Maximum Precipitation

The Hudson River Flood records at Green Island show that in the past most major floods have occured in the spring months as a result of an early spring storm combined with melting snow. The hydrologic history of the Hudson River at Green island was discussed in Chapter II. The probable maximum precipitation for the spring months, however, is about 50 per cent of the all season probable maximum precipitation[2]. Moreover, the adjacent Connecticut River Drainage Basin study[3] indicated that the maximum storm would be at least equal to the maximum combination of rainfall and snow melt. Therefore, a summer or fall storm producing a runoff at least as great as a spring storm with melting snow was used in this study.

The probable maximum precipitation for this study was developed from the 72-hour precipitation and depth-duration-drainage area (DDA) curves prepared by the U.S. Weather Bureau for the adjacent Susquehanna River basin[1]. A copy of the Susquehanna DDA Curves is reproduced in Figure III-6.

The PMP curves of Figure III-6 were obtained by transposing storms of record into the Susquehanna Basin and adjusting them for maximum moisture at the transposed location.

III-14a

Figure III-6

III-15

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The general level of these curves has been found to be consistent with that of recent estimates for the Potomac & Delaware River Basins[1].

On the basis of these curves, a probable maximum precipitation depth of 14 inches associated with a duration of 72 hours was used for the entire 12,650 square miles of the Hudson River Basin above Indian Point.

1. Selection of Pattern & Areal Distribution of the Probable Maximum Precipitation Storm

The storm selected for application to the Hudson River Basin is

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similar in areal distribution to that of the Susquehanna River at Danville. The Danville pattern was developed by the U.S. Weather Bureau[1] and is reproduced in Figure III-7.

This pattern was considered in this study due to the similarity in shape and drainage area size between the Susquehanna River at Danville (11,220 Sq. Miles) and the Hudson River at Indian Point (12,650 Sq. Miles). The two rain centers of the Danville pattern were shifted and rotated so as to give best fit to the Hudson River Basin. The resulting pattern is shown in Figure III-8. The letters in Figure III-8 represent different values of the pattern storm Isohyets as will be shown later. One of the selected storm centers

III-16 Figure III-7
III-17 Figure III-8

III-18

-25-

was located on the Sacandaga River Basin (Subbasin 8) in the Upper Hudson River Basin and the other one on the Catskill Creek Basin (Subbasin 18.3) in the Lower Hudson River Basin. The direct distance between these two centers is about 80 miles.

Because of the comparatively small differences between the average elevation of the Hudson River Basin and the elevations of the terrain covered by the selected storm isohyets, no adjustment was made for difference in elevation.

2. Computation of the Isohyet Values for the Total Duration of the Probable Maximum Storm

The purpose of this step is to define rain concentration associated with the 72 hour total storm within the Hudson River Basin. The procedure employed in this step was also used to compute the isohyet values for incremental durations as discussed in subsequent sections of this chapter.

The computational procedure is described below. Due to the presence of two storm centers, the values of the lettered isohyets of Figure III-8 were determined as follows:

a. The two centers were treated as separate storms and the areas within Isohyets A_1, B_1, C_1, A_2, B_2, C_2, were measured.

III-19

b. The average depth of rainfall corresponding to each one of the areas of Step a was obtained from the appropriate DDA curves of Figure III-6. The 72 hour curve was used for the computation of the isohyet values for the total duration of the probable maximum storm.

c. To insure that the precipitation averaged along the individual isohyets must equal the basin average PMP, an auxilary isohyet C'_1 enveloping the A_1, B_1, C_1, center, extending down to the A_2, B_2, C_2, center, and forming a border line with the northern end of C_2, isohyet was established.

d. The area enclosed by C'_1 isohyet was conveniently selected equal to that of C_2 isohyet. The average depth of rainfall corresponding to C'_1 isohyet is therefore equal to its C_2 counterpart.

e. The average rainfall depth over an area equivilent to $C'_1 + C_2$ was computed using the appropri- DDA curves of Figure III-6.

f. The average rainfall depths of "Step b," were then adjusted by multiplying them by the ratio of the average depth computed in "Step e" to that of "Step d." As mentioned earlier, this was done to insure that the precipitation averaged along the individual

isohyets must equal the basin average PMP.

g. The average depths of rainfall over the areas bounded by isohyets D and E which envelop both centers were computed in a manner similar to that outlined in Steps a and b.

h. The individual values for the selected isohyets were determined graphically as shown in Figure III-9 and summarized below:

i. compute the average value of any two adjacent isohyets using the following equation which assumes linear depth-area relationship.

$$\frac{V_n + V_{n+1}}{2} = \frac{P_{n+1}A_{n+1} - P_nA_n}{A_{n+1} - A_n}$$

in which

V_n, V_{n+1} = Values of isohyets number n and n+1. These numbers refer to the lettered isohyets shown in Figure III-8.

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A_n, A_{n+1} = Area bounded by isohyets number n and n +1. (Steps a and g above).

P_n, P_{n+1} = Average depth of rainfall over the areas bounded by

III-21

Figure III-9

III-22

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isohyets 1 and 2 (steps f and g above).

ii. Plot the computed average values of the isohyets versus the corresponding average areas, i.e. $\frac{V_n + V_{n+1}}{2}$ vs. $\frac{A_n + A_{n+1}}{2}$

iii. Locate the area enclosed by the individual isohyets on the plot of item ii above and draw straight lines between any two locations passing through the individual points and following the general shape of the curve of step ii above.

iv. Obtain the values of the isohyets corresponding to the straight lines of step iii.

v. Check the values obtained in step iv by computing the average values of any two isohyets (step iv) and comparing it against the values computed in step i.

vi. Repeat until the computed values of step iv

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are the same as those obtained using equation 1.

This trial and error procedure was found to be

very stable and its convergance was remarkable.

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The results of the areas (step a), separate storm depths of rainfall (step b), adjusted depths (step f and g), and isohyet values (step h) for the total duration of the probable maximum storm are summarized in Table III-4.

3. Time Distribution of PMP

The purpose of this step is to distribute the volume of the rain associated with the total duration of the probable maximum storm in time. A procedure similar to that outlined in Hydrometeorological Report No. 40 was employed. The procedure consists of the following two steps:

a. <u>Time concentration of PMP</u>: This expresses how much of the total rainfall in 72 hours is concentrated in the maximum (1st) 6-hour increment, the next highest (2nd) 6-hour increment, 3rd, 4th, etc. The total basin rain volume was distributed proportional to the 6-hour incremental values from the PMP depth-duration curve of Figure III-6. Thus the maximum (1st) 6-hour increment is the 6-hour PMP at the area of the basin, the next highest (2nd) 6-hour increment is the difference between 12-hour and 6-hour PMP etc.

This procedure is conservative in that it combines PMP for all durations in one storm event.[1]

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TABLE III-4

ISOHYET VALUES FOR THE TOTAL DURATION OF THE PROBABLE MAXIMUM STORM

Isohyet[1] No.	Area Enclosed[2] by Isohyet (sq.mi.)	Two Storm[3] Ave. Rainfall Depth (in.)	Adjusted[4] Ave. Rainfall Depth (in.)	Isohyet[5] Value (in.)
A_1	56	28.6	25.8	22.6
B_1	330	24.2	21.8	19.4
C_1	1,060	21.2	19.1	16.8
C_1	2,130	19.2	17.3	14.2
A_2	140	26.2	23.6	20.8
B_2	710	22.3	20.1	17.6
C_2	2,130	19.2	17.3	14.2
D	11,020	14.7	14.7	12.0
E	17,210	13.3	13.3	9.6

1 See Figure III-8
2 Step a of the procedure
3 Step b and g of the procedure
4 Step f of the procedure
5 Step h of the procedure

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b. Sequence of increments: The 6-hour incremental values of PMP over the entire Hudson Basin at Indian Point were rearranged in accordance with the following criteria recommended in Hydrometeorological Report No. 40:

i. Group the four highest 6-hour increments of the 72-hour PMP in a 24-hour period, the middle four increments in a 24-hour period, and the lowest four increments in a 24-hour period.

ii. Within each of these 24-hour periods, arrange the four increments in accordance with sequential requirements; that is, the second highest next to the highest, the third highest adjacent to these, and the fourth highest at either end.

iii. Arrange the three 24-hour periods in accordance with the sequential requirements; that is, the second highest 24-hour period next to the highest with the third at either end. Any possible sequence of three 24-hour periods is acceptable with the exception of placing the lowest 24-hour period in the middle.

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4. Computation of 6-hour increments of PMP

The incremental average depth for the 12 6-hour subdurations were obtained using the DDA curves of Figure III-6 and within-basin depth-area curves. The within-basin curves were derived for several subdurations and sizes of areas in a manner similar to that recommended in Hydrometeorological Report No. 40. The within-basin curves gave results higher than those based upon the entire basin ratio.

The results of the incremental average depth for the first 4 6-hour increments and for the second and third days are summarized in Table III-5. For successive 6-hour values within the second and third days the recommended U.S. Weather Bureau[1] percentages shown in Table III-5 were used.

The incremental values of the isohyets for the 12 subdurations were computed using the procedure outlined in the preceding item. The results are given in Table III-6.

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5. Computation of Subbasin Probable Maximum Precipitation & Runoff
The values of the probable maximum precipitation corresponding to the 12 subdurations for each one of the 28 subbasins shown in Figure II-1 were computed by planimetering the area between the isohyets within the subbasin boundaries.

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INSERT TABLE III-5-10 (DOCUMENT TABLE III5-10)

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The results of one of the subbasins (Catskill Creek – Subbasin 18.3) are summarized in Table III-7.

These increments were rearranged in accordance with the recommended sequence. The following rainfall losses, as recommended by Mr. Nunn of the AEC,[17] were then subtracted to obtain rainfall excesses for each subbasin and subduration:

Initial Losses = 1 in.
Infiltration Losses = .05 in./hour

The recommended rainfall losses agreed reasonably well with the October, 1955 storm and resulting stream flows at several U.S.G.S. gaging stations as will be discussed in the next section of this chapter.

The results of the probable maximum precipitation and runoff computations are given in Tables 8, 9, and 10 for the subbasins in the Upper Hudson, Mohawk and Lower Hudson Rivers respectively.

The incremental PMP values and related runoff for the entire Hudson River Basin above Indian Point are depicted in figure III-10. It is interesting to note that the runoff coefficient corresponding to these results in about 70 per cent more than twice the value used in the February, 1969 report.

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Figure III-10

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D. Unit Hydrograph Analysis
Six-hour unit hydrographs were established for each of the Hudson River subbasins shown in Figure II-1.

Unit hydrographs for 12 subbasins in the Upper Hudson were taken from the Stone & Webster Report[4] on the Upper Hudson. A copy of that report is appended to this report. A discussion of these

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unit hydrographs is reproduced from reference below: [4]

> "Unit hydrographs were determined from records of nine gaging stations located on the various streams and reservoirs within the basin having drainage areas varying in size from 90 to 1,044 square miles. In general, the unit hydrographs were developed from the hurricane storm of September 1938 and compared with the next largest storm for which data were available. The second storm used varied from area to area. The storm of September 1938 produced the largest floods without snow melt for which adequate records are available. It was not possible to develop a unit hydrograph for the Indian Lake Reservoir and Batten Kill from the September 1938 storm because of inadequate rainfall data, and other storms were used. A computer program described by D. W. Newton and J. W. Vinyard, was used in developing the unit hydrographs for the gaged areas.

> Unit hydrographs for approximately 70 percent of the total drainage area were developed from rainfall and runoff records. The unit hydrographs for the Sacandaga Reservoir and Indian Lake inflows and the Hudson River at the USGS gage at Gooley were used without modification. The unit hydrographs developed at the gaging stations on the Schroon River, Batten Kill and Kayaderosseras Creek were transposed to their respective mouths using Snyder's coefficients and the method outlined in Unit Hydrographs – Part I Principles and Determinations."

Synthetic unit hydrographs were derived for the three subbasins in the Mohawk Basin, Subbasins 9, 13, and 14 in the Upper Hudson and all the Lower Hudson Subbasins (excluding Subbasin 18.5). Taylor & Schwarz's method[5] was used for this purpose. This method was developed for basins in the north and middle Atlantic states. The basic data used in the development includes the Hoosic River, one of the major tributary streams of the Hudson River.

No unit hydrograph was derived for subbasin 18.5. This subbasin has a small drainage area and the runoff from the precipitation over this area was assumed to be instantaneous.

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TABLE III-11

III-39

Figure III-11

III-40

TABLE III-12

III-41

Figure III-15

III-45

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The basic equations used in the derivation of the synthetic unit hydrographs are listed in Table III-11.

The construction of the shape of the synthetic unit hydrographs was guided by the empirical Corps of Engineers[6] relationship between the peak discharge rate and the width of unit hydrographs at ordinates exceeding approximately 50 per cent of the maximum. This relationship corresponding to ordinates equal to 50 and 75 per cent of the peak ordinate is reproduced in Figure III-11.

The area under the synthetic unit hydrographs which should be equivalent to one inch of direct runoff provided another control.

The basic subbasin characteristics used in deriving the unit hydrographs are summarized in Table III-12. The computed unit hydrograph peak flows and lag time are also listed in Table III-12.

Three typical unit hydrographs for three subbasins in the Upper Hudson, Mohawk and Lower Hudson Basins are depicted in Figure III-12, III-13, and III-14 respectively.

Figure III-15 compares the unit hydrograph peaks used in this study with those prepared by the Corps of Engineers through September, 1961.

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The Corps of Engineers results are represented by a mean, upper and lower limit lines which were drawn to envelop some 200 observed peak discharge values of 6-hour unit hydrographs.

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The Hudson River Basin synthetic unit hydrograph values are represented by open circles. Notice that the synthetic peaks used in this study are higher than those represented by the Corps of Engineers mean line.

The Upper Hudson unit hydrograph peaks which were developed from the hurricane storm of September, 1938 are also shown in Figure III-15. These points fall within the upper and lower limit lines with the majority of them above the mean line. Only subbasin #5 (Schroon River) value is below the lower limit line. Since the Schroon River Subbasin has a very small slope ($6x10^{-4}$) and large natural storage capacity and since its unit hydrograph was developed from observed data, it was not found necessary to adjust its peak.

A more elaborate verification of the developed Hudson River Basin unit hydrographs based upon observed precipitation and runoff data was conducted. The flood of October 1955[18] was used for this purpose because of availability of adequate rainfall data in the Hudson River Basin.

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Storms swept the New York and southern New England area in three periods; October 6-8, 13-17, 30-31. The worst floods occurred in mid-October, 1 or 2 days before or after October 16. The areas most affected were western Massachusetts, western Connecticut, southeastern New York, and a separated area in south-central New York and north-central Pennsylvania.

Maximum 4-day rainfall (October 14-17) at Esopus Creek and Schoharie Creek Subbasins was 17.72 and 16.77 inches respectively. October rainfall at individual stations ranged from 3.14 inches at Plattsburgh, New York to 25.27 inches at West Shokan. This last figure is about 60 per cent of the mean annual rainfall in New York.

The Mohawk River discharge observed at Cohoes reached a peak of 100,000 cfs on October 17. The corresponding Hudson River peak at Green Island and Wallkill River peak at Gardiner in the Lower Hudson Basin were 113, 000 cfs on October 17 and 30,800 cfs on October 16 respectively.

The unit hydrographs developed for the above-mentioned basins (all of the Upper Hudson, Mohawk and Wallkill River unit hydrographs) and the rainfall losses used in this study were verified using the October 1955 storm rainfall and runoff data in the following manner:

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1. Isohyetal maps corresponding to 12 6-hour increments of rainfall commencing at 24:00 on October 13, 1955 were constructed. Hourly precipitation data observed at 25 weather bureau station in the Hudson River Basin were used for this purpose. A sample of these isohyetal maps is depicted in Figure III-16.
2. The average depth of rainfall for the 12 subdurations for each subbasin was obtained.
3. The rainfall losses used previously were then subtracted to obtain the rainfall excesses for each subbasin; i.e. initial losses of 1 inch and infiltration losses of 0.05 inch/hour. The rainfall and runoff results for the Upper Hudson, Mohawk and Wallkill River Basins are listed in Tables III-13, III-14 and III-15 respectively.
4. Using the beginning of the storm (24:00 October 13) as a common time base, the runoffs were applied to the developed 6-hour unit hydrographs to produce successive 6-hour flood hydrographs for each subbasin.
5. The subbasin flood hydrographs were then combined and routed downstream to the gaging stations; i.e. Cohoes, Green Island and Gardiner. The routing procedure employed for this purpose will be presented in the next section of this chapter.
6. The generated floods of "Step 5" above are compared to the observed Hudson, Mohawk and Wallkill River data in Figures III-17, III-18 and III-19 respectively. The observed data were obtained from the U.S.G.S. hourly records. The agreement between the

III-49

Figure III-16

Figure III-17

III-55

computed Mohawk and Upper Hudson River floods and their measured counterparts is remarkable. The computed values are somewhat higher than the observed data.

The agreement between the observed and computed results for the Wallkill River is reasonable. The generated peak is about 10 per cent highter than its measured counterpart. This and the other minor differences in the Upper Hudson and Mohawk River results may be due to the non-uniform aerial distribution over large subbasins, base flow and losses estimates. The non-uniform aerial distribution over the long and narrow Wallkill River Subbasin (100 miles x 15 miles) was significant and maybe the reason behind the 10 per cent disagreement.

The good agreement between the measured and computed results indicate that the use of the developed unit hydrographs and recommended rain-

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fall losses is justified.

E. Development of the Hudson River Probable Maximum Flood at Indian Point

1 Description of Procedure: The procedure followed for determination of the probable maximum flood at Indian Point was delineated earlier and may be summarized as follows:

a. Flood hydrographs for all of the Hudson River Subbasins were established using the probable maximum

precipitation (PMP) amounts from the selected probable maximum storm (PMS) together with rainfall losses (Section C above) and the derived unit hydrographs (Section D above).

b. With base flows added the flood hydrographs were combined and/or routed downstream to give the inflows into the Hudson River from its tributaries. Inflows to the Sacandaga and Ashokan reservoirs were routed through the reservoirs. The reservoirs' outflows were then combined with the appropriate subbasin flood hydrographs to define the inflows in the main channel.

c. The main river inflows were then combined with the Hudson River bank subbasin flood hydrographs and routed downstream to Indian Point using the beginning of the probable maximum precipitation as the time origin.

The specific details of this procedure, excluding reservoir flood routing, are presented in this section. When possible, the Mohawk River Basin computations will be used in a detailed manner to illustrate the adopted procedure.

The results of the reservoir outflows are used in this section. How-

ever, the details of the reservoir flood routing procedure, are discussed in the next chapter in connection with a thorough treatment of dam failure analysis.

2. Mohawk River Flood Development

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a. Mohawk River Subbasins
As indicated earlier, the Mohawk River Basin was divided into three subbasins. These are designated as subbasins numbers 15, 16 and 17 in Figure II-1.

Subbasin No. 15 envelops an area of 2170 square miles and includes the Mohawk River drainage area above the mouth of Schoharie Creek.

Subbasin No. 16 includes Schoharie Creek basin which drains an area of 926 square miles.

Subbasin No. 17 includes the remaining 366 square miles of the Mohawk River Basin and extends from the mouth of Schoharie Creek to Cohoes at the junction of the Mohawk and Upper Hudson Rivers.

b. Subbasin Flood Hydrographs

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To illustrate the procedures employed in this study, a detailed description of the basic flood data and results of a typical subbasin is presented below. The Schoharie Creek Subbasin was arbitrarily selected for this purpose.

Figure III-20 summarizes the necessary basic data for Schoharie Creek. An isohyetal map of the subbasin appears in the upper right hand corner. Isohyets D and C^2 of the selected PMS (Figure III-8) are shown in Figure III-20. Several values interpolated between the pattern isohyets are also shown. The values shown in the Figure correspond to the 72-hour duration. The values of the 12 6-hour subdurations are listed in Table III-6.

The subbasin precipitation, determined by planimetering the area between the isohyets within the subduration is depicted in the upper left hand hyetograph. The rainfall losses (1" initial and .05"/hr. infiltration) as well as the rainfall excesses (shaded area) are also shown. The rainfall excesses were obtained by subtracting the rainfall losses from the incremental rainfall

III-61

FIGURE III-20

III-62

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values. This procedure is outlined in Tables III-9 through III-12.

The synthetic 6-hour unit hydrograph for this sub-basin is represented by the broken line in the lower left hand side of the figure. The method used to derive this as well as the other synthetic unit hydrographs is outlined in Table III-11. The results are summarized in Table III-12. The Schoharie basin characteristics required for the derivation of the unit hydrograph are listed in Figure III-20.

The Schoharie Creek flood hydrograph is also given in the figure. This hydrograph was obtained by applying the rainfall excesses to the 6-hour unit hydrograph.

c. Flood Hydrograph Combination and River Channel Flood Routing

A base flow of 1 cfs/square mile was added to the product of the rainfall excesses and unit hydrograph values to obtain the subbasin outflow. This base flow value which represents the stream flow main-

tained by ground water return and subsurface storm flow represents a long term average fore the Hudson River Basin. For the Upper Hudson River Subbasins, base flows obtained from observed flood hydrographs were used. These base flows are listed in Table 4 of the Stone & Webster Report[4].

The generated flood curve in Figure III-20 includes the appropriate base flow value for the Schoharie Creek Subbasin, i.e. 926 cfs.

The generated flood hydrographs for the three Mohawk River Subbasins were combined and routed downstream to the Hudson River. The procedure used for this purpose is depicted in Figure III-21.

The individual flood hydrographs for the three sub-basins are represented by the solid curves in Figure III-21. The Mohawk River Flood downstream of its

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conjunction with the Schoharie Creek was obtained by combining the flood hydrographs of subbasins 15 and 16. The resultant is represented by (15 + 16) curve in Figure III-21.

III-64 Figure III-21
III-65

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The resultant flood hydrograph was then routed through the Mohawk River to Cohoes. The output is represented by the broken curve in Figure III-21. The classical Muskingum coefficient method for routing[7] was used for this purpose. This method will be described later.

The routed hydrograph was then combined with the flood hydrograph of subbasin #7 to obtain the total Mohawk River flood hydrograph at Cohoes. The resultant hydrograph is shown in Figure III-21

The foregoing procedure and the one outlined in the previous section of this chapter were programmed for solution of RAPIDATA time-sharing facilities. A listing of the Mohawk River Program is given in Plate III-1.

Plate III-2 is a listing of the input or data file for the Mohawk River Program which includes rainfall excess (Table III-9), unit hydrograph ordinates (Figure III-21) above flows for the three subbasins.

The solution printouts including the flood hydro-

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graph for each subbasin and the results used to construct Figure III-20 are shown in Plate III-3. A more detailed description of the unit hydrographs for all subbasins is also included in Plate III-3.

A description of the flood routing procedure follows.

The Muskingum Method expresses the outflow as the summation of products of routing constants and inflows. The relationship is expressed in equation III-2

$$o_2 - o_1 = \left[\frac{2\ \Delta t}{2k\ (1-x) + \Delta t}\right] \left[I_1 - o_1\right]$$

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$$+ \left[\frac{\Delta t - 2kx}{2k(1-x)+\Delta t}\right]\left[I_2 - I_1\right] \qquad \text{III-2}$$

in which

o_1, o_2 = Outflow from reach 1 and 2

I_1, I_2 = Inflow at upstream end of reach 1 and 2

Δt = Length of the routing period having a maximum value of 2kx and may be taken as equal to k. The routed hydrograph is relatively insensitive to the value of Δt.

k = Time of travel of flood wave and also

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the change of storage per unit change of discharge.

x = A dimension less constant representing an index of the wedge storage in a routing reach.

The routing coefficients x and k used in this study are summarized in Table III-16. The Upper Hudson River coefficients were determined by Stone & Webster[4] from available flood records.

The October, 1955 storm flood records described in the previous section of this chapter were analyzed to determine the Mohawk River routing coefficients.

The Lower Hudson River routing coefficients were evaluated using a step backwater calculation as will be shown in the next section of this chapter.

Except for two subreaches in the Upper Hudson a value of 0.3 was used for the dimensionless constant x. This value is conservative and is representative of wide rectangular channels.[7] The influence of variation in x on the Hudson

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River routed hydrographs is discussed in the last section of this chapter.

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3. Upper Hudson River Flood Development

A procedure similar to the one outlined in the preceding section was used to derive the Upper Hudson River flood hydrograph at Green Island.

Because of the presence of the Sacandaga Reservoir which controls nearly 30 per cent of the total Upper Hudson basin area, the basin was divided into two segments. The first portion extends from the source to the mouth of Sacandaga River and the second includes the remaining portion of the basin, i.e. from Sacandaga River to Mechanicville.

A computer program, data file, and printouts similar to their Mohawk River counterparts were developed for both segments and are presented in Plates III-4 through III-9. The results are presented graphically in Figure III-22.

The upper portion of this basin includes subbasins 1 through 8. the outflow from subbasin 8 (Sacandaga River Basin) was obtained by routing the flood hydrograph through Sacandaga Reservoir using

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a detailed reservoir flood routing procedure. The procedure is described in the next chapter.

The flood hydrographs of subbasins 1 through 7 were combined and/or routed downstream to Sacandaga River. The Sacandaga Reservoir outflow was then combined with the resultant of the upstream subbasins. This new flood hydrograph was combined and/or routed with the hydrographs of subbasin 9 through 14 to obtain the resultant Upper Hudson River flood hydrograph at Mechanicville.

The Upper Hudson unit hydrographs at Mechanicville, upstream and downstream of Sacandaga River are shown in Figure III-22. The outflow from Sacandaga Reservoir is also depicted.

The routing coefficients used were obtained from the Stone & Webster Report. A description of the method used to determine these coefficients is reproduced from reference #4 below:

> "The interrelated constants X and K, used to determine the coefficients for routing by the coefficient me method, were evaluated on the section of the river above Thurman Station, because of lack of adequate flood data, evaluation of these constants was made from the stage-discharge-volume relationship for each reach. This

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procedure required determination first of K, which is the travel time for an elemental discharge wave to traverse the reach. Then an x, which is a measure of the wedge storage in the reach, was assumed and the flood was routed.

Finally, to check the correctness of the assumed X, the actual wedge storage was calculated using a step backwater calculation with varying flow within the reach corresponding to the inflow and outflow hydrographs. Trials were continued until the assumed and calculated X's were substantially in agreement."

The Mechanicville flood hydrograph was then combined with the Cohoes hydrograph to obtain the flood hydrograph at Green Island. The Mechanicville hydrograph was not routed to Cohoes since Subbasin 14 of the Upper Hudson Basin extends all the way down to the junction of the Mohawk and Hudson Rivers near Troy. The results are shown in Figure III-23.

4. Lower Hudson River Flood Development

A procedure similar to the Mohawk and Upper Hudson River procedures was employed to route the river hydrographs to the Indian Point site. The Lower Hudson River computer program, input or data file and

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printouts are listed in Plates III-10 through III-12. The input or data file includes the resultant flood hydrograph at Green Island, Ashokan Reservoir outflow, rainfall excess and unit hydrograph data for the Lower Hudson Subbasins.

The specific details of the Ashokan Reservoir flood routing is presented in the next chapter.

The coefficients used in the Lower Hudson River Channel routing are described below.

A conservative value of 0.3 representative of a wide rectangular channel was used for the dimensionless coefficient x. A lower x value corresponding to wide parabolic channel results in a small reduction in the probable maximum flood at Indian Point. The Lower Hudson River Channel is somewhere in between these two shapes. To be on the conservative side, however, the value of 0.3 was adopted. The results of this study, as will be shown later, indicate that the probable maximum flood at Indian Point is relatively insensitive to the value of x.

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The celerity method suggested by the Corps of Engineers[7, 19] was

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employed to determine the coefficient k. This method is based upon Seddon's principle, which may be expressed by the following equation:

$$V_w = \left[\frac{1}{B}\right] \left[\frac{dq}{dy}\right] \qquad \text{III-3}$$

in which

v_w = Rate of movement of flood wave

B = Breadth of channel at water surface

q = discharge

y = Height above bed

$\frac{dq}{dy}$ = Slope of discharge rating curve of a station whose cross-section is representative of the reach for steady flow

The value of k is then the ratio of reach length to wave celerity V_w.

The relationship of Equation III-3 was discovered experimentally by James A. Seddon in studies of flood movements on the Mississippi and Missouri Rivers in 1889. The same relationship was developed mathematically by Kleitz in 1877.[19]

Several Lower Hudson River water surface profiles corresponding to flows ranging from 20,000 cfs to 1,100,000 cfs were developed using an accepted backwater program. The results are shown in

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Figure III-24. The backwater program used is described in detail in Chapter V of this report.

The Lower Hudson River between Green Island and Indian Point was then divided into 11 reaches. A discharge rating curve was established for each reach using the surface profile curves of Figure III-24. The discharge rating curves are shown in Figure III-25.

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As suggested by the Corps of Engineers,[7] a single value of k corresponding to the average of the initial flow and anticipated peak outflow for each subreach was used. The anticipated peak outflow was determined by trial and error.

The results of this step indicate that the total time of travel of the flood wave between Green island and Indian Point (a distance of more than 150 river miles) is 16 hours. The values for the individual subreaches are listed in Table III-16.

Several other x and k values were used to examine the influence of the routing coefficients on the probable maximum flood at Indian Point. The results are depicted in Figure III-26. It can be clearly seen that the PMF is relatively insensitive to the value of x.

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The dependence of the Indian Point PMF on k is due to the degree of synchronization of the Green Island peak and Esopus Creek peak. In other words, the shorter the time of travel between Green Island and Esopus Creek the smaller the lag time between the two peaks and vice versa.

The use of a weighted average k equivalent to the ratio of the area under k vs. time curve to the total time for each subreach resulted in a longer time of travel, i.e. lower PMF at Indian Point. However, as an added measure of conservatism, the averaging technique suggested in reference number 7 was adopted.

The routed hydrographs at several locations along the Hudson River are shown in Figure III-27.

5. Probable Maximum Flood at Indian Point

The peak discharge thus obtained at the Indian Point site amounts to 1,100,000 cfs and represents the probable maximum flood at Indian Point.

Onn the basis of drainage area size, this value agrees reasonably well with its Susquehana, Delaware and Potomac Rivers counterparts shown in Figure III-2.

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It is also about 25 per cent and 60 per cent higher than the dimensional analysis value of 880,000 cfs of Figure III-5 and

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the statistically evaluated 10,000 year flood at Indian Point of Figure II-5.

As a final verification step, the unit hydrographs corresponding to the Green Island flood hydrograph and the Indian Point probable maximum flood were derived and compared to the Corps of Engineers generalized curves of Figure III-15. The 72-hour unit hydrographs were developed by inversing the flood hydrograph procedure, i.e. deducting the base flow from the flood hydrograph and dividing the resultant by the appropriate rainfall excess. The 6-hour unit hydrographs were then developed using the standard S-hydrograph method.[5]

The Green Island and Indian Point unit hydrographs thus obtained are shown in Figures III-28 and III-29 respectively. As shown in Figure III-15 these unit hydrographs produced peak discharges in excess of the Corps of Engineers mean line values. It was then concluded that the Indian Point PMF of 1,100,000 cfs represents a conservative estimate.

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IV. FLOOD ROUTING & DAM FAILURE ANALYSIS OF HUDSON RIVER BASIN RESERVOIRS

A. Introduction

The purpose of this chapter is to present the results of the dam failure analysis of the Hudson river Basin Reservoirs and to determine the effect on the maximum possible stage at Indian Point 3 point.

A conservative estimate of the combined failure effect is presented first. The combined effect was obtained by locating the major Upper Hudson and Mohawk Basin reservoirs at Sacandaga and the Lower Hudson Basin reservoirs at Ashokan.

Following this is a detailed evaluation of the probable maximum effect of the largest two reservoirs in the basin (Sacandaga and Ashokan).

Reservoir flood routing procedures used in this study and referred to in the previous chapter are also presented in this chapter.

B. Combined Failure Effect

The purpose of this section is to show that the effect of failure of dams other than Sacandaga and Ashokan is insignificant.

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IV-1

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The major reservoirs located upstream of the Indian Point site and their characteristics are summerized in Table II-5. The table values clearly indicate that the largest two reservoirs are the Sacandaga in the Upper Hudson and the Ashokan in the Lower Hudson Basin. The total capacity of both of these reservoirs is more than 250 per cent in excess of all other reservoirs combined. Moreover, most of these reservoirs are located in the Upper Hudson and Mohawk River basins and are all farther from Indian Point than is Sacandaga or Ashokan.

to obtain a conservative, but realistic, estimate of the failure effect of these dams, the Delta, Hinckley, Schoharie and Indian Lake were located at Sacandaga and Rondout at Ashokan.

A formula developed by Lishtwan[20] and a wave profile expression derived by Chow[21] were then used to compute the flow resulting from dam failure of the hypothetical Sacandaga and Ashokan Reservoirs, i.e. including the relocated capacity of the other reservoirs.

Chow's[21] equation of the wave profile, resulting from the failure of a dam is in the form of:

$$X = 2t\sqrt{gh} - 3t\sqrt{gy} \qquad \text{(IV-1)}$$

in which
X = The distance measured from the dam site
IV-2

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h = The depth of the impounding water

y = The depth of the wave profile

t = The time after dam failure

Equation IV-1 may also be written as follows:

$$y = \frac{1}{g}\left[\frac{2\sqrt{gh} - \frac{x}{t}}{3}\right]^2 \qquad \text{(IV-2)}$$

Integrating this equation from the origin at the dam site to the

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wave front $2t\sqrt{gh}$ and solving for the total volume of water $\int yBdx$,

$$\text{Volume} = \frac{8}{27}\sqrt{g}\, Bh^{3/2}t \qquad \text{(IV-3)}$$

where

B = channel width

the initial discharge after dam failure Qo may be computed by dividing the volume of equation IV-3 by a very conservative failure time of one second. Therefore,

$$Qo = \frac{8\sqrt{g}}{27}\, Bh^{3/2t} \qquad \text{(IV-4)}$$

Lishtwan's [20] equation of the flow downstream of the dam site may be expressed as follows:

$$Q' = \frac{Wo\ Qo}{Wo + QoLa} \qquad \text{(IV-5)}$$

where

Q' = The flow resulting from dam failure at a distance L downstream

of the dam site

Qo = The initial dam failure discharge.

W_o = The volume of the stored water.

a = $\frac{1}{2}\left(\frac{1}{v_2} - \frac{1}{v_1}\right)$ = 0.224 for basins having a drainage area of up to about 20,000 square miles and river slopes ranging from .1 x 10^{-4} to 5 x 10^{-4}

v_1 = The wave front velocity.

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v_2 = The wave tail velocity.

Equations IV-4 and IV-5 were used to determine the flow at Indian Point 3 Point resulting from failure of the hypothetical reservoirs at Sacandaga and Ashokan. The computations and results are summarized in plate IV-1.

Comparison of these results with those resulting from the failure of Sacandaga and Ashokan Dams alone indicated that the effect of the smaller dams represents a small percentage of the total effect. this, coupled with the fact that these dams are farther from Indian Point 3 Point then the two major reservoirs, concentrated the detailed analy-ysis on the failure of Sacandaga and Ashokan Dams alone.

Furthermore, no advantage was taken of the storage available in the the smaller reservoirs, (Delta, Hinckley, Gilboa, Indian Lake and

IV-4
IV-5

IV-6 -60-

Rondout) from the standpoint of PMP flood routing. In other words, the runoff resulting from the probable maximum precipitation was routed through Sacandaga and Ashokan Reservoirs only. The flood storage capacity of these two reservoirs is 12 and 2.3 billion cubic feet respectively.

C, Sacandaga Reservoir Studies

1. Description of Reservoir and Dam

As indicated earlier, the Sacandaga reservoir is the largest regu-lating feature in the entire Hudson River Basin. It controls all of the Sacandaga River drainage area of 1044 square miles – Subbasin No.8 in Figure II-1. The reservoir is of a multiple use – stream regulation and flood control – type and was formed by con-struction of the Conklingville Dam in 1930. The Conklingville Dam is lcoated on the Sacandaga River which discharges into the Upper Hudson River some 70 miles upsteam of Troy. The Sacandaga Reservoir and Conklingville Dam are located in Figure IV-1.

At elevation 771.0 feet above mean sea level (Crest of Conklingville Dam Spillway), the reservoir has a surface area of 27,000 acres and a total capacity of 37.8 billion cubic feet. The total capacity consists of the following componenets:

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IV-8

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Use	Elevation	Storage Capacity Billion Cubic Feet
Dead Storage	700 – 735	4.60
Stream Regulation Storage	736 – 768	29.67
Flood Control Storage	768 – 771	3.45

Figure IV-2 shows the capacity curve of Sacandaga Reservoir.

Other interesting characteristics are summarized below:

Length = 27 miles

Width = 2,000 to 28,000 feet

Length of Shore Line = 125 miles

Flow Line Elevation = 771.0' above sea level

Storage Above Elevation 768.0 = 12 billion cubic feet (total

flood storage)

A typical cross-section of the Conklingville Dam is depicted in Figure IV-3.
The dam was built by the state of New York and is maintained and operated by the Hudson River-Black River Regulating District.
It is an earth dam with a relatively impermeable core founded on rock and earth, with a concrete gravity spillway built on rock.
The earth dam was built by the semi-hydraulic fill method. The
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earth fill was obtained from borrow pits located at the south abutment and consisted of glacial drift that proved to be ideal for the purpose.

Core samples, taken after completion of the dam construction, showed that the entire dam material is very well packed and the non-core material is an impermeable as the core material itself.[16]
The soil characteristics of the core material are listed in table IV-1A and documented in Figure IV-3A. Subsequent to award of the construction contract, the rock excavation for the spillway channel

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was increased. The extra rock material was added to the downstream toe of the dam, providing an unusually large rock toe section.

The dam is very stable and has proven its ability to withstand severe earthquakes[4] and floods.[16]

A two-day inspection of the dam and reservoir, conducted after occurrence of the highest recorded earthquake on April 20, 1931 at Lake George, new York, showed no evidence of damage.[4]

The dam is relatively impervious and seepage is reduced by the use of a very broad base. The seepage water collects in the unusually large rock toe and moves to a point where it is safely discharged. the maximum measured seepage and leakage through the dam is about 20 cfs.[16]

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IV-14

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The upstream slope of the dam is protected against wave action by a sizable cover of riprap. The riprap cover extends to above the crest of the core.
Major dam dimenstions and information are summarized below.

Crest Width = 43 feet

Crest Elevation = 795.0 feet

Base Width = 650 feet

Maximum Height = 96 feet

Length of Dam = 1100 feet

Length of Spillway = 405 feet

Crest of Spillway Elevation = 771.0 feet

Crest of Core Elevation = 780.0 feet

Record High Reservoir Stage Elevtion = 769.72 feet (on May 21, 1969)

The outlet works consist of three 8 foot diameter Dow Valves an two 18 foot by 8 foot siphons. The spillway and diversion canal are located in a rock section away from and to the left of the earth dam. The spillway is ungated free overflow section 405 feet long. In addition, a power house with a pumping capacity of some 5600 cfs

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at elevation 769.0 feet is also available.[16]

Figure IV-4 shows the combined discharge variation with the stage in the Sacandaga Reservoir. The curve values were based upon re-

IV-15

IV-16

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sults of the January – February, 1931 test and information supplied by the Hudson River – Black River Regulating District. The curve values do not include the power house pumping capacity.

2. Flood Routing through Sacandage Reservoir

The outflow pattern that is produced by the Sacandaga River Basin (Subbasin #8) inflow into the reservoir was determined by an analytical procedure based upon a stepwise analysis of the various hydraulic occurrences: varying inflow, changing water level and varying outflow. The equation used for this purpose may be expressed follows:

$$\left[2\left(\frac{I_n + I_{n+1}}{2}\right) - \left(\frac{O_n + O_{n+1}}{2}\right)\right]\left(t_{n+1} - t_n\right) = S_{n+1} - S_n \qquad \text{(IV-6)}$$

in which,

I = Rate of inflow into the reservoir
O = Rate of outflow from the reservoir
S = Available storage above spillway level
n, n+1 = Subscripts denoting successive intervals of time of length $t_{n+1} - t_n$

Equation IV-6 was programmed as shown on Plate IV-2 and the input or data file included:

a. Subbasin #8 unit hydrograph – see Table III-11.

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b. Subbasin #8 inflow storm hydrograph computed using the procedure of Section E of Chapter III.
c. Reservoir stage – discharge curve – see Figure IV-4. This curve does not take advantage of the available power house pumping capacity of 5600 cfs. This added

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discharge capacity was not considered to be on the safe side and to account for the possibility of inoperative pumps during probable maximum conditions.

d. Reservoir storage curve – see Figure IV-2. A straight line extrapolation was used for elevations in excess of crest of spillway elevation. This was done in accordance with the data supplied by the HR-BRRD.[16]

e. A time interval of six hours.

It was assumed that the outlet works would start operation when the reservoir level reaches elevation 771.0, the crest of Spillway elevation. It was also assumed that the initial reservoir level, prior to the storm, would be at elevation 768.0 (bottom of flood storage – Figure IV-2). These assumptions are very conservative since the PMP considered in this report is a late summer or fall storm and the reservoir would usually be below the flood storage elevation and the outlet work would probably be started before the

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level reaches elevation 771.0. The long term late summer-fall average reservoir elevation ranges between 758.37 in August and 752.71 in November.

The program output includes subbasin #8 flood hydrograph (inflow to the reservoir), reservoir outflow hydrograph, and reservoir water surface elevation.

The program, data file and printout are shown in plates IV-2 through IV-4 respectively.

The results are shown graphically in Figures IV-5 and IV-6. Figure IV-5 depicts the reservoir inflow and outflow hydrographs. The variation in water surface elevation is documented in Figure IV-6. The computed reservoir inflow reached a peak of 262,362 cfs some 42 hours subsequent to the beginning of the probable maximum precipitation. The corresponding maximum reservoir outflow is 69,500 cfs occurring 78 hours after the beginning of PMP.

The maximum reservoir stage resulting frm routing subbasin #8 flood hydrograph is elevation 783.94. This elevation is about 11 feet below the crest of the Conklingville Dam and less than 4 feet above

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IV-24

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the crest of the core. This value is almost the same as the maximum possible level, elevation 784.0, determined by Stone & Webster.[4]

These findings were discussed with Mr. Robert Forrest, Chief Engineer of the Board of Hudson River – Black River Regulating District, on April 27, 1970. Mr. Forrest stated that the Conklingville Dam is capable of "taking this load without damage." to support this conclusion, he gave the following reasons:

a. The dam was designed for large surcharge and has a sufficient freeboard (24 feet above the spillway crest).
b. The dam has a substantial cross-section and has proven its ability to withstand sever earthquakes.
c. The excellent material used and the settlement and consolidation during the past 40 years have resulted in an inherently stable dam. Core samples taken after completion of construction showed that the dam material is homogenous and relatively impermeable.
d. The unusually large rock toe section downstream of the dam provided a safe disposal system.
e. Perculation from the top will not occur because of the paved highway which has its crest at elevation 795.6

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These comments and the dam failure analysis of the next section of this chapter indicate that the Conklingville Dam will not fail.

3. Conklingville Dam Failure Analysis

The results of the previous section showed that the Conklingville Dam will not be overtopped. Another dam stability criterion involving the position of the free water surface within the dam was considered. This criterion was evaluated to determine the effect of the computed maximum reservoir elevation on the safety of the dam.

In general, earth dams will fail when seepage water escapes at the

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downstream face of the embankment in sufficient volume to erode the surface and carry soil grains with it. This phenomenon is usually called "piping." To avoid this, seepage is usually deflected from the face of a dam by the proper use of an earth core on a rock toe.

As indicated earlier, the Conklingville Dam is provided with an unusually large rock toe as well as an earth core. The position of the free water surface within the dam was estimated for the following two downstream conditions:

Condition 1 No water downstream of the dam

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Condition 2 A more conservative case involving a water depth of 20 feet downstream of the dam.

the assumptions and equations used to determine the location of the seepage curve are summarized in Plate IV-5.

For the purpose of computing the seepage curve, the dam cross-section was simplified by the sketch shown in Plate IV-5 and divided into three parts A, B and C. No advantage was taken of the deflection in curve caused by the presence of the core. In other words, the material was assumed to be homogenous. The foundations were assumed to be practically impervious. These assumptions are justifiable,[16] conservative and realistic

The equations are based upon Darcy's equation:

$q = kiy$ IV-7

or $q/k = iy$ IV-8

in which

q = seepage per unit width

y = depth of seepage curve

i = hydraulic quadient and is equivalent to the slope of the seepage curve $\frac{dy}{dx}$

k = permeability coefficient

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The product (iy) for parts A, B and C may be determined as follows:

Part A:

The flow lines in this portion may be approximated by circles having a common origin at the location of maximum water elevation (Point A in Plate IV-5) using a head loss of (a), the maximum depth at Point A may be taken as the difference between the total depth (H) and the head loss (a). The hydraulic quadient is equivalent to (a) divided by the arc length $\pi (90 - \theta) h/360$ where $\tan \theta$ is the slope of the up-stream face of the dam. Substitution in equation IV-8 yields:

$$q/k = \frac{ah}{\pi \ (90 - \theta)h} \cong \frac{115 \ (H-h)}{90 - \theta}$$

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Part B:
Integration of Darcy's equation and the use of the boundary conditions y(x-o) = h and y(x=s) = h_1 yields

$$q/k = \frac{h^2 - h_1^2}{2s} \qquad \text{IV-10}$$

and

$$x = \frac{k}{q}\left(\frac{h^2 - y^2}{2}\right) \qquad \text{IV-11}$$

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Part C:
When the distance s_1\ is small by comparison to s, the slope of the flow line at point B may be taken as 1, and Equation IV-8 becomes:

$$q/k = h_1 \qquad \text{IV-12}$$

When a downstream water depth of h_o is considered, equation IV-12 becomes:

$$q/k = h_1 - h_o \qquad \text{IV-13}$$

Since the dam is assumed to be homogeneous, the variable q/k for the three parts is the same under steady state conditions. Equations IV-9, IV-10 and IV-12 can be solved simultaneously to obtain the three unknowns h_1, h and q/k and equation IV-11 gives the profile within the middle part of the dam "B". The parameters used in solving these equations include:

H = 80 feet
$\theta = 18.43^o$
h_o = o and 20 feet for conditions 1 and 2 respectively
s = 169 feet and 189 feet for conditions 1 and 2 respectively

The results for both conditions are depicted in Figure IV-7. In both cases, the downstream ordinate of the seepage curve is well below the crest of the rock toe.

This indicates that the seepage water would collect in the rock toe

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IV-33

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and would be safely discharged without carrying soil grains with it. In other words piping would not occur.

As a conclusion, therefore, the safety of the Conklingville Dam under the probable maximum precipitation conditions considered in

this study would not fail from earthquake, overtopping or piping.

C. Ashokan Dam Studies

1. Introduction

This section presents the results of the Ashokan Reservoir studies. As in the previous section, the presentation begins with a general description of the reservoir and dam followed by delineation of the reservoir flood routing procedure used in connection with the Indian Point PMF determination. Evaluation of the Ashokan Dam failure is given next.

The work required to achieve the dam failure analysis objective includes:

a. Determination of the effect of routing the Esopus Creek flood hydrograph resulting from the selected probable maximum storm (Figure III-8) through the reservoir on the dam.

b. Since the results of item (a) above showed that the dam will not fail, a more critical condition involving

IV-34
IV-35

the effect of a local probable maximum storm (over Esopus Creek Basin only) on the dam was investigated. This condition resulted in a partial failure of the dam.

The last item in this section documents the results of the combined effect at Indian Point resulting from simultaneous occurrence of the Ashokan Dam failure and a Hudson River Standard Project flood.

2. Description of Ashokan Reservoir and Dam

The Ashokan Reservoir is the second largest regulating feature in the Hudson River Basin. It controls some 257 square miles of the Esopus Creek Basin which drains the southerly slope of the Catskill Mountains. This area is designated as Subbasin No. 19 in Figure II-1.

The reservoir is an artificial lake about 12 miles long, located about six miles west of the city of Kingston, and is part of the New York City Water Supply System. It was formed by construction of the Ashokan Dam during 1907-11. The dam is located on Esopus Creek which discharges into the Hudson River some 60 miles upstream of Indian Point. The layout of the reservoir is shown in Figure IV-8.

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The reservoir has a surface area of 8180 acres and consists of two basins connected by a dividing weir. The west basin controls about 90 per cent of the drainage area and has a storage capacity of 6.3 billion cubic feet at elevation 590.0. The east basin is 3 feet lower than the west basin and has a capacity of 10.8 billion cubic feet at elevation 587.0

The reservoir has a maximum surface width of three miles with an average of about 1 mile. The reservoir depth averages about 50 feet and reaches a maximum of 190 feet.

the reservoir is provided with a 900 foot long spillway having a crest elevation of 587.0. The spillway is located at the eastern end of the east dike. The stored water flows through the east basin and is carried by the Catskill Aqueduct to the New York City Water Supply System. The aqueduct has a capacity of some 600 million gallons per day. Except for the dividing weir, the west basin is not provided with any kind of outlet works.

The capacity curves of the east and west basins are shown in Figure IV-9.

As shown in Figure IV-8 the Ashokan Dam consists of several parts. The length and elevation of each part are summarized in Table IV-1.

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TABLE IV-1
LENGTH AND ELEVATION
OF ASHOKAN DAM PART

Part	Length (ft)	Elevation, MSL	
		Crest of Dam	Crest of Core
Main Dam			
Masonry Part	1000	610	
Earth Part	3650	610	596
West Dike	1700	610	596
Middle Dike	6700	606	593
East Dike	2600	602	593
Spillway	900	587	

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Typical crosss-sections of the main dam (both the masonry and earth parts), west, middle and east dikes are shown in Figures IV-10 and IV-10a.

3. Flood Routing through Ashokan Reservoir

The outflow pattern that is produced by the Esopus Creek Basin (Subbasin No. 19) inflow into the Ashokan Reservoir was determined by an analytical procedure similar to the one used for Sacandaga Reservoir routing. Because the Ashokan Reservoir consists of two basins, Equation IV-6 was adjusted to account for the various occurrences as follows:

$$\left[\left(\frac{I_{wn} + I_{wn+1}}{2} - \frac{O_{wn} + O_{wn+1}}{2}\right) + \left(\frac{I_{en} + I_{en+1}}{2} + \frac{O_{wn} + O_{wn+1}}{2} - \frac{O_{en} + O_{en+1}}{2}\right)\right] (t_{n+1} - t_n) = S_{wn+1} - S_{wn} + S_{en+1} - S_{en} \qquad \text{IV-14}$$

in which

e and w = subscripts denoting east basin and west basin respect-
tively.

Equation IV-14 was programmed as shown on Plate IV-6 and the input or data file included:

a. Subbasin No.19 unit hydrograph – see Table III-11
b. Subbasin No. 19 inflow storm hydrograph comupted using

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the procedure of Section E of Chapter III. This flood hydrograph was divided between the east and west basin on the basis of their drainage areas.
c. Reservoir storage curves of Figure IV-9. A straight line extrapolation was used for elevations in excess of elevations 587.0 and 590.0 for the east and west basins respectively.
d. A time interval of 2 hours.
e. Reservoir state – discharge formulas. No advantage was taken of the 600 million gallons per day capacity of the Catskill Aqueduct.

The outflow from the east basin, O_e, (spillway discharge only) was determined using the following equation:

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$O_e = CLH^{'3/2}$ (IV-15)

in which

C = Constant equal to 4 feet ½ /second determined from data supplied by new York City Department of Water Resources.
L = Length of Spillway (900 feet).
H = Water head above the spillway in feet.
The west basin outflow, O_w (over the dividing weir) was computed using the following equation:

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$$O_w = C\acute{L}H_w^{3/2}\left[1 - \left(\frac{H_e}{H_w}\right)^{3/2}\right] \quad \text{(IV-16)}$$

in which
C = Constant equal to 4 feet ½ /second
L = Length of weir in feet
H_w = Water head above the crest in the west basin in feet
H_e = Water head above the crest in the east basin in feet.

The Ashokan Reservoir outflow was then combined with subbasin No. 20 flood hydrograph (directly downstream of the dam) to obtain the total Esopus Creek flood hydrographs.

The program, data file and printout are shown in Plates IV-6 through IV-8 respectively.

Figure VI-11 depicts the reservoir inflow and outflow hydrographs. The computed inflow reached a peak of about 58,000 cfs some 44 hours after the beginning of the storm. The corresponding maximum reservoir outflow is about 26,000 cfs occurring 8 hours later.

The variation in the water surface elevation in the east, as well as in the west basin, is shown in Figure IV-11a. The maximum elevation in the east basin is less than 591. This is more than two feet below the crest of the east dike core and more than 11 feet below

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the crest of the east dike. The maximum elevation in the west basin is about 595 feet above mean sea level. This value is about one foot below the crest of the concrete wall core and 15 feet below the crest of the dike.

4. Ashokan Dam Failure Analysis

a. Under Hudson River Probable Maximum Flood Conditions

The results of the previous section showed that under the Hudson River PMF conditions, the water level in the east and west basin would reach a maximum elevation of 590.74 and 595.14 respectively. These values are below the elevations of the crest of the concrete wall core and well below the dike crest elevations. Therefore, seepage or overtopping of the dam would not occur. this conclusion is supported by the events of the March, 1951 flood. The elevation of the water in the east basin reached a record high of 592.23 feet at 4:45 A.M. on March 31, 1951.[22] This value is about 1.5 feet higher than the above presented value of 590.74. The March, 1951 flood was the highest of records extending back to 1904.

Moreover, structural analysis of the Ashokan Dam indicated that the dam is stable under these conditions. Details of the structural analysis under a more critical condi-

tion are presented in the subsequent section.

Therefore, there is no reason to believe that the Ashokan Dam would fail, under the Hudson River PMF conditions from overtopping or seepage.

b. Under Esopus Creek Probable Maximum Flood Conditions

As indicated earlier, a more critical condition suggested by the AEC personnel was investigated. It was considered because the lower center of the selected PMS (Figure II-8) covers asn area to the east of the dam site. Another storm pattern would have resulted in a storm center over the Esopus Creek Basin. It was then decided to evaluate a probable maximum storm over Esopus Creek Basin, study its effect on Ashokan Dam and determine the water surface elevation at Indian Point as a result

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of this flood and a standard project flood over the rest of the Hudson River Basin.

The Esopus Creek PMF and its effect on the dam are presented in this section The effect on the Indian Point site is considered in the next section of this chapter.

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Much of the discussion on the determination of the probable maximum flood at Indian Point presented in Chapter III applies here, mutatis mutandis.
The depth-duration values of PMP for a basin having a drainage area of 257 square miles were obtained from Figure III-6. The results are shown in Figure IV-12. The total rainfall for such basin is about 25 inches in 72 hours.
The 6-hours incremental values of PMP over this subbasin were then rearranged in accordance with the Hydrometeorological Report No. 40 criteria.
The rainfall excesses were obtained by substracting the rainfall losses of Chapter III from the PMP values. the incremental PMP values and related runoff for subbasin No.19 are depicted in Figure IV-13. The results are also summarized in Table IV-2.
Subbasin No.19 unit hydrograph – see Table III-12 – was then applied to the rainfall excess to obtain the flood hydrograph.
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this hydrograph was routed through the Ashokan Reservoir following the procedure described earlier.
Computer outputs of the reservoir inflow and outflow hydrographs as well as the related water surface elevations in both basins are shown in Plate IV-9.
The reservoir inflow hydrograph is shown in Figure IV-14. The computed inflow reached a peak of about 200,00 cfs some 44 hours after the beginning of the Esopus Creek probable maximum flood. This peak is about four times the peak resulting from the Hudson River probable maximum flood and resulted in a water surface elevation higher than the crest of the masonry wall in the west dike and earth part of the main dam.

The variation in water elevation caused by runoff generated by the Esopus Creek PMP in both basins is shown in Figure IV-15.

It was assumed that failure of the dikes and earth part of the main dam would occur with the reservoir water level at a foot higher than the crest of the

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masonry core wall. In other words, failure of the west dike and earth part of the main dam would occur when the west basin water level reaches elevation 597.0. The corresponding east basin failure elevation is 594.0.

The results of Figure IV-15 indicate that the failure elevation would be reached in the west basin 40 hours after the beginning of the storm. The corresponding maximum water elevation in the east basin is about 590.5 or 2.5 feet below the crest elevation of the east basin dikes.

Structural analysis of the main dam, presented in the next section of this chapter, showed that the masonry part would not fail under the Esopus Creek PMP conditions.

It was concluded, therefore, that only the west dike and the earth part of the main dam would fail 40 hours after the beginning of the Esopus Creek PMP.

The influence of this failure on the reservoir outflow

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and elevation is depicted in Figures IV-14 and IV-15 respectively. The specific details and effect of dam failure on the Indian Point site are presented in the last part of this chapter.

c. Structural analysis of the masonry Part of the Main Dam (Olive Bidge Dam)

The purpose of this section is to show that the masonry part of the main dam shown in Figure IV-10

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would not fail under the Esopus Creek PMP conditions.

the following two conditions were evaluated:

Condition i – An upstream water level at elevation 610.0 and no water downstream of the dam.

Condition ii – An upstream water level equivalent to the east basin failure elevation of 597.0 and a water depth of 20 feet downstream of the dam.

Condition i:

The major forces acting on the dam are indicated in Figure IV-16 and defined below:

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F_H = Horizontal component of hydrostatic pressure, acting along a line H/3 feet above the base

= $1/2H^2$, where y = specific weight of water.

F_V = vertical component of hydrostatic pressure

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= weight of fluid mass vertically
above the upstream face, acting
through the centroid of that mass.

W = Weight of dam = (area of cross-
section of dam) (Sy) where S =
specific gravity of masonry,
approximately 2.4 or 2.5, acting
through centroid of cross-section

F_u = Uplift force on base of dam, as
determined by foundation seepage
analysis, and integration of point
pressure intensities over base area;
if foundation is homogeneous and
permeable, pressure varies approxi-
mately linearly from full hydro-
static head at the heel to full
tailwater head, and F_u is approxi-

mately 1/2yHB, acting at B/3 from
the heel. This value is often
multiplied by some fraction less
than 1 if the foundation is rela-
tively impermeable, but it is on

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the safe side to assume uplift over the entire base area.

F_E = Vertical component of earth pressure acting on downstream face, i. e. weight of earth mass vertically above the downstream face, acting through the centroid of that mass.

R = Resultant of foundation shear and bearing pressures: horizontal component, $R_H = F_H + F_E$ acting along the base; vertical component, $R_v = W + F_V - F_u$ acting at a distance x from the toe that can be determined by the requirement for rotational equilbrium of the dam, by equating to zero the sum of the moments of all the foregoing forces about the toe of the dam.

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G = Resultant of all forces acting on the dam, equal to R but in the opposite direction.

For further explanation of these formulas, the reader is refered to reference 23. The

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equations used to determine these forces and the results are listed in Table IV-3. The horizontal and vertical components of the resultant of all forces are 1,510 x 10^3 and 2542 x 10^3 pounds respectively. The location of the resultant is in the middle third of the base about 72.5 feet from the toe.

The factor of safety against sliding and overturning for this condition are 1.09 and 1.87 respectively. The maximum normal stress and shear stress are less than the allowable limits.

These results indicate that the masonry Part of the main dam would not fail.

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Condition ii

The equations used to compute the various forces and the results for this condition are listed in Table IV-4.

The horizontal and vertical components of

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the resultant are 1088 x 10^3 and 2044 x 10^3 pounds respectively. The location of the resultant is in the middle third of the base about 91 feet from the toe.

The factors of safety against sliding and overturning for this condition are 1.22 and 1.86 respectively. These results indicate that condition i is more critical.

Therefore failure of the masonry part of the Olive Bridge Dam from overturning or sliding would not occur.

5. Combined Effect at Indian Point resulting from Simultaneous Occurrence of Ashokan Dam Failure and Hudson River Standard Project Flood

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The following procedure was employed to determine the flood hydro-Graph at Indian Point resulting from Ashokan Dam failure and Hudson River standard project flood:

a. Computation of Reservior Outflow Hydrograph after Dam Failure

The work required to achieve this objective includes:

i. The use of Equation IV-4 to determine

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the initial dam failure discharge. The affected area of the west basin dike and main dam is 238,670 feet with an average depth of 47 feet. This area was conservatively approximated by a rectangular cross-section. The computed initial dam failure flow is about 2.6 million cubic feet per second.

ii. Determination of west basin emptying time. The dam break hydrograph was conservatively assumed to have a triangular shape with the initial flow as the height and emptying time as the base. The emptying time is, therefore, equivalent to $\frac{2v}{Q_o}$ where v is the total volume of water stored in the west basin (6.765

billion cubic feet) and Q_o is the initial flow (2.6 million cfs). These values give an emptying time of 5200 seconds or 1.45 hours.

iii. Combination of dam break hydrograph with

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the east basin outflow over the main spillway to obtain the reservoir outflow hydrograph. the east basin was computed in a manner similar to that outlined in Section 3 of this chapter.

The reservoir outflow hydrograph including dam break wave is shown in Figure IV-14. Subsequent to the emptying time the resultant hydrograph coincides with the reservoir inflow hydrograph. This is a conservative assumption since no advantage is taken of the "cushioning effect" of the east basin.

b. Routing of Esopus Creek Flood Hydrograph to the Hudson River.

This step requires the following:

i. Combination of Ashokan reservoir outflow Hydrograph (including dam break wave) of

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previous step with the flood hydrograph of the remaining part of the Esopus Creek Basin, i.e. Subbasin No. 20. Since the contribution of

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Subbasin No. 20 represents only a fraction of the reservoir outflow hydrograph, the resultant is essentailly the same as the reservior outflow hydrograph shown in Figure IV-14.

ii. Computation of Valley storage in Sub-basin No. 20 (between the dam and Hudson River). It was assumed, as suggested by reference 21, that the wave front moves in a steeply inclined wall of water whose profile is unchanging as long as the channel conditions remain fixed and the source of supply is constant. Therefore, this can be considered a special case of uniformly progressive flow, known specifically as the roll wave. The wave front discharge, Q_W may be obtained using Manning's Equation:

$$Q = \frac{1.49}{n} BY'^{5/3} S_o^{1/2} \qquad \text{IV-17}$$

In which

S_o = Channel slope

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Y' = Depth at the crest of the wave

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B = Channel width

n = Manning's coefficient (.035 was used for this purpose)

As a conservative estimate of the wave profile, a constant flow equivalent to the maximum initial failure flow of 2.6 million cfs and the valley characteristics (B and S_o) obtained from the **U.S.G.S.** maps were used.

A careful study of the topography of Esopus Creek Valley indicated that the flood generated by the Esopus Creek PMP and Ashokan Dam failure would create a lake extending from the dam to Glenerie Falls some 5 miles west of the Hudson River. The lake would have a total length of some 14 miles and a surface width ranging from 4000 to 1800 feet. The lake would have a control section at Glenerie Falls where width decreases

suddenly from 6000 feet to 500 feet. A sketch showing the size, direction of flow

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through the lake and the location of the control section is presented in Figure IV-17.

The maximum water surface level at Glenerie Falls would be at elevation 180.0. This value was computed using Equation IV-17 and the standard step method of computing gradually varied steady flow profiles. Elevation 180, therefore, delineate the shore lines of the lake at this elevation is about 7 billion cubic feet. This value is almost the same as the storage capacity of the Ashokan west basin. This provides an added confirmation of the computed elevation of 180.0.

The storage capacity curve of this lake is shown in Figure IV-18.

The outflow from the lake at Glenerie Falls was computed using the standard critical depth formula:

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$$Q = \sqrt{\frac{q A^3}{B}} \qquad \text{IV-18}$$

A stage-discharge curve was established for this lake and is shown in Figure IV-18.

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ii. Routing of Esopus Creek Flood Hydrograph through the lake was accomplished by using a computer program similar to the one described in Section 3. The program, data file and printouts are given in Plates IV-10 through IV-12. The input includes the initial lake outflow of 620,000 cfs at elevation 180.0, Subbasin No. 19 flood hydrographs generated by Esopus Creek PMP and Subbasin No. 20 flow caused by runoff generated by the Hudson River Standard Project Flood.

The 620,000 cfs represents the effect of the dam failure at Glenerie Falls and is equivalent to the lake outflow at elevation 180.0. This is a conservative, but realistic, estimate of the failure effect since the volume of the stored water in the lake is

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essentially the same as the Ashokan west basin

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capacity.

c. Routing of Esopus Creek Flood Hydrograph and Hudson River Standard Project Flood to Indian Point

The Esopus Creek flood hydrograph resulting from Ashokan dam failure, Subbasin No. 19 PMP and Subbasin No. 20 SPF as computed above is represented by Curve I in Figure IV-19.

The Hudson River Standard Project Flood upstream of Esopus Creek was obtained by dividing the probable maximum flood values of Figure III-27 by two. The factor two is within the range of 40 per cent to 60 per cent developed by the Corps of Engineers for the Susquehanna River Basin[24] and was suggested by Mr. Nunn of the AEC.[17] Curve II of Figure IV-19 shows the SPF above Esopus Creek.

Curve III in Figure IV-19 represents the combined effect (Curve I + Curve II) just downstream of the mouth of Esopus Creek.

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The combined hydrograph was then routed through the Hudson River Channel to Indian Point using the procedures developed in Chapter III. The resultant at Indian Point is shown as Curve IV in Figure IV-19.

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The program, data file and printouts for this condition are listed in Plates IV-13 through IV-15 respectively. The maximum discharge at Indian Point resulting from the conditions considered in this chapter is 705,000 cfs. The probable maximum flood at Indian Point is about 50 per cent higher than this value.

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V. MAXIMUM RIVER ELEVATION FOR FLOODING CONDITIONS AT INDIAN POINT

A. Introduction and Summary of Procedures Employed

The purpose of this chapter is to determine the maximum water surface elevation at the Indian Point site resulting from several flooding conditions including:

1. Runoff generated by the Hudson River probable maximum precipitation presented in Chapter III.
2. Flooding caused by the occurrence of the Ashokan Dam failure concurrent with the standard project flood for the Hudson River

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Basin considered in Chapter IV.

3. Flooding resulting from the probable maximum hurricane for the New York Harbor area concurrent with spring high tide discussed in Appendix A.

To achieve this objective, backwater curves were calculated using the well known standard step method[21] as well as a newly developed numerical method of computing gradually varied flow profiles,[25] starting with several assumed water-surface elevations at a control point on the river. The hydrodynamic and physical characteristics of the Hudson River dictated the use of the ocean entrance at the battery as the control section. Therefore, the backwater computa-

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tions were initiated at this location and the following boundary conditions, at the Battery, were evaluated:

1. Mean sea level
2. High tide water elevation
3. Low tide water elevation
4. Standard project hurricane
5. Probable maximum hurricane

The tidal variation in the Hudson River Estuary discussed in Chapter II influenced the choice of the proper boundary conditions as well as the value of the discharge for which the flow profile is desired. As in the case of flooding from dam failure and unlike the previous attempts, the tidal variation was treated as

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an integral part of the system and its influence was simultaneously coupled with the other relevent hydraulic occurrences.

In the evaluation of the maximum water surface elevation at Indian Point resulting from the above delineated flooding and boundary conditions, the wind produced local oscillatory short period waves were also considered. The computed Indian Point stages corresponding to the various flooding conditions were conservatively increased by one foot to account for this effect. A detailed discussion of this effect appears in Appendix A.

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A detailed description of the standard step method employed in this study as well as the newly developed flow profile numerical method is presented first. Following this is a discussion of the influence of the Hudson River tidal variation. The results of the various flooding and boundary conditions are discussed and presented last.

B. Methods of Backwater Computation

The so-called "Standard Step Method"[21] was employed in this study to determine the water elevation at Indian Point as well as the time of travel of the flood hydrograph between Troy and Indian Point Point.

However, a new numerical method of computing gradually varied steady flow profiles developed by Prasad[25] was used to verify the Standard Step Method results.

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A discussion of both methods follows.

1. Standard Step Method

This method is characterized by dividing the channel into several short reaches and carrying the computation step by step from one end of the reach to the other. The basic equation that defines the procedure may be expressed as

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follows:

$$H_{i+1} = H_i + h_f + h_e \qquad \text{(V-1)}$$

in which

H = Total head (elevation of the energy line) above a horizontal datum. The mean sea level was selected in this study as the datum.

h_f = Friction loss between two end sections i and i + 1

h_e = Eddy loss. For convenience of computation, this loss was considered part of the friction loss h_f

The friction loss h_f may be computed as follows:

$$h_f = \overline{S}_f \mathrm{D}x = \tfrac{1}{2}(S_i + S_{i+1})\mathrm{D}x \qquad \text{(V-2)}$$

where

$\overline{S}_f$ = Friction slope taken as the average of the slopes at the two end stations, i.e. $\overline{S}_f = \tfrac{1}{2}(S_i + S_{i+1})$

Δx = Longitudinal distance between the two end sections.

Manning's formula was used to compute the friction slope

$$S_f = \left(\frac{nv}{1.49}\right)^2 \frac{1}{R_h^{4/3}} \qquad \text{(V-3)}$$

where

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n = Manning's roughness coefficient

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v = River velocity and is equal to $\frac{Q}{A}$

R_h = Hydraulic radius

Q = River discharge

A = Cross-sectional area

The total head or elevation of the energy line above mean sea level may be expressed as follows:

$$H_i = Y_i + \alpha \frac{v_i^2}{2g} \qquad \text{(V-4)}$$

in which

Y = Water surface elevation above mean sea level

α = Energy coefficient. This coefficient was assumed to be unity because of the fairly straight alignment and regular cross-section of the river between the Battery and Indian Point.

The river channel was conservatively approximated by a rectangular cross-section defined by the mean water surface width B and the mean depth D. Thus

$$A_i = B_i D_i = B_i (Y_i + Z_i) \qquad \text{(V-5)}$$

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in which

Z_i = Water depth below mean sea level

No advantage was taken of the increase in surface width

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above mean sea level. Since the channel is of a wide rectangular shape, the hydraulic radius R_h was approximated by the mean depth $Y_i + Z_i$.

The foregoing equations were programmed for solution on RAPIDATA time-sharing facilities. The program is listed in Plate V-1.

The Lower Hudson River Channel was divided into 136 reaches ranging in length from 0.3 to 2 miles. The wide variation in channel geometry influenced the selection of these subreaches. The surface width at mean water B and mean depth below mean sea level Z corresponding to the 136 sections are listed in Plate V-2. The locations of these stations expressed in miles above the ocean entrance at the Battery are also shown in Plate V-2.

An estimated average value 0.025 was used for Manning's roughness coefficient n in the Lower Hudson. Limited information is available for an accurate determination of this coefficient. The 0.025 value was selected on

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the basis of several U.S.G.S. estimates of the Lower

Hudson River flow resistance coefficient. These estimates were derived from velocity measurements made by the U. S. Geological Survey at the Poughkeepsie gaging site. Values ranging from .019 to .024[26] were computed using available, but inconclusive, data and several computer programs developed by the U.S.G.S.[27-32]

Moreover, the selected n value falls within the range recommended by Dronkers[33] for tidal rivers. (Chezy coefficients of 50 to 70 meter ½/sec for shallow waters and deep inlets respectively. The corresponding Manning n values, for a channel having an average hydraulic radius of 30 feet are 0.029 and .021 respectively.)

The procedure used to compute the backwater profiles may be summarized below:

a. Select a boundary condition, i.e. water surface elevation above MSL at the control section (the Battery).

b. Select a flooding condition in the Lower Hudson River such as flooding resulting from PMP, dam failure etc. Use a constant flow

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value Q equivalent to the peak of the flood hydrograph. This is a conservative assumption since the flow, prior and subsequent to the peak time, is substantially less than this value.

c. Compute the control section velocity corresponding to the selected water surface elevation – Step a – and flooding condition - Step b – using the continuity equation

$$V_i = Q/B_i (Y_i + Z_i)$$

d. Compute the friction slop S_f^i using Equation V-3.

e. Determine the energy line elevation H_i above MSL using Equation V-4.

f. Estimate the water surface elevation at the upstream section of the first reach i.e. section i + 1. As a first approximation use

$$Y_{i+1} = Y_i = S_f^{(i)} \Delta x$$

g. Using the estimated Y_{i+1}- Step f – repeat steps c, d and e to determine V_{i+1}, $S_f (i + 1)$ and H_{i+1}.

h. Compute the difference between the elevation of the energy line at sections i and i + 1, i.e.

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$\Delta H = H_i + 1 = H_i$

i. Compute the friction loss h_f between the two sections using Equation V-2.

j. If the difference between ΔH and h_f is greater than .05 feet, assume a new value of Y_{i+1} and repeat steps g through i until agreement with the control $|\Delta H - h_f| \leq .05$ feet is obtained.

k. Proceed to the next reach using the final Y_{i+1} value of the first reach as the downstream control for the second reach.

l. Repeat until the water surface elevation at the last section is determined.

2. Numerical Method of Computing Gradually Varied Flow Profiles

This method was developed by Prasad in 1968.[25] Prasad describes his method as "simple, fast, efficient, and does away with most of the approximations and limitations of other methods of computing gradually varied flow profiles."

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The method is based on the following two equations:

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$$D' = \left[S_o - \frac{n^2 Q^2}{2.22 A^2 R_h^{4/3}} \right] \Big/ \left[1 - \frac{\alpha Q^2 B}{g A^3} \right] \qquad \text{(V-6)}$$

and

$$D_{i+1} = D_i + \left(\frac{D'_{i+1} + D'_i}{2} \right) \Delta x \qquad \text{(V-7)}$$

in which

D = River Depth

D' = $\frac{dD}{dx}$

S_o = Slope of river bed

Equation V-6 is the well-known differential equation of gradually varied flow expressed in terms of channel geometry and hydraulic characteristics. Equation V-7 utilizes the classical trapezoidal method of integration to express the depth in terms of its derivative with respect to the longitudinal distance x. This equation requires a very small reach length Δx. These two equations can be solved numerically to determine the two unknowns D and D'. The procedure suggested by Prasad is reproduced below.

a. Compute D'_i from equation V-6 in which D_i is given as an initial condition or from

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a previous calculation.

b. Assume $D_{i+1} = D¢$ as a first approximation

c. Calculate an approximate value of D_{i+1} from equation V-7 using the value of $D¢_{i+1}$ obtained from step b or d.

d. Compute a new value of D_{i+1} obtained in step c.

e. If this new value of $D¢_{i+1}$ is not very close to the previously assumed value in step b, repeat steps c through e. Otherwise repeat the whole procedure for another i, or, advance the solution by one integration step.

The foregoing procedure was programmed for solution on RAPIDATA time-sharing facilities. A listing of the program appears on Plate V-3. The input data are identical to those used in the Standard step method. However, because of the use of the trapezoidal integration method in this program, the reach size Δx was reduced by 400 per cent.

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C. Influence of Hudson River Tidal Variation on Water Surface Level at Indian Point

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As indicated earlier, the influence of tidal variation in the Hudson River during probable maximum flood conditions was treated as an integral part of the problem and simultaneously coupled with the other hydraulic occurrences. In the past studies, a somewhat piecemeal method consisting of adding a constant component flow equivalent to the maximum observed ebb flow to the the flood hydrograph at Indian Point was employed. A careful evaluation of this effect during high runoff conditions revealed the following findings:

1. The tides and currents in the river are subject to large seasonal variations due primarily to fluctuations in the fresh-water discharge.
2. Near the mouth of the river this variation is relatively small, but in advancing up the river the variations become more important and reach their maximum at the head.
3. At the time of high runoff from the Upper Hudson and Mohawk Basins, the tide may be completely masked in the upper portion of the river, the water continuing to rise or fall for a period of several days without any tidal oscillation.
4. The ocean derived salinity intrusion length is influenced, to a great extent, by the upland runoff. The relationship between

the intrusion length and the fresh water flow in the Lower Hudson River is shown in Figure II-12. Extrapolation based

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upon the equations of Figure II-12 shows that under the PMP conditions the salinity concentration in the Lower Hudson would be less than 250 ppm. This indicates that only a small reach of negligible length would be influenced by the ocean derived salt.

On the basis of these observations, a more realistic approach was adopted. The new approach may be illustrated by the following steps:

1. Assume that the tidal variation at the ocean entrance is not influenced by the probable maximum precipitation. This assumption is supported by the USC and GS tidal measurements over a very long period (1899-1932).[34] A comparison between the tidal characteristics at the mouth (the Battery) and head (Albany) for the year 1922 is shown in Table V-1. This year was selected for this purpose because of the high runoff that occurred in April (154,000 cfs at Troy).

   The table values clearly indicate the influence of land runoff at the Battery is relatively small. The April values are within 1 to 2 per cent of their annual average counterparts.

In the vicinity of Albany, on the other hand, the April values are 60 to 150 per cent higher than the annual average.

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2. For a given flooding condition, such as the probable maximum flood at Indian Point (1,100,000 cfs) and a known boundary condition, such as mean water level at elevation 0.0, compute the Lower Hudson River flow profile. The standard step method may be used for this purpose. Designate time (t) to this profile.

3. At the beginning of the flood cycle, the state at the Battery starts to increase until it reaches a maximum elevation of +2.2 some three hours later. This increase in elevation causes a decrease in the slope of the Lower Hudson River water surface profile. This change results in a stage increase along the river ranging from a maximum of 2.2 feet at the Battery to zero at some upstream location. The amount of water stored within the reach of influence (ΔV) is equivalent to the product of the time interval (Δt) and decrease in river flow caused by channel storage (ΔQ).

The process is then reversed during the second half of the flood cycle, i.e. the water stored during the first half of

of the flood cycle is released causing an increase in the flow and a decrease in water surface elevation within the same influenced reach. About three hours later the flow profile approaches its initial condition (before the beginning of

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the flood cycle)

4. During the first half of the ebb cycle (from mean water to low water), the stage at the Battery starts to decrease until it reaches a minimum elevation of –2.2 during maximum ebb. This control section stage variation causes an increase in the water surface gradient resulting in a stage decrease along the influenced reach. The water volume released during this interval is again equivalent to the product of the time interval and the difference between the initial and final flows.

The process then continues until the flow profile returns to its original shape.

The tidal cycle at the Battery was divided into 16 intervals having a duration of about 45 minutes. The boundary conditions corresponding to these intervals were obtained from the USC and GS tidal measurements at the Battery. A trial and error procedure was then employed to determine the length of influenced reach, amount of

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water stored or released during each interval and related channel flow values. The standard step method was then employed using the computed flows and given boundary conditions to obtain the flow profiles in the Lower Hudson. The final results are presented in the next part of this chapter.

D. Water Surface Level at Indian Point

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Several severe Hudson River flooding conditions and water-surface elevations at the Battery were presented in the previous chapters. These conditions were conservatively grouped in seven different ways on the basis of a simultaneous occurrence of an appropriate set of flooding, hurricane and tidal conditions.

The seven groups are outlined in the following tabulation:

Case No.	Flooding Condition at Indian Point	Water-surface Elevation at the Ocean Entrance
1	Probable Maximum Flood	Mean Tide, i.e. M.S.L.
2	Probable Maximum Flood concurrent with tidal flow	High Tide
3	Probable Maximum Flood concurrent with tidal flow	Low Tide
4.	Dam failure concurrent with standard project flood	Mean Tide
5.	Standard Project Flood	Standard Project Hurricane

Case No.	Flooding Condition at Indian Point	Water-surface Elevation at the Ocean Entrance
6.	Dam Failure concurrent with standard project flood	Standard Project Hurricane
7.	Probable Maximum Hurricane concurrent with Spring high tide	Probable Maximum Hurricane

A detailed description of these cases and their significance follows:

Case 1

This case considers the Indian Point stage at peak discharge due

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to runoff generated by the probable maximum precipitation over the Hudson River drainage area upstream of the site. The mean tide at the ocean entrance which corresponds to the mean sea level was selected as a boundary condition for this case.

The probable maximum flood of 1.1 million cubic feet per second, discussed in Chapter III, was considered to prevail in the Lower Hudson River for a long period. In other words, a steady state flow condition of 1.1 million cfs in the reach between the ocean entrance and the site was assumed. This assumption is conservative since the flood hydrograph under PMP conditons (Figure III-27) indicates that the duration of the peak discharge (PMF) is only several hours.

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The mean sea level at the ocean entrance (the Battery) was selected as the boundary condition for this case since the tidal variations under PMP conditions would be relatively small. Examination of the USC & GS measurements[35] indicate that under such conditions the tide would be completely masked in a significant portion of the river. The results of cases No. 2 and 3 suggest that only the lower 27 miles of the river would be subjected to significant tidal oscillation.

The standard step method for water surface profiles presented earlier was employed to determine the stage at the Indian Point site for

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the above-delineated conditions, i.e. flow of 1.1. million cfs and mean sea level at the Battery. The results are listed in Plate V-4 and shown in Figure V-1. The results of Cases No. 2 and 3 are also shown in Figure V-1.

The flow profile corresponding to Case No. 1 conditions was verified using the newly developed Prasad's method discussed in Section B of this chapter. These results are shown in Plate V-5 and compared to their Standard Step Method counterparts in Table V-2. The agreement between the two methods is remarkable. In general the new method gives slightly lower stage values with a maximum difference of –0.5 feet.

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The probable maximum flood would result in a river stage of elevation 12.7 feet above the mean sea level at Indian Point.

The account for the influence of the local oscillatory wave at Indian Point referred to in Section A of this Chapter, the computed stage must be increased by one foot. This estimate is conservative and discussed in detail in Appendix A.

Therefore, the Indian Point river stage corresponding to Case No. 1 conditions and including the influence of the local oscillatory wave would be 13.7 feet above mean sea level.

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Cases No. 2 and 3

These cases were considered to evaluate the influence of the tidal variation on the Indian Point stage during the probable maximum conditions. The procedure outlined in the previous section of this chapter was used for this purpose. The tidal variation at the ocean entrance, its influence on the probable maximum flood and river stage at Indian Point are depicted in Figure V-2. The specific details of the procedure are listed in Table V-3.

Figure V-1 documents the flow profiles corresponding to cases No. 2 and 3 conditions, probable maximum flood concurrent with

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tidal flow at Indian Point and high water and low water elevations at the Battery respectively.

As indicated earlier, these results suggest that the river above the Tappan Zee Bridge, some 27 river miles above the Battery, would experience relatively small tidal variations under the PMP conditions. The upstream extent of the reach influenced by tidal variations was located using the material balance procedure described earlier.

The conclusion seems to be in good agreement with the lack of tidal oscillation observed in the upper portion of the river during the floods of March 28, 1913 and March 19, 1936. The maximum discharge

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measured at the head of the estuary during the 1913 and 1936 floods were 223,000 and 215,000 cfs respectively. These floods pushed the tide back downstream resulting in a flattening of the slope of the measured flood profiles some 30 miles downstream of Troy.[35]

Thus, a peak discharge of more than one million cfs would be expected to mask the tide in a significantly longer reach. Figure V-1 estimates the length of this reach to be some 120 miles (between Troy and the Tappan Zee Bridge).

Theoretically speaking, therefore, the ocean-derived tidal influence would not be experienced at Indian Point under the probable maximum precipitation conditions.

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However, to be on the conservative side, the reach of tidal influence was extended upstream to the site at Indian Point. The tidal effect resulted in the following changes:

Case No.	Stage at the Battery (M.S.L.)	Peak Flow at Indian Point (cfs)	Water-surface elevation at Indian Point
1	Mean Water	1,100,000	12.70
2	High Water	1,013,500	12.37
3	Low Water	1,164,800	13.05

As a conclusion, the water-surface elevation at the site would range between 12.37 and 13.05 depending upon the tidal phase at the Battery.

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As before, these values must be increased by one foot to account for the local oscillatory wave at Indian Point.

Case No. 4

This case considers the maximum river stage resulting from a failure of the Ashokan Dam concurrent with the Hudson River Standard Project Flood at Indian Point. This flooding condition was discussed in detail in Chapter IV.

It should be also recalled that one of the centers of the probable maximum storm was placed over the largest reservoir in the basin. Due to the cross-shaped watershed of the basin, the reshaping of

the isohyets recommended by the Weather Bureau placed the lower center several miles upstream of the second largest reservoir. As indicated in Chapter IV, the runoff generated by the probable maximum precipitation over the entire basin as described above would not result in failure of the basin dams. However, because the Ashokan Dam contains the second largest volume of stored water in the basin and is closer to Indian Point than any other dam in the basin, a flooding condition resulting in a failure of this dam was considered. As suggested by the AEC, the flooding condition consisted of the simultaneous occurrence of a probable maximum flood over the Esopus Creek Basin upstream of Ashokan and a standard project flood over the rest of the Hudson

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River Basin.

As far as the Ashokan Dam failure is concerned, the suggested condition proved to be more critical than a hypothetical condition placing the lower center of the selected PMS directly over the Ashokan Reservoir.

The suggested condition resulted in a failure of the Ashokan Dam and a peak flow of 705,000 cfs at Indian Point. This flow would

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result in a river stage of Elevation 7.2 above mean sea level at the Indian Point site. This value corresponds to a mean sea level elevation at the ocean entrance. Because this condition is less critical than the probable maximum flood conditions (Cases No. 1 through 3), additional evaluation concerning the influence of the tidal variation was not undertaken.

Case 4 results, as well as Cases 5 and 6, are depicted in Figure V-3.

Cases No. 5 and 6

To carry the degree of severity a step further, a more critical boundary condition was imposed on two flooding conditions. The peak storm surge at the Battery resulting from the Standard Project Hurricane for the New York Harbor area was used as the boundary condition for these two cases. The flooding condition of Case No. 4, as well as a similar flooding condition, in which the effect of Ashokan Dam failure was excluded, were used for these two cases.

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The New York Harbor hurricane studies[36, 37] undertaken by the New York District Corps of Engineers showed that the peak storm surge height at the Battery resulting from the Standard Project Hurricane is 11.0 feet. This value was based on the transposition of the September, 1944 hurricane to a path critical to the New York Harbor

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area, using parameters given in U.S. Weather Bureau Memoranda HUR 7-60 and HUR 7-60a dated 27 March 1959 and 7 April 1959 respectively. The parameters finally selected for this storm were a maximum wind speed of 116 miles per hour, a central pressure range from 27.55 to 27.95 inches of mercury with a normal pressure of 30.12 inches, a radius to maximum winds of 30 nautical miles, and a forward speed of 40 knots. The corresponding peak storm surge height at Sandy Hook, some 17 miles downstream of the Battery, is approximately 12 feet.

Similar computations were made by the Corps of Engineers for a 1938 hurricane transposed to a path critical to New York Harbor. The computations, which were based on parameters given in the U.S. Weather Bureau Memorandum HUR 7-25, dated 25 February 1957, and a forward storm speed of 35 knots, yielded surges of 8.9 feet at Sandy Hook and 8.8 feet at Fort Hamilton. The maximum surge of record at Fort Hamilton, which is 8.2 feet, was experienced during the extratropical storm of 25 November 1950.

To maintain the selection of severe conditions and in accordance with the high degree of conservatism adopted in this study, the value based upon the transposed September 1944 hurricane was selected. This value is 75 per cent higher than the maximum storm surge during

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hurricane "Donna" that struck on September 12, 1960. Hurricane

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"Donna" was the latest storm to have the greatest effect in the New York Harbor area since the September 1821 hurricane. Simultaneous occurrence of the standard project hurricane with a flooding condition resulting from runoff generated by a standard project precipitation over the entire basin would result in a river stage of elevation 13.0 at the site.

Case No. 6 considers an even more critical set of occurrences consisting of the simultaneous occurrence of the following three severe conditions:

1. Probable Maximum Precipitation over the Ashokan Reservoir drainage basin resulting in a failure of the Ashokan Dam.

2. Runoff generated by the standard project precipitation over the rest of the basin.

3. Peak storm surge at the Battery resulting from the Standard Project Hurricane for the New York Harbor area.

This case would result in a river stage of elevation 14.0 at Indian Point. This value is higher than any of the previous five estimates. The simultaneous occurrence of the above-delineated three severe conditions is extremely remote. Moreover, each one

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of these three individual conditions was based upon several conservative assumptions discussed in previous chapters. Combination of these conditions, therefore, increases the degree of conservatism substantially.

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The resulting flow profiles corresponding to this case as well as Case No. 5 are shown in Figure V-3.

For convenience, the results of the six cases are summarized and compared in Table V-4. The computer printouts for the individual cases are presented in Plates V-2 and V-6 through V-10 respectively.

Case No. 7 considers the water surface elevation at the site resulting from the occurrence of a PMH concurrent with spring high tide. This case was the main subject of Q.L. & M's report of February, 1969 which is appended to the main body of this report. The results are included in Table S-1 for convenience and comparison purposes. A summary of the determination of the results follow.

Using the general equations for surge in an estuary, presented in a paper by J. Proudman, an internationally respected oceanographer, a procedure to route the hurricane surge through the Hudson Estuary was developed. This procedure considered channel frictional resis-

tance, geometric changes in cross-section, and estuary storage of the surge flow. The large storage capacity of Haverstraw Bay acts as an on-line reservoir to attenuate the hurricane surge as it flows toward Indian Point.

A conservative estimate of the routed hurricane surge height at Indian Point is 8.8 feet above mean tidal level, with an additional surge due to wind setup of 1.0 feet, and combined with a spring

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high tide of 3.0 feet. The total calculated elevation is 13.5 feet above mean sea level.

In addition, estimated local oscillatory waves will reach a height of 1.0 feet above the calculated level in the rive. Provision should be made for protecting any structures and appurtenances from this 1.0 foot wave action above the 13.5 foot mean sea level calculated elevation. This value was added to all of the computed stages corresponding to the seven cases and the results are listed in the last column of Table S-1.

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APPENDIX A

MAXIMUM RIVER ELEVATION AT INDIAN POINT RESULTING FROM PROBABLE MAXIMUM HURRICANE AND SPRING HIGH TIDE *

A. Hurricane Parameters

The probable maximum hurricane (pmh) for zone 4, latitude 41°N, as defined by the United States Weather Bureau, has the following meteorological characteristics.

Central Pressure	27.26 inches of Hg
Radius of Maximum Winds	24 nautical miles (weighted mean)
	8 nautical miles (min. limit)
	48 nautical miles (max. limit)
Forward Speed of Hurricane	34 knots (weighted mean)
	15 knots (min. limit)

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	51 knots (max. limit)
Maximum Wind Speed	124 mph to 136 mph
	127 mph (at mean forward speed)

Inspection of published data showed that the vast majority of the hurricane parameters cluster around the mean values reported above.

The minimum and maximum ranges represent the limits of the frequency distribution of the parameters as defined by isolated storms.

Parameters used in the analysis included a forward speed of 34 knots, a radius of maximum wind of 24 nautical miles, and a maximum wind speed of 127 mph. These values represent a probable maximum

* The material included in Appendix A is essentially that of Chapter III of the February, 1969 report.

hurricane with a recurrence interval estimated within the range of 1,000 to 10,000 years.

I should be noted that the very low probability of occurrence of the pmh must be multiplied by the probability of occurrence of spring tides, which occur twice in a lunar cycle of approximately one month, in order to determine the probability of occurrence of the hurricane surge concurrent with maximum high tides. The resulting flooding condition is, therefore, very conservative.

For this hurricane, procedures published by the U.S. Army Coastal Engineering Research Center were used to determine the surge height at the Battery.

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B. Summary of Procedures Employed

The initial step involved the construction of the isovel field for the meteorological characteristics of the pmh. The field was constructed using standard procedures developed by the U.S. Weather Bureau in the aforementioned report.

Following the construction of this field, the wind stresses for several lines passing through the hurricane and parallel to the direction of its movement were computed. The stress coefficient for a hurricane moving over a sloping bottom was used. The maximum

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wind stresses occur on a line which passes through the radius of maximum winds.

the maximum wind stresses were routed along several tracks along the Atlantic Coast, to determine which tract would produce the highest surge at Sandy Hook. After selecting this critical hurricane track, the maximum hurricane stress was then routed through New York Harbor to the Battery.

The greatest surge height resulting from the probable maximum hurricane was computed to be approximately 15.6 feet in New York Bay. This calculation agrees with a published estimate of a maximum surge of 15.3 feet by B. Wilson.

Using an additional stage increase of 1.9 feet due to a condition of atmospheric pressure reduction, the estimated surge in New York Bay is 17.5 feet above mean sea level. A stage-time hydrograph was

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estimated according to the variation predicted by Wilson.

J. Proudman solved the general equations of motion and continuity for a long progressive wave in an estuary. He developed an equation to compute stage at any point in the estuary as a function of an input stage variable at the estuary's mouth.

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Using Proudman's equation for the case of an estuary where the product of base width multiplied by celerity at various cross-sections are similar, a routing of a critical hurricane stage hydrograph was made from the Battery to Dobbs Ferry. This reach is essentially uniform. However, the reach from Dobbs Ferry through Haverstraw Bay, to Indian Point is approximately double the width of the former reach. This allows considerable attenuation of the hurricane surge as a result of the increased channel storage. In this latter reach, a more complex procedure was used to route the Dobbs Ferry stage-time hydrograph upstream to Indian Point.

The surge routing procedure employed considers the effect of friction in the channel, and the large storage capacity of Haverstraw Bay. Haverstraw Bay acts as an on-line reservoir to attenuate any surge condition.

The attenuated surge stage at Indian Point is approximately 8.8 ft. above mean tidal level. The calculated subsidence is affected somewhat by variations in the channel cross-section and the assumed resistance formula. Values of depth and width were computed at

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channel sections where the river's geometry changed. Linear variations were assumed at intermediate sections. The channel was divided into eleven reaches between the Battery and Indian Point.

Prior calculations show that the additional increase in surge height due to the forward advance of the hurricane from the Battery to Indian Point was about 1.0 feet. These calculations assumed that the hurricane would essentially follow a direction parallel to the Hudson, but that winds would be reduced somewhat by the sheltering topography surrounding the river.

The basic theory used was the Corps of Engineers procedure for surge routing in open seas.

In the evaluation of the maximum surge height, it should be remembered that the wind which produces the storm surge, also produces oscillatory short period waves. Calculations for the area downstream from Indian Point show that the maximum fetch length for wave development is 6.85 miles. In order to transfer energy fully from wind to water, a high speed wind must exist for a minimum duration of many hours or days. The short duration of high speed winds as the hurricane passes the critical Indian Point fetch reduces the height of waves. Increased stage is conservatively estimated at 1 foot.

It is possible for the hurricane surge to occur at high tide con-

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ditions. The spring high tide at Indian Point is estimated to increase the stage by 3.0 feet above M.T.L. (very conservative),

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The routed surge and water elevations for the probable maximum hurricane moving along a critical track to Indian Point are summarized below:

	Elevation
Routed Hurricane Stage	8.8 feet
Additional surge due to wind set up from the Battery to Indian Point	1.0 feet
Spring high tide	3.0 feet
	12.8 feet above M.T.L. or 13.5 feet above M.S.L.
Estimated maximum local oscillatory wave height above M.S.L.	1.0 feet

The routed hurricane surge elevation of 13.5 feet exceeds the 11.7 feet elevation computed for the flood runoff condition and is therefore considered to be the controlling elevation for flood protection. Historical records of flooding at Indian Point show that the extreme high water mark of 7.4 feet above M.S.L. occurred during the November 25, 1950 hurricane Unfortunately, data concerning water elevations at both the Battery and Indian Point are meager. A general idea of the magnitude of surge reduction in the Hudson Estuary can be obtained from the depth increases between mean tidal level to mean high water. At the Battery, the depth increases by

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2.3 feet, and at Indian Point, by 1.5 feet. This is a ratio of about 1.5. The corresponding ratio of the 17.5 feet hurricane surge at the Battery to the attenuated 9.8 surge at Indian Point is about 1.8. The 1.8 ratio is reasonable, since a higher surge travels at a faster velocity, with greater friction losses that a lower surge. The necessary calculations, figures and tables for the hurricane surge routing are presented below.

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APPENDIX A'

Selected Calculations and Figures from
the Preliminary Report

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APPENDIX B

FLOOD STUDY

OF

UPPER HUDSON RIVER BASIN

MARCH 21, 1969

IPEC00034968

QUIRK, LAWLER & MATUSKY ENGINEERS

STONE & WEBSTER ENGINEERING CORPORATION

BOSTON, MASS.

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QUIRK, LAWLER & MATUSKY ENGINEERS

REPORT ON
FLOOD STUDY
UPPER HUDSON RIVER BASIN

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INTRODUCTION

PURPOSE AND SCOPE

In order to develope hydrological data for the Upper Hudson River Basin during flood conditions, an investigation was made to determine the maximum predictable stage of the Hudson River at Stillwater less than five miles upstream of Mechanicville and immediately above the mouth of the Hoosic River.
A determination of stage was made for the following two conditions:

1. Probable Maximum Flood – Stage at peak discharge due to runoff from the probable maximum precipitation over the drainage area upstream of Stillwater.

2. Dam Failure – Stage due to an assumed failure of the Conklingville Dam at the time the Sacandaga Reservoir is at its highest stage from the probable maximum flood.

SUMMARY OF RESULTS

The results of the investigation fro the two conditions is as follows:

1. The probable maximum flood would have a peak flow of

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300,000 cfs and result in a river stage of El. 110.0 (USGS Datum) at the Stillwater.

2. The maximum river stage resulting from a failure of the Conklingville Dam would be El. 124.0 (USGS Datum) at the Stillwater.

DESCRIPTION OF THE BASIN

The basin covered by this study is that portion of the Upper Hudson River Basin north of Stillwater, New York and is located in the central part of the great trough that extends northward from New York harbor to the St. Lawrence River. Its drainage area, lying principally in east central New York, is about 3,760 square miles. (See Figure 1.)

The major topographical feature of the Watershed is the rugged eastern portion of the Adirondack Mountains which has peaks rising up to abut 5,000 ft. The northern part of the basin is wilderness country which has dense forests and contains many lakes and small streams. The valleys of the basin are covered by extensive deposits of glacial material produced by the ice sheets which formerly covered the area.

The Hudson River has its source in the Adirondack Mountains at Lake Henderson at an elevation of 1,808 ft above sea level and flows southerly for a distance of about 315 river miles to enter New York Bay. From its source to Stillwater Dam the river drops about 1,725 ft in a distance of about 140 river miles. Abrupt drops of 80, 45 and 60 ft occur at Palmer Falls, Glens Falls and Bakers Falls, respectively.

The tributaries of the Upper Hudson are numerous, the more important in order of size being the Sacandaga River uniting with the main stream at Hadley, the Schroon River at Thurman, Batten Kill and Fish Creeks at Schuylerville, and the Indian River near Gooley. The flow into the Hudson from these tributaries is controlled by dams which form reservoirs that are maintained at relatively fixed levels.

The climate of the Upper Hudson River Basin is characterized by long, cold and snowy winters and short, mild summers. The average annual rainfall varies from about 40 in. in the central portion to over 50 in. in the headwaters of the Sacandaga River. The precipitation is normally fairly evenly distributed throughout the year, with only a slight rise during the summer.

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The average annual runoff from the basin is about 55 percent of the average rainfall, with the runoff from the easterly half being a lower percentage than the westerly portion.

STREAMFLOW DATA

Stream gaging stations have been maintained by the U.S. Geological Survey and others for various periods of time at over 30 locations within the upper Hudson River watershed above Stillwater.

The main river gaging station with the longest period of published record is located at Mechanicville, New York, four and one-half miles downstream of Stillwater The drainage area above the gage is 4,500 square miles, of which approximately 650 square miles is in the Hoosic River watershed which enters the Hudson River one and one-half miles above the gage. Continuous stream flow data at Mechanicville have been published for the period October 1887 to September 1956. The USGS have maintained a gaging station on the Hoosic River at Eagle Bridge with a drainage area of 510 square miles from 1910 to date. Using the records of both these gages, an estimate of past stream flow at Stillwater can be made withr easonable confidence.

A summary of runoff data at 10 stations representative of the basin in general is shown in Table 1.

STREAMFLOW REGULATION

The Upper Hudson River is well regulated from the standpoint of flood control. The most important regulating feature is the Sacandaga Reservoir, operated by the Hudson River – Black River Regulating District. This reservoir, controlling the Sacandaga River drainage area of 1,044 square miles or nearly 30 percent of the total upper basin area, has a storage capacity of 866,000 acre-ft at spillway crest level. The high crest and relatively

small spillway at Conklingville Dam also provide the reservoir with a large amount of flood detention capacity above the rated storage. Additional tributary regulation is provided by Indian Lake Reservoir, operated by the Indian River Company. This reservoir has a capacity of 114,000 acre-ft and controls 131 square miles of Indian River drainage.

Natural storage for reducing flood flows is also provided by widespread lake areas such as Schroon Lake, Chain Lakes and

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Saratoga Lake. Small dams at the outlet of Schroon Lake and on Fish Creek downstream of Saratoga Lake effectively increase the natural lake storage and reduce the flood potential of drainages of about 560 and 250 square miles on the Schroon River and Fish Creek, respectively.

On the main stem of the Upper Hudson there are a number of man-made controls, principally in the form of power dams. The river profile is shown on Figure 2. From Corinth to Fort Edward, a distance of about 23 river miles, there is a series of eight privately owned power dams. In this distance, the river drops about 420 ft. Although the storage capacity of none of these dams is very large, their combined effect is to retard flood flows and reduce flood stage downstream.

Proceeding downstream, the Champlain Canal, part of the New York State Barge Canal System, makes use of the Hudson between Port Edward and Troy. The river is channeled and controlled for navigation by a series of dams and locks. The river drops about 60 ft between Fort Edward and the Upper Mechanicville Dam. This section of the river channel, generally larger in cross sectional area than upstream, provides additional flood control regulation through natural valley storage.

RECORD FLOODS

The highest flood on record on the Hudson River at Mechanicville occurred in March 1913, prior to construction of the Sacandaga Reservoir. The flow reached a peak of 120,000 cfs on March 28. The rainfall causing this flood was general over the entire north-central and northeastern United States. The storm was preceded by a thaw with temperatures rising to 70 F, followed by over 5 in. of rain in five days. The estimated maximum mean daily flow at Stillwater was 107,000 cfs during this storm.

The second greatest flood at Mechanicville was the New Year flood of 1949, which had a peak flow of 118,000 cfs on January 1, 1949. The runoff was the result of over 6 inc. of precipitation which fell as a combination of rain, snow and sleet. The estimated maximum mean daily flow at Stillwater was 61,5000 cfs. The significantly lower flood flow at the Stillwater in comparison

with the 1913 storm is due to large measure to the flood storage provided by the Sacandaga Reservoir. The flood flows in the Hudson River above the Sacandaga River and in the Sacandaga above Conklingville for both storms were approximately equal. However,

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the discharge of the Sacandaga during march 1913 reached a peak of 35,5000 cfs, while during the 1949 flood the two-day average discharge was less than 500 cfs.

Following is a list of the major floods and maximum mean daily flows recorded at Mechanicville and estimated at Stillwater:

Date	Recorded Flow at Mechanicville, cfs	Estimated Flow at Stillwater cfs
March 1913	113,500	107,000
April 1914	64,800	59,000
April 1922	72,900	67,000
March 1936	72,700	54,500
September 1938	65,600	38,000
January 1949	94,400	61,500

Maximum stages and discharge for key gaging stations within th watershed are shown on Table 2.

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5.

DEFINITION

The probable maximum flood has been defined as an estimate of the hypothetical flood characteristics that are considered to be the most severe "reasonably possible" at a particular location, based on comprehensive hydrometeorological analyses of critical runoff producing precipitation and hydrologic factors favorable for maximum flood runoff. (1) It has been further described as the estimate of the boundary between possible floods and impossible floos. The objective, therefore, is to obtain a flood that has a chance of occurance approaching zero or a return period of infinity.

Using the above definition as a guide the probable maximum flood for the Hudson River at Stillwater was developed as follows:

1. The basin was divided into subbasins or hydrologice units on the basis of tributary drainage area and location of hydraulic controls and unit hydrographs developed for each subbasin.

2. The probable maximum precipitation was applied to the unit hydrographs with the appropriate infiltration losses to develop the flood hydrograph for each subbasin.

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3. Starting at the uppermost reach in the Hudson River, the subbasin flood hydrographs were combined in their proper time sequence and routed downstream. The process of combining inflows and routing continued in the downstream direction until the flow at Stillwater was determined.

A detailed description of the development of the hydrometeorological characteristics of the basin and synthesis of the probable maximum flood are given in the following sections.

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6.

PROBABLE MAXIMUM PRECIPITATION

The records show that a number of large floods on the Hudson river are the results of an early spring storm combined with melting snow. However, the data contained in Hydrometeorology Report No. 33[2] indicate that the probable maximum precipitation for the spring months would be about 50 percent of the all season probable maximum precipitation. In addition a study made by the U. S. Army Corps of Engineers for the adjacent Connecticut River Basin concluded that the maximum storm rainfall would be at least equal to the maximum combination of rain and melting snow[3]. It was therefore concluded that use of a summer or fall storm would produce a runoff of at least as great as a spring storm with snow melt.

The probable maximum precipitation used in this study was developed from the 72-hour precipitation and depth-duration-drainage area curves prepared by the Hydrometeorologoical Section, U.S. Weather Bureau, for the Delaware River at the U.S. Army Corps of Engineers dam site at Tocks Island. [4]

Tocks Island rainfall data include two subtropical storm patterns A and B, which have been given transposition limits. However, since there is a general similarity in orientation, size, shape and topography of the Delaware River Basin above Tocks Island and the Upper Hudson River Basin, both storms were transposed over the Upper Hudson River Basin within the orientation limits prescribed by the Weather Bureau so as to produce the maximum rainfall on critical areas. It was found that transposed storm pattern B produced the greater rainfall and it was used in this study. This storm pattern is of the extra-tropical type where rainfall centers are associated with convergence in the vicinity of waves on frontal boundaries. In its early stages, it was of tropical character.

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The 72-hour isohyets for storm pattern B in the transposed location used in this study are shown in red on Fig. 1A. The pattern produces a 72-hour total precipitation equivalent to an average of 14,3 in. over the entire drainage area above Stillwater. Using applicable rainfall data contained in Hydrometeorological Report No. 28, [(5)] and Hydrometeorological Report No. 40, [(6)] it was determined that the probable maximum 72-hour precipitation for the area under study would be 13.0 and 14.4 in., respectively.

Incremental average depths for subdurations were obtained by applying the time ratio from the Tocks Island DDA curves to the 72-hour basin probable maximum precipitation. The precipitation for each subbasin was determined by planimetering the area between isohyets within the subbasin boundaries. The incremental

average depth for the subduration for each subbasin was obtained by applying the same ratio as found for the entire basin. The six-hour incremental probable maximum values were rearranged in accordance with the following criteria recommended in Hydrometeorological Report No. 40:

a. Group the four highest six-hour increments of the 72-hour PMP in a 24-hour period, the middle four increments in a 24-hour period, and the lowest four increments in a 24-hour period.

b. Within each of these 24-hour periods, arrange the four increments in accordance with sequential requirements; that is, the second highest next to the highest, the third highest adjacent to htese, and the fourth highest at the either end.

c. Arrange the three 24-hour periods in accordance with the sequential requirements: that is, the second highest 24-hour period next to the highest, with the third at either end. Any possible sequence of three 24-hour periods is acceptable with the exception of placing the lowest 24-hour period in the middle.

The 6-hour incremental precipitation depths for the entire area under study arranged in order according to the above criteria are as follows:

Hour	Incremental Depth-in	Accumulative Depth-in
6	0.2	0.2
12	0.4	0.6

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18	0.6	1.2
24	0.2	1.4
30	1.3	2.7
36	7.8	10.5
42	2.3	12.8
48	1.1	13.9
54	0.1	14.0
60	0.1	14.1
66	0.1	14.2
72	0.1	14.3

UNIT HYDROGRAPHS

Unit hydrographs were derived for 19 subbasins in the basin by the methods described in "Unit Hydrographs – Part – Principles and Determination,"[7] and "Hydrology Guide for Use in Watershed Planning".[8]

Unit hydrographs were determined from records of nine gaging stations located on the various streams and reservoirs within the basin having drainage areas varying in size from 90 to 1,044 square miles. In general, the unit hydrographs were developed from the hurricane storm of September 1938 and compared with the next largest storm for which data were available. The second storm used varied from area to area. The storm of September 1938 produced the largest floods without snow melt for which adequate records are available. It was not possible to develop a unit hydrograph for the Indian Lake Reservoir and Batten Kill from the September 1938 storm because of inadequate rainfall data, and other storms were used. A computer program described by D. W. Newton and J. W. Vinyard,[9] was used in developing the unit hydrographs for the gaged areas. Data for the observed unit hydrographs are shown on Table 3.

Unit hydrographs for approximately 70 percent of the total drainage area were developed from rainfall and runoff records. The unit hydrographs for the Sacandaga Reservoir and Indian Lake inflows and the Hudson River at the USGS gage at Gooley were used without modification. The unit hydrographs developed at the gaging stations on the Schroon River, Batten Kill and Kayaderosseras Creek were transposed to their respective mouths using Snyder's coefficients and the method outlined in "Unit Hydrographs – Part I Principles and Determination."

Synthetic unit hydrographs were developed for the large ungaged subbasins using Snyder's relations and coefficients developed from the gaged area unit hydrographs and data developed by Taylor

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and Schawarz, [10] for basins in the north and middle Atlantic states.

The remaining unit hydrographs were developed by the method contained in "HYdrology Guide for Use in Watershed Planning." These unit hydrographs are for the short, very steep streams immediately adjacent to the Hudson River. In general, the drainage areas for the individual streams are small, and they have been combined into large subbasins extending between routing points on the Hudson River.

No unit hydrographs have been developed for a few small, mostly water surface subbasins adjacent to the Hudson River. It was assumed that runoff from the precipitation is instantaneous for these areas.

The data for the 19 subbasin unit hydrographs are shown on Table No. 4 and the 6-hour unit hydrographs are listed on Table No. 5.

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9

STORM HYDROGRAPHS

In order to define the time discharge relationship for inflows to the major reservoirs and to the Hudson from its tributaries and the drainage areas along its banks, storm hydrographs were developed using the unit hydrographs previously described. The time distribution of the rainfall in each subbasin was the same as was used for the entire basin. For a subbasin, the amount of direct runoff resulting from the rainfall during each time increment was then determined using the method described in Chapter II, Section 53, "Design of Small Dams."[11] the curve numbers used in the above method were selected after analysis of precipitation and runoff data in the basin. Where these data were available for more than one storm, the curve number was used which corresponded to the largest runoff-to-precipitation ratio.

The direct runoff increments were applied to the unit hydrograph to produce the flood hydrograph for beach subbasin. With base flows added where applicable, these storm hydrographs defined the inflows into the Hudson River from its tributaries and bank drainage areas.

RESERVOIR FLOOD ROUTING

Inflows to Indian Lake, Sacandaga Reservoir, Stewarts Bridge Reservoir and Saratoga Lake resulting from the probable maximum storm were routed through these reservoirs using a graphical

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solution of the step method of flood routing. The data used for routing were: (1) inflow storm hydrographs, except for Stewarts Bridge; (2) reservoir storage curves, and (3) spillway rating curves. At Stewarts Bridge, the inflow used for routing was the discharge from Conklingville Dam with a small adjustment for inflow from the intervening drainage area.

Reservoir conditions assumed prior to the storm and predictions of reservoir operation during routing of the probable maximum flood are shown in the following tabulation:

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10.

Reservoir	Water Surface Elevation Before Storm	Conditions During Routing: Max W.S. El.	Max Discharge Cfs	Time Hours
Indian Lake	1651.35 (spillway crest)	1660.5	10,950	72
Sacandaga	768.0 (bottom of flood storage)	784.0	44,000	90
Stewarts Bridge	705.0 (normal H.W.)	710.5	43,000	90
Saratoga Lake	202.4 (top of flash-boards at Winnies Reef)	215.4	14,200	84

*Time from beginning of P.M.P.

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11.

The Sacandaga Reservoir flood control storage pool between Elev. 768 and 771 is rarely used except during the spring months when an effort is made to store the maximum volume of snow melt and spring precipitation. In over 45 years of operation the maximum reservoir state was Elev. 769.43 on April 19, 1954, and at that elevation less than half of the flood control storage pool was used. The probable maximum precipitation for this study is a late summer or fall storm and the Sacandaga Reservoir would normally be below the flood pool. However it is possible that a prior storm could result in an abnormally high level and it is judged that an initial reservoir level of Elev. 768.0 for this study is conservative and reasonable. The other reservoirs have no specified volume allocated for flood storage and the initial

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water surfaces were chosen as being the maximum that could exist at the time.

Discharges were calculated at Indian River Dam with the sluice gates closed, at Conklingville Dam with the Dow Valves open and no flow through the E. J. West hydroelectric Station, and at Stewarts Bridge with the five Tainter gates open and no flow through the turbines.

At Indian Lake, the maximum water surface elevation predicted is approximately at the crest of the earth dike section of the dam. it was assumed that for a flood of this magnitude and duration, precautionary measures would be taken at the dam to prevent overtopping of the earth sections and overflow discharge would be confined to the masonry sections. Minimum freeboards at Conklingville and Stewarts Bridge are 10.5 ft and 3.5 ft, respectively. Winnies Reef, the control for Saratoga Lake, would be submerged at the flow predicted, but overtopping this low concrete structure presents no serious structural hazard.

Outflows derived from the preceding reservoir routings were then routed down their respective channels to the Hudson River using the stream flood routing techniques described in the following sections.

CHANNEL ROUTING METHODS

Two general methods were used in routing the probable maximum flood down the main stem of the Hudson River. The first method, used for the upper portion of the river between North Creek and Hadley and for the lower portion between Fort Edward and Stillwater, was based on what is commonly called the coefficient method of routing. The procedures used generally followed the methods outlined in Corps of Engineers Engineering and Design Manual on Flood Routing.[12] The second method used was the step method of reservoir routing. This method was used for the
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12.

portion of the river between Hadley and Fort Edward because of the many dams within the reach which control the river flow.

RIVER CHANNEL CHARACTERISTICS

Most of the effort required to perform the flood routing was connected with the collection and evaluation of physical data necessary to define the river flow and valley storage characteristics. The task of determining these characteristics at the stages anticipated was greatly complicated by the fact that there are no floods of record approaching the maximum

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discharges being studied.

In order to route the flood using the two methods mentioned above, several river parameters had to be defined. Reduced to simplest form, these were:

a. Hydraulic characteristics of the river, which include area, top width and hydraulic radius at enough river cross sections to adequately define the channel.

b. Manning's "n"

c. State-discharge-volume relationships for the various reaches of the river

c. X and K, which are constants used to determine routing coefficients

The methods used to determine the above parameters will be described briefly.

<u>Hydraulic Characteristics</u>

The hydraulic characteristics of the river were determined mainly from river cross section data. Approximately 120 river cross sections were used between Mechanicville and Thurman Station, a distance just under 75 river miles. The majority of the above water portions of these sections were taken from large scale topographic maps furnished by Niagara Mohawk Power Corporation. Large scale topographic maps were not available for a section of the river below Fort Edward and another section between Palmer and Hadley, and field cross sections were made in the fall of 1968. Underwater soundings, by fathometer, for most cross sections were obtained from the Albany office of the U.S. Geological Survey, which were combined with the above data to obtain a complete cross section of the river channel and banks. Additional valuable information on the section of the river below Fort Edward was obtained from the New York State Barge Canal charts of the Chamblain Canal. Use was made of all available

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13.

U.S. Geological Survey maps of the area for general river channel topography and to fill in any gaps in the above-mentioned information.

In the fall of 1968, prior to making field surveys, a field reconnaissance of the river and its major tributaries was made by two engineers from Stone & Webster Engineering Corporation.

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Accompanied by engineers from Niagara Mohawk Power Corporation, they visited all the major hydraulic structures in the drainage basin and collected hydraulic data and pertinent drawings of the structures wherever available from state and federal agencies and private industry. In addition, a determination was made of where field survey data were required and a photographic record made of these areas and at the locations of available cross sections.

Mannings's "n"

Limited data were available for a determination of Manning's "n" from velocity measurements made by the U.S. Geological Survey. These data were used to determine an "n" value at three locations using the velocity distribution method. (reference 13, p.207). An additional value for "n" was calculated from flood stage discharge data. Based on these calculations and the visual observations recommended by Barnes[14] made during the field reconnaissance, an estimated average value of 0.025 was used in the final calculations. However, because of uncertainty in the determination of average "n" values at flood stages, trial calculations were made assuming n=0.035. The higher value, while tending to increase river stage for a particular discharge, also increased the valley storage which resulted in a somewhat lower maximum discharge at the site. Therefore, from these calculations it was judged that the stage variation which would result from varying "n" within reasonable limits would be less than the tolerances of the other physical data upon which the calculations were based.

Stage-Discharge-Volume

Both of the methods used for flood routing required determination of the state-discharge-volume relationships for certain reaches of the river. To facilitate this, the river from Mechanicville to Thurman Station was divided into 13 reaches, each reach starting at a control point on the river. Backwater curves were calculated using the standard step method (reference 13, p.265) for each reach, starting with assumed discharges at the downstream control. A digital computer program was used to make these calculations. Input required included the hydraulic characteristics, Manning's "n," a discharge rating curve for the downstream control and an estimate of eddy and form losses from bends and contractions in the river channel. The output

consisted of a water surface profile and volume of water stored in the reach at a particular steady state discharge.

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At high flows, the river stage would in many cases overflow the banks. A precise determination of the amount of flow in the overbank under these conditions in the absence of records is impossible. A solution to this problen was effected by the use of two different cross sections at each river station where hydraulic characteristics were determined. One section was made for the complete river valley including overbank. The second section, which was in most cases less than the first, included only that portion of the river valley in which appreciable flow was judged to take place. The determination of this second or flow section was based largely on judgement after careful consideration of topography, vegetation and all physical features, including those which were man-made, in the river valley. All storage values were based on the overbank sections, while backwater profiles for steady state flow were determined using the flow sections.

Rating curves for most of the dams in this section of the river were furnished by the owners. Theoretical rating curves were calculated at several points and all rating curves were extended theoretically when the discharge exceeded the design capacity of the spillways. The rating curves were developed using the fixed crest of the structures, assuming no flash boards in place, all spillway gates open, and no flow through turbines or supplementary outlets.

In computing the valley storage for the reach between Stillwater and Hadley, it was assumed that all dams could pass the probable maximum flood without failure. It was further assumed that the volume below the fixed dam crest under consideration could be ignored and that only the volume above the crest was usable.

<u>X and K</u>

The interrelated constants S and K, used to determine the coefficients for routing by the coefficient method, were evaluated on the section of the river above Thurman Station by the inverse flood routing process. Below Thurman Station, because of lack of adequate flood data, evaluation of these constants was made from the stage-discharge-volume relationship for each reach. This procedure required determination first of K, which is the travel time for an elemental discharge wave to traverse the reach. Then an X, which is a measure of the wedge storage in the reach, was assumed and the flood was routed.

Finally, to check the correctness of the assumed X, the actual wedge storage was calculated using a step backwater calculation

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with varying flow within the reach corresponding to the inflow and outflow hydrographs. Trials were continued until the assumed and calculated X's were substantially in agreement.

Values used for X and K were as follows:

River Reach	X	K(Hr)
Gooley to North Creek	0.4	3
North Creek to Thurman Station	0.3	3
Thurman Station to Hadley	0.1	6
Fort Edward to Thomson	0.3	3
Thomson to Stillwater	0.3	6

The routing coefficients used were those tabulated in reference 12 for the above values of X and K when t=3 hrs.

ROUTING PROCEDURE

Routing the maximum probable flood from the headwaters to Hadley was accomplished using the coefficient method and the procedure described in Reference 12. The routing started with the inflow flood hydrograph at Gooley and proceeded to Hadley in two steps. Tributary inflow and riverbank runoff were combined with main stream flow at the appropriate times. Discharge from Indian Lake was routed to the Hudson River using the coefficient method also.

From Hadley to Fort Edward the river was divided into reaches varying from about two to seven miles long, each reach terminating at one of the eight dams shown on Figure 2 for that stretch of the river. Inflow from the Sacandaga River was combined with flow in the Hudson at Hadley and routed from dam to dam using the step method of reservoir routing. Using the storage curves developed from backwater computations, routing was accomplished in three-hour time steps with addition at appropriate locations of the flood hydrographs of the inflows along the riverbanks from the subareas of subbasin 9 shown on Figure 1. Checks of wave travel times for the various reaches were made by calculating the change in storage with discharge for the various discharges considered. These travel times were found to vary from a few minutes to about one and a half hours, considerably less than the time step used.

Flow characteristics and water surface profiles for the reaches below Fort Edward were determined starting at the Upper

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Mechanicville Dam, because it was not known initially where river

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control might be for establishing stage at Stillwater. It was found that critical depth occurred at Stillwater Dam for flow up to at least 300,000 cfs. It was also determined that critical depth did not occur at Crockers Reef except at flows below 50,000 cfs. Therefore, this section of the river was divided into three reaches terminating at the dams at Fort Miller, Thomson and Stillwater. Calculations of change in storage with discharge were made for these reaches and the travel times were determined as being about three hours from Fort Edward to Thomson and about six hours from Thomson to Stillwater. The discharge at Fort Edward was accordingly routed to Thomson and then to Stillwater using the coefficient method, again adding tributary and riverbank inflow at appropriate times and locations.

STAGE-DISCHARGE RELATION

the stage-discharge relation shown on Figure 4 is for the river section at the northern end of the Easton Site, approximately 4 ½ miles upstream of Stillwater Dam.

The rating curve was developed from backwater curves calculated from Stillwater or Upper Mechanicville Dams. Because Stillwater Dam is a relatively low structure, the backwater computations were initiated at Upper Mechanicville and flow conditions checked for control at Stillwater Dam. It was found that at flows greater than 500,000 cfs Stillwater Dam became submerged and control shifted downstream to Upper Mechanicville Dam. The backwater curves were computed as described in the section under flood routing and verified, where possible, with Champlain Barge Canal gage records.

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17.

DAM BREAK WAVE

UPSTREAM RESERVOIRS

The major existing dams with significant storage located above Stillwater are listed in Table No. 6. In addition, there are a number of smaller dams with minor storage capacities located on the various tributaries above Stillwater. It is obvious that the largest single block of storage is contained in the

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Sacandaga Reservoir behind the Conklingville Dam, with a volume greatly in excess of all other reservoirs combine. Failure of the Conklingville Dam would release the largest confined volume of water in the basin and would result in the highest conceivable stage at Stillwater if it should occur coincident with flood conditions.

CONKLINGVILLE DAM

The Conklingville Dam, completed in 1930, is located on the Sacandaga River, as shown on Figures 1 and 2. It is an earth dam founded on rock and earth, with a concrete gravity spillway built on rock. The earth dam was built by the semi-hydraulic fill method. It has a crest width of 43 ft. at El. 794.5 and a base width of 650 ft. at its maximum height of 96 ft., the width-to height ratio being 6.75 to 1. At spillway crest El. 771.0, the reservoir has a total capacity of 37.8 billion cubic feet. The reservoir volume above El. 768.0 is reserved for flood control purposes. The diversion canal and spillway are located in a rock section away from and to the left of the earth dam. The outlet works consists of three 8 ft. diam. Dow Valves and two 18 ft. by 8 ft. siphons. The spillway is an ungated free overflow section 405 ft. long.

The dam was designed for large surcharge, having a freeboard height of 23.5 above the spillway crest. The maximum level attained by the reservoir during the 38 years of operation was El. 769.43, 1.57 ft below the spillway crest. When routing the probable maximum flood through the reservoir, the maximum stage reached was El. 784.0, 10.5 ft below the crest of the earth dam.

The dam was built by the State of New York and is maintained and operated by the Hudson River – Black River Regulating District. Subsequent to award of the construction contract, the rock excavation for the spillway channel was increased. This rock material was spoiled at the downstream toe of the dam, providing an unusually large rock toe section. The dam has an ample cross section, is inherently stable, and has proven its ability to withstand severe earthquakes, as mentioned below.

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A series of significant earthquakes have occurred in the area since completion of the dam. The highest recorded earthquake occurred on April 20, 1931, at Lake George, New York. This earthquake was recorded at Intensity VII (Modified Mercalli Scale) and was perceptible over an area of 60,000 square miles. The Sacandaga Reservoir level was at E. 752.2 at the time.

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Following the earthquake, a two-day inspection of the dam and reservoir was conducted and no damage was found.

there is no reason to believe that the Conklingville Dam would fail from earthquake, overtopping, or any other natural cause. However, because it does contain the largest volume of stored water in the entire basin, it was used in determining the stage at Stillwater from an assumed dam failure.

It was assumed that failure would occur with the reservoir at its maximum possible level, El. 784.0, which is caused by runoff generated by the probable maximum precipitation. At this surcharged elevation, the reservoir contains approximately 54 billion cubic feet of water.

It was further assumed that Stewarts Bridge Dam would fail prior to Conklingville Dam. This assumption is based on the fact that, in routing the maximum flood through the tow reservoirs, it is found that the freeboard at the Conklingville Dam would be 10.5 ft as compared to 3.5 ft at the Stewart Bridge Dam. While this shows that neither of the dams would be overtoppped during the maximum probable flood, in the event of a catastrophe, the smaller freeboard at the Stewart Bridge Dam would make it more likely to fail first.

MODE OF FAILURE

In a hypothetical study of this type, the first assumption to be made concerns the mode in which the structure fails, for this will greatly effect the resulting hydrograph. Earth fill structures such as Conklingviell Dam generally fail progressively, failure starting from an initial breach which increases in size by erosion of material under influence of the current. This mode of failure produces a hydrograph with an initially low discharge, increasing with time to a maximum, then decreasing as the reservoir elevation drops. The quantitative determination of this type hydrograph depends on the assumption of size and location of the initial breach, and rate of erosion, both of which are subject to question.

The maximum discharge rate would be obtained if failure were assumed to be instantaneous and complete and, for a discrete
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volume of water, produces a hydrograph of shortest duration, maximizing the flow concentration. This is the most conservative mode of failure that can be assumed, and was used for this study.

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DAM BREAK HYDROGRAPH

The physical laws governing unsteady flow in natural channels caused by a dam failure are among the most complex in the field of hydraulics. The first attempt to solve the problem was carried out by Saint-Venant who gave two differential equations bearing his name. These are based on a series of hypotheses which render them applicable only to gradually unsteady flow. While no integration of the equations is possible in the general case, certain simplifications and additional hypotheses have been used which have allowed integration and furnished solutions of limited applicability. Contributions based on theory and experiments have been made by many researchers including Ritter[15] Schocklitsch [16], Re[17], Pohle[18], Levin[19], Dressler[20-21], Stoker[22], Snyder[24], and U.S. Army Engineers[25].

Essentially, the sudden destruction of a dam results in a positive wave, advancing in downstream direction in the river channel, and a negative wave propagating in upstream direction into the storage reservoir. The wave velocity and profile depend on many factors including height of dam, channel and reservoir cross-section, channel resistance initial stream flow conditions, and length of storage reservoir. The simplifications commonly adopted by most researchers with a view to reducing the mathematical complexity of the problem included consideration of a prismatic, rectangular channel, horizontal channel bottom, and negligible flow resistance. The analytical methods which have been developed based on these simplifications have been confirmed by model studies, and as such can now be used as an engineering tool for determining flow conditions generated a sudden dam failure.

The objective of the Conklingville Dam failure study, as related to determination of the maximum possible stage at Stillwater, was to calculate a dam-break hydrograph to be used in flood routing. The hydropraph was determined by two independent approaches.

The first approach is essentially based on Stoker's method[22] which is the outgrowth of most of the theoretical and analytical work carried out to date. According to this method, the water depth in a rectangular channel at the dam site is 4/9 of the head in the reservoir until the negative wave reaches the end of the reservoir. To apply this method to they hypothetical failure of Conklingville Dam, the channel cross section at the Dam was approximated by three rectangles with a total area equaling that

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of the actual section. The downstream depth of flow was taken equal to that determined for the total flow at any time. The total releases were determined by the summation of the flows from each rectangle under a given head in the reservoir. The water surface in the reservoir was considered horizontal at any time and the drops in water level ranged between 0.5 ft and 5 ft, depending on stage. By plotting the calculated releases versus time, the Dam-break hydrograph was obtained. The results have been closely checked by the more recent work of the U.S. Army Corps of Engineers[24].

The second approach was developed as an independent check of the previously discussed method and was aimed at determining a conservative upper limit for the releases after hypothetical dam failure. It is assumed that flows are controlled only by the reservoir stage and channel characteristics and no energy is expended for negative wave motion. Critical depth is assumed to prevail at the dam section throughout the period under consideration and for all the flows after dam break. Essentially, this would mean that the channel bottom at the control section forms a broad-crested weir over which the water from the reservoir must flow. This would not be inconsistent with the relative steepness and geometry of the Sacandaga River channel below the dam. This assumption is the most conservative, since for a given head, the maximum flow is always released at critical depth. In determining the releases, a minor adjustment was made for head losses due to a change in channel cross section upstream of the dam. The results were again based on the assumption that the water surface in the reservoir is horizontal at any time. By plotting the calculated releases versus time, the dam-break hydrograph was obtained.

The hydrographs obtained with the two independent approaches described above are as follows:

	Stoker's Method		Critical Depth Method	
Time, Hr	Discharge, Cfs x 10^3	Total Outflow, Cu ft x 10^9	Discharge, Cfs x 10^3	Total Outflow, Cu ft x 10^9
0	990	0	1,410	0
5	780	14.5	980	20.0
10	616	25.9	690	33.3
15	482	34.3	479	42.1
20	380	40.8	336	48.2
25	300	46.0	215	52.0

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As anticipated, the critical depth method, which was used as an upper limit verification for Stoker's method, leads to higher flows, with the peak discharge 42 percent greater, and the volume released in the first 25 hours 11 percent greater.

ROUTING OF DAM BREAK WAVE

The downstream movement of the dam break wave is described as unsteady flow governed by the principles of conservation of energy and conservation of matter. Continuity and dynamic equations which mathematically describe the phenomenon of unsteady flow were first presented in the 19th century by Saint-Venant and are found in most texts dealing with unsteady flow. The equations have been verified by observations and experiments. However, owing to their mathematical complexity, an exact integration of the equations is almost impossible. Solutions therefore must be made by methods based on simplifying assumptions and approximate step methods.

The stream channels of the Sacandaga River and Hudson River between Conklingville Dam and Stillwater are of widely varying characteristics. The river has many sharp bends, man-made and natural constrictions and abrupt drops, all of which made the use of wave theory impracticable. The method used was the same as described in the section on routing the maximum probable flood. This method neglects the energy relationship and is based on the conservation of matter and, in effect, is a storage routing procedure. Because the energy relationship is neglected, the results obtained by using this method are very approximate for locations a short distance downstream of a breached dam. However as the reach length increases, the storage relationship becomes the more predominate factor in wave attenuation and results are of greater accuracy. Stillwater is approximately 60 miles downstream of the Conklingville Dam and it is believed that the storage routing procedure produces results within the accuracy of available physical data.

In routing, no advantage was taken of the storage available in the Sacandaga River. This reach is about six miles long, with a relatively narrow channel, without flood plains, containing a very small amount of valley storage, and it was conservatively assumed that the dam outflow hydrograph would be transposed to the confluence of the Sacandaga and Hudson Rivers undiminished.

The stage-storage-discharge relationships for the reaches downstream of the confluence were determined as described under the probable maximum flood routing section with the exception of the reach above Palmer Falls, which includes the mouth of the Sacandaga. It was recognized that a large flow from the

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Sacandaga would divide when it reached the Hudson River and flow

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22.

would be in the upstream as well as the downstream direction. This storage upstream of the confluence was calculated from the stage at the confluence determined by steady state backwater calculations from Palmer Falls, assuming a horizontal water surface upstream of the confluence. An adjustment in this volume was made by subtracting the volume occupied by the flow in the Hudson River at the time the dam break wave arrives.

It was assumed that all the dams downstream of the mouth of the Sacandaga River would pass the Conklingville Dam break wave without failure. It is realized that for some of the dams this assumption is not valid. However the combined volume impounded by all the dams on the Hudson River above Stillwater is about 1 billion cubic feet. This is about 2 percent of the total volume in the Sacandaga Reservoir at the time of the hypothetical failure and considerably less than the difference in the two dam break hydrographs discussed in the previous section. To include this volume in the computations would greatly increase the compexity of the solution without increasing the validity of the results.

The dam break hydrographs determined by both methods were routed to Stillwater using a time increment of 20 minutes. Tributary inflow and river bank runoff from the probable maximum precipitation were combined with the dam break wave in the proper time sequence as the wave was routed.

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23.

DISCUSSION OF RESULTS

PROBABLE MAXIMUM FLOOD

The probable maximum flood at Stillwater reaches a peak discharge of 300,000 cfs and a river stage elevation of 110 ft approximately 64 hours after the start of the precipitation. The maximum 24 hour mean discharge is 260,000 cfs. Figure 3 shows the hydrograph of the flood at the site.

At the existing Bakers Falls Dam the probable maximum flood has a peak discharge of 230,000 cfs and a maximum headwater elevation of 221 ft.

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It has been said that no method has been devised which can accurately indicate the frequency of large floods in the absence of stream flow records over long periods [25]. If the large flood being considered is several times larger than any observed evetn, as is the case for the predicted probable maximum flood of this study, the above statement can hardly be questioned. In fact, the probable maximum flood by definition has a frequency which is extremely large. However, in an attempt to bring some degree of perspective to the question of flood probability on the Hudson River at Stillwater, discharge-frequency curves based on a statistical evaluation of the available date were prepared, as shown on Figure 5, and extended to 10,000 years.

The data used for plotting the curves were based on USGS flow records at the Mechanicville gage from 1911 to 1956. Maximum daily discharge at Stillwater was obtained by correcting the Mechanicville flow for discharge from the Hoosic River and, prior tp 1930, for discharge from the Sacandaga River which would have been impounded by the Conklingville Dam.

the two curves, plotted on extreme value paper, are based on two of the numerous methods which have been suggested for estimating discharge-frequency relationships. These curves are thought to define the extremes of these relationships for the Hudson River at Stillwater. Curve A, based on a Type I extreme value distribution as suggested by Gumbel, represents an unsymmetrical data distrubution with a fixed skew. When the data are plotted, the flood of record falls considerably above the curve. Curve B is based on a graphical fit of the data distributed according to a method used by the U.S. Geological Survey[26]. The points plotted are for the latter distribution, but they are located approximately in the same position for both methods. The graphical fit shown by Curve B assumes a difference in the distribution parameters for the four floods with return periods exceeding 10 years. It has been suggested that outstanding

floods may in fact follow some law of their own which comes into operation at very long interval.,[25].

From Figure 5, the estimated discharge for a flood with a frequency of 10,000 years is as follows:

	Discharge – Cfs	
	Mean Daily	Peak=Mean Daily x 1.15

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Curve A	121,000	139,000
Curve B	232,000	267,000

The factor of 1.15 used to determine the peak discharge from the mean daily, is the ratio of these discharges found for the probable maximum flood.

Based on the above analysis, the peak discharge for a flood with a return period of 10,000 years would fall between 139,000 and 267,000 cu. ft. per second. These results indicate that the maximum flood predicted has a return period well in excess of 10,000 years and meets the requirements of obtaining a flood that has a change of occurance approaching zero.

It should be noted that the predicted maximum flood produces stages along the river which would inundate large areas, including many existing communities. However, it is recognized by most experts that all work can not be economically protected against such remote occurance and lesser floods are normally used for most design purposes.

Often the U.S. Army Corps of Engineers use a lesser flood designated as the Standard Project Flood for design. This flood represents critical concentration of runoff from the most severe combination of precipitation that is considered “reasonably characteristic” of the drainage basin involved. The Standard Project Flood Peak discharge and volume is usually equal to 40 percent to 60 percent of the probable maximum flood for the same drainage basin(1). There are some other design floods criteria used, which consider the degree of risk, hazzards involved and consequences of failure. The use of probable maximum flood as a design flood is not always justified or warranted for all projects and the design flood should be selected only after a complete study of all the factors involved.

DAM BREAK WAVE

The decision to assume that Conklingville Dam would fail in determining the effect of a dam break wave at Stillwater was based only on determining a hypothetical stage Stillwater. We believe that the probability of a

failure of Conklingville Dam is extremely remote. The probable maximum flood study clearly indicates the reservoir has sufficient storage and the dam ample freeboard to pass this flood safely without danger of overtopping. The dam has successfully withstood a significant earthquake without damage. However, because a hypothetical failure of Conklingville Dam would cause

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the highest possible stage at Stillwater this possibility was included in the study.

The determination of the maximum river stage at a point almost 60 miles downstream from a hypothetical dam failure is, at best, a rough estimate, greatly dependent on a large number of assumptions for solution. A conscientious effort was made to make all assumptions as reasonably conservative as possible. Two different approaches were used in determining the initial dam break hydrograph. Stoker's method is the most reasonable based on the present state of knowledge. The critical depth approach was used only as an upper limit verification, since it is admittedly ultraconservative. By routing the dam-break hydrographs obtained with the two approaches to Stillwater the following results were obtained:

	Stoker's Method	Critical-Depth Method
Max Discharge at Conklingville	990,000 cfs	1,410,000 cfs
Max Discharge at Stillwater	670,000 cfs	810,000 cfs
Max Stage at Stillwater	El. 124	El. 128

From the above tabulation it is apparent that in terms of maximum state at Stillwater the results obtained with the two independent approaches are reasonably close. In our opinion, this confirms the validity of Stoker's method which is itself based on many conservative assumptions. As pointed out before, the results obtained with this method were closely checked with a more recent work of the U.S. Army Corps of Engineers. Therefore, the maximum possible river stage at Stillwater resulting from the failure of Conklingville Dam coincident with the maximum flood is El. 124.

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REFERENCE LIST

1. Definition of Terms, "Survey Conducted by the Committee on Failure and Accidents to Large Dams Other Than in Connection with the Foundations," United States Committee on Large Dams, Feb. 1968

2. Hydrometerological Report No. 33 "Seasonal Variation of the Probable Maximum Precipitation East of the 105th Meridian for Areas from 10 to 1,000 Square Miles and Durations of 6, 12, 24 and 48 Hours" U.S. Department of Commence, Weather

IPEC00034995

Bureau, U.S. Government Printing Office, 1956

3. "Report on Project Flood for Review of Reports on Flood Controls for the Connecticut River Basin" Unpublished Report, U.S. Army Corps of Engineers, Providence District, August 1946

4. Gilman, C. S., Chief Hydrometeorological Section, U.S. Weather Bureau, Memorandum to Mr. A. L. Cochran, Civil Works, Office of Chief of Engineers, "Preliminary Estimate of Probable Maximum Precipitation for the Delaware River at Tocks Island Dam Site," July 3, 1956

5. Hydrometeorological Report No. 28, "Generalized Estimate of Maximum Possible Precipitation Over New England and New York," U.S. Department of Commerce, Weather Bureau, U.S. Government Printing Office, 1952

6. Hydrometeorological Report No. 40, "Probable Maximum Precipitation – Susquehanna River Drainage Above Harrisburg, Pa.", U.S. Department of Commerce, Weather Bureau, U.S. Government Printing Office, 1965

7. "Unit Hydrographs, Part I – Principles and Determinations," Civil Works Investigation, Project 152, U.S. Army Engineer District, Baltimore: Corps of Engineers, Baltimore, Md., 1963

8. "Hydrology Guide for Use in Watershed Planning," National Engineering Handbook, Sec. 4, Supplement A, U.S. Department of Agriculture, Soil Conservation Service Central Technical Unit, Beltsville, Md., 1957

9. Newton, D. W., and Vinyard, J. W., "Computer-Determined Unit Hydrograph from Floods," Proceeding ASCE, Journal of the Hydraulic Division, Vol. 93, Hy-5, September 1967

10. Taylor, A. B., and Schwartz, H. E. , "Unit-Hydrograph Lag and Peak Flow Related to Basin Characteristics," Trans. Amer. Geophysical Union, Vol. 33, No. 2, April 1952

11. "Design of Small Dams," U.S. Department of Interior, Bureau of Reclamation, U.S. Government Printing Office, Washington, D.C., 1965

12. "Routing of Floods Through River Channels" U.S. Army Corps of Engineers, Engineering and Design Manuals EM 1110-2-1408, U.S. Government Printing Office, Washington, D.C., March

IPEC00034996

1960

13. Chow, V. T., "Open Channel Hydraulics," McGraw-Hill Book Company, New York 1959

14. Barnes, H. H., Jr., "Roughness Characteristics of Natural Channels," Geological Survey Water Supply Paper 1849, U.S. Government Printing Office, Washington, D.C., 1967

15. Ritter, A., "Die Fortpflanzung der Wasserwellen," (Propogation of Waves) , Zeitschrift des Verelines deutscher Ingenieure, Vol. 36, No. 33, pp. 947-954, Aug. 13, 1892

16. Shocklitsch, A., "Uber Dambruchwellen," (On Waves Produced by Broken Dams), Stilzungsberichte, Mathematisch-naturwess-enschaftliche Klasse, Akademie der Wissenschaften in Wien, Vol. 126, Part IIa, pp. 1489-1514, Vienna, 1917

17. Re., R., "Etude de Lacher Instantane d'une Retenue d'eau Dans un Canal par La methods Graphique," (Study of the Instantaneous Release of Water in a Reservoir to a Canal by the Graphic method), La Houille Blanche, 1st year, No. 3, pp. 181-187, Grenoble, May 1946

18. Pohle, F. V., "Motion of Wave Due to Breaking of a Dam and Related Problems," Paper No. 8, in Symposium on Gravity Waves, U.S. National Bureau of Standards, Circular 521, pp. 47-53, 1952

19. Levin, L., "Mouvement Non Permanent Sur Le Cors d'eau a la Suite de Ruptrue de Barrage' (Unsteady Flow in Channels Following the Rupture of Dam), Revue Generale de l'Hydraulique, Vol. 18, No. 72 pp. 293-315, Paris, Dec. 1952

20. Dressler, R. F., "Hydraulic Resistance Effect Upon the Dam-Break Functions," Paper 2356, Journal of Research, U.s. National Bureau of Standards, Vol. 49, No. 3, pp. 217-225, Sept. 1952

21. Dressler, R. F., "Comparison of Theories and Experiements for the Hydraulic Dam-break Wave," Assemblee Generale de Rome, 1954, International Association of Scientific Hydrology, Publication No. 38, Vol. 3, pp. 319-328, 1954

22. Stoker, J. J., "Water Waves, " Vol. IV of "Pure and Applied Mathematics," Interscience Publishers, Inc. New York, 1957

23. Snyder, F. F. "Hydrology of Spillway Design; Large Structures Adequate Data" Proceedings of ASCE, Journal of the Hydraulics Division, Vol. 90, HY-3, May 1964

IPEC00034997

24. "Floods Resulting from Suddenly Breached Dams" Miscellaneous Paper 2-374, Conditions of Minimum Resistance Report No. 1, February 1960, Condition of High Resistance Report No. 2, November 1961, U.S. Army Engineers Waterways Experiment Station, Viksburg, Mississippi

25. "Review of Flood Frequency Methods," Final Report of the Subcommittee of the Joint Division Committee on Floods, Transactons, ASCE vol. 118, pp. 1220-1230, 1953

26. Riggs, H. C., Frequency Curves, Chap. A2, "Techniques of Water Resources Investigations of the United States Geological Survey," U.S. Department of the Interior, U.S. Government Printing Office, Washington, D.C. 1968

B-31

OTHER SOURCES

Chow, V. T., "Handbook of Applied Hydrology," McGraw-Hill Book Company, New York, 1964

Lensley, R. K., Jr., Kohler, M.A., and Paulkus, J.C. H. "Applied Hydrology," McGraw-Hill Book Company, New York, 1949

Davis, C.V., "Handbook of Applied Hydraulics,"McGraw-Hill Book Company, New York, 1952

"Backwater Curves in River Channels," U.S. Army Corps of Engineers, Engineering and Design Manual EM 1110-2-1409, U.S. Government Printing Office, Washington, D.C., December 1952, Change 1, September 1960

Topographic Maps – Entire Basin, 7.5 Min and 15 Min Quadrangles and 1:250,000 Scale, U.S. Department of Interior Geological Survey, Topographic Division, Washington, D.C.

Topographic Maps – Hudson River, Fort Edward Dam to Palmer Falls Dam, Scale 1 In. = 200 ft, Niagara Mohawk Power Corporation

Topographic Maps – Hudson River, Palmer Falls Dam to Curtis Falls Dam, Scale 1 In. – 100 Ft, International Paper Company, 1966

Topographic Maps – Hudson River, Hadley to Mouth of the Schroon River, Scale 1 In. = 400 Ft, Niagara Mohawk Power Corporation, 1922

U. S. Lake Survey Chart No. 180, New York State Barge Canal System, U. S. Army Corps of Engineers, Detroit, Michigan,

IPEC00034998

QUIRK, LAWLER & MATUSKY ENGINEERS

1964

River Soundings, Fathometer Readings, Hudson River, Troy Locks to Corinth Bridge, U.S. Geological Survey, Albany, New York, 1967

Field Cross Sections and River Soundings, Clarkeson and Clough Associates, Albany, New York, 1968

Precipitation Records of Stations Within and Adjacent to Basin, National Weather Records Center, Asheville, North Carolina

Water Supply Papers, U.S. Department of Interior Geological Survey, Water Resources Division, U. S. Government Printing Office, Washington, D.C.

Unpublished Stage Recordings and Rating Tables, U.S. Department of Interior, Geological Survey, Water Resources Division, Albany, New York

B-32

2

Unpublished Gage Readings, Champlain Barge Canal, Lock 3c to Lock 7C, New York State Barge Canal, Canal Engineer, Waterford, New York

"Report on the Water Power and Storage Possibilities of the Hudson River" New York Water Power Commission, State of New York, 1922

U. S. Army Corps of Engineer "308 Report," Hudson River Basin, House Document No. 149, 72nd Congress, 1st Session, December 1931

Plans and data on the following dams were obtained from the listed owners:

Dam	Owner
Indian Lake Dam	Indian River Company
Conklingville Dam	Board of Hudson RIver-Black River Regulating District
Stewarts Bridge Dam	Niagara Mohawk Power Corporation
Curtis Falls Dam	International Paper Company

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Palmer Falls Dam	International Paper Company
Spiers Falls Dam	Niagara Mohawk Power Corporation
Sherman Island Dam	Niagara Mohawk Power Corporation
Feeder Dam	State of New York, Moreau Manufacturing Corporation
Glens Falls Dam	Finch, Pruyn & Company, Inc., Niagara Mohawk Power Corporation
Bakers Falls Dam	Niagara Mohawk Power Corporation
Crockers Reef	State of New York

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Dam	Owner
Fort Miller Dam	Fort Miller Pulp & Paper Company
Thomson Dam	United paperboard Corporation, Thomson Paper Company
Stillwater Dam	Niagara Mohawk Power Corporaiton
Upper Mechanicville Dam	State of New York, West Virginia Pulp and Paper Company
Winnies Reef	Niagara Mohawk Power Corporation

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APPENDIX C

NOTATIONS & SYMBOLS USED IN THE REPORT

IPEC00035000

A	=	Cross-sectional area
a	=	$0.5(1/v_2 - 1/v_1)$
$A_n A_n+1$	=	Area bounded by isohyets number n and n+1
B	=	Channel width
C	=	Constant equal to 4 feet ½/second determined from data supplied
D	=	River depth
D'	=	$\frac{dy}{dx}$
dg/dy	=	Slope of discharge rating curve at a station whose cross-section is representative of the reach for steady flow
F_E	=	Vertical component of earth pressure acting on downstream face, i.e. weight of earth mass vertically above the downstream face, acting through the centroid of that mass.
F_H	=	Horizontal component of hydrostatic pressure, acting along a line H/3 feet above the base. $1/2yH^2$, where y = specific weight of water
F_u	=	Uplift force on base of dam, as determined by foundation seepage analysis, and integration of point pressure intensities over base area; if foundation is homogeneously permeable, pressure varies approximately linearly from full hydrostatic head at the heel to full tailwater head, and F_u is approximately 1/2yHB, acting at B/3 from the heel. This value is often multiplied by some fraction less than 1 if the foundation is relatively impermeable, but it is on the safe side to assume uplift over the entire base area.
F_v	=	Vertical component of hydrostatic pressure. Weight of fluid mass vertically above the upstream face, acting through the centroid of that mass.

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NOTATIONS & SYMBOLS USED IN THE REPORT (Continued)

IPEC00035001

H	=	Resultant of all forces acting on the dam, equal to R but in the opposite direction.
H	=	Total head (elevation of the energy line) above a horizontal datum. The mean sea level was selected in this study as the datum.
H'	=	Water head above the spillway crest in feet
h	=	Depth of impounding water
h_e	=	Eddy loss. For convenience of computation, this loss was considered part of the friction loss h_f.
h_f	=	Friction loss between two end sections i and i+1
h_o	=	Depth of water below the dam
I	=	Rate of inflow into reservoir
i	=	Hydraulic quadient. It is equivalent to the slope of the seepage curve dy/dx.
I_1,I_2	=	Inflow at upstream end of reach 1 and 2
K	=	Permeability coefficient (ft/sec)
k	=	Time of travel of flood wave and also the change of storage per unit change of discharge
L	=	Distance from dam
L'	=	Length of Spillway or weir
n	=	Manning's roughness coefficient
n, n+1	=	subscripts denoting successive intervals of time of length $t_{n+}1 - {}^{t}n$
O	=	Rate of outflow from the reservoir

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$0_1'0_2$	=	Outflow from reach 1 and 2
$P_1'P_{n+1}$	=	Average depth of rainfall over the areas bounded by Isohytes 1 and 2
Q	=	River discharge
Q'	=	Flow resulting from dam failure at a distance L Downstream of the dam site
Q_o	=	Initial discharge after dam failure
$\mathcal{L}$	=	Seepage (cfs/ft)
R	=	Resultant of foundation shear and bearing pressures: Horizontal component, $R_H = F_H + F_E$ acting along the base; Vertical component, $R_V = W + F_V - F_U$ acting at a distance X from the toe that can be determined by the requirement For rotational equilibrium of the dam, by equating to zero the sum of the moments of all the foregoing forces about the toe of the dam.
R_h	=	Hydraulic radius
S	=	Available storage above spillway level
$\overline{S_f}$	=	Friction slope taken as the average of the slopes at the two end stations, i.e. $\overline{J}_f = ½ (S_i + S_{i+1})$
So	=	Slope of river bed
T	=	Time after dam failure
V	=	River velocity and is equal to Q/A
V_1	=	Velocity of wave front
V_2	=	Velocity of wave tail
Vn'Vn+1	=	Values of isohyets number n and n+1. These numbers refer to the lettered isohyets shown in Figure III-8
V_W	=	Rate of movement of flood wave.
W	=	Weight of dam – (area of cross-section of dam) (Sy) where S = specific gravity of masonry, approximately 2.4 or 2.5, acting through centroid of cross-section

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IPEC00035003

NOTATIONS & SYMBOLS USED IN THE REPORT (Continued)

W_O	=	Amount of water stored in reservoir
X	=	A dimensionless constant representing an index of the wedge storage in a routing reach
x	=	Distance measured from dam site
Y	=	Water surface elevation above mean sea level
Y'	=	Depth at the crest of the wave
y	=	Depth of seepage curve
Z_1	=	Water surface elevation above mean sea level
θ°	=	Angle between horizontal line and upstream face of the dam.
Δt	=	Length of the routing period having a maximum value of $2K_x$ and may be taken as equal to k. The routed hydrograph is relatively insensitive to the value bf Δt
α	=	Energy coefficient. This coefficient was assumed to be unity because of the fairly straight alignment and regular cross-section of the river between the Battery and Indian Point

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APPENDIX D

REFERENCES

1. U. S. Department of Commerce, Weather Bureau, Hydrometeorological Report No. 40, PROBABLE MAXIMUM PRECIPITATION, SUSQUEHANNA RIVER DRAINAGE BASIN ABOVE HARRISBURGH, PA., Washington, May 1965

2. U. S. Department of Commerce, Weather Bureau, Hydrometeorological Report No. 33, SEASONAL VARIATION OF THE PROBABLE MAXIMUM PRECIPITATION EAST OF THE 105TH MERIDIAN FOR AREAS FROM 10 TO 1,000 SQUARE MILES AND DURATION OF 6, 12, 24, AND 48 HOURS, U. S. Government Printing Office, 1956.

3. U. S. Army Corps of Engineers, Unpublished Report, REPORT ON PROJECT FLOOD FOR REVIEW OF REPORTS ON FLOOD CONTROLS FOR THE CONNECTICUT RIVER BASIN, Providence District, August, 1946.

IPEC00035004

4. Stone & Webster Engineering Corporation, FLOOD STUDY OF UPPER HUDSON RIVER BASIN, Boston, Mass., March 21, 1969.

5. U. S. Army Engineer District, Baltimore Corps of Engineers, Civil Works Investigations, Project 152, UNIT HYDROGRAPHS, PART I, PRINCIPLES AND DETERMINATIONS, Baltimore, Maryland, 1963.

6. U. S. Army Corps of Engineers, Engineering and Design Manuals EM 1110-2-1405, FLOOD HYDROGRAPH ANALYSIS AND COMPUTATIONS, U. S. Government Printing Office, Washington, D. C., August 31, 1959.

7. U. S. Army Corps of Engineers, Engineering and Design Manuals EM 1110-2-1408, ROUTING OF FLOODS THROUGH RIVER CHANNELS, U. S. Government Printing Office, Washington, D. C., March 1, 1960.

8. State of New York Hudson River Valley Commission, THE HUDSON WATER RESOURCES, New York, 1966

9. New England, New York Inter-Agency Committee, THE RESOURCES OF THE NEW ENGLAND-NEW YORK REGION, Part Two, Chapter XXXVII Subregion E (Hudson River Basin), New York, Vermont, Mass., 1955

10. Water Pollution Control Board, New York State Department of Health, LOWER HUDSON RIVER DRAINAGE BASIN SURVEY SERIES REPORT NO. 5, New York, 1953

D-1

11. Water Pollution Control Board, New York State Department of Health, LOWER HUDSON RIVER DRAINAGE BASIN SURVEY SERIES REPORT NO. 8, New York, 1960.

12. Summary Report of the Hudson River Valley Commission, THE HUDSON, New York, February 1, 1966.

13. Hudson River Valley Commission of New York, HUDSON RIVER ECOLOGY, New York, October, 1966.

14. Water Resources Commission, State of New York Conservation Department, Bulletin 61, THE HUDSON RIVER ESTUARY, A Preliminary Investigation of Flow and Water-Quality Characteristics, New York, 1967.

15. Quirk, Lawler & Matusky, HUDSON RIVER DISPERSION CHARACTERISTICS; Memo Report to Con Ed, October, 1965.

16. Several telephone conversations with Mr. Robert Forrest, Chief Engineer, Board of Hudson River-Black River Regulating District, March-April, 1970.

17. Several meetings and telephone conversations with Mr. Dwight E. Nunn, the Atomic Energy Commission, and Mr. P. Carpenter, FWPCA, March-April, 1970.

IPEC00035005

18. United States Geological Survey, FLOODS OF AUGUST-OCTOBER 1955, NEW ENGLAND TO NORTH CAROLINA, Water Supply Paper No. 1420.

19. Gilcrest, B. R., FLOOD ROUTING, Chapter X in Engineering Hydraulics, John Wiley & Sons Inc., New York, 1950.

20. Listvan, L.L., RASCOT MAKSIMALNOGO RASCHODA VODY OT PRORYVA NEKAPITALNYCH PLOTIN PRI PROJEKTIROVANII MOSTOVYCH PERECHODOR in Russian, or Computation of Flow after Dam Break for Design of Bridges, STPMS, Moscow, 1948.

21. Chow, Ven Te, OPEN-CHANNEL HYDRAULICS, McGraw-Hill Book Company, Inc., New York, 1959.

22. Data and Correspondence supplied by New York City Department of Water Resources concerning Ashokan Dam and Reservoir.

23. Morris, Henry M., APPLIED HYDRAULICS IN ENGINEERING, the Ronald Press Company, New York, 1963.

D-2

24. A meeting with Mr. Andrew Matusky of the U. S. Army Engineers District, Baltimore, February, 1970.

25. Prasad, Ramanand, A NUMERICAL METHOD OF COMPUTING GRADUALLY VARIED FLOW PROFILES, Presented to the Specialty Conference of the Hydraulics Division ASCE, at the Massachusetts Institute of Technology, Cambridge, Massachusetts, August 21, 1968.

26. Several telephone conversations between the author, Karim A. Abood of Q. L. & M. and Mr. Kenneth I. Darmer, Supervisory Hydrologist, Water Resources, Division, U.S.G.S., Albany, N.Y., March-April, 1970.

27. Baltzer, R. A. and Shen, J., FLOWS OF HOMOGENEOUS DENSITY IN TIDAL REACHES, U.S.G.S., Washington, September, 1961, Reprinted July, 1966.

28. Lai, Chintu, FLOWS OF HOMOGENEOUS DENSITY IN THE REACHES, SOLUTION BY THE METHOD OF CHARACTERISTICS, U.S.G.S., open file report, Washington, D.C., 1965.

29. Lai, Chintu, , FLOWS OF HOMOGENEOUS DENSITY IN TIDAL THE REACHES, SOLUTION BY THE IMPLICIT METHOD, U.S.G.S., open file report, Washington, D.C., 1965.

30. Lai, Chintu, , NUMERICAL SIMULATION OF WAVE-CREST MOVEMENT IN RIVERS AND ESTUARIES, Extract of "The Use of Analog and Digital Computers in Hydrologie," Symposium of Tucson, December, 1968.

IPEC00035006

31. Lai, Chintu, COMPUTATION OF TRANSIENT FLOWS IN RIVERS AND ESTUARIES BY THE MULTIPLE-REACH IMPLICIT METHOD, U.S.G.S. Prof. paper 575-B, 1967.

32. Lai, Chintu, COMPUTATION OF TRANSIENT FLOWS IN RIVERS AND ESTUARIES BY THE MULTIPLE-REACH METHOD OF CHARACTERISTICS, U.S.G.S. Prof. paper 575-D, 1967.

33. J. J. Dronkers, TIDAL COMPUTATIONS IN RIVERS AND COASTAL WATERS, Interscience publishers, Division of John Wiley and Sons, N. Y., 1964.

34. U.S.C. & G.S., TIDAL AND CURRENTS IN HUDSON RIVER, Special publication No. 180 by Paul Schureman, Washington, 1934.

D-3

35. Darmer, Kenneth I., HYDROLOGIC CHARACTERISTICS OF THE HUDSON RIVER ESTUARY, presented at the 2nd HRVC Hudson River Symposium, 1969.

36. Gofseyeff, S. and Panuzio, Frank L., HURRICANE STUDIES OF NEW YORK HARBOR, Journal of the Waterways and Harbors Division, proceedings of the ASCE, February 1962.

37. Wilson, B., THE PREDICTION OF HURRICANE STORM-TIDES IN NEW YORK BAY, Technical Memorandum No. 120, Beach Erosion Board Corps of Engineers, 1960.

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IPEC00035007

2.6 Meteorology

The description of site meteorology is given in Section 2.6.1 while a brief discussion of specific site meteorological and atmospheric diffusion studies are given in section 2.6.2. The technical reports pertaining to these site specific studies are also include. The safety analysis presented in section 2.6.3 and 2.6.4 is based on the site specific studies discussed in section 2.6.2. Section 2.6.5 provides a brief description of the onsite meteorological monitoring program.

The discussion of site meteorology given in section 2.6.1 is based on selected individual years in which analysis of meteorological data was performed. It should be pointed out that although the years selected are representative of the site meteorology, at least some year to year variability in the meteorological parameters will occur.

2.6.1 Site Meteorology

Winds

An important meteorological characteristic of the Indian Point Environment is that both northerly and southerly winds occur at maximum frequency. This is evident in all meteorological data collected at Indian Point from 1955 to the present.

Figures 2.6-1 a, b, and c present some constructed wind roses prepared using meteorological data collected during 1984 from the onsite 122 meter meteorological tower [1,2,3,4]. These wind roses provide an example of typical wind direction and frequency distributions that occur at Indian Point, on a quarterly basis for the 10 meter, 60 meter, and 122 meter levels of the tower. These wind roses show clearly the bidirectional frequencies in the wind directions, with frequency maximas in the north and south direction.

A comparison of the 10 meter level wind roses between each of the four quarterly periods during 1984 (Figure 2.6-1a) shows that north winds had the highest frequency during the period January-March, while northeast winds dominated during the remaining three quarterly periods. The period July-September had the highest frequency of northeast and south winds. South winds occurred with the lowest frequency during the period January-March.

At both the 60 meter and 122 meter levels (Figures 2.6-1b, 2.6-1c), a distinct peak in frequency of north winds occurred for all four quarterly periods. The 60 meter level, like the 10 meter level also displayed a peak in frequency of northeast winds particularly during the July-September period. This peak in northeast winds was not nearly as pronounced at the 122 meter level. The frequency of south and southeast winds was lowest during the period January-March and more pronounced during the remaining three quarterly periods. These figures also indicate a smaller third peak in the frequency of northwest winds which was most pronounced during the January-March period at all three tower levels. The relatively low frequency of south winds and the third peak in the frequency of northwest winds is likely to be the result of the stronger large scale (gradient) winds during the January-March period.

These wind characteristics for 1984 are generally consistent with wind observations collected during other years, with the most significant feature being the tendency for air flow along the axis of the valley. Differences in wind distributions that do occur between years can be attributed to year to year variability in the strength and movement of synoptic scale weather systems (cyclones and anticyclones).

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The 1984 data, as well as analysis of meteorological data from other years (see section 2.6.2), suggests that winds in the region are controlled primarily by topography. It appears that both terrain channeling and a thermally driven valley wind is contributing to the observed wind direction frequency distribution.

Terrain channeling occurs when surface air flow at some angle to the valley, is deflected by the elevated valley walls and forced to flow along the valley axis. Terrain channeling is dependent only on the orientation of the valley, and the strength and direction of large scale winds. The thermally driven valley winds are induced by differential heating between one region of the valley and an adjacent region with different topography. The differential heating induces an along valley pressure gradient which drives the up or down-valley wind. Up-valley winds are confined during the daytime when surface heating is occurring while down-valley winds are primarily nocturnal, when there is significant surface radiative cooling. Consequently, up-valley winds will occur during hours with unstable stability classes while down-valley winds are characteristic of hours when low level inversions are occurring and stability classes are stable. These up and down-valley winds are most prevalent during the summer and fall season when the large scale (gradient) winds are weakest. Under these conditions it is common to observe north or northeast winds during the night and early morning at Indian Point, with a shift to southerly winds occurring within a few hours of sunrise, when surface heating commences. Thus, diurnal variations in winds at Indian Point will have the highest frequency of occurrence during the summer and fall season.

The diurnal variation of the vector mean wind as measured 70-feet above river during September-October 1955 is shown in Figure 2.6-2 for conditions in which the large scale flow was virtually zero (12 days) and in Figure 2.6-3 for conditions in which the large scale flow (gradient wind) was less than 16 MPH (35 days). It may be seen that for these virtually stagnant prevailing wind conditions, there is a regular diurnal shift in wind direction and that the mean vector wind associated with the down-valley flow is on the order of 6 MPH.

A measure of the reliability of the diurnal shift in wind direction is shown in figure 2.6-4 where the steadiness of the wind (vector) mean speed over the mean scalar speed is shown as a function of time and the strength of the prevailing flow. Where the steadiness is close to one (an extraordinarily high value for meteorological wind systems in this latitude), the reliability of a given wind direction is very high, It may be seen that the down-valley nocturnal flow is extremely reliable from 20-08 hours while the up-valley flow is as reliable from about 14-16 hours under zero pressure gradient conditions. For weak pressure gradient conditions the nocturnal flow direction is very probable from 24 to 08 hours and thereafter becomes quite unreliable. In short, these data indicate that a consecutive 24 hours down-valley flow with light wind speeds and inversion conditions is extremely improbable.

Atmosphere Stability

Tables 2.6-1, 2.6-2 and 2.6-3 provide the wind direction percent frequency distribution as a function of stability at the 10 meter level of the 122 meter meteorological tower.(5) The Pasquill stability classes are based on vertical temperature gradients (0C/100 meters) and are the same as the NRC classification of atmospheric stability (6). Table 2.6-1 shows the joint frequency distribution for a one year period while Table 2.6-2 and 2.6-3 give distributions for the summer season (May – October) and winter season (November – April), respectively.

Inspection of tables 2.6-1, 2.6-2 and 2.6-3 show that stability Class E occurred with the highest frequency for all wind directions (total) both for the one year period and for the summer and

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winter seasons. For the one year period it occurred 37.17% of the time. Similar percent frequencies are shown for the summer and winter seasons. This stability class occurred most frequently during south southwest winds with a second peak in frequency occurring for northeast winds. The total percent frequencies show stability Class D occurs with the second highest frequency while Class G had the lowest frequency occurring only 1.69% of the time for the one year period. Again similar percent frequencies are indicated for the summer and winter seasons.

Joint Frequency Distributions

Tables 2.6-4 (sheets 1 through 28) provides recent joint frequencies of wind direction, wind speed and atmospheric stability for the quarterly periods in 1986. Sixteen wind directions, seven wind speed categories including calm winds and seven Pasquill stability classes (A-G) are used. The stability classes are determined from 61-10 meter vertical temperature difference (delta-T). Data recovery during 1986 was 99 percent (13 missing hours) during the April-June period and 100 percent for the remaining quarterly periods.

Thunderstorms

Thunderstorms, although not unique to the Indian Point Site, are important since they can produce wind and precipitation patterns in the Indian Point environment that have considerable spatial and temporal variability. An important characteristic of thunderstorms is a downdraft of relatively cold air which spreads radially outward at the earth's surface. This cold air outflow, commonly called a gust front, can at times travel significant distances from the immediate storm environment. A typical gust front will appear as a sharp change in wind speed and direction and a drop in ambient air temperature.

Figure 2.6-5 shows the mean annual distribution of days with thunderstorms for the northeast United States. (7) This map is based on data from the period 1952-1962. Figure 2.6-5 shows that in the vicinity of Indian Point an average of between 20 and 30 days per year will have thunderstorms. Most of these thunderstorm days will occur during the summer season.

2.6.2 Meteorological and Atmospheric Diffusion Studies at Indian Point

New York University under a contract with Consolidated Edison Company made extensive tests on the meteorological conditions at the Indian Pont site. The testing program started in 1955 and was completed in 1957. Site meteorology (wind direction, wind speed and vertical temperature gradient) and atmospheric diffusion characteristics as determined from this testing program are described in three technical reports prepared by the New York University staff under the immediate direction of Professor Benjamin Davidson. The original New York University reports, or applicable excerpts there from, which were reviewed by Professor Davidson and the Consolidated Edison staff, are provided on pages Q1-Q44 and R1-11. In addition, information on precipitation, the prevalent wind directions associated with precipitation, a table of wind directions during thunderstorms and associated wind roses are given on pages R12-R20.

Due to questions concerning the relevancy of certain meteorological data obtained in the 1956-1957 period a new meteorological monitoring program in the Hudson River Valley was initiated to try to verify the results of the old study. The locations of the meteorological towers erected for the new program did not correspond to the locations of the towers used in the earlier program, and the data were not reliable, due to instrumentation difficulties. The two sets of data

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were, therefore, difficult to compare although it was evident that no substantial change occurred in the valley meteorology from 1956-1969.

The experimental program was reorganized in the fall of 1969, and a new meteorological tower site was selected as close to the original 1956 one as was possible under current conditions. Wind observations were made at this 100-foot tower at Indian Point and at a ship anchored in the Hudson River northwest of Indian Point. The results of the program for the period 26 November 1969 to 1 October 1970 are presented in Dr. Halitsky's report NYU-TR-71-3 (see pages Y-1 to Y-32).

The conclusions, as stated in the report, are:

1) Annual average statistics of wind speed direction and vertical temperature differences were substantially the same for 1956 and 1970.

2) Average wind hodographs, as the ships exhibited the same diurnal reversal pattern and the same 2.5 m/sec nighttime down-valley speed in both years. The average wind hodograph at the tower showed a similar pattern of reversal, but the nighttime down-valley speed was about 2 m/sec.

3) All sixteen daily wind hodographs used for calculating the average hodograph at the tower showed the diurnal reversal and exhibited considerable variability in speed and direction from day to day through a complete cycle.

4) Maximum persistencies of low-speed inversion winds in the critical 005-020 sector were 2 hours, 4 hours and 3 hours for 1, 1.5 and 2 m/sec speeds, respectively, during the entire ten-month data record.

Additional data acquired from 1 January 1970 to 31 December 1971 is presented in NYU-TR-73-1 (see pages Z-1 to Z-82).

In addition to these meteorological studies, several diffusion studies pertaining to atmospheric diffusion modeling applied to the Indian Point Site were conducted. The final reports pertaining to these diffusion studies are given on pages 2.6.K-1 to 2.6.K-15, 2.6.L-1 to 2.6.L-67 and 2.6.M1 to 2.6.M-11.

2.6.3 Application of the Site Meteorology to the Safety Analysis of the Loss-of Coolant Accident

The atmospheric dispersion factors required for the safety analysis of Chapter 14 have been computed for the worst possible meteorological conditions which could prevail at the Indian Point site.

A search of the records indicates that the most protracted consecutive period during which the wind direction was substantially from the same directions was five days. The winds in this case were from the northwest and speeds ranged from 15 to 30 MPH. In view of the large wind speeds and slightly unstable to adiabatic range of thermal stability associated with this period of maximum wind direction duration, this case does not represent the most conservative meteorology associated with the Loss-of-Coolant Accident.

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The most frequent wind flow at low heights under inversion conditions is down the axis of the valley. This direction, roughly 010-030o, is also the direction of maximum wind frequency at the 100 ft. tower level. Because of the relatively high frequency of inversion conditions associated with this wind direction, the safety analysis assumed that the distribution of wind speed and thermal stability during the hypothetical accident is exactly that measured at the 100 ft. tower level for the 005-020o wind direction. The valley wind is diurnal in nature, i.e., up-valley during unstable hours and down-valley during stable hours.

The safety analysis of the Loss-of-Coolant Accident assumed that the accident occurs during down-valley inversion flow conditions and that these conditions persisted for 24 hours with average wind speeds slightly less than 2 m/sec. Figures 2.6-2 and 2.6-3 indicate that the duration of the down-valley flow is about 12 hours rather than 24 hours, and that the vector mean wind speeds are on the order of 2.5 m/sec.

In view of the discussion above, it must be concluded that the safety analysis for the first 24 hours was conservative to within a factor of about two.

The remainder of the safety analysis assumed that for the next 30 days, 35% of the winds are in the 20o sector corresponding to the nocturnal down-valley flow and that wind speed and thermal stability were as observed over the period of one year as measured at the 100 ft. tower location. If the observations were distributed uniformly throughout the year, slightly over 100 hours per month of 005-020o winds could be expected to occur. The analysis assumes 276 hours of 005-020o winds occur in the first 31 days after the accident, and that about 130 of these hours are characterized by inversion conditions. Approximately 35 weak pressure gradient days were observed in September-October 1955 or about 430 hours per month. From Figure 2.6-4, the hours during which the down-valley flow is quite reliable under weak pressure gradient conditions are from 00-08 hours. Assuming that the reliability is 1.0 during these hours (it is fact about 0.9 or less), the number of down-valley inversion winds per month during September and October is on the order of 140 hours per month. This indicates that the meteorology assumed in the safety analysis beyond the first 24 hours is about right for the worst months (September and October) and is undoubtedly conservative, with varying degrees of conservatism, for about ten months of the year.

The inversion frequency assumed for the 30-day accident case is conservative because the evaluation was made from joint assumptions concerning the postulated meteorological conditions viz.,

1) Inversion conditions prevail for 42.4% of the time

2) The wind direction is within a narrow 20o sector, for 35% of the time

This is equivalent to assuming that in the model 20o sector, the inversion frequency is 14.8 percent for the 30-day period. The observed annual maximum inversion frequency for a 20o sector is 6.2% (p.29, Table 3-3, NYU Tech. Report 372.3, Section 1.6). If we assume that the inversion frequency is spread uniformly throughout the year, almost three months worth of inversion in the model 20o sector are considered to occur in the first 31-day month after the accident. The assumptions of uniform spread of inversion frequency over the year are examined above, where an attempt was made to isolate those local meteorological conditions at Indian Point which might yield the highest 30-day dose. It is concluded that the "worst"

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meteorological conditions are associated with the nocturnal down-valley flow which is most frequent during September and October.

2.6.4 Conservatism of Indian Point Site Meteorology with Respect to Calculation of Off-Site Doses

The conservatism of the site meteorology was evaluated with respect to wake dilution factors, Pasquill categories for stability classification and site shaping characteristics.

Building wake dilution factors, documented in reports by Dr. Halitsky titled "An Analysis of the Con Edison and AEC-DRL Wake Diffusion Models as Applied to the Indian Point Site", (see pages 2.6.K-1 to 2.6-K-15) and "An Analysis of the Con Edison and AEC-DRL Accident meteorology models as Applied to the Indian Point Site", (see pages 2.6.L-1 to 2.6.L-67) demonstrate that limiting the building wake dispersion correction factor to a value of 3, as required by AEC Safety Guide No. 4, is overly conservative. Both the Con Edison wake model and the Safety Guide model, without limiting the building wake dispersion correction factor, are realistically conservative when compared to actual field and wind tunnel measurements. The reports also evaluate the overall conservatism of the Con Edison accident diffusion model. Specific investigations of the turbulence characteristics and wind persistence for the site are presented.

In addition, these two reports show that the classification of atmospheric stability using the criteria documented in Safety Guide No. 23 is not appropriate for the Indian Point site. The significance of the valley influence in generating lateral dispersion, and meandering of the wind, create horizontal standard deviations of greater magnitude than those determined by using vertical temperature gradients. The data indicate Pasquill categories measured under inversion conditions with horizontal wind fluctuations similar to a Pasquill D category while, vertically, Pasquill categories are E, F, or G.

Pickard, Lowe and Garrick of Washington D.C., in the report, "A Study of Atmospheric Diffusion Condition Probabilities using the Composite Year of Indian Point Site Weather Data" (see pages 2.6.M-1 to 2.6.M-11), illustrate the effects of the site shaping technique for estimating the 95% confidence level of the annual average dispersion coefficient at the exclusion area envelope. In addition, the report shows the effect of using the "split sigma" model to account for the lateral wind meander observed in the valley.

The composite year of measured meteorological data was compiled in a form compatible with AEC Safety Guide No 23 in sheets 8 to 14 of Table 2.6-5. In order to conform with the sensor heights specified in Safety Guide No. 23 the measured ΔT was multiplied by a ΔT correction factor. The method used to determine this factor assumes an exponential relationship between temperature and height, such that measured temperature difference between any two heights can be represented as a temperature difference between two other heights, according to the following relationship:

$$\Delta T \text{ correction factor} = 1n\,(h_{ue}/h_{le})/\ 1n\,({}^{h}um/{}^{h}lm)$$

where:

h_{ue} =	height of upper extrapolated temperature (ft)
h_{le} =	height of lower extrapolated temperature (ft)

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h_{um} = height of upper measured temperature (ft)

h_{lm} = height of lower measured temperature (ft)

Sheets 1 to 7 of Table 2.6-5 show data normalized to the sensor heights specified in Safety Guide No. 23.

The composite year reflects those periods in which data recovery was the greatest. The composite year consists of January through July of 1970, August 1971, September and October 1972 and November and December of 1970.

Incorporation of the aforementioned characteristics unique to the Indian Point valley site into diffusion calculations, insure that off-site doses following a loss-of-coolant accident are within the limitations outlined in 10 CFR 100.

2.6.5 Onsite Meteorological Measurements Program

The meteorological measurement program consists of three instrumented towers, redundant power and ventilation systems, redundant communication systems, and a mini-computer processor/recorder. The meteorological measurement program complies with the acceptance criteria stated in Section 2.3.3. and in Section 17.2 of NUREG-75/087 Revision 1 (superseded by NUREG-0800, Rev. 2, July 1981) with the former section dealing with meteorological sensors and recorders, and the latter dealing with the Quality Assurance Program. The meteorological measurements program consists of primary and backup systems. The accuracy of the meteorological sensor and recording systems meet the system specifications given in the Section C.4 of proposed Revision 1 to Regulatory Guide 1.23.

Primary System

A 122-meter, instrumented tower is located on the site and provides:

1. Wind direction and speed measurement at a minimum of two levels, one of which is representative of the 10-meter level:
2. Standard deviations of wind direction fluctuations as calculated at all measured levels;
3. Vertical temperature difference for two layers (122-10 meters and 60-10 meters);
4. Ambient temperature measurements at the 10-meter level;
5. Precipitation measurements near ground level;
6. Pasquill stability classes as calculated from temperature difference.

To assure acceptable date recovery, the meteorological measurements system and associated controlled environmental housing is connected to a power supply system which has a redundant power source. A diesel generator has been installed to provide immediate power to the

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meteorological tower system in the event of a power outage. The generator becomes fully powered within 15 seconds after an automatic transfer switch is tripped. Various support systems include an uninterruptible power supply, dedicated ventilation systems, halon fire protection, and dedicated communications.

The meteorological data is transmitted simultaneously to two data loggers located at the Primary Tower site. One data logger transmits 15-minute average meteorological data to a computer to determine joint frequency distributions, and the second data logger transmits 15-minute average meteorological data to a computer located in the Buchanan Service Center, which provides the capability for accessing the meteorological data remotely.

Meteorological data can be transmitted simultaneously to the IP3 / IP2 emergency response organization and the NRC in a format designated by NUREG-0654/FEMA-REP-1.

Fifteen minute averages of meteorological parameters covering the 12-hour period previous to a recall command is available upon interrogation of the system.

Backup Systems

In the event of a failure of the primary meteorological measurement system, a backup meteorological system is used at the Indian Point site. This system is independent of the primary system and consists of an instrumented meteorological tower (a backup tower located approximately 2700 feet north of the primary tower). The associated data acquisition system for the backup tower is located in the Emergency Operations Facility. The backup system provides measurements of the 10-meter level of wind direction and speed, and an estimate of atmospheric stability (Pasquill category using sigma theta which is a standard deviation of wind fluctuation). The backup system provides information in the real-time mode. Changeover from the primary system to the backup system occurs automatically. In the event of a failure of the backup meteorological measurement system, a standby backup system exists at the 10-meter level of the Buchanan Service Center building roof. It also provides measurements of the 10-meter level of wind direction and speed, and an estimate of atmospheric stability (Pasquill category using sigma theta which is a standard deviation of wind fluctuations). The changeover from the backup system to the standby system also occurs automatically.

As in the case of the primary system, the backup meteorological measurements system and associated controlled environmental housing system is connected to a power system which is supplied form redundant power sources.

In addition to the backup meteorological measurements system, a backup communications line to the meteorological system is operational. During an interim period, the backup communications is provided via telephone lines routed through a telephone company central office separate from the primary circuits.

Atmospheric Dispersion Factors for Routine Releases

Extensive analyses and calculations were carried out in 1991[8] to reevaluate the atmospheric dispersion factors for routine releases at Indian Point. The primary objective was to ensure the applicability of the dispersion factors in the Offsite Dose Calculation Manual (ODCM) in view of potential changes in the meteorological conditions at the site.

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In the analyses, consideration was given to an extended meteorological database and up-to-date analytical models. Briefly, use was made of the following:

1. 10 years' worth of hourly meteorological data collected on site for the period 1981 through 1990.

2. Mixed-mode releases, whereby released plumes can be totally elevated, totally at ground level, or in between,

3. Valley-flow considerations for the assessment of channeling and recirculation effects (based on a site specific study[9]), and

4. Analytical models which permit the computation of radiation exposures due to inhalation and due to immersion in finite clouds of radioactive material.

The locations of interest were the site boundary, the nearest residences in each sector, and milking animals at 5 miles. See Ref. 8 for complete details and results.

References

1. Kaplan, Edward J. and B. Wuebber 1984(a), Quarterly Summary of Meteorological Data from Indian Point Meteorological Systems, First Quarter, January 1 – March 31, 1984, Prepared for the New York Power Authority, May 1984, Project No. 01-4251-02-1.

2. Kaplin, Edward J. and B. Wuebber, 1984(b), Quarterly Summary of Meteorological Data from Indian Point Meteorological Systems, Second Quarter, April 1 – June 30, 1984. Prepared for the New York Power Authority, September 1984, Project No. 01-4251-02-1.

3. Kaplin, Edward J. and B. Wuebber, 1984(c), Quarterly Summary of Meteorological Data from Indian Point Meteorological Systems, Third Quarter, July 1 – September 30, 1984. Prepared for the New York Power Authority, December 1984, Project No. 01-4251-02-1.

4. Kaplin, Edward J. and B. Wuebber, 1985, Quarterly Summary of Meteorological Data from Indian Point Meteorological Systems, Fourth Quarter, October 1 – December 31, 1984, prepared for the New York Power Authority, June 1985, Project No. 01-4251-05-1.

5. Kaplin, Edwar J., B. Wuebber (1981) Facility Safety Analysis Report (FSAR), consolidated Edison Company of New York, Inc., Indian Point Nuclear Generating Unit No. 2, Meteorological update, September 1981, YSC Project No. 01-4122.

6. Nuclear Regulatory Commission, 1980, Proposed Revision 1 to Regulatory Guide 1.23, Meteorological Programs in Support of Nuclear Power Plants. U.S. Nuclear Regulatory Commission, Washington D.C.

7. Bryson Reid A. and F. K. Hare, 1974, World Survey of Climatology, volume 11, Climatology, volume 11, Climate of North America, Helmut Landsbert, Editor in Chief.

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8. NYPA Corporate Radiological Engineering Calculation IP3-CALC-RAD-00001, "IP3 – Revised ODCM Atmospheric Dispersion Parameters (Multi-Year Hourly Data, Mixed-Mode Releases and Valley Effects" (10/1191)

9. Kaplin, Edwar J., "Wind Field Analysis at Indian Point," York Services Corporation, Stamford, CT, Technical Report No. 4873-02 (3/19/91)

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Table 2.6-1

ANNUAL SUMMARY OF WIND DIRECTION PERCENT FREQUENCY DISTRIBUTION
AS A FUNCTION OF STABILITY - 10M LEVEL
(JANUARY 1, 1979 - DECEMBER 31, 1980)

Wind Direction	Stablility Class A	B	C	D	E	F	G
N	1.28	0.36	0.48	3.39	2.67	0.50	0.09
NNE	1.76	0.40	0.46	3.15	3.33	0.80	0.17
NE	0.63	0.35	0.58	4.22	4.66	2.12	0.40
ENE	0.06	0.07	0.17	1.59	2.61	1.84	0.43
E	0.01	0.03	0.03	0.64	1.49	0.59	0.11
ESE	0.01	0.01	0.01	0.27	0.73	0.21	0.04
SE	0.03	0.01	0.02	0.23	0.67	0.26	0.02
SSE	0.09	0.03	0.04	0.45	1.04	0.31	0.05
S	2.04	0.25	0.29	1.74	3.39	0.76	0.11
SSW	2.58	0.51	0.38	2.14	5.04	0.72	0.05
SW	1.16	0.33	0.35	1.89	3.03	0.51	0.03
WSW	0.49	0.17	0.16	0.96	1.44	0.39	0.02
W	0.56	0.22	0.17	1.40	1.64	0.43	0.06
WNW	0.47	0.15	0.26	1.64	1.49	0.21	0.03
NW	0.70	0.31	0.32	2.36	1.85	0.10	0.01
NNW	0.80	0.40	0.49	3.26	1.60	0.17	0.04
CALM	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MISSING	0.12	0.05	0.03	0.21	0.51	0.15	0.02
TOTAL %	12.80	3.66	4.23	29.56	37.17	10.08	1.69
NO. OF HOURS	2244	641	742	5183	6519	1768	297

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Table 2.6-2
[Historical Information]

SUMMARY OF WIND DIRECTION PERCENT FREQUENCY
DISTRIBUTION AS A FUNCTION OF STABILITY
SUMMER SEASON - 10M LEVEL
(MAY 1, 1979, 80 - OCTOBER 31, 1979, 80)

Wind Direction	Stability Class A	B	C	D	E	F	G
N	1.68	0.26	0.37	1.25	2.06	0.57	0.07
NNE	2.65	0.42	0.43	2.90	2.41	1.01	0.18
NE	0.58	0.31	0.46	3.46	4.44	3.17	0.35
ENE	0.11	0.10	0.24	1.38	2.66	2.62	0.39
E	0.02	0.07	0.01	0.57	1.57	0.61	0.05
ESE	0.01	0.01	0.00	0.31	1.01	0.36	0.06
SE	0.05	0.02	0.01	0.17	0.84	0.40	0.02
SSE	0.15	0.06	0.05	0.50	1.07	0.40	0.08
S	3.32	0.36	0.43	2.47	3.58	0.85	0.05
SSW	4.10	0.75	0.59	2.93	5.70	0.85	0.01
SW	1.84	0.49	0.48	2.23	3.03	0.51	0.05
WSW	0.87	0.20	0.18	0.94	1.05	0.34	0.00
W	0.88	0.28	0.19	1.38	1.42	0.34	0.07
WNW	0.80	0.09	0.25	0.94	1.03	0.15	0.05
NW	1.05	0.19	0.17	0.84	0.63	0.10	0.02
NNW	0.78	0.19	0.24	0.97	0.74	0.20	0.02
CALM	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MISSING	0.22	0.06	0.01	0.31	0.68	0.22	0.03
TOTAL %	19.11	3.86	4.11	23.54	33.92	12.69	1.48
NO. OF HOURS	1687	341	363	2078	2994	1120	131

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Table 2.6-3

SUMMARY OF WIND DIRECTION PERCENT FREQUENCY DISTRIBUTION AS A FUNCTION OF STABILITY WINTER SEASON - 10M LEVEL (NOVEMBER 1, 1979,80 - APRIL 30, 1979,80) [Hisotrical Information]

Table 2.6-4
(Sheet 1 of 28)

JOINT FREQUENCY DISTRIBUTION
INDIAN POINT JAN-MAR 1986
10 METER WIND SPEED & DIR. WITH 61-10 METER DELTA T
PASQUILL CLASS A

WIND DIRECTION	WIND SPEED (MPH)					
	01-03	04-07	08-12	13-18	19-24	>24	TOTAL
N	7.	49.	20.	0.	0.	0.	76.
NNE	3.	7.	0.	0.	0.	0.	10.
NE	7.	2.	0.	0.	0.	0.	9.
ENE	1.	0.	0.	0.	0.	0.	1.
E	2.	0.	0.	0.	0.	0.	2.
ESE	2.	1.	0.	0.	0.	0.	3.
SE	6.	4.	0.	0.	0.	0.	10.
SSE	12.	21.	0.	0.	0.	0.	33.
S	8.	18.	6.	0.	0.	0.	32.
SSW	7.	11.	7.	0.	0.	0.	25.
SW	2.	2.	1.	0.	0.	0.	5.
WSW	0.	0.	0.	0.	0.	0.	0.
W	4.	6.	5.	1.	0	0.	16.
WNW	0.	29.	13.	0.	0.	0.	42.
NW	4.	35.	16.	0.	0.	0.	55.
NNW	12.	33.	7.	0.	0.	0.	52.
TOTAL	77.	218.	75.	1.	0.	0.	371.
CALM	0.						

Table 2.6-4
(Sheet 2 of 28)

JOINT FREQUENCY DISTRIBUTION
INDIAN POINT JAN-MAR 1986
10 METER WIND SPEED & DIR. WITH 61-10 METER DELTA T
PASQUILL CLASS B

WIND
DIRECTION WIND SPEED (MPH)

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