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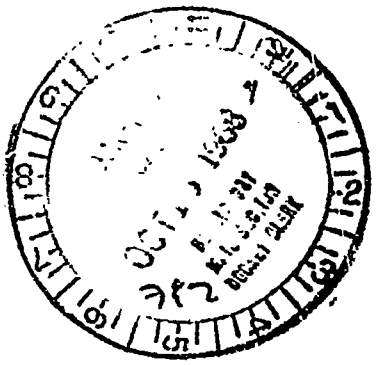
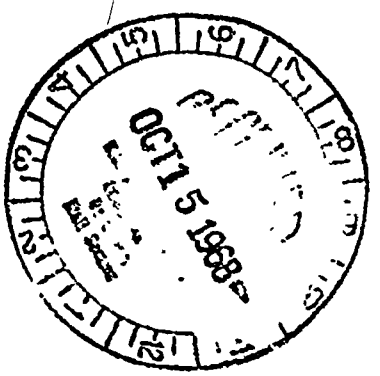


CONSOLIDATED EDISON COMPANY OF NEW YORK, INC.

INDIAN POINT NUCLEAR GENERATING UNIT NO. 2

FINAL FACILITY DESCRIPTION AND SAFETY ANALYSIS REPORT



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Information in this record was deleted in accordance with the Freedom of Information Act.
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November 2007

This version of the Indian Point Unit 2 Final Facility Description and Safety Analysis Report (FSAR) is the licensee's original version, issued in October 1968, through Supplement 15, dated November 12, 1970. Certain redactions of sensitive information have been made by staff of the Nuclear Regulatory Commission (NRC) to allow release to the public. The redactions are made under 10 CFR 2.390(d)(1). The material included within is classified as publicly available information.

The redactions were made due to meeting the NRC's criteria on sensitive information, as specified in SECY-04-0191, "Withholding Sensitive Unclassified Information Concerning Nuclear Power Reactors from Public Disclosure", dated October 19, 2004, ADAMS ML042310663, as modified by the NRC Commissioners Staff Requirements Memorandum on SECY-04-0191, dated November 9, 2004, ADAMS ML043140175.

The following information was redacted by NRC staff:
Figures 1.2-5, 1.2-6, 1.2-7, 1.2-8, 1.2-9
Figures 5.1-2, 5.1-3, 5.1-4, 5.1-5, 5.1-6
Figures 6.2-2, 6.2-3
Figure 7.7-1
Figures 10-7, 10-7a
Figure 11.2-3
Figure 5-16 of Appendix 14a
Figure Q9.5-1

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AMEND. # 20---- SUPPL. # 11 TO FSAR - TRANSMITTAL OF PROP. TECH. SPEC.

AMEND. # 21---- TRANS. OF 1969 ANNUAL RPT.

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1. INTRODUCTION AND SUMMARY

1.1 INTRODUCTION

This final Safety Analysis Report is submitted in support of an application by Consolidated Edison Co. for a license to operate the Indian Point Unit #2 nuclear power plant (AEC Docket 50-247 Permit #CPPR-21). It provides pertinent technical information in accordance with Section 50.34 of 10 CFR 50 requirements to obtain a nuclear power plant's operating license.

Westinghouse Electric Co. is the primary contractor and has turnkey responsibility for the design, construction, testing and initial start-up of the facility. Westinghouse has contracted with United Engineers and Constructors as architect-engineer to provide engineering assistance in the design of and construction of the structural and civil works.

Upon successful completion of construction, preoperational tests, and initial start-up Consolidated Edison will assume responsibility for the operation of the plant on a commercial basis.

The following paragraphs contain a summary of the report's scope:

The unit employs a pressurized water reactor nuclear steam supply system furnished by Westinghouse Electric Corporation. It is similar in design to several projects now in operation or currently under review by the U.S. Atomic Energy Commission.

The reactor thermal power objective is 2758 MWt which corresponds to a turbine-generator output of 873 MW(e) net. The license application is for this power rating. Calculations indicate that hot channel factors will be considerably less than those used for design purpose in this application.

The plant heat removal systems have been designed for the turbine guarantee rating of 3083 MWt and the portions of the safety analysis dependent on heat removal capacity of plant and safeguards systems have assumed the higher the maximum calculated power of 3216 MWt as have the evaluations of activity release and radiation exposure.

The remainder of Section I of this report summarizes the principal design features and parameters of the plant, pointing out the similarities and differences with respect to other pressurized water nuclear power plants presently in operation. A general description of the plant is included as well as a statement and summary of all General Design Criteria.

Section 2 contains a description and evaluation of the site and environs, supporting the suitability of the site for a reactor of the size and type described. Section 3 and 4 describe and evaluate the reactor and the Reactor Coolant System; Section 5, the Containment System, Section 6, the Engineered Safety Features; Section 7, Plant Instrumentation and Control; Section 8, the Electrical System; Section 9, the Auxiliary and Emergency System; Section 10, the Steam and Power Conversion System; Section 11, Radioactive Wastes and Radiation Protection. Sections 12 and 13 are Conduct of Operations and Initial Test and Operation respectively. They describe plant organization, training programs, and start-up administrative procedure. Section 14 is a safety evaluation summarizing the analysis which demonstrates the adequacy of the reactor protection system and the containment and engineered safety features, and which shows that the consequences of various postulated accidents are within the guidelines set forth in 10CFR100. The design criteria for structures and equipment are summarized in Appendix A. Appendix B provides a description of the quality assurance organization. A report on the containment liner design and analysis is included in Appendix C.

There have been a number of design modifications incorporated into this plant since submission of the preliminary report and issuance of the construction permit. The more significant are as follows:

1. Increased capacity of emergency core cooling.
2. Deletion of the reactor pit crucible.
3. Selection of sodium hydroxide rather than sodium thiosulfate as the additive to the containment spray for purpose of fission product removal.
4. Addition of burnable poison rods into the first core loading.
5. Addition of part length control rods for power distribution control.
6. Valving and piping modifications in the emergency core cooling system to give added assurance of core and containment cooling in the very unlikely event of a passive component failure long term cooling following a loss of coolant accident.
7. Addition of a boron injection tank into the safety injection path.

The increased capacity of the emergency core cooling system results from the addition of a pressurized accumulator to each coolant loop which provide rapid core reflooding capability with borated water after a major loss of coolant accident. As a result of the increased cooling system capacity, clad melting is effectively prevented for rupture sizes up to and including the double-ended severance of a main reactor coolant pipe. The detailed analysis of such breaks is shown is Section 14.3.3.

In the prior design of the emergency core cooling system core reflooding following a loss of coolant accident was accomplished by three high head and two high flow safety injection pumps and by the two high flow residual heat removal pumps. The reflooding rates with this design was not sufficient to prevent the fuel clad temperature on the highest power fuel rods from rising to the clad melting temperature, hypothetically assuming instantaneous severance of a coolant loop. Further, the additional pumping capacities and emergency power requirements necessary to provide reflooding times that would not result in clad melting for a loop severance were prohibitively large. Because of this and the uncertainties involved in demonstrating that the fuel pellets released from the melted clad could not fall to the bottom of the reactor vessel and melt through the reactor vessel a provision was proposed for containing the melted fuel in this unlikely event. This provision was

a refractory lined crucible to be located directly beneath the reactor vessel. Extensive research and development efforts were initiated in the areas of; 1) designing a core reflood system that would limit clad temperature to below melting, and 2) the design of a crucible that could contain the molten core. Because of the success in developing a highly improved core reflooding system and the continuing uncertainties associated with the behavior and containment of molten fuel, more stringent core cooling criteria were adopted to preclude fuel clad melting and prevent significant clad water reaction and hence insure the preservation of the core heat transfer geometry. The increased capability of the emergency core cooling system to meet these new criteria are reflected in the design by the inclusion of four pressurized accumulator tanks containing a large volume of borated water held back from the reactor system by check valves which open (without requiring a signal) to discharge into the reactor coolant system when the system pressure decrease associated with a loss of coolant falls below their discharge pressure. These four accumulator tanks supplement the two high flow safety injection pumps. The rapid water discharge from these accumulators greatly reduces the core reflooding time thereby supplying earlier core cooling and limiting the clad temperature increase to a value well below the melting temperature. As a measure of effectiveness of the accumulators, the core midplane reflood time after a loop severance is less than 35 seconds with the revised design as compared to about 300 seconds with the initial design assuming one high head and one low head pump ineffective in both cases. This direct approach of reducing the core reflood times and retaining the core intact eliminates the problem of containing the fuel pellets and the possibility of core migration and thus the need for the reactor pit crucible associated with the slower reflooding rates provided by the initial emergency core cooling system design. Hence, the reactor pit crucible has been deleted from the plant design details of the design of the revised emergency core cooling system including the accumulators are presented in Section 6.2. A complete analysis of the capability of the revised emergency core cooling system to accomodate the loss of coolant accidents, including supporting basis, assumptions and results which show that the new emergency core cooling system design meets the revised criteria is included in Section 14.3.

Further research and development work by Westinghouse and the industry on spray additives for iodine removal has resulted in the conclusion that sodium hydroxide solution is more suitable for the accident application than is sodium thiosulfate.

Burnable poison rods have been added to the first core cycle to reduce the coolant boron concentration such that the moderator temperature coefficient of reactivity will be negative at power operation conditions. This modification lessens the severity of a rod ejection or loss of coolant accident and improves the stability of the core power distribution. In core cycles after the first no burnable poison rods are needed to maintain the moderator coefficient negative.

Further study on the subject of spatial Xenon oscillations indicates that oscillations are to be expected in the axial direction. Part length absorber rods are provided to give the operator means to control the axial power distribution.

The valving and piping modifications in the emergency core cooling system give capability to maintain core cooling and containment cooling in the event of a passive component failure in the safety injection system or service water system long term after a loss of coolant. The design also has sufficient component redundancy to accomodate an active component failure.

The boron injection tank modification consists of an addition to the safety injection system which causes an initial charge of high concentration boric acid to be injected into the Reactor Coolant System on activation of the safety injection system. The system gives increased emergency shutdown capability for this condition by supplementing the poison insertion from the control rods. This greatly limits the time the reactor could be critical following a hypothetical steam break accident.

1.2 SUMMARY PLANT DESCRIPTION

1.2.1 SITE

Indian Point Unit No. 2 is being built adjacent to and north of Unit No. 1 on a site of approximately 239 acres of land on the east bank of the Hudson River at Indian Point, Village of Buchanan in upper Westchester County, New York. The proposed Indian Point Unit No. 3 will be adjacent to and south of Unit No. 1. The site is about 24 miles north of the New York City boundary line. The nearest city is Peekskill, 2.5 miles northeast of Indian Point, with a population of about 19,000. An aerial photograph, Figure 2.2-1, shows the site and about 58 square miles of the surrounding area.

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Meteorology

Meteorological conditions in the area of the site were determined during a two-year test program. These data have been used in evaluating the effects of gaseous discharges from the plant during normal operations and during the postulated loss-of-coolant accident. In addition, data supplied by the U. S. Weather Bureau at the Bear Mountain Station, regarding the meteorological conditions during periods of precipitation, have been used to evaluate the rainout of fission gases into surface water reservoirs following the postulated loss-of-coolant accident. The evaluations indicate that the site meteorology provides adequate diffusion and dilution of any released gases.

Geology and Hydrology

Geologically, the site consists of a hard limestone in a jointed condition which provides a solid bed for the plant foundation. The bedrock is sufficiently sound to support any loads which could be anticipated up to 50 tons per square foot, which is far in excess of any load which may be imposed by the plant. Although it is hard, the jointed limestone formation

is permeable to water. Thus, if water from the plant should enter the ground (an improbable event since the plant is designed to preclude any leakage into the ground) it would percolate to the river rather than enter any ground water supply. Additional studies by Consolidated Edison's geology consultant, Thomas W. Fluhr, and examination of recent soil borings confirm the above conclusions.

In the Hudson River, about 80,000,000 gallons of water flow past the plant each minute during the peak tidal flow. This flow provides additional mixing and dilution for liquid discharges from the facility. In fact, however, this aspect is superfluous since the assumption in the plant design is to treat the river water as if it were used for drinking (which it is not) and thus to reduce radioactive discharges, by dilution with ordinary plant effluent, to concentrations that would be tolerable for drinking water. There is no danger of flooding at the site as discussed in Section 2.5.

Seismology

Seismic activity in the Indian Point area is rare and no damage has resulted therefrom. As stated by Applicant's consultant on seismology, the site is "practically non-seismic" and is "as safe as any area at present known." Notwithstanding such assurance, the plant is designed to withstand an earthquake of the highest intensity which can reasonably be predicted from geologic and seismic evidence developed for the site.

Environmental Radiation Monitoring

Environmental radioactivity has been measured at the site and surrounding area for nearly ten years in association with the operation of Indian Point Unit No. 1. These measurements will be continued and reported. The radiation measurements of fallout, water samples, vegetation, marine life etc. have shown no significant post-operative increase in activity. Noticeable increases in fallout have coincided with weapons testing programs and appear

to be related almost entirely to those programs. The New York State Department of Health in an independent two-year post-operative study, Section 2.9, found that environmental radioactivity in the vicinity of the site is no higher than anywhere else in the state of New York.

Conclusions

Consideration of all the items mentioned above, plus the containment design and the engineered safety features included in the plant design lead to the conclusion of appropriate suitability of the site for safe operation of the Indian Point Unit #2 nuclear power plant. Accident analysis presented in Section 14 verify that the maximum expected doses at or beyond the site boundary are well within the 10 CFR 100 suggested guidelines.

1.2.2 PLANT DESCRIPTION

The unit incorporates a closed-cycle pressurized water nuclear steam supply system, a turbine-generator and their necessary auxiliaries. A radioactive waste disposal system, fuel handling system and all auxiliaries, structures, and other on-site facilities required for a complete and operable nuclear power plant are provided for the unit.

The general arrangement of the plant is shown on Figures 1.1-1 through 1.1-4.

NUCLEAR STEAM SUPPLY SYSTEM

The nuclear steam supply systems consist of a pressurized water reactor, Reactor Coolant System, and associated auxiliary fluid systems. The Reactor Coolant System is arranged as four closed reactor coolant loops, containing a reactor coolant pump and a steam generator, connected in parallel to the reactor vessel. An electrically heated pressurizer is connected to one of the loops.

The reactor core is composed of uranium dioxide pellets enclosed in Zircaloy tubes with welded end plugs. The tubes are supported in assemblies by a spring clip grid structure. The mechanical control rods consist of clusters of stainless steel clad absorber rods and guide tubes located within the fuel assembly. The core is initially loaded in three regions of different enrichments with new fuel being introduced into the outer region at successive refuelings, moved, and discharged to spent fuel storage.

The steam generators are vertical U-tube units employing Inconel tubes. Integral separating equipment reduces the moisture content of the steam leaving the steam generators to 1/4 per cent or less.

The reactor coolant pumps are vertical, single stage, centrifugal pumps equipped with controlled leakage shaft seals.

Auxiliary systems are provided to perform the following functions:

a) Charge the reactor coolant system
b) Add makeup water
c) Purify reactor coolant water
d) Provide chemicals for corrosion inhibition and reactor control
e) Cool system components
f) Remove residual heat when the reactor is shutdown
g) Cool the spent fuel storage pool
h) Sample reactor coolant water
i) Provide for emergency core cooling
j) Collect reactor coolant drains
k) Provide containment spray
l) Provide containment ventilation and cooling
m) Dispose of liquid, gaseous and solid wastes.

Reactor and Plant Control

The reactor is controlled by a corrdinated combination of chemical shim and mechanical control rods. The control system allows the unit to accept step load changes of ± 10% and ramp load changes of ± 5% per minute over the load range of 15% to, but not exceeding, 100% power under nominal operating conditions subject to xenon limitations.

Turbine and Auxiliaries

The turbine is a tandem-compound, comprising one high pressure and three low pressure cylinders, 1800 rpm unit having 44 inch exhaust blading in the low pressure cylinders. Six combination moisture separator-reheater units are employed to dry and super heat the steam between the high and low pressure turbine cylinders. The turbine-generator is capable of a 50% loss of external electrical load without turbine or reactor trip. The turbine auxiliaries include deaerating surface condensers, steam jet air ejector, turbine driven main feed pumps, motor driven condensate pumps, and six stages of feedwater heating.

The turbine generator has a guaranteed capability of 1,021,793 kw at 1.5 in Hg absolute exhaust pressure with zero per cent make up and six stages of feedwater heating.

Electrical System

The main generator feeds electrical power through an isolated phase bus to two half-sized main power transformers. Station auxiliaries receive power during normal operation from auxiliary transformer connected to the isolated phase bus.

The Auxiliary Electrical System provides power to those auxiliary components required to operate during any plant's normal or emergency condition of operation.

Standby power required for plant start-up and after reactor trip is supplied from the Consolidated Edison to 138 kv system from the Buchanan substation.

Emergency power supply for vital instruments and controls is from two 125 d.c. station batteries.

The system design provides sufficient independence, isolation capability, and redundancy between the different power sources to avoid complete loss of auxiliary power.

Control Room

The plant is provided with a reactor and turbine-generator control room containing all necessary instrumentation for plant's operation under normal and accident conditions.

Adequate shielding and air conditioning facilities permit occupancy during all operating or accident conditions.

Diesel Generators

Three diesel generator sets supply emergency for shutdown or essential safeguards operation power in the event of a loss of all other a.c. auxiliary power.

Waste Disposal System

The Waste Disposal System collects and processes liquids, gaseous and solid waste from plant operation for removal from plant site. All removals are made in accordance with government guidelines for the process. It also provides hydrogen and nitrogen to the primary system as required by the plant's normal operating condition.

Fuel Handling System

The fuel handling system provides the ability to fuel and refuel the reactor core. Administrative procedures carefully established plus the design of the sytem minimizes the probability of potential fission product release during the refueling operation.

The system also includes the following features:

Safe accessibility for operating personnel. Provisions to prevent fuel storage criticality. Visual monitoring of the refueling procedures at all times.

Engineered Safety Features

The Engineered Safety Features for this plant have sufficient redundancy of component and power sources such that under the conditions of a hypothetical loss-of-coolant, the system can, even when operating with partial effectiveness, maintain the integrity of the containment and keep the exposure of the public below the limits of 10 CFR 100.

In addition to the pressurizer penetrations and seal water system the following systems are also part of the ESS.

The major engineered safeguards systems are as follows:

a) The Containment System which incorporates continuously pressurized and monitored penetrations and liner weld channels and a seal water injection system which provides a highly reliable, essentially leak-tight barrier against the escape of radioactivity which might be released to the containment atmosphere.

b) The Safety Injection System (which constitutes the Emergency Core Cooling System) which provides borated water to cool the core in the event of a loss of coolant accident.

c) The Containment Air Recirculation Cooling and Filtration System which provides a heat sink to cool the containment atmosphere and provides filtration of the containment atmosphere to remove airborne particulate and halogen fission products which form the source for potential public exposure.

d) The Containment Spray System which provides a spray of cool, chemically treated borated water to the containment atmosphere to provide backup heat sink and iodine removal capability to the Containment Air Recirculation Cooling and Filtration System.

Structures

The major structures are the reactor containment building, the auxiliary building, the control building, and the turbine building. General layouts of the reactor containment interior components arrangement are shown on Figures 5.1-2 through 5.1-7. General layouts and interior components arrangement of the auxiliary building, control building, spent fuel pit building, and holdup tank building are shown on Figures 1.2-5 through 1.2-9. Figure 10-7 shows the general arrangement of the turbine building.

Containment

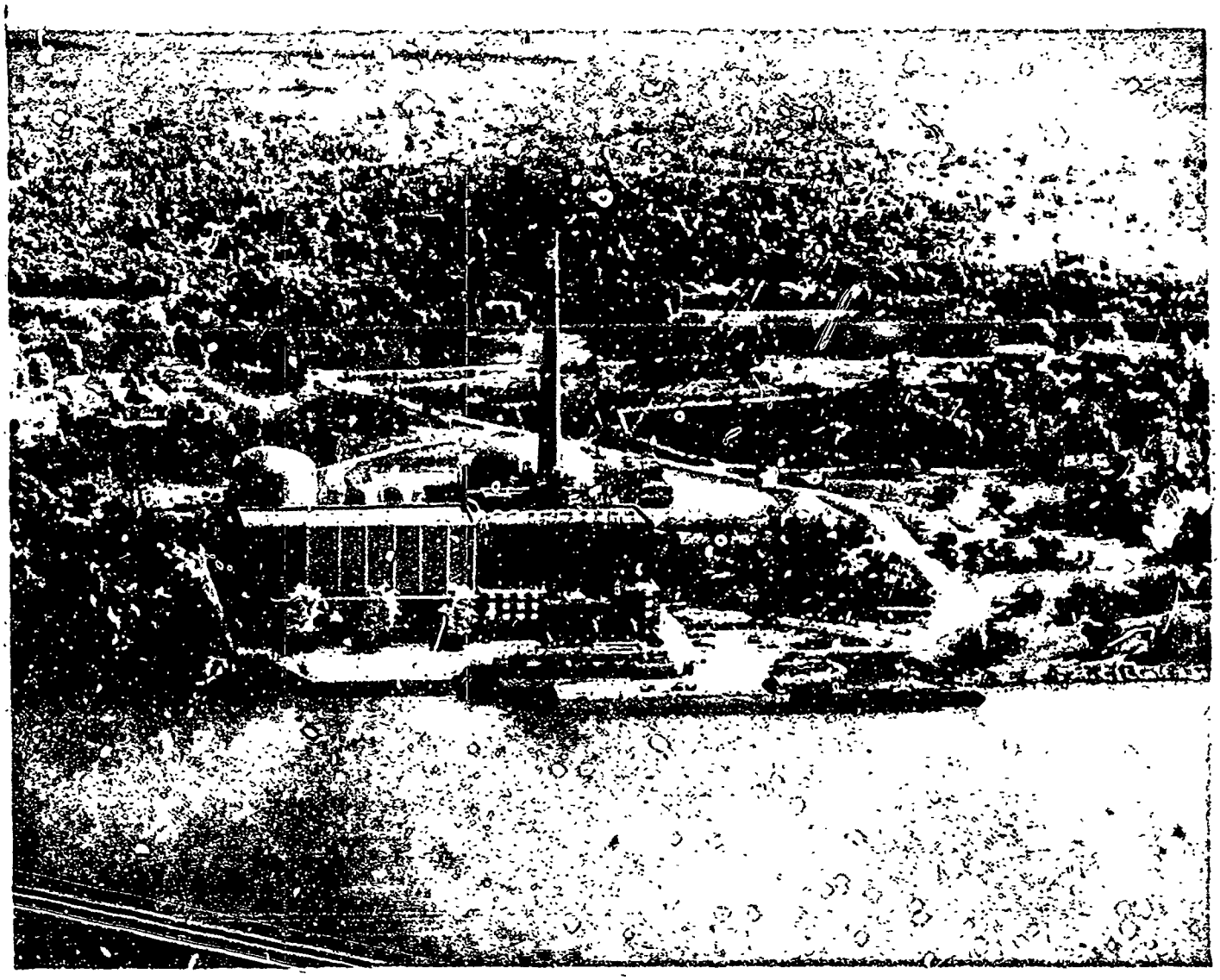
The reactor containment is a steel lined reinforced concrete cylinder with a hemispherical dome and a flat base. The containment is designed to withstand the internal pressure accompanying a loss-of-coolant accident, is virtually leaktight and provides adequate radiation shielding for both normal operation and accident conditions.

Continuously pressurized double containment is provided at all liner seams and penetrations including access opening and ventilation ducts.

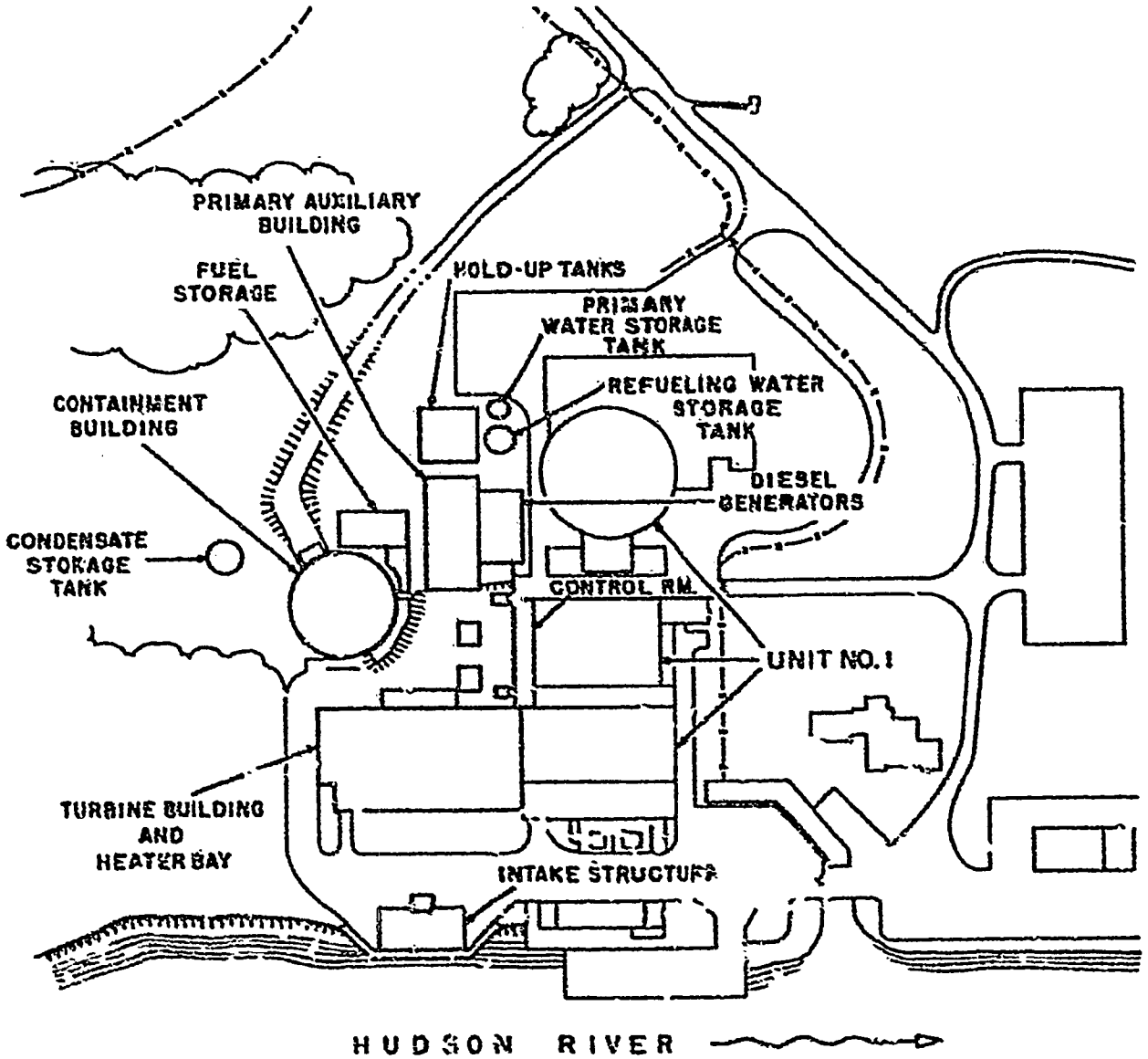
When required, the containment isolation valve seal water system permits automatic rapid sealing of pipes which penetrate the containment so that in the event of any loss-of-coolant accident, there will be no leakage from containment to the environment.

Ground acceleration for design earthquake used for containment design purposes and all Seismic Class I structures (Appendix A) are .1g applied horizontally and .05g applied vertically. In addition ground accelerations for the maximum hypothetical earthquake of .15g horizontal and .10g vertical are used to analyze the no loss-of-function concept.

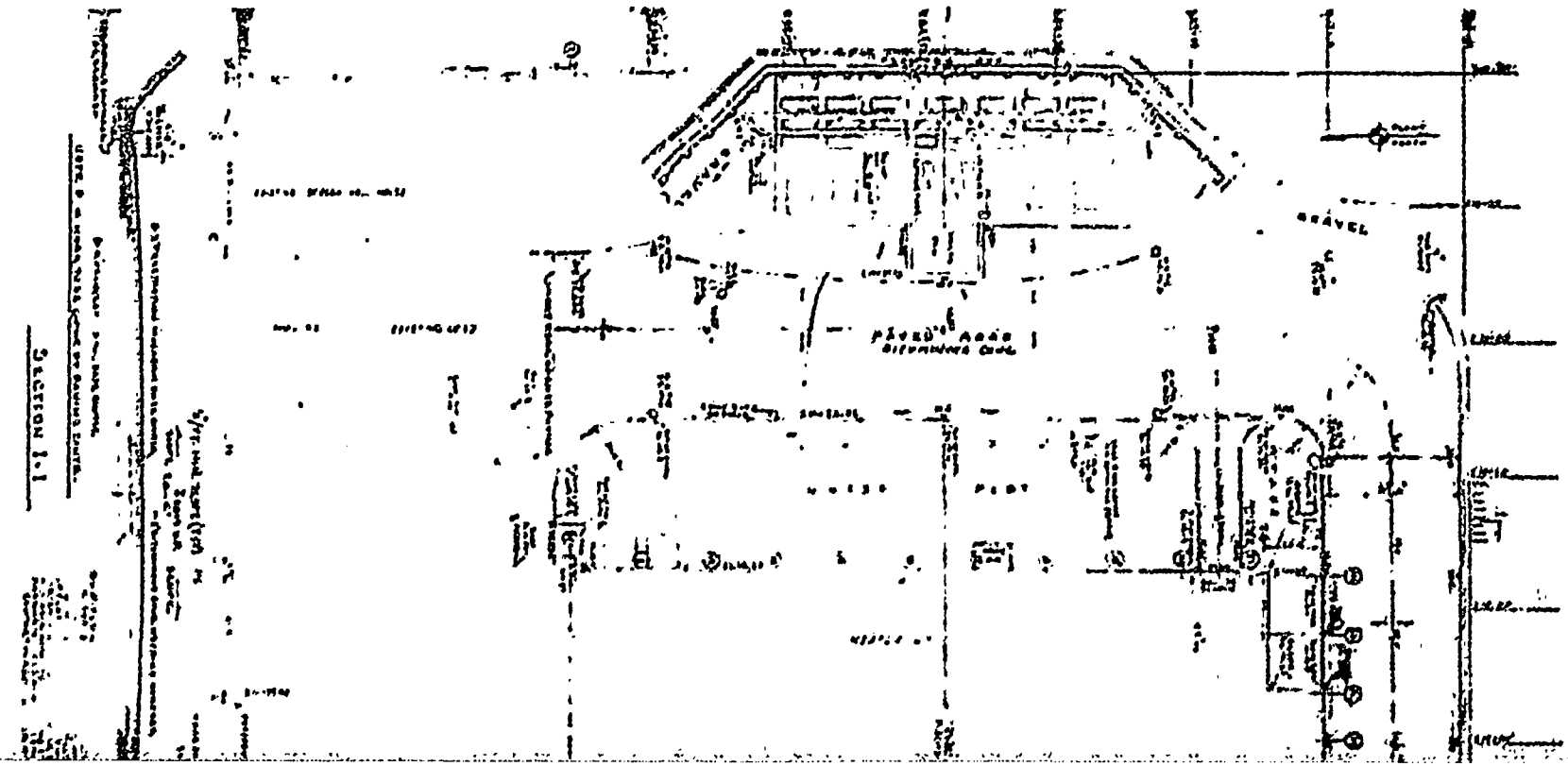
The primary auxiliary building (except for the steel superstructure) is a Seismic Class I structure. The spent fuel pit is a Seismic Class I Structure. The control building is a Seismic Class I structure. The turbine building is a Seismic Class III structure.

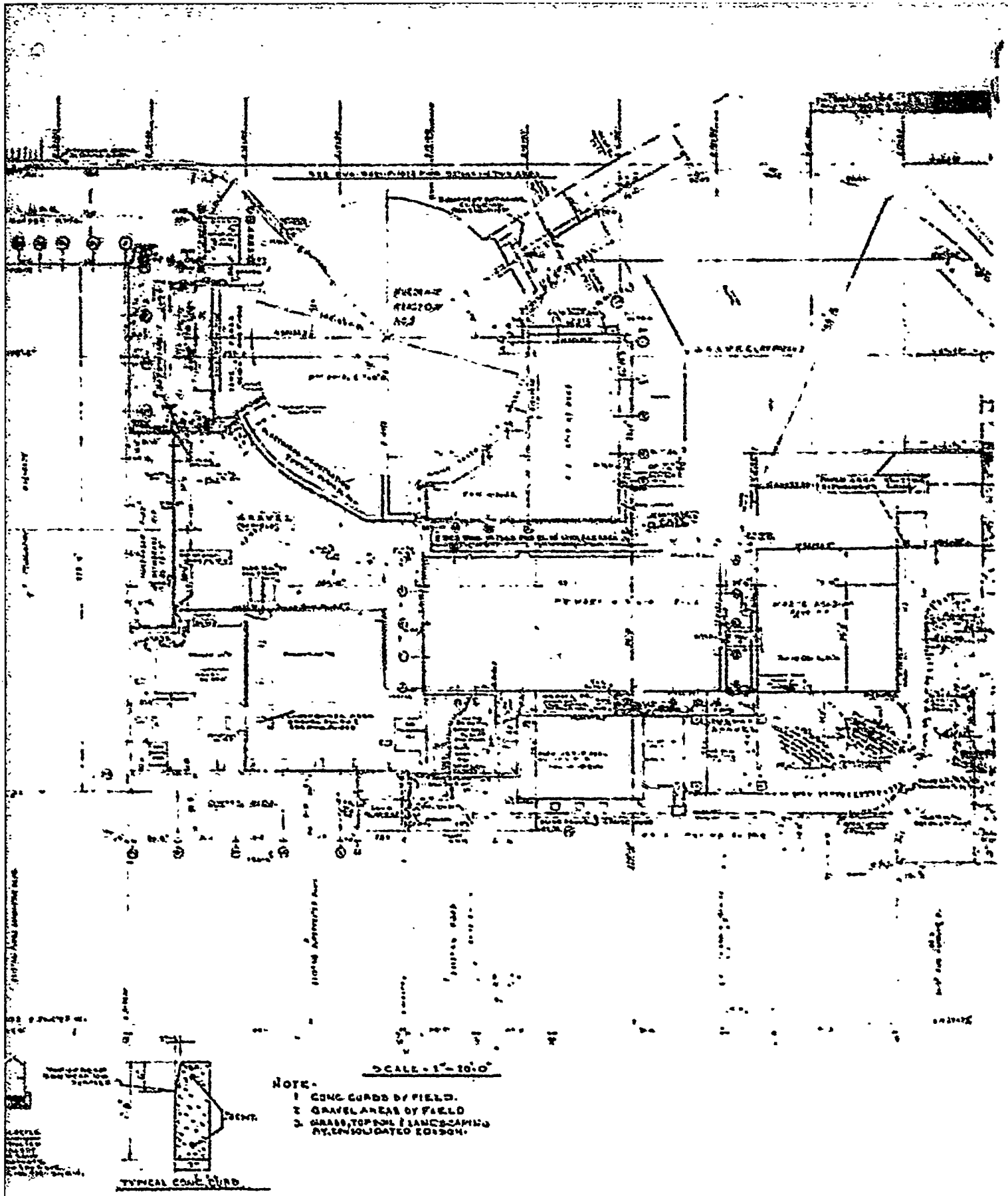


Indian Point Nuclear Generating Units No. 1 and No. 2
Figure 1.2-1

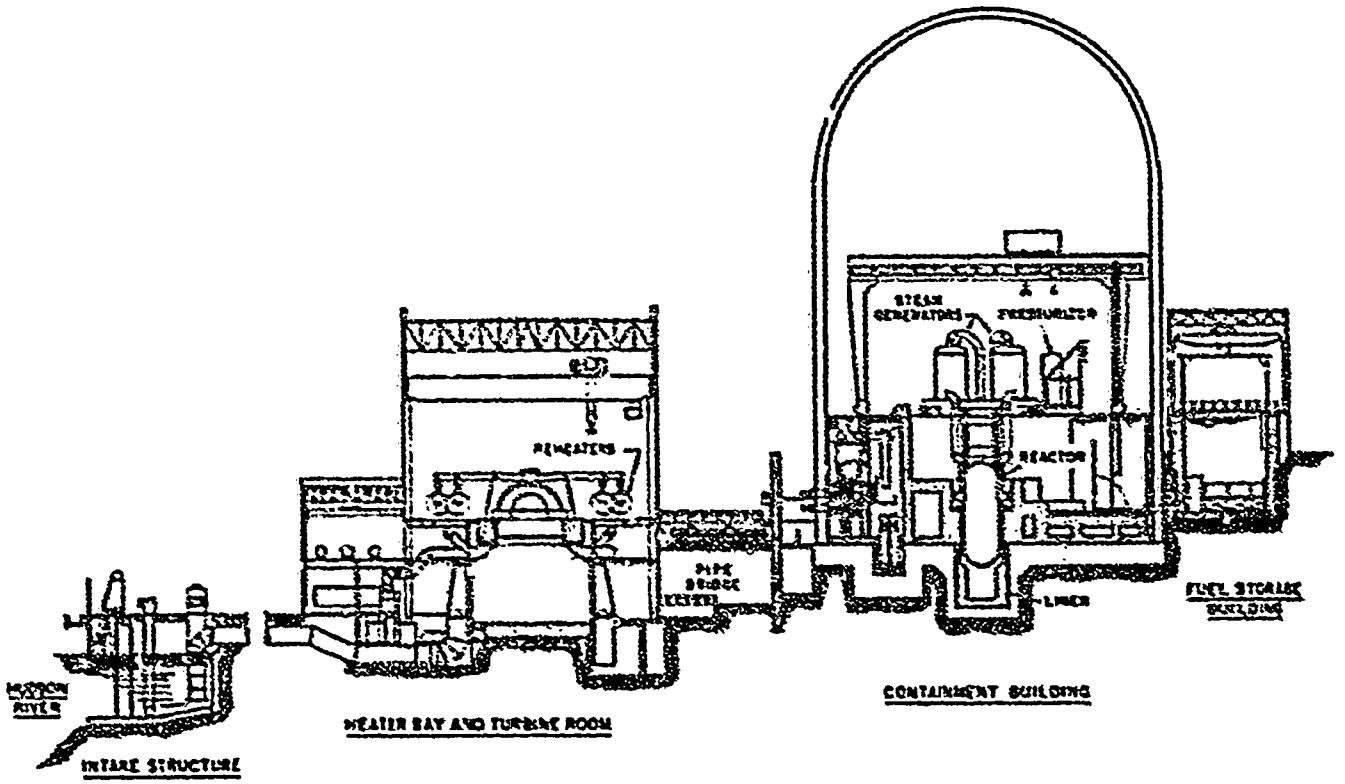


General Plot Plan
Figure 1.2-2


SECTION 1-1
GRAVEL



Site Plot Plan
Figure 1.2-1



Cross Section of Plant
Figure 1.2-4

Ex 4

Figure 1.2-5, Titled "Primary Auxiliary Building General Arrangement Plans"

The above was **redacted** by NRC staff as **sensitive information** to allow release of this document to the public.

E4

Figure 1.2-6, Titled "Primary Auxiliary Building General Arrangement Elevation"

The above was **redacted** by NRC staff as **sensitive information** to allow release of this document to the public.

Figure 1.2-7, Titled "Control Building General Plan" Ex 4

The above was **redacted** by NRC staff as **sensitive information** to allow release of this document to the public.

Figure 1.2-8, Titled "Spent Fuel Pit General Arrangement" 24

The above was **redacted** by NRC staff as **sensitive information** to allow release of this document to the public.

Figure 1.2-9, Titled "Holdup Tank Building General Arrangement" Ex 4

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1.3 GENERAL DESIGN CRITERIA

The general design criteria define or describe safety objectives and approaches incorporated in the design of this plant. These general design criteria, tabulated explicity in the pertinent systems section in this report, comprise the proposed Atomic Industrial Forum versions of the criteria issued for comment by the AEC on July 10, 1967. The remainder of this section, 1.3, presents a brief description of related plant features which are provided to meet the design objectives reflected in the criteria. The description is developed more fully in those succeeding sections of the report indicated by the references.

The parenthetical numbers following the section headings indicate the numbers of its related proposed General Design Criterion (GDC).

1.3.1 OVERALL PLANT REQUIREMENTS (GDC 1-GDC 5)

All systems and components of the facility are classified according to their importance. Those items vital to safe shutdown and isolation of the reactor or whose failure might cause or increase the severity of a loss-of-coolant accident or result in an uncontrolled release of excessive amounts of radioactivity are designated Class I. Those items important to reactor operation but not essential to safe shutdown and isolation of the reactor or control of the release of substantial amounts of radioactivity are designated Class II. Those items not related to reactor operation or safety are designated Class III.

Class I systems and components are essential to the protection of the health and safety of the public. Consequently, they are designed, fabricated, inspected and erected and the materials selected to the applicable provisions of recognized codes, good nuclear practice and to quality standards that reflect their importance.

All systems and components designated Class I are designed so that there is no loss of function in the event of the maximum potential ground acceleration acting in the horizontal and vertical directions simultaneously. The working stresses for both Class I and Class II items are kept within code allowable values for the design earthquake. Similarly, measures are taken in the plant design to protect against high winds, sudden barometric pressure changes, flooding, and other natural phenomena.

Reference sections:

Section Title	Section
Site and Environment; Meterology, Seismology	2.6, 2.8
Reactor Coolant System; Design Bases	4.1
Containment System Structure; Design Bases	5.1
Electrical System; Design Bases	8.1
Design Criteria for Structures and Equipment	Appendix A

Fire prevention in all areas of the nuclear-electric plant is provided by structure and component design which maximizes the use of fire-resistant materials, optimizes the containment of combustible materials and maintains exposed combustible materials below their ignition temperature in the design atmosphere. Fixed or portable fire fighting equipment are provided of capacities proportional to the energy that might credibly be released by fire.

The Fire Protection System provided has the design capability to extinguish any probable combination of simultaneous fires which might occur at the station. The system is designed in accordance with the standards of the National Fire Protection Association.

Reference section:

Section Title	Section
Operating Control Station	7.7
Facility Services	9.6

A complete set of as-built facility plant and system diagrams including arrangements plans and structural plans and records of initial tests and operation are maintained throughout the life of the reactor A set of all the quality assurance data generated during fabrication and erection of the essential components of the plant, as defined by the quality assurance program, is retained.

Reference section:

Section Title	Section
Records	12.4
Initial Tests and Operation	13
Quality Assurance Organization	Appendix B

1.3.2 PROTECTION BY MULTIPLE FISSION PRODUCT BARRIERS (GDC 6 - GDC 10)

The reactor core, with its related control and protection system, is designed to function throughout its design lifetime without exceeding acceptable fuel damage limits. The core design, together with reliable process and decay heat removal systems, provides for this capability under all expected conditions of normal operation with appropriate margins for uncertainties and anticipated transient situations.

The reactor control and protection instrumentation is designed to actuate a reactor trip for any anticipated combination of plant conditions, when necessary to ensure a minimum Departure from Nuclear Boiling (DNB) ratio equal to or greater than 1.30 and fuel center temperature below the melting point of UO_2.

Reference sections:

Section Title	Section
Reactor, Design Basis, Nuclear Design	3.1, 3.2
Instrumentation and Control, Protective Systems	7.2
Safety Analysis	14

The design of the reactor core and related protection systems ensures that power oscillations which could cause fuel damage in excess of acceptable limits are not possible or can be readily suppressed.

Low frequency spatial xenon oscillations may occur in the axial dimension, and part length control rods are provided to supress these oscillations. The core is expected to be stable to xenon oscillations in the X-Y dimension. Out of core instrumentation is provided to monitor any xenon induced oscillations. In core instrumentation is used to periodically calibrate and verify the information provided by the out of core instrumentation.

The moderator temperature and overall power coefficient in the power operating range is maintained negative by inclusion of burnable poison shims in the first core loading. The overall power coefficient in the power operating range is always negative.

Reference section:

Section Title	Section
Reactor Design	3.1, 3.2

The Reactor Coolant System in conjunction with its control and protective provisions is designed to accommodate the system pressures and temperatures attained under all expected modes of plant operation or anticipated system interactions, and maintain the stresses within applicable code stress limits.

The materials of construction of the pressure retaining boundary of the Reactor Coolant System are protected by control of coolant chemistry from corrosion phenomena which might otherwise reduce the system structural integrity during its service lifetime.

System conditions resulting from anticipated transients or malfunctions are monitored and appropriate action is automatically initiated to maintain the required cooling capability and to limit system conditions so that continued safe operation is possible.

The system is protected from overpressure by means of pressure relieving devices, as required by Section III of the ASME Boiler and Pressure Vessel Code.

Isolable sections of the system are provided with overpressure relieving devices discharging to closed systems such that the system code allowable relief pressure within the protected section is not exceeded.

Reference sections:

Section Title	Section
Design Basis, Reactor Coolant System	4.1

The design pressure of the containment exceeds the peak pressure occurring as the result of the complete blowdown of the reactor coolant through any pipe rupture of the Reactor Coolant System up to and including the hypothetical severance of a reactor coolant pipe.

All piping systems which penetrate the vapor barrier are anchored so that the pentration is stronger than the piping system and that the vapor barrier will not be breeched due to a hypothesized pipe rupture. The lines connected to the Primary Coolant System that penetrate the vapor barrier are anchored in the secondary shield walls and are each provided with at least one valve between the anchor and the coolant system. These anchors are designed to withstand the thrust moment and torque resulting from a hypothesized rupture of the attached pipe.

All isolation valves are supported to withstand, without impairment of valve operability, the combined loadings of the design basis accident and design seismic conditions.

Reference section:

Section Title	Section
Containment System Structure	5.1

1.3.3 NUCLEAR AND RADIATION CONTROLS (GDC 11 - GDC 18)

The plant is equipped with a control room which contains all controls, and instrumentation and facilities necessary for continuous operation of the reactor and turbine generator under normal and accident conditions.

Sufficient shielding, distance, and containment integrity are provided to assure that control room personnel shall not be subject to doses under postulated accident conditions during occupancy of the control room which, in the aggregate would exceed the suggested limits of 10 CFR 100.

Instrumentation and controls essential to avoid undue risk to the health and safety of the public are provided to monitor and maintain neutron flux, primary coolant pressure, flow rate, temperature, and control rod positions within prescribed operating ranges.

The non-nuclear regulating, process and containment instrumentation measures temperatures, pressure, flow, and levels in the Reactor Coolant System, Steam Systems, Containment and other Auxiliary Systems. The quantity and types of process instrumentation provided ensures safe and orderly operation of all systems and processes over the full operating range of the plant.

The operational status of the reactor is monitored from the control room. When the reactor is subcritical the neutron source multiplication is continuously monitored and indicated by proportional counters located in instrument wells in the primary shield adjacent to the reactor vessel. The source detector channels are checked prior to operations in which criticality may be approached by the use of an in-core source. Any appreciable increase in the neutron source multiplication, including that caused by the maximum physical boron dilution rate, is slow enough to give ample time to start corrective action (boron dilution stop and/or emergency boron injection) to prevent the core from becoming critical.

When the reactor is critical, means for showing the relative reactivity status of the reactor is provided by display of all rod bank positions in the control room. Periodic samples of the coolant boron concentration are taken, the variation of which provides a further check on the reactivity status of the reactor including core depletion during life.

Instrumentation and controls provided for the protective systems are designed to trip the reactor when necessary to prevent or limit fission product release from the core; to limit energy release; to signal closure of containment isolation valves; and to control the operation of engineered safety features equipment.

During reactor operation in the startup and power modes, redundant safety limit signals will automatically actuate two reactor trip breakers which are in series with the rod drive mechanism coils. This action would interrupt power and initiate reactor trip.

Reference section:

Section Title	Section
Instrumentation and Controls	7.1, 7.2, 7.4, 7.7

The Reactor Protection System Receives, from plant instrumentation, signals which are indicative of an approach to an unsafe operating condition, actuates alarms, prevents control rod motion, initiates load cutback, and/or opens the reactor trip breakers, depending on the severity of the condition.

The basic reactor tripping philosophy is to define a region of power and coolant temperature conditions allowed by the primary tripping functions. The overpower high ΔT trip, the over-temperature high ΔT trip and the nuclear

overpower trip. The allowable operating region within these trip settings is shown to prevent any combination of power, temperatures and pressure which would result in DNB with all reactor coolant pumps in operation. Additional tripping functions such as a high pressurizer pressure trip, low pressurizer pressure trip, high pressurizer level trip, loss of flow trip, steam and feedwater flow mismatch trip, steam generator low-low level trip, turbine trip, safety injection trip, source and intermediate range level trips, and manual trip are provided to back up the primary tripping functions for specific accident conditions and mechanical failures.

Overpower and over-temperature rod stops are provided to prevent abnormal power conditions which could result from excessive control rod withdrawal initiated by a malfunction of the Reactor Control System or by operator violation or administrative procedures.

Reference sections:

Section Title	Section
Engineered Safety Features	6.2
Reactor Protection System	7.2

Positive indications in the Control Room of leakage of coolant from the Reactor Coolant System to the containment are provided by equipment which permits continuous monitoring of containment air activity and humidity, and of runoff from the condensate collecting pans under the cooling coils of the containment air recirculation units. The basic design criterion is the detection of deviations from normal containment environmental condition including air particulate activity, radiogas activity, humidity, condensate runoff and in addition, in the case of gross leakage, the liquid inventory in the process systems and containment sump.

The containment atmopshere, the plant vent, the containment fan-coolers service water discharge, the Waste Disposal System liquid effluent and the component cooling loop are monitored for radioactivity concentration during all normal operations, anticipated transients, and accident conditions.

For the case of leakage from the reactor containment under accident conditions the plant area radiation monitoring system supplemented by portable survey equipment to be kept in the control room provides adequate monitoring of accident releases.

Monitoring and alarm instrumentation is provided for fuel and waste storage and handling areas to detect excessive radiation levels. The permanent record of activity releases is provided by radiochemical analysis of known quantities of waste.

A controlled ventilation system removes gaseous radioactivity from the atmosphere and fuel storage and waste treating areas of the auxiliary building and discharges it to the atmosphere via the plant vent. Radiation monitors are in continuous service in these areas to actuate high-activity alarms on the control board annunciator.

Reference sections:

Section Title	Section
Engineered Safety Features	6.2, 6.5
Auxiliary Coolant System	9.3
Radiation Protection	11.2

1.3.4 RELIABILITY AND TESTABILITY OF PROTECTION SYSTEMS (GDC 19 GDC 26)

Upon a loss of power to the magnetic-type control rod drive mechanisms, the rod cluster control assembly is released and falls by gravity into the core. The reactor internals, fuel assemblies, RCC assemblies and

drive system components are designed as Class I equipment. The RCC assemblies are fully guided through the fuel assembly and for the maximum travel of the control rod into the guide tube. Furthermore, the RCC assemblies are never fully withdrawn from their guide thimbles in the fuel assembly. Due to this and the flexibility designed into the RCC assemblies, abnormal loadings and misalignments can be sustained without impairing operation of the RCC assemblies.

All reactor protection channels are supplied with sufficient redundancy to provide the capability for channel calibration and test at power. Bypass removal of one trip circuit is accomplished by placing that circuit in a half-tripped mode; i.e., a two-out-of-three circuit becomes a one-out-of-two circuit. Testing does not trip the system unless a trip condition exists in a concurrent channel. The trip signal furnished by the two remaining channels would be unimpaired in this event.

Two reactor trip breakers are provided to interrupt power to the rod drive mechanisms. The breaker main contacts are connected in series with the power supply to the mechanism coils. Opening either breaker interrupts power to all full length rod mechanisms. Each breaker is opened through an undervoltage trip coil. Each protection channel actuates two separate trip logic trains, one for each reactor trip breaker undervoltage trip coil. The protection system is thus inherently safe in the event of a loss of rod control power.

Channel independence is carried throughout the system extending from the sensor to the relay providing the logic. In most cases, the safety and control functions when combined are combined only at the sensor. A failure in the control circuitry does not affect the safety channel. This approach is used for pressurizer pressure and water level channels, steam generator water level, T_{avg} and ΔT channels, steam flow-feedwater flow and nuclear power range channels.

The power supplied to the channels is fed from four instrument buses. Two of the buses are supplied both by constant voltage transformers and two are supplied by inverters.

The initiation of the engineered safety features provided for loss-of-coolant accidents, e.g., high head safety injection and residual heat removal pumps, and containment spray systems, is accomplished from redundant signals derived from reactor coolant system and containment instrumentation. The initiation signal for containment spray comes from coincidence of two sets of two-out-of-three high containment pressure signals. On loss of voltage of the safety features equipment bus, the diesel generator will be automatically started and connected to the bus.

The containment isolation valves trip signals are derived from either a coincidence of 2/3 containment high pressure signals or a safety injection signal.

Redundancy is provided in that there are three diesel generator sets capable of supplying separate 480 volt buses. The minimum complement of safety features equipment is supplied from any two of the three diesel generators.

The components of the protection system are designed and laid out so that the mechanical and thermal environment accompanying any emergency situation in which the components are required to function does not interfere with that function.

Each protection channel in service at power is capable of being calibrated and tripped independently by simulated signals for test purposes to verify its operation.

The ability of the diesel-generator sets to start within the prescribed time and to carry load can periodically be checked.

The generator breaker is not closed automatically after starting during this testing. The generator may be manually synchronized to the 480 volt bus for loading. Blocking the closing of the generator breaker causes the bus tie breaker to close after a time delay sufficient for normal diesel generator starting.

An open circuit or loss of reactor trip channel power causes the system to go into its trip mode. In a two-out-of-three circuit, the three channels are equipped with separate primary sensors and each channel is energized from independent electrical buses.

The signal for containment isolation is developed from a two-out-of-three circuit in which each channel is separate and independent and which signals for containment isolation upon loss of power. The failure of any channel to de-energize when required does not interfere with the proper functioning of the isolation circuit.

Diesel engine cranking is accomplished by a stored energy system supplied solely for the associated diesel-generator. The undervoltage relay scheme is designed so that loss of 480 volt power does not prevent the relay scheme from functioning properly.

Reference sections:

Section Title	Section
Instrumentation and Control; Protection Systems	7.2
Electrical System	8

1.3.5 REACTIVITY CONTROL (GDC 27-GDC 32)

Reactivity control is achieved by two independent systems, the rod cluster control assemblies (RCCA) and the Chemical and Volume Control System which regulates the concentration of boric acid solution neutron absorber in

the Reactor Coolant System. The system is designed to prevent, under anticipated system malfunction, uncontrolled or inadvertent reactivity changes which might stress the system beyond design limits.

The Reactivity Control Systems provided are capable of making and holding the core subcritical from any hot standby or hot operating condition, including those resulting from power changes. The maximum excess reactivity expected for the core occurs for the cold, clean condition at the beginning of life.

The full length and part length Rod Cluster Control (RCC) assemblies are divided into three categories comprising control banks, shutdown banks, and a part length rod bank. The control banks of RCC assemblies are used to compensate for short term reactivity changes at power produced due to variations in reactor power or in coolant temperature. The chemical shim control is used to compensate for the more slowly occurring changes in reactivity throughout core life such as those due to fuel depletion and fission product buildup and decay.

The shutdown banks are provided to supplement the control banks of RCC assemblies to make the reactor at least one per cent subcritical ($k_{eff} = 0.99$) following trip from any credible operating condition to the hot, zero power condition assuming the most reactive RCC assembly remains in the fully withdrawn position.

Boron injection from the Safety Injection System actuation supplements rod insertion and prevents return to critical in the event of the maximum credible steam break, namely opening of a safety valve. This is accomplished with maximum worth rod fully withdrawn.

Any time that the plant is at power, the quantity of boric acid retained in the boric acid tanks and ready for injection will always exceed that quantity required for the normal cold shutdown.

The boric acid solution is transferred from the boric acid tanks into the reactor coolant by the boric acid transfer pumps and charging pumps which can be operated from diesel generator power on loss of primary power. Boric acid can be injected by one of the three charging pumps at a rate sufficient to shut the reactor down from full power with no rods inserted in less than sixteen minutes. In sixteen additional minutes, enough boric acid can be injected by one pump to compensate for xenon decay although xenon decay below the equilibrium operating level will not begin until approximately 15 hours after shutdown. Additional boric acid is employed if it is desired to bring the reactor to cold shutdown conditions.

The Reactor Protection Systems will limit reactivity transients to DNBR $\geq$ 1.30 due to any single malfunction in the reactor coolant deboration controls.

Limits, which include considerable margin, are placed on the maximum reactivity worth of control rods by limiting position of insertion as a function of power and on rates at which reactivity can be increased to ensure that the potential effects of a sudden or large change of reactivity cannot (a) rupture the reactor coolant pressure boundary or (b) disrupt the core, its support structures, or other vessel internals so as to lose capability to cool the core.

The rod cluster drive mechanisms are wired into preselected groups, and are prevented from being withdrawn in other than their respective groups. The rod drive mechanism is of the magnetic latch type and the coil actuation is sequenced to provide variable speed rod travel. The maximum reactivity

insertion rate is analyzed in the detailed plant analysis assuming two of the highest worth banks to be accidentally withdrawn at maximum speed, yielding reactivity insertion rates of the order of 8×10^{-4} $\Delta k/sec$ which is well within the capability of the overpower-over temperature protection circuits to prevent core damage.

Reference sections:

Section Title	Section
Reactor Design Bases	3.1
Protection Systems	7.2
Regulating Systems	7.3
Chemical and Volume Control System	9.2

1.3.6 REACTOR COOLANT PRESSURE BOUNDARY (GDC 33-GDC 36)

The reactor coolant boundary is shown to be capable of accommodating without further rupture, the static and dynamic loads imposed as a result of a sudden reactivity insertion such as a rod ejection.

The operation of the reactor is such that the severity of an ejection accident is inherently limited. Since control rod clusters are used to control load variations only and core depletion is followed with boron dilution, only the rod cluster control assemblies in the controlling groups are inserted in the core at power, and at full power these rods are only partially inserted. A rod insertion limit monitor is provided as an administrative aid to the operator to ensure that this condition is met.

By using the flexibility in the selection of control rod groupings, radial locations and position as a function of load, the design limits the maximum fuel temperature for the highest worth ejected rod to a value which precludes any resultant damage to the primary system pressure boundary.

The failure of a rod mechanism housing causing a rod cluster to be rapidly ejected from the core is evaluated as a theoretical, though not a credible, accident. While limited fuel damage could result from this hypothetical event, the fission products are confined to the Reactor Coolant System and the reactor containment.

In the core region of the reactor vessel the notch toughness of the material will change during operation as a result of fast neutron exposure, which results in a shift in the Nil Ductility Transition Temperature. This is factored into the operating procedures in such a manner that full operating pressure is not obtained until the affected vessel material is above the increased Design Transition Temperature and in the ductile material region. The pressure during startup and shutdown at the temperature below NDTT is maintained below the threshold of concern for safe operation.

The DTT is a minimum of NDT temperature plus 60°F and dictates the procedures to be followed in the hydrostatic test and in station operations to avoid excessive cold stress. The value of the DTT is increased during the life of the plant as required by the expected shift in the NDT temperature, and as confirmed by the experimental data obtained from irradiated specimens of reactor vessel materials during the plant lifetime.

The design of the reactor vessel and its arrangement in the system provides the capability for accessibility during service life to the entire internal surfaces of the vessel and the external zones of the vessel including the

nozzle to reactor coolant piping welds and the top and bottom heads. The reactor arrangement within the containment provides sufficient space for inspection of the external surfaces of the reactor coolant piping, except for the area of pipe within the primary shielding concrete.

Monitoring of the Nil Ductility Transition Temperature properties of the core region plate forgings, weldments and associated heat treated zones are performed in accordance with ASTM E185 (Recommended Practice for Surveillance Tests on Structural Materials in Nuclear Reactors). Samples of reactor vessel plate materials are retained and catalogued in case future engineering development shows the need for further testing.

The material properties surveillance program includes not only the conventional tensile and impact tests, but also fracture mechanics specimens. The observed shifts in NDTT of the core region materials with irradiation will be used to confirm the calculated limits to startup and shutdown transients.

To define permissible operating conditions below DTT, a pressure range is established which is bounded by a lower limit for pump operation and an upper limit which satisfies reactor vessel stress criteria. Since the normal operating temperature of the reactor vessel is well above the maximum expected DTT, brittle fracture during normal operation is not considered to be a credible mode of failure.

Reference sections:

Section Title	Section
Reactor Coolant System	
System Design and Operation	4.2
Safety Limits and Conditions	4.4
Vessel NDTT	Appendix 4A

1.3.7 ENGINEERED SAFETY FEATURES (GDC 37-GDC 65)

The design, fabrication, testing and inspection of the core, reactor coolant pressure boundary and their protection systems give assurance of safe and reliable operation under all anticipated normal, transient, and accident conditions. However, engineered safety features are provided in the facility to backup the safety provided by these components. These engineered safety features have been designed to cope with any size reactor coolant pipe break up to and including the circumferential rupture of any pipe in that boundary assuming unobstructed discharge from both ends, and to cope with any steam or feedwater line break up to and including the main steam or feedwater headers. The total loss of all outside power is assumed concurrent with this accident.

The release of fission products from the reactor fuel is limited by the Safety Injection System which, by cooling and the core and limiting the fuel clad temperature, keeps the fuel in place and substantially intact with its essential heat transfer geometry preserved and limits the metal water reaction to an insignificant amount.

The basic design criteria for assuring that the core geometry remains in place and substantially intact so that effective cooling of the core is not impaired following a loss-of-coolant accident are:

1. The cladding temperature is to be less than:

    a. the melting temperature of Zircaloy-4
    b. the temperature at which gross core geometry distortion, including fragmentation may be expected.

2. The total core metal-water reaction will be limited to less than 1%.

The Safety Injection System (which constitutes the Emergency Core Cooling System) consists of high and low head centrifugal pumps driven by electric motors, and passive accumulator tanks which are self energized and which act independently of any actuation signal or power source.

The release of fission products from the containment is limited in three ways:

1. Blocking the potential leakage paths from the containment. This is accomplished by:

   a. A steel-lined concrete reactor containment with continuously pressurized double penetrations and liner weld channels which forms a virtually leaktight barrier to the escape of fission products should a loss-of-coolant occur.

   b. Isolation of process lines by the Containment Isolation System which imposes double barriers in each line which penetrates the containment.

2. Reducing the fission product concentration in the containment atmosphere. This is accomplished by:

   a. Air recirculation filters which provide for rapid removal of particles from the containment atmosphere.

   b. Chemically treated spray which removes elemental iodine vapor from the containment atmosphere by washing action.

3. Reducing the containment pressure and thereby limiting the driving potential for fission product leakage. This is accomplished by cooling the containment atmosphere by the following independent systems of equal heat removal capacity:

   a. Containment Spray System

   b. Containment Air Recirculation Cooling System

A comprehensive program of plant testing is performed for all equipment systems and system control vital to the functioning of engineered safety features. The program consists of performance tests of individual pieces of equipment in the manufacturer's shop, and integrated tests of the system as a whole, and periodic tests of the actuation circuitry and mechanical components to assure reliable performance, upon demand, throughout the plant lifetime. In the event that one of the components should require maintenance as a result of failure to perform during the test according to prescribed limits, the necessary corrections or minor maintenance will be made and the unit retested immediately.

The plant is supplied with independent and redundant normal, standby and emergency power sources as follows:

1. The normal source of auxiliary power during plant operation is the generator. Power is supplied via the unit auxiliary transformer that is connected to the main leads of the generator.

2. Standby power required during plant startup, shutdown and after reactor trip is supplied from the high tension transmission terminal which has multiple lines running to the interconnected system.

3. Three diesel generator sets are connected to the engineered safety features buses to supply emergency shutdown power in the event of loss of all other a.c. auxiliary power. Any two diesel engine-electrical generator sets is capable of supplying automatically the engineered safety features load required for an acceptable post-blowdown containment pressure transient for any loss-of-coolant accident.

4. Emergency power supply for vital instruments and control and for emergency lighting is supplied from the two 125 V d.c. station batteries.

The emergency bus electrical power arrangement and logic network provides the capability to manually transfer component loads to the remaining diesel following the failure of one diesel engine-generator unit to start.

For such engineered safety features as are required to ensure safety in the event of an accident, protection from dynamic effects or missiles is considered in the layout of plant equipment and missile barriers.

Layout and structural design specifically protect injection paths leading to unbroken reactor coolant loops against damage as a result of the maximum reactor coolant pipe rupture. Injection lines penetrate the main missile barrier, and the injection headers are located in the missile-protected area between the missile barrier and the containment outside wall. Individual injection lines, separated to the maximum extent practicable, are connected to the injection header, pass through the barrier and then connect to the loops. Movement of the injection line, associated with rupture of a reactor coolant loop, is accommodated by line flexibility and by the design of the pipe supports such that no damage outside the missile barrier is possible.

Each engineered safety feature provides sufficient performance capability to accommodate any single failure of an active component and still function in a manner to avoid undue risk to the health and safety of the public.

Under the hypothetical accident conditions, the air recirculation system and the containment spray system are designed and sized so that either system, operating alone, is able to supply the necessary iodine removal and post-accident cooling capacity to rapidly reduce the containment pressure following blow-down and cooling of the core by safety injection.

All active components of the Safety Injection System (with the exception of injection line isolation valves) and the Containment Spray System are located outside the containment and not subject to containment accident conditions.

Instrumentation, motors, cables and penetrations located inside the containment are selected to meet the most adverse accident conditions to which they may be subjected. These items are either protected from containment accident conditions or are designed to withstand, without failure, exposure to the worst combination of temperature, pressure, and humidity expected during the required operational period.

The reactor is maintained subcritical following a primary system pipe rupture accident. Introduction of borated cooling water into the core results in a large negative reactivity addition. The control rods insert and remain inserted. The delivery of cold safety injection water to the reactor vessel following accidental expulsion of reactor coolant was evaluated to assure that it does not cause further loss of integrity of the Reactor Coolant System boundary.

Design provisions are made to the extent practical to facilitate access to the critical parts of the reactor vessel internals, injection nozzles, pipes, valves and safety injection pumps for visual or boroscopic inspection for erosion, corrosion and vibration wear evidence, and for non-destructive test inspection where such techniques are desirable and appropriate.

Design provisions are made so that active components of the Safety Injection System can be tested periodically for operability and functional performance. The safety injection pumps can be tested periodically during plant operation using the minimum flow recirculation lines provided. The residual heat removal pumps are used every time the residual heat removal loop is put into operation.

An integrated system test can be performed during the late stages of plant cooldown when the residual heat removal loop is in service. This test would not introduce flow into the Reactor Coolant System but would demonstrate the operation of the valves, pump circuit breakers, and automatic circuitry upon initiation of safety injection.

The accumulator tank pressure and level are continuously monitored during plant operation and flow from the tanks can be checked at any time using test lines.

The accumulators and the safety injection piping up to the final isolation valve is maintained full of borated water at refueling water concentration while the plant is in operation. Flow in each of the high head injection headers and in the main flow line for the residual heat removal pumps is monitored by flow and pressure instrumentation.

The design provides for capability to test initially, to the extent practical, the full operational sequence up to the design conditions for the Safety Injection System to demonstrate the state of readiness and capability of the system.

These functional tests provide information to confirm valve operating times, pump motor starting times, the proper automatic sequencing of load addition to the diesel generators, and delivery rates of injection water to the Reactor Coolant System.

The following general criteria are followed to assure conservatism in computing the required containment structural load capacity:

a) In calculating the containment pressure, rupture sizes up to and including a double-ended severance of reactor coolant pipe are considered.

b) In considering post-accident pressure effects, various malfunctions of the emergency systems are evaluated. Contingent mechanical or electrical failures are assumed to disable one of the diesel-electric generators, two of the fan coolers and one of the containment spray pumps.

c) The pressure and temperature loadings obtained by analyzing various loss-of-coolant accidents, when combined with operating loads and maximum wind or seismic forces, do not exceed the load-carrying capacity of the structure, its access opening or penetrations.

Discharge of reactor coolant through a double-ended rupture of the main loop piping, followed by operation of only those engineered safety features which can run simultaneously with power from two of the three emergency on-site diesel generator results in a sufficiently low radioactive materials leakage from the containment structure that there is no undue risk to the health and safety of the public.

The concrete containment is not susceptible to a low temperature brittle fracture. The containment liner is enclosed within the containment and thus is not exposed to the temperature extremes of the environs. The containment ambient temperature during operation is between 50 and 120°F which is well above the NDT temperature + 30°F for the liner material. Containment penetrations which can be exposed to the environment are also designed to the NDT + 30°F criterion.

The reactor coolant pressure boundary does not extend outside of the containment. Isolation valves for all fluid system lines penetrating the containment provide at least two barriers against leakage of radioactive fluids to the environment in the event of a loss-of-coolant accident. These barriers, in the form of isolation valves or closed systems, are defined on an individual line basis. In addition to satisfying containment isolation criteria, the valving is designed to facilitate normal operation and maintenance of the systems and to ensure reliable operation of other engineered safety features.

After completion of the containment structure and installation of all penetration and weld channels, an initial integrated leakage rate test is conducted at the peak calculated accident pressure, maintained for a minimum of 24 hours, to verify that the leakage rate is not greater than 0.1 per cent by weight of the containment volume per day. This leakage rate test will be performed using the reference vessel method.

A leak rate test at the peak calculated accident pressure using the same method as the initial leak rate test can be performed at any time during the operational life of the plant, provided the plant is not in operation and precautions are taken to protect instruments and equipment from damage.

Penetrations are designed with double seals so as to permit pressurization of the interior of the penetration whenever a leak test is required. The system utilizes a supply of clean, dry, compressed air which places all the penetrations under an internal pressure equal to the peak calculated accident pressure.

Leakage from the system is checked by measurement of the integrated makeup air flow. In the event excessive leakage is discovered, each penetration can then be checked separately.

Capability is provided to the extent practical for testing the functional operability of valves and associated apparatus during periods of reactor shutdown.

Initiation of containment isolation employs coincidence circuits which allow checking of the operability and calibration of one channel at a time.

The main steam and feedwater barriers and isolation valves in systems which connect to the Reactor Coolant System are hydrostatically tested to measure leakage.

Design provisions are made to the extent practical to facilitate access for periodic visual inspection of all important components of the Containment Air Recirculation and Containment Spray Systems.

The containment pressure-reducing systems are designed to the extent practical so that the spray pumps, spray injection valves, spray nozzles and additive injection valves can be tested periodically and after any component maintenance for operability and functional performance.

Permanent test lines for the containment spray loops are located so that all components up to the isolation valve at the spray nozzles may be tested. These isolation valves are checked separately.

The air test lines for checking that spray nozzles are not obstructed connects upstream of the isolation valve. Air flow through the nozzles is monitored by tell-tale devices attached to each nozzle or by use of a smoke generator.

Capability is provided to test initially to the extent practical the operational startup sequence beginning with transfer to alternate power sources and ending with near design conditions for the Containment Spray and Containment Air Recirculation Cooling Systems.

Reference sections:

Section Title	Section
Containment Systems	5
Engineered Safety Features	6
Electrical Systems	8.1, 8.2

1.3.8 FUEL AND WASTE STORAGE SYSTEMS (GDC 66-GDC 69)

The new and spent fuel storage racks are designed so that it is impossible to insert assemblies in .her than the prescribed locations. The spent fuel storage pit is filled with borated water at the same concentration as that used in the reactor cavity and refueling canal during refueling operations. The fuel is stored vertically in an array with sufficient center-to-center distance between assemblies to assure $K_{eff} \leq 0.90$ even if unborated water were used to fill the pit.

During reactor vessel head removal and while loading and unloading fuel from the reactor, the boron concentration is maintained at not less than that required to shutdown the core to a $k_{eff} = 0.90$. This shutdown margin maintains the core at $k_{eff} > 0.99$, even if all control rods are withdrawn from the core. The refueling water boron concentration is periodically checked to ensure the proper shutdown margin.

The design of the fuel handling equipment incorporating built-in interlocks and safety features, the use of detailed refueling instructions and observance of minimum operating conditions provide assurance that no incident could occur during the refueling operations that would result in a hazard to public health and safety.

The refueling water provides a reliable and adequate cooling medium for spent fuel transfer. Heat removal from the refueling water is provided by an auxiliary cooling system.

Adequate shielding for radiation protection is provided during reactor refueling by conducting all spent fuel transfer and storage operations under water. This permits visual control of the operation at all times while maintaining low radiation levels, less than 2.5 mr/hr, for periodic occupancy of the area by operating personnel. Pit water level is indicated, and water to be removed from the pit must be pumped out as there are no gravity drains. Shielding is provided for waste handling and storage facilities to permit operation within requirements of 10 CFR 20.

Gamma radiation is continuously monitored in the auxiliary building. A high level signal is alarmed locally and is annunciated in the control room.

Auxiliary shielding for the Waste Disposal System and its storage components is designed to limit the dose rate to levels not exceeding 1 mr/hr in normally occupied areas, to levels not exceeding 2.5 mr/hr in intermittently occupied areas and to levels not exceeding 15 mr/hr in controlled occupancy areas.

All fuel and waste storage facilities are contained and equipment designed so that accidental releases of radioactivity directly to the atmosphere are monitored and will not exceed the guidelines of 10 CFR 100.

The refueling canal and spent fuel storage pit are reinforced concrete structures with a seam-welded stainless steel plate liner. These structures are designed to withstand any anticipated earthquake loadings as Class I structures so that the liner should prevent leakage even in the event the reinforced concrete develops cracks.

Reference sections:

Section Title	Section
Fuel Handling System	9.4
Waste Disposal System	11.1
Radiation Protection	11.2

1.3.9 PLANT EFFLUENTS (GDC 70)

Liquid, gaseous, and solid waste disposal facilities are designed so that discharge of effluents and off-site shipments are in accordance with applicable governmental regulations.

Radioactive fluids entering the Waste Disposal System are collected in sumps and tanks until determination of subsequent treatment can be made. They are sampled and analyzed to determine the quantity of radioactivity, with an isotopic breakdown if necessary. Before any attempt is made to discharge, they are processed as required and then released under controlled conditions. The system design and operation are characteristically directed toward minimizing releases to unrestricted areas. Discharge streams are appropriately monitored and safety features are incorporated to preclude excessive releases.

The bulk of the radioactive liquids discharged from the Reactor Coolant System are processed and retained inside the plant by the Chemical and Volume Control System recycle train. This minimizes liquid input to the Waste Disposal System which processes relatively small quantities of generally low-activity level wastes.

Radioactive gases are pumped by compressors through a manifold to one of the gas decay tanks where they are held a suitable period of time for decay. Cover gases in the nitrogen blanketing system are re-used to minimize gaseous wastes. During normal operation, gases are discharged intermittently at a controlled rate from these tanks through the monitored plant vent. The system is provided with discharge controls so that environmental conditions do not restrict the release of radioactive effluents to the atmosphere.

Liquid wastes are processed to remove most of the radioactive materials. The spent resins from the demineralizers, the filter cartridges and the concentrates from the evaporators are packaged and stored on-site until shipment off-site for disposal.

Reference sections:

Section Title	Section
Waste Disposal System	11.1

1.4 DESIGN PARAMETERS AND PLANT COMPARISON

1.4.1 DESIGN HIGHLIGHTS

The design of the plant is based upon proven concepts which have been developed and successfully applied in the construction of pressurized water reactor systems. In subsequent paragraphs, the design features of the plant are indicated. These represent variations or extrapolations from units presently in operation such as San Onofre and Connecticut Yankee.

POWER LEVEL

The license application power level for Indian Point Unit No. 2 is 2758 MWt. The increase in this power rating over 1473 MWt for Connecticut Yankee is achieved by a 45 per cent increase in heat transfer surface area and a 29 per cent increase in average heat flux as indicated in Table 1.4-1. The increased heat transfer surface area is due to 22 per cent more fuel rods, each 20 per cent longer.

The increase in maximum heat flux and the 18.4 KW/ft linear power rating resulting are justified by the results of in-core experiments by Westinghouse and others.

REACTOR COOLANT LOOPS

The Reactor Coolant System for the Indian Point Unit No. 2 will consist of four loops as compared with three loops for San Onofre and four loops for Connecticut Yankee. The use of four loops in the Indian Point Unit No. 2 for the production of 2758 MWt requires an attendant increase in the size and capacity of the Reactor Coolant System components such as the reactor vessel, reactor coolant pumps, piping and steam generators. These increases represent reasonable engineering extrapolations of existing and proven designs.

PEAK SPECIFIC POWER

Based on the design hot channel factors, operation at 2758 MWt yields a steady state peak specific power of 18.4 kw/ft and a corresponding peak power of 20.6 kw/ft for the 112% overpower condition.

Based on values for hot channel factors, reactivity coefficients and other design parameters, which were established in the PSAR and are supported by the previous experience with other plants of the same type, this reactor can be operated safely at power levels at least as high as the license application rating.

FUEL CLADDING

The fuel rod design for the plant employs Zircaloy as a cladding material. This clad has proven successful in the CVTR and Saxton reactors and Yankee (Rowe) test assemblies. The fuel rod dimensions are identical to those in Ginna, Diablo Canyon, Salem and Zion Station Units 1 and 2.

FUEL ASSEMBLY DESIGN

The fuel assembly incorporates the rod cluster control concept in a canless 15 x 15 fuel rod assembly using a spring clip grid to provide support for the fuel rods. Extensive out-of-pile tests have been performed on this current concept; successful in-pile tests have been performed in the Saxton reactor; and operating experience is available now from the San Onofre and Connecticut Yankee plants.

MODERATOR TEMPERATURE COEFFICIENT OF REACTIVITY

The reactor has a negative moderator temperature coefficient of reactivity at operating temperature at all times throughout core life.

1.4.2 PLANT DESIGN PARAMETERS AND COMPARISON

Table 1.4-1 presents a summary of the design parameters for Indian Point Unit No. 2 in the first column. It gives a comparison of these data with the data for the San Onofre Plant, Ginna Power Plant and for Connecticut Yankee Power Plant. The differences existing for the Indian Point Unit No. 2 are due to the differences in power level and associated thermal and hydraulic parameters. The physical characteristics of all of these units are very similar.

LINE ITEM COMPARISON

IPP#2 - GINNA - SAN ONOFRE - CONNECTICUT YANKEE

Line item	Notes
1.	The increase in power level between Indian Point #2 and the other 3 plants results from the increased core size and an increase in the specific power.
2.	Directly related to Item 1 by conversion.
3.	No change in the fraction of the total heat generated in the core.
4.	This limitation applies only to prevention of temperatures of the fuel rods and coolant corresponding to power in excess of this overpower limit. As demonstrated by the detailed examination of the rod withdrawal accident at power, presented in Section 14.2.1 nuclear overpower can be 18 percent without exceeding this limit. The primary consideration in overpower protection is not the actual value of the trip set point but rather the error allowances that make up the margin to trip. The set point is selected so that a minimum DNB ratio of 1.3 is maintained at the condition of the maximum overpower when all errors are taken in the adverse direction and with the most adverse pressure and temperature allowed by the high ΔT trip. The combination of these protection channels (variable high ΔT and overpower) limit the range of allowable plant conditions to a region of temperature, pressure and power which preclude DNB or core damage for credible accidents. The error allowances are subject to verification by performance tests of the installed system. The errors due to drift and set

point reproducibility for Indian Point Unit No. 2 station are errors quoted by many instrumentation manufacturers and are demonstrated in actual performance tests on the equipment before shipment. The improved performance is attributable to the use of a solid state system.

The errors due to rod motion result from variations in axial flux distribution with rod motion. Because of this variation, ion chamber reading at a given axial location may differ for the same core average power level. These errors are reduced by the use of long ion chambers with top and bottom detectors, each equal in length to about one-half the core height. The detectors yield an average reading over one-half the axial length.

5-6 The reactor coolant system design pressure for the four plants is 2500 psia. For all conditions the system pressure is limited by code safety valves set to open at design pressure and sized to prevent system pressure from exceeding code limitations. Equipment capabilities for overpressure protection are established by the complete loss of load without an immediate reactor trip. The maximum overpressure for this transient is therefore a function of the safety valve capacity and the maximum pressurizer surge rate and is not dependent on the value of the nominal operating pressure.

The operating pressure is selected to ensure that desired thermal conditions are maintained in the core. The operating pressure is established and maintained between the upper and lower reactor trip limits to permit transient variations in either direction with the assistance of the Pressure Control System.

The normal operating pressure for the Indian Point Unit No. 2 and the Ginna Plants (2250 psia) has been increased over the previous designs; e.g., San Onofre (2100 psia) and Connecticut Yankee (2165 psia) and provides sufficient operating margin between the upper and lower pressure limits. Increasing the

operating pressure allows higher reactor coolant operating temperatures and hence improves the secondary plant performance.

The change to a higher nominal operating pressure in the IPP No. 2 design neither increases the probability of reaching the design pressure in operation, nor reduces the margin of safety in effect when design pressure is reached.

7. The heat flux hot channel factor is consistent with that of San Onofre and Connecticut Yankee.

8. The reduction in the enthalpy rise hot channel factor is due to a reevaluation of the enthal rise engineering hot channel factor. For a detailed discuss see Section 3.

9. The differences in the DNBR at nominal conditions is due to the slight differences in operating conditions.

10. Same for all plants.

11-12 The flow varies from plant to plant due to pump design and number of loops.

13-23 There are no signifi ant changes for these parameters from the previous plants.

24. The active surface area is determined by the mechanical design of the core.

25-28 The heat transfer parameters are determined by the required heat output, the heat transfer surface area and the design peaking factors for the core. They are related to clad integrity in that these conditions must be within the capability of the fuel and must also meet the thermal-hydraulic design criteria of DNB and fuel temperature. Extensive experience indicates that no problem exists at these thermal outputs.

29. The maximum clad surface temperature is higher for Indian Point #2 and Ginna due to the increased nominal operating pressure.

30-31 The fuel central temperatures are not significantly different that those for the other plants. The temperatures are well below the UO_2 melting temperature of 5000°F.

32. The increase in linear power at maximum overpower results from increased power density.

33-35 The fuel assembly design is not significantly changed with respect to type, rod pitch and overall dimensions.

36-37 The significant increase in fuel weight is due to the increased core size. The total weight of each fuel assembly includes the weight of the fuel, clad, grids, RCC guide tubes, and top and bottom nozzles.

38. The number of grids per assembly is increased because of the longer core design. The grid spacing provides slightly closer support spacing than provided in the other two plants.

39. The total number of fuel rods is consistent with the fuel assembly design and number of fuel assemblies.

40. There is no change in the fuel rod outside diameter.

41. The diametral gap is greater in IPP #2 and Ginna plants to allow for the greater differential thermal expansion between the fuel and the zircaloy clad.

42. More clad thickness is provided so the zircaloy clad meets the same design ground rules as the stainless steel clad of San Onofre and Connecticut Yankee Plants.

The cladding thickness of 0.0243 in. is chosen so that the stresses in the cladding material are below the yield strength of the material throughout the life of the core for normal operating conditions and design transient conditions. The maximum cladding strains are limited to 1/2 to 1 per cent throughout life when considering internal fission gas pressure, reactor coolant pressure, fuel thermal expansion, swelling and clad creep. The diametral gap of 0.0065 inches is sufficient to accommodate the fuel thermal expansion.

43. The cladding material is changed to Zircaloy-4. The clad thickness and internal diametral gap are revised to accommodate its properties.

44-47 The design of the fuel pellets is not substantially changed. The diameter is less to accommodate the large diametral gap and thicker clad.

48-52 The rod cluster control design is similar for all four plants. The number of RCC assemblies for each plant is determined based upon the control requirements. Twenty control rods per cluster are used in the 15 x 15 fuel assembly used in Indian Point #2.

53-54 The core barrel and thermal shield diameters are consistent with the core diameter.

55-56 The same comments as for line items 36 and 37 apply here.

57. The core equivalent diameter is greater due to the use of more fuel assemblies.

58. The core is two feet longer than San Onofre and Connecticut Yankee. Discussion of the long period xenon-induced axial instability and its control can be found in WCAP 7208.

59-61 The calculated characteristics of the reflector are not changed.

62. The water to uranium ratio is increased becau e this produces better economy with the Zircaloy clad core.

63. More fuel assemblies are required, consistent with core design.

64. The number of fuel rods per assembly is consistent with the number of absorber rods per RCC and the bottom mounted in-core instrumentation. Any fuel assembly can be placed over an in-core instrumentation penetration and can accept a neutron flux probe.

65. Three enrichments are used.

66-67 The average first cycle and first burnups are not significantly different but are affected by the burnable poison.

68-70 The core enrichment requirements are reduced due to the use of Zircaloy clad and the accompanying improvement in neutron economy.

71-73 The change is largely due to burnable poison.

74-76 The same comments as for Line Items 48, 49 and 50 apply here.

77. The total control rod worth is not significantly different.

78-81 The boron requirements for reactor shutdown and control are changed substantially from San Onofre and Connecticut Yankee because of burnable poison.

82-85 With the use of burnable poison, the moderator coefficient is always negative throughout core life at power operating conditions. The pressure coefficient, the moderator void (density) coefficient and the Doppler coefficient are not significantly different.

86-91 Section III of the ASME Boiler and Pressure Vessel Code is considered to be the better design guide because it has significantly upgraded Section VIII and its associated Nuclear Code Cases. It presents the latest skills in the analytical techniques of pressure vessel design and improved knowledge of pressure vessel failure patterns.

92. The code requirements for piping design are essentially the same as for Connecticut Yankee. The San Onofre requirement is different because of local state requirements.

93-94 Comments are the same as for Line Items 1 and 2.

95. Comments are the same as for Line Items 5 and 6.

96-97 There is no significant change.

98. More coolant loops are used because of the increased power.

99-100 The reactor coolant system design pressure and temperature are the same.

101. The hydro test pressure reduction is due to the change of codes.

102. The reactor coolant system volume is greater due to more loops.

103. The same comment as for Line Item 11.

104-111 The physical dimensions of the reactor vessel are changed to be consistent with the core size and requirements.

112-117 The steam generator design bases are the same. The number of generators is consistent with the number of coolant loops.

118. This is one-quarter the value of reference line 11.

119. The shell side design pressure is higher because of the increase in the maximum steam pressure. The value permits an increase in steam pressure and corresponding improvement in the heat rate of the turbine plant.

120. For Indian Point #2 and RGE, the design temperature is chosen closer to the actual maximum steam temperature.

121. Operating pressure of the tube side is higher because of the increase in the operating pressure of the reactor.

122. The shell side maximum operating pressure corresponds to the steam pressure at no load.

123. No change in the maximum moisture at the outlet at full load.

124. The hydro test pressure reduction is due to the change of code.

125-128 The type of reactor coolant pump (shaft seal) and the design conditions are the same. The number of pumps is consistent with the number of reactor coolant loops.

129. The increased nominal operating pressure corresponds to the increased system operating pressure.

130. There is no significant change in the suction temperature to the pumps.

131. The pump capacity is the same as for Ginna.

132. The design head of the pumps meets the requirements of the component and piping pressure losses.

133. The hydro test pressure reduction is due to change of code.

134. The type and design of the pump motors is the same.

135. The horsepower rating accommodates the increased flow and head requirements.

136-139 The reactor coolant piping is the same as for Ginna. The hot leg pipe has increased in diameter to maintain the same flow velocity limitation (<50 ft/sec) as used in the other plants. The pipe between the steam generator and the pump has increased in diameter to meet the allowable velocity limits at the pump inlet.

140. The piping design pressure is the same as that for other components of the Reactor Coolant System.

141-161 Steam and power conversion system components are standard commercial items with a generally higher rating than the other plants to accommodate the higher power of Indian Point Unit No. 2.

HYDRAULIC AND THERMAL DESIGN PARAMETERS

Total heat output, MWt

Total heat output, Btu/hr

Heat generated in fuel, %

Maximum thermal overpower, with
design hot channel factors, %

System pressure, nominal, psia

System pressure, minimum steady state, psia

Hot Channel Factors
Heat Flux, F_q
Enthalpy rise, $F_{\Delta H}$

DNB ratio at nominal conditions (Min)

Minimum DNBR for designs transients

Coolant Flow
Total flow rate, lb/hr
Effective flow rate for heat transfer, lb/hr
Average velocity along fuel rods, ft/sec
Average mass velocity, $lb/hr\text{-}ft^2$

Coolant Temperature, °F
Nominal inlet
Maximum inlet due to instrumentation,
error and deadband
Average rise in vessel
Average rise in core
Average in core
Average in vessel
Nominal outlet of hot channel

Average film coefficient, $Btu/hr\text{-}ft^2$ - °F

TABLE 1.4.1

COMPARISON OF DESIGN PARAMETERS

INDIAN POINT #2 FINAL REPORT	SAN ONOFRE FINAL REPORT	GINNA FINAL REPORT	CONNECTICUT YANKEE FINAL REPORT	REFERENCE LINE NO.
758	1347	1300	1473	1
413 x 10^6	4595 x 10^6	4437 x 10^6	5027 x 10^6	2
7.4	97.4	97.4	97.4	3
2	18	12	18	4
250	2100	2250	2165	5
220	2070	2220	2140	6
.23	3.23	3.38	3.09	7
.77	1.88	1.77	1.78	8
.00	2.07 (W-3)	2.15 (W-3)	2.82 (W-3)	9
.30 (W-3)	1.30 (W-3)	1.30 (W-3)	1.30 (W-3)	10
36.3 x 10^6	78 x 10^6	67.3 x 10^6	101.4 x 10^6	11
30 x 10^6	71 x 10^6	64.3 x 10^6	92.3 x 10^6	12
5.4	13.3	14.7	14.3	13
.53 x 10^6	2.02 x 10^6	2.38 x 10^6	2.36 x 10^6	14
43	553	551.9	545	15
47.0	557	555.9	549	16
3.0	44.2	49.5	38.5	17
5.5	48.5	52	42.0	18
71.0	578.0	578.0	566.0	19
69.5	575.1	577.0	564.3	20
33.5	637.5	634.0	615.5	21
790	5100	5590	5230	22

Average film temperature drop, °F

Heat Transfer at 100% Power
- Active heat transfer surface area, ft^2
- Average heat flux, Btu/hr-ft^2
- Maximum heat flux, Btu/hr-ft^2
- Average thermal output, kw/ft
- Maximum thermal output, kw/ft

Maximum clad surface temperature at nominal pressure, °F

Fuel Central Temperature, °F
- Maximum at 100% power
- Maximum at overpower

Thermal output, kw/ft at maximum overpower

CORE MECHANICAL DESIGN PARAMETERS

Fuel Assemblies
- Design

- Rod Pitch, in.
- Overall dimensions, in.
- Fuel Weight (as UO_2), pounds
- Total weight, pounds
- Number of grids per assembly

Fuel Rods
- Number
- Outside diameter, in.
- Diametral gap, in.
- Clad thickness, in.
- Clad material

Fuel Pellets
- Material
- Density (% of theoretical)
- Diameter, in.
- Length, in.

TABLE 1.4.1 (Cont'd)

(Sheet 2 of 9)

INDIAN POINT #2 FINAL REPORT	SAN ONOFRE FINAL REPORT	GINNA FINAL REPORT	CONNECTICUT YANKEE FINAL RSPORT	REFERENCE LINE NO.
30.3	28.2	26.9	26.1	22
52,200	31,222	28,715	35,900	23
175,600	143,350	150,500	136,400	24
567,300	463,350	508,700	421,500	25
5.7	4.65	4.88	4.25	26
18.4	15.0	16.5	13.7	27
657	648	657	652	29
4090	~3900	3880	3880	30
4390	~4100	4100	~4100	31
20.6	17.7	18.5	16.3	32
RCC Canless	RCC Canless	RCC Canless	RCC Canless	
15 x 15	14 x 14	14 x 14	15 x 15	33
0.563	0.556	0.556	0.563	34
8.426 x 8.426	7.763 x 7.763	7.763 x 7.763	8.426 x 8.426	35
216,000	143,600	120,872	165,000	36
276,000	167,100	152,895	222,000	37
9	7	9	7	38
39,372	28,260	21,659	32,128	39
.422	0.422	0.422	0.422	40
.0065	0.0055	0.0065	0.0055	41
.0243	0.0165	0.0243	0.0165	42
Zircaloy-4	Stainless Steel	Zircaloy-4	Stainless Steel	43
UO_2 Sintered	UO_2 Sintered	UO_2 Sintered	UO_2 Sintered	44
94-92-91	94-93	94-93	94-93	45
.3669	0.3835	0.3669	0.3835	46
.600	0.6000	0.600	0.600	47

	INDIAN POINT #2 FINAL REPORT
Rod Cluster Control Assemblies	
Neutron absorber	5% Cd; 15% In; 80% A
Cladding material	Type 304 SS-Cold Wor
Clad thickness, in.	.019
Number of clusters, full/part length	53/8
Number of control rods per cluster	20
Core Structure	
Core barrel ID/OD, in.	148.0/152.5
Thermal shield ID/OD, in.	158.5/164.0

FINAL NUCLEAR DESIGN DATA

Structure Characteristics

Fuel Weight (as UO_2), lb.	216,000
Clad weight, pounds	44,600
Core diameter, in.	132.75
Core height, in.	144
Reflector Thickness and Composition	
Top - water plus steel, in.	10
Bottom - water plus steel, in.	10
Side - water plus steel, in.	15
H_2O/U, unit cell (cold volume ratio)	4.01
Number of fuel assemblies	193
UO_2 rods per assembly	204

Performance Characteristics

Loading technique	3 region, non-unifor
Fuel Discharge Burnup, MWD/MTU	
Average first cycle	14,200
First core average	24,700
Feed Enrichments, w/o	
Region 1	2.2
Region 2 (first core with burnable poison	2.7
Region 3	3.2
Equilibrum	3.2

TABLE 3.4.1 (Cont'd)

(Sheet 3 of 9)

SAN ONOFRE FINAL REPORT	GINNA FINAL REPORT	CONNECTICUT YANKEE FINAL REPORT	REFERENCE LINE NO.
5% Cd; 15% In; 80% Ag	5% Cd; 15% In; 80% Ag	5% Cd, 15% In; 80% Ag	48
Type 304 SS-Cold Worked	Type 304 SS-Cold Worked	Type 304 SS-Cold Worked	49
0.019	0.019	0.0195	50
45/0	29/4	45/0	51
16	16	20	52
125/128.5	109.0/112.5	131/134.6	53
129/134.0	115.3/122.5	137.6/145.6	54
1a3,600	120,130	165,000	55
22,000	22,440	24,300	56
1°1	96.5	119.5	57
..0	144	121.8	58
10	∿10	10	59
10	∿10	10	60
15	∿15	15	61
3.04	3.35	3.76	62
157	121	157	63
180	179	204	64
3 region, non-uniform	3 region, non-uniform	3 region, non-uniform	65
13,500	∿14,900	14,100	66
20,000	∿24,400	21,800	67
3.15	2.44	3.00	68
3.40	2.78	3.24	69
3.85	3.48	3.67	70
3.00			

	INDIAN POINT #2 FINAL REPORT
Control Characteristics (Beginning of life)	
Effective Multiplication (With Burnable Poison)	
Cold, no power, clean	1.257
Hot, no power, clean	1.199
Hot, full power, Xe and Sm equilibrium	1.152
Rod Cluster Control Assemblies	
Material	5% Cd; 15% In; 80% A
Number of RCC assemblies	53/8
Number of absorber rods per RCC assembly	20
Total rod worth (calculated)	8.5%
Boron Concentrations (First Cycle with Burnable Poison)	
To shut reactor down with no rods inserted, clean (k_{eff} = .99) cold/hot	1370/1405
To control at power with no rods inserted, clean/equilibrium xenon and samarium	1160/780
Boron worth, hot	1% δk/k / 89 ppm
Boron worth, cold	1% δk/k / 72 ppm
Kinetic Characteristics	
Moderator temperature coefficient	-0.25×10^{-4} to -3.00×10^{-4} ($°F^{-1}$)
Moderator pressure coefficient	$+.2 \times 10^{-6}$ to 3.00×10^{-6} (Psi)$^{-1}$
Moderator void (density) coefficient	-.1 to +.30 (Δk/g/c
Doppler coefficient	-1.1×10^{-5} to 1.8×10^{-5} ($°F^{-1}$)

TABLE 1.4.1 (Cont'd)

(Sheet 4 of 9)

SAN ONOFRE FINAL REPORT	GINNA FINAL REPORT	CONNECTICUT YANKEE FINAL REPORT	REFERENCE LINE NO.
1.254	1.188	1.280	71
1.210	1.137	1.202	72
1.160	1.080	1.163	73
5% Cd; 15% In; 80% Ag	5% Cd; 15% In; 80% Ag	5% Cd; 15% In; 80% Ag	74
45	33	45	75
16	16	20	76
7.1%	6.8%	8.2%	77
2600 ppm/2900 ppm	1160 ppm/820 ppm	3210 ppm/3220 ppm	78
2500 ppm/2050 ppm	1310 ppm/890 ppm	2800 ppm/2300 ppm	79
1% δk/k / 150 ppm	1% δk/k / 120 ppm	1% δk/k / 150 ppm	80
1% δk/k / 120 ppm	1% δk/k / 90 ppm	1% δk/k / 120 ppm	81
+1 to -3.1 x 10^{-4} δk/k / °F	+0.5 to -3.5 x 10^{-4} δk/k / °F	+1 x 10^{-4} to 3.5 x 10^{-4} δk/k / °F	82
-1 to +3.1 x 10^{-6} δk/k / psi	-0.5 x 10^{-6} to 3.5 x 10^{-6} δk/k / psi	-1.0 to +3.0 x 10^{-6} δk/k / psi	83
+1 to -3 x 10^{-3} δk/k / % void	-0.10 to -0.30 δk/k / gram/cc	0 to -2 x 10^{-3} δk/k / % void	84
-1 to -2.5 x 10^{-5} δk/k / °F	-1.1 x 10^{-5} to 1.8 x 10^{-5} δk/k / °F	-0.5 x 10^{-5} to 2.1 x 10^{-5} δk/k / °F	85

REACTOR COOLANT SYSTEM - CODE REQUIREMENTS

Component	
Reactor vessel	ASME III Class A
Steam Generator	
Tube Side	ASME III Class A
Shell Side	ASME III Class C
Pressurizer	ASME III Class A
Pressurizer relief tank	ASME III Class C
Pressurizer safety valves	ASME III
Reactor coolant piping	USAS B31.1

PRINCIPAL DESIGN PARAMETERS OF THE REACTOR COOLANT SYSTEM

Reactor heat output, MWt	2758
Reactor heat output, Btu/hr	9413×10^6
Operating pressure, psig	2235
Reactor inlet temperature, °F	543
Reactor outlet temperature, °F	596
Number of loops	4
Design pressure, psig	2485
Design temperature, °F	650
Hydrostatic test pressure (cold), psig	3110
Total reactor coolant system volume, cu. ft.	12,600
Total reactor flow, gpm	358,800

TABLE 1.4.1 (Cont'd)

(Sheet 5 of 9)

SAN ONOFRE FINAL REPORT	GINNA FINAL REPORT	CONNECTICUT YANKEE FINAL REPORT	REFERENCE LINE NO.
ASME VIII, 1270N, 1273N	ASME III Class A	ASME VIII, 1270N, 1273N	86
ASME VIII, 1270N, 1273N	ASME III Class A	ASME VIII, 1270N, 1273N	87
ASME VIII, 1270N, 1273N	ASME III Class C	ASME VIII, 1270N, 1273N	88
ASME VIII, 1270N, 1273N	ASME III Class A	ASME VIII, 1270N, 1273N	89
ASME VIII, Par. UW-2	ASME III Class C	ASME VIII, Par. UW-2	90
ASME I, Case 1271N	ASME III	ASME I, Case 1271N	91
ASME I	USAS B31.1	ASA B31.1	92
1347	1300	1473	93
4595×10^6	4437×10^6	5027×10^6	94
2085	2235	2150	95
553	551.9	545	96
597.2	601.4	583.5	97
3	2	4	98
2485	2485	2485	99
650	650	650	100
3735	3110	3750	101
7082	6245	8635	102
208,500	180,000	268,200	103

PRINCIPAL DESIGN PARAMETERS OF THE REACTOR VESSEL

Material	SA 302 Grade B, 1 alloy steel, inte clad with Type 30 tenitic stainless
Design pressure, psig	2485
Design temperature, °F	650
Operating pressure, psig	2235
Inside diameter of shell, in.	173
Outside diameter across nozzles, in.	262-7/16
Overall height of vessel and enclosure head, ft-in.	43'-9 11/16
Minimum clad thickness, in.	5/32

PRINCIPAL DESIGN PARAMETERS OF THE STEAM GENERATORS

Number of units	4
Type	Vertical, U-tube integral-moisture separator
Tube material	Inconel
Shell material	SA 302 Grade B
Tube side design pressure, psig	2485
Tube side design temperature, °F	650
Tube side design flow, lb/hr	34.07×10^6
Shell side design pressure, psig	1085
Shell side design temperature, °F	556
Operating pressure, tube side, nominal, psig	2235
Operating pressure, shell side, maximum, psig	1005.3
Maximum moisture at outlet at full load, %	1/4
Hydrostatic test pressure, tube side (cold), psig	3110

TABLE 1.4.1 (Cont'd)

(Sheet 6 of 9)

	SAN ONOFRE FINAL REPORT	GINNA FINAL REPORT	CONNECTICUT YANKEE FINAL REPORT	REFERENCE LINE NO.
lly aus- teel	SA 302 Grade B, low alloy steel, internally clad with Type 304 austenitic stainless steel	SA 302 Grade B, low alloy steel, internally clad with Type 304 austenitic stainless steel	SA 302 Grade B, low alloy steel, internally clad with Type 304 austenitic stainless steel	104
	2485	2485	2485	105
	650	650	650	106
	2085	2235	2150	107
	142	132	154	108
	227	219-5/16	243-3/16	109
	44-4	39-1 5/16	41-6	110
	5/32	5/32	5/32	111
	3	2	4	112
ch	Vertical, U-tube with integral-moisture separator	Vertical, U-tube with integral-moisture separator	Vertical, U-tube with integral-moisture separator	113
	Inconel	Inconel	Inconel	114
	Carbon steel	Carbon steel	Carbon steel	115
	2485	2485	2485	116
	650	650	650	117
	26×10^6	33.63×10^6	25.35×10^6	118
	985	1085	985	119
	600	556	600	120
	2085	2235	2150	121
	910	989	910	122
	1/4	1/4	1/4	123
	3725	3110	3750	124

PRINCIPAL DESIGN PARAMETERS OF THE REACTOR COOLANT PUMPS

Number of units	4
Type	Vertical, single radial flow with suction and horiz discharge
Design pressure, psig	2485
Design temperature, °F	650
Operating pressure, nominal, psig	2235
Suction temperature, °F	556
Design capacity, gpm	90,000
Design head, ft	252
Hydrostatic test pressure (cold) psig	3110
Motor type	A-C induction single speed 6000 Hp

PRINCIPAL DESIGN PARAMETERS OF THE REACTOR COOLANT PIPING

Material	Austenitic SS
Hot leg - ID, in.	29
Cold leg - ID, in.	27-1/2
Between pump and steam generator - ID, in.	31
Design pressure, psig	2485

STEAM AND POWER CONVERSION DESIGN PARAMETERS

Turbine-Generator	
Turbine type	Four element, tandem-compound, six-flow exhaust

TABLE 1.4.1 (Cont'd)

(Sheet 7 of 9)

SAN ONOFRE FINAL REPORT	GINNA FINAL REPORT	CONNECTICUT YANKEE FINAL REPORT	REFERENCE LINE NO.
3	2	4	125
Vertical, single stage radial flow with bottom suction and horizontal discharge	Vertical, single stage radial flow with bottom suction and horizontal discharge	Vertical, single stage radial flow with bottom suction and horizontal discharge	126
2485	2485	2485	127
650	650	650	128
2085	2235	2150	129
553	551.9	545	130
65,900	90,000	67,200	131
206	252	214	132
3735	3110	3750	133
A-C induction single speed	A-C induction single speed	A-C induction single speed	134
4100 HP	6000 HP	4600 HP	135
Austenitic SS	Austenitic SS	Austenitic SS	136
27-1/2	29	27-1/2	137
27-1/2	27-1/2	27-1/2	138
29	31	29	139
2485	2485	2485	140
Three element, tandem-compound, four-flow exhaust	Three element, tandem-compound, four-flow exhaust	Three element, tandem-compound, four-flow exhaust	141

Turbine capacity, kw	
Maximum guaranteed	1,021,793
Maximum calculated	1,068,701
Generator rating, kva	1,125,600
Turbine speed, rpm	1800
Condensers	
Type	Radial flow, sin pass, divided wa box, deaerating
Number	3
Condensing capacity, lb of steam/hr	7,230,000
Condensate pumps	
Type	8 stage, vertica pit type, centri
Number	3
Design capacity, (each) gpm	7860
Motor type	Vertical, induct
Motor rating, hp	3000
Feedwater pumps	
Type	High speed, band casing, single s centrifugal

TABLE 1.4.1 (Cont'd)

(Sheet 8 of 9)

SAN ONOFRE FINAL REPORT	GINNA FINAL REPORT	CONNECTICUT YANKEE FINAL REPORT	REFERENCE LINE NO.
450,000	496,322	616,200	142
450,000	516,739	646,135	143
500,000	608,400	667,000	144
1800	1800	1800	145
Single pass, horizontal divided water box, deaerating	Radial flow, semi-cylindrical water boxes, deaerating	Single pass, divided water box, deaerating	146
2	2	2	147
3,293,000	3,448,805	-	148
vertical, wet pit	multi-stage, vertical pit-type, centrifugal	seven-stage, vertical pit-type	149
4	3	2	150
2900	6600	6200	151
Vertical, induction	Vertical	Vertical, induction	152
700	1500	1500	153
Two-stage horizontal split case, double volume, centrifugal	High-speed, barrel-type, single stage, double flow, centrifugal	Two-stage, horizontal centrifugal	154

	INDIAN POINT #2 FINAL REPORT
Number	2
Design capacity (each), gpm	15,600
Motor type	horizontal stea
Motor rating, hp	8350
Emergency feedwater source	360,000 gallons reserve in 600, condensate tank Alternate suppl 500,000 gal. tank
Emergency feedwater pumps Number	3 (one steam d two electric m driven)
Design capacity, gpm	800 (steam dri
	400 (motor-dri

TABLE 1.4:1 (Cont'd)

(Sheet 9 of 9)

	SAN ONOFRE FINAL REPORT	GINNA FINAL REPORT	CONNECTICUT YANKEE FINAL REPORT	REFERENCE LINE NO.
	2	2	2	155
	7000 (10,500 during safety injection)	7400	9600	156
urbine	horizontal induction	horizontal	horizontal, induction	157
	3500	5000	4500	158
sured gal. rom y water	240,000 gallon condensate storage tank	30,000 gallons in each of two condensate storage tanks; service water	100,000 gallon demineralizer water storage tank	159
n,	2 (one steam-driven and one motor-driven)	3 (one steam-driven and two motor-driven)	1	160
	300 (steam-driven)	400 (steam-driven)	450	161
	235 (motor-driven)	200 (motor-driven)		

1.5 RESEARCH AND DEVELOPMENT REQUIREMENTS

Research and development was conducted relating to finalization of core design details and parameters, air recirculation system halogen filters, failure of core cooling systems and means to ameliorate consequences, emergency core cooling system, control rod ejection analysis and reactor coolant pump controlled leakage seals.

The detailed final core design and thermal-hydraulics and physics parameters have been completed. The nuclear design including fuel configuration and enrichments, control rod pattern and worths, reactivity coefficients and boron requirements are presented in Section 3.2.1 and the final thermal-hydraulics design parameters are in Section 3.2.2. Section 3.2.3 presents the final fuel, fuel rod, fuel assembly and control rod mechanical design. The core design incorporates fixed burnable poison rods[1] in the initial loading to ensure a negative moderator reactivity temperature coefficient at operating temperature. This improves reactor stability and lessens the consequences of a rod ejection or loss of coolant accident. The nuclear design configuration and mechanical design is presented in Sections 3.2.1 and 3.2.3 respectivley.

Core stability has been analyzed[2],[3],[5] and design provisions for detection and control of potential Xenon oscillations have been finalized.[3] The core design incorporates part length control rods for controlling these Xenon oscillations and shaping the axial power distribution. X-Y control is not required and therefore not provided. Tests in operating reactors demonstrate the ability of the out-of-core instrumentation to give accurate indication of power redistribution and provide the operator information necessary to monitor redistributions and control axial oscillations by moving the rods in a prescribed pattern[3]. This capability will be verified during startup tests in the Indian Point Unit No. 2 Plant.

Full-size filter tests were conducted for the Connecticut Yankee Atomic Power Company to demonstrate the efficiency for iodine removal under the most extreme conditions anticipated in the post-accident containment environment. The results of these tests[4] filed with the Atomic Energy Commission under Docket No. 50-213, are directly applicable to the charcoal filter system employed in this plant. The continued effectiveness of the charcoal filters during plant use will be demonstrated by periodic tests in the Haddam Neck Plant and in this plant as required by the technical specifications.

A program for development of a crucible system design, which would contain the reactor core assuming failure of the core cooling system to prevent a core meltdown was undertaken. A scheme for containing the molten core in a water submerged high melting point refractory lined steel crucible resulted. Refractory materials and crucible physical design were investigated along with analysis of the temperature distribution expected with the molten core and crucible refractory, and steam and water recirculation paths. As a result of uncertainties in material properties at the high application temperature, the lack of experimental proof that the boiling core mass would dissipate its heat upward through the water cover, and the possibility of violent liquid metal-water reactions it became apparent that the proper emphasis for research and development on the loss of coolant accident should be increased emphasis placed on research and development for emergency core cooling system improvement in order to eliminate need for a crucible. This is supported by the conclusions of the Report of Advisory Task Force on Power Reactor Emergency Cooling," Emergency Core Cooling," USAEC.

This additional development effort on emergency core cooling system design has resulted in the modification of the system to include pressurized accumulator tanks for rapid core reflooding. This increased flooding capability limits

the clad temperature after a loss of coolant accident to well below the melting temperature of Zircaloy-4, minimizes metal-water reaction and ensures that the core remains in place and intact thereby assuring preservation of essential heat transfer geometry. The system design incorporates redundancy of components such that the minimum required water addition rates can be met assuming any active component to fail concurrent with the loss-of-coolant accident or, over the long term period of post-accident core decay heat removal a passive or active component failure in either the safety injection or service water systems, or an active failure in the component cooling water system. Details of system design and operation are in Sections 6.2, 9, and 9.8, and analysis of the loss of coolant accident is presented in Section 14.3. Because of the incorporation of this revised emergency core cooling system the Reactor Pit Crucible has been deleted from the plant design. Although it is not required to provide cooling for molten fuel in the bottom of the reactor vessel with the upgraded ECCS performance, the clearance between the insulation and the instrumentation penetrations with the pressure relief holes in the insulation at the top of the vessel provide assurance that water in the flooded reactor vessel cavity will be in contact with the vessel. No other provisions for direct vessel cooling are provided or required.

A control rod ejection analysis was performed for the final core design, rod worths, rod position limits, and moderator reactivity temperature coefficient. As mentioned above the addition of the burnable poison rods eliminates power operation with a positive moderator temperature coefficient and reduces the severity of the ejected rod accident, hence, lessening the need for research and development on this subject. The analysis, presented in Section 14.2, shows that, with the final core design and the insertion limits for the control rods, no consequential damage to the Reactor Coolant System and no fuel or clad melt will occur for ejection of the highest worth rod.

The reactor coolant pump controlled leakage seal design for this plant has been fully developed. A full scale mock-up of this seal was operated for over 100 hours to confirm that seal deflection and leak rate under

load ware acceptable. The full scale mock-up has been used during the development of the controlled leakage seal to provide information related to long-term performance. One of the two seals used in this plant was operated about 300 hours and the other about 100 hours, each in its pump motor unit. During hot functional testing in the plant, before the core is loaded, additional operation will bring the total operating time for each seal to well over 500 hours.

Successful operation of similar seals has been demonstrated with over 5000 hours total running time in San Onofre and over 3000 hours in Haddam Neck.

REFERENCES

1. Wood, P.M., Bassler, E. A., et. al., "Use of Burnable Poison Rods in Westinghouse Pressurized Water Reactors," WCAP-7113 (October 1967).

2. Wood, P.M., Gallagher, J. M., Metz, R.M., Dean, R.A., "Use of Part Length Absorber Rods in Westinghouse Pressurized Water Reactors," WCAP-7072 (June 1967).

3. WCAP-7208, "Power Distribution Control of Westinghouse Pressurized Water Reactors (October 1968).

4. CYAP-101, "Connecticut Yankee Charcoal Filter Tests, December," (1966).

5. WCAP-7407-L "Power Maldistribution Investigations," Westinghouse Proprietary (Jan. 1970).

1.6 IDENTIFICATION OF CONTRACTORS

Consolidated Edison, as owners, engaged Westinghouse Electric Corporation to design and construct the Indian Point Unit No. 2. However, irrespective of the explanation of contractual arrangements offered below, Consolidated Edison is the sole applicant for licenses as owner and applicant is responsible for the design, construction, and operation of the Indian Point Unit No. 2.

The Indian Point Unit No. 2 was designed and is being built by the Westinghouse Electric Corporation as prime contractor for Consolidated Edison. Westinghouse has undertaken to provide a complete, safe, and operable nuclear power plant ready for commercial service in 1969. The project is directed by Westinghouse from the offices of its Atomic Power Division in Pittsburgh, Pennsylvania and by Westinghouse representatives at the plant site during construction and plant startup. Westinghouse has engaged United Engineers and Constructors of Philadelphia, Pennsylvania to provide the design of certain portions of the plant.

The plant construction is under the general direction of Westinghouse through United Engineers and Constructors which is responsible for the management of all site construction activities and either performs itself or subcontracts the work of construction and equipment erection. Preoperational testing of equipment and systems at the site and initial plant operation will be performed by Consolidated Edison personnel under the technical direction of Westinghouse.

Chap 2

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Information in this record was deleted in accordance with the Freedom of Information Act. Exemptions 4 FOIA/PA 2001-0343

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ADDITIONAL INFORMATION SUPPLIED
BY QUESTION RESPONSES IN
VOLUMES 5 AND 6

Section	Title	Question
2.5	Maximum probable hurricane and resultant highest potential flooding levels	2.1

2.0 SITE AND ENVIRONMENT

2.1 SUMMARY OF CONCLUSIONS

This Section of the FSAR sets forth the site and environmental data which together form a basis for the criteria for designing the facility and for evaluating the routine and accidental release of radioactive liquids and gases to the environment. These data support the conclusion that there will be no undue risk to public health and safety with the plant designed as planned and the environmental characteristics described. The strength of this conclusion rests not only upon the data themselves but upon the favorable opinions (also included in this report) of several independent consultants to the Applicant, each speaking within his particular area of expertness, — health physics, demography, geology, seismology, hydrology or meteorology, as the case may be.

The task of evaluating the environmental characteristics of the area has been facilitated by the significant fact that for more than 12 years studies and measurements of these characteristics have been made, whereas for over five years measurements have been made of the effects on environment of releases from an operating nuclear power facility, the facility the subject of AEC License No. DPR-5.

Careful projections have been made of the probable growth of population in the area and these projections have been taken into account in plant design both as to control of accidents and as to assumptions about operation.

Only forty-six people reside within 1/2 mile of Unit No. 2 and only 1080 live within one mile. Approximately 53,000 people now reside within a 5-mile radius of the facility. The largest concentration of population is in the City of Peekskill (Population 19,000; estimated 1960 population, 30,000) the center of which is about 2-1/2 miles northeast of the site. The most densely populated 15 degree sector, within 5 miles, is toward Peekskill to the northeast and contains 12,120 people.

The 1960 population within a 15-mile radius of the site was 326,9?? whereas the 1980 estimated population is 670,210. The projections do not indicate, and there is no reason otherwise to conclude, that the land usage within this radius will shift appreciably during the intervening period. (The land is now zoned principally for residential and state park usage although there is some industrial activity and a little agricultural and grazing activity.)

The outer boundary of the low population zone (inhabited by about 66 people) has been set at 1,100 meters from Unit No. 2.

Geologically, the site consists of a hard limestone in a jointed condition which will provide a solid bed for the plant foundation. The bedrock is sufficiently sound to support any loads which could be anticipated up to 50 tons per square foot, which is far in excess of any load which may be imposed by the plant. Although it is hard, the jointed limestone formation is permeable to water. Thus, if water from the plant should enter the ground (an improbable event since the plant is designed to preclude any leakage into the ground) it would percolate to the river rather than enter any ground water supply. Additional studies by Consolidated Edison's geology consultant, Thomas W. Fluhr, and examination of recent soil borings confirm the above conclusions.

In the Hudson River, about 80,000,000 gallons of water flow past the plant each minute during the peak tidal flow. This flow will provide additional mixing and dilution for liquid discharges from the facility. In fact, however, this aspect is superfluous since the assumption in the plant design is to treat the river water as if it were used for drinking (which it is not) and thus to reduce radioactive discharges, by dilution with ordinary plant effluent, to concentrations that would be tolerable for drinking water. There is no danger of flooding at the site.

Seismic activity in the Indian Point area is rare and no damage has resulted therefrom. As stated by Applicant's consultant on seismology, the site is "practically non-seismic" and is "as safe as any area at present known." Notwithstanding such assurance, the plant is designed to withstand an earthquake of the highest intensity ever recorded in this area.

Meteorological conditions in the area of the site were determined during a two-year test program. These data have been used in evaluating the effects of gaseous discharges from the plant during normal operations and during the postulated loss-of-coolant accident. In addition, data supplied by the U.S. Weather Bureau at the Bear Mountain Station, regarding the meteorological conditions during periods of precipitation, have been used to evaluate the rainout of fission gases into surface water reservoirs following the postulated loss-of-coolant accident. The evaluations indicate that the site meteorology provides adequate diffusion and dilution of any released gases.

Environmental radioactivity has been measured at the site and surrounding area for nearly ten years in association with the operation of Indian Point Unit No. 1, and the construction of the Indian Point Unit No. 2. These measurements will be continued and reported. The radiation measurements of fallout, water samples, vegetation, marine life, etc. have shown no perceptible post-operative increase in activity. Noticeable increases in fallout have coincided with weapons testing programs and appear to be related almost entirely to those programs. The New York State Department of Health in an independent two-year post-operative study[1] found that environmental radioactivity in the vicinity of the site is no higher than anywhere else in the State of New York.

Consultants participating in the preparation of the various reports, measurements and conclusions appearing in this Section include

Dr. Merril Eisenbud, Director of Environmental Radiation Laboratory, Institute of Industrial Medicine, New York University; Dr. Benjamin Davidson, Meteorologist and Director, Geophysical Science Laboratory, New York University College of Engineering; Dr. Edgar M. Hoover, Regional Economic Development Institute, Inc.: Metcalf & Eddy Engineers, hyrology specialists; Rev. J. J. Lynch, S. J., Director of the Seismic Observatory, Fordham University; Mr. Sidney Paige, Consulting Geologist; Quirk, Lawler and Matusky Engineers, Environmental Science and Engineering Consultants; Mr. Karl R. Kennison, Consulting Civil and Hydraulic Engineer; and Mr. Thomas W. Fluhr, P. E., Consulting Engineering Geologist. Reports made by Captain Elliott D. Roberts, Chief of the Geophysics Division, U.S. Department of Commerce and by Mr. James Dorman, Lamont Geological Observatory, Columbia University have also been utilized.

(1) Consolidated Edison Indian Point Reactor Post Operational Survey - August, 1965, Division of Environmental Health Services, New York State Department of Health, Hollis S. Ingraham, M.D., Commissioner.

Consolidated Edison Indian Point Reactor Environmental and Post Operation Survey - July, 1966, Division of Environmental Health Services, New York State Department of Health, Hollis S. Ingraham, M.D., Commissioner.

2.2 LOCATION

2.2.1 GENERAL

Indian Point Unit No.2 is being built adjacent to and north of Unit No.1 on a site of approximately 239 acres of land on the east bank of the Hudson River at Indian Point, Village of Buchanan in upper Westchester County, New York. The proposed Indian Point Unit No.3 will be adjacent to and south of Unit No.1. The site is about 24 miles north of the New York City boundary line. The nearest city is Peekskill, 2.5 miles northeast of Indian Point, with a population of about 19,000. An aerial photograph, Figure 2.2-1, shows the site and about 58 square miles of the surrounding area. 15

2.2.2 ACCESS

The site is accessible by several roads in the Village of Buchanan. A paved road links the eastern boundary of the site to the existing plant. The existing wharf is being used to receive heavy equipment during the construction period. The site is not served by rail.

2.2.3 SITE OWNERSHIP AND CONTROL

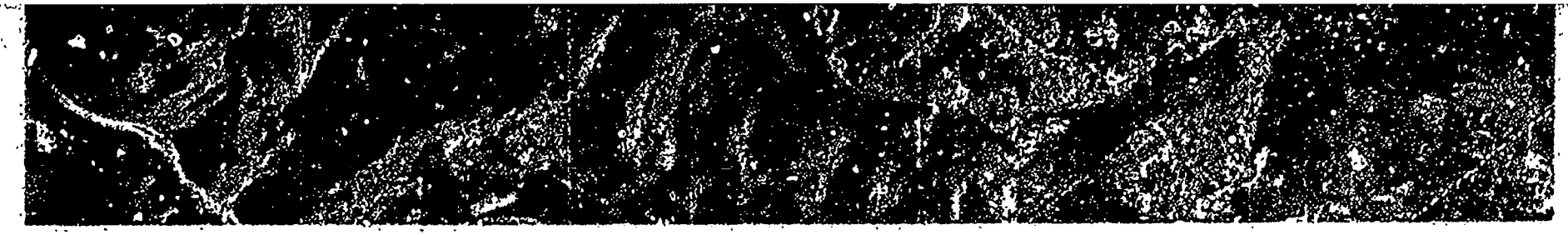
The consolidated Edison Company is the sole owner of the entire property. The Algonquin Gas Transmission Company has a right-of-way running east to west through the property, 2840 feet long and 65 feet wide. Unit 2 is 1450 feet north of the 26 inch Algonquin gas main. The Georgia-Pacific Corporation has an easement, 1610 feet long and 30 feet wide, along the southerly property line of the Indian Point site. The Georgia-Pacific easement is used for overhead electrical power and telephone lines, and underground gas, water and sewer lines. A permanent easement for the Village of Buchanan sewer crosses the eastern corner of the property. It is 20 feet wide, 900 feet long and 2300 feet east of Unit No.2.

Units No. 1, 2, 3 will be fenced by an eight-foot chain link type fence
15| surmounted by three-strand barbed wire. The gates to this area will be either secured or attended by plant personnel. In addition, a fence of the same type will separate the conventional and nuclear parts of the units, isolating the control area. The site is fenced in part with agricultural type fencing and the access road is continuously controlled by Company guards. A scale plot plan of the site is shown on Figure 2.2-2.

2.2.4 ACTIVITIES ON THE SITE

The principal activities on the site will be the generation, transmission and distribution of steam and electrical energy; associated service activities; activities relating to the controlled conversion of the atomic energy of fuel to heat energy by the process of nuclear fission; and the storage, utilization and production of special nuclear, source and by-product materials. Possible future activities include the addition of other nuclear and conventional electrical generating units.

An observation building is located about 1175 feet from the center line of the reactor to the nearest point of the fence southeast of Unit No. 2, and is open to the public during the day and attended by Company personnel. Limited public recreational grounds under Company control will also be provided on the site in the future.



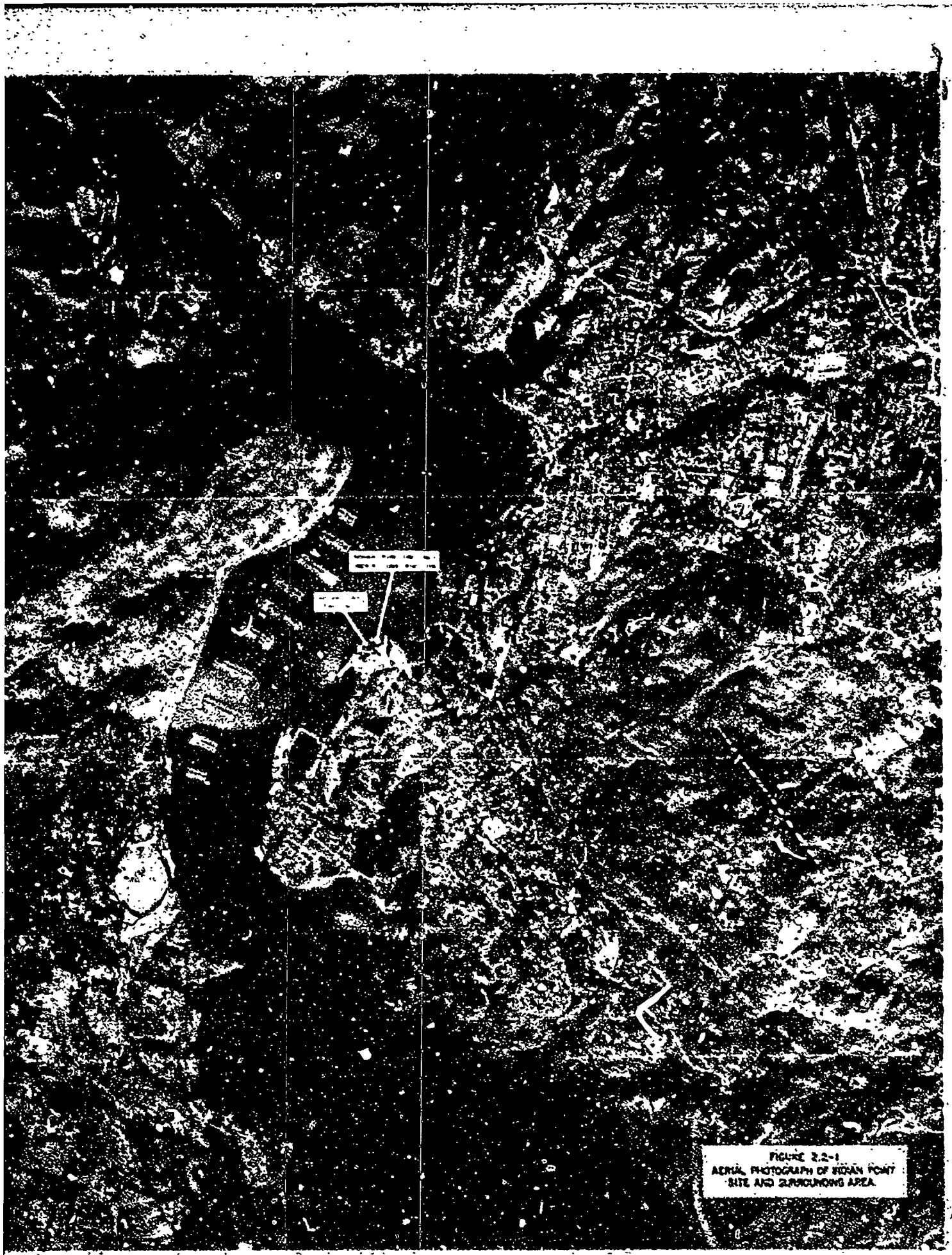
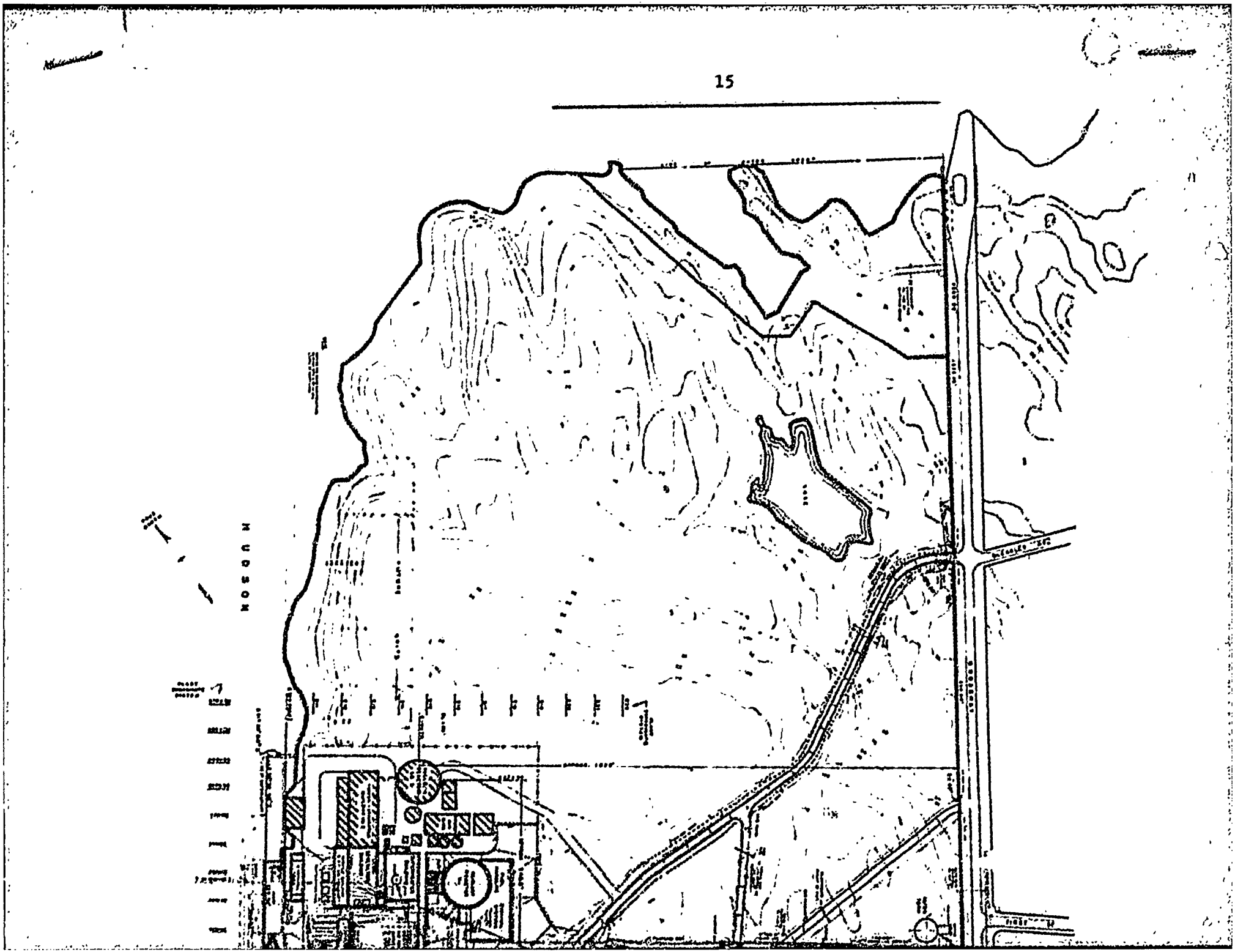


FIGURE 2.2-1
AERIAL PHOTOGRAPH OF MOAB POINT SITE AND SURROUNDING AREA


HUDSON

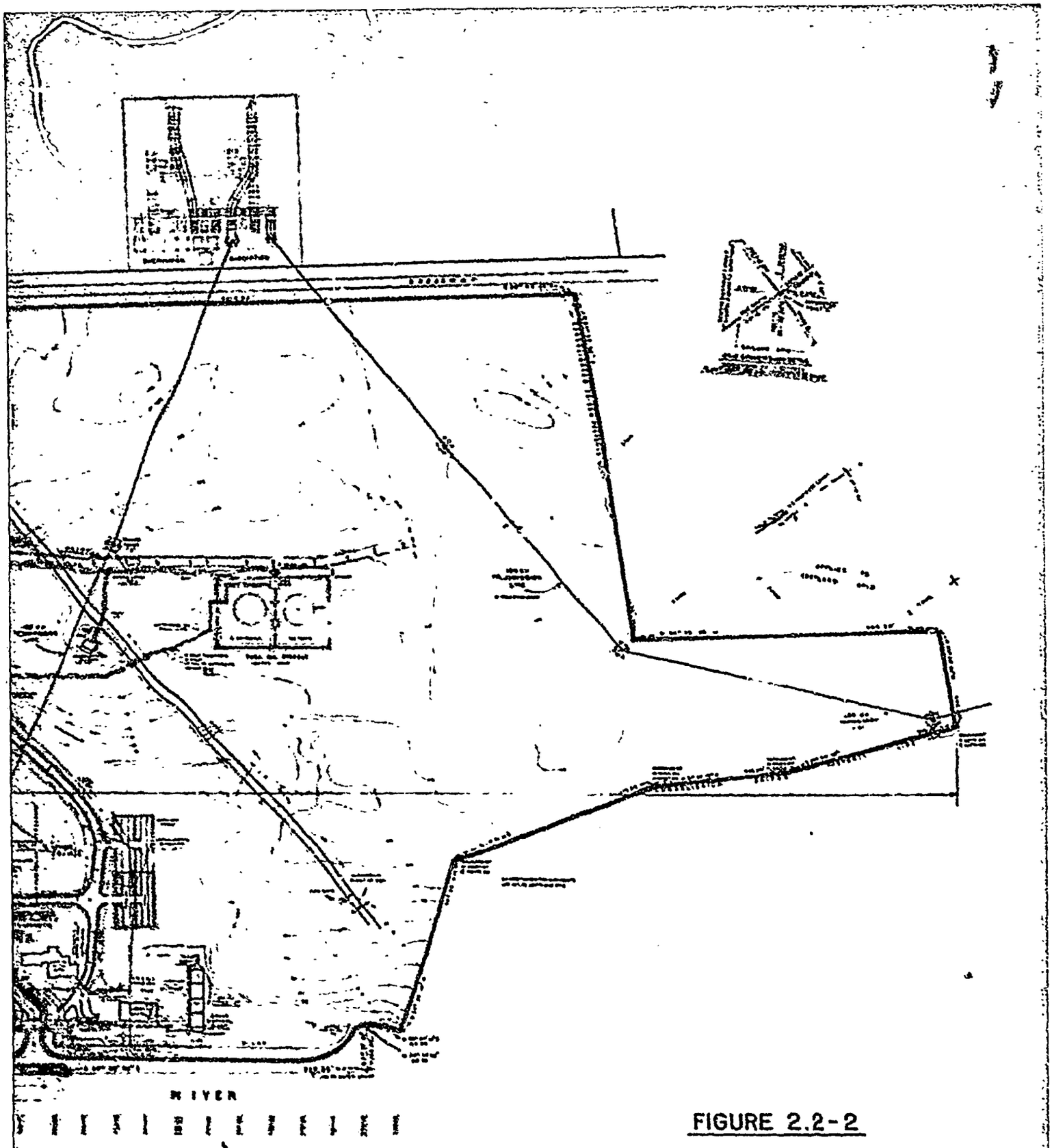


FIGURE 2.2-2

SCALE PLOT PLAN OF THE SITE
SHOWING LOCATION OF THE FACILITY
INDIAN POINT STATION
BUCHANAN, N. Y.

SCALE

NOVEMBER 1965

Supplement 15
11/70

2.3 TOPOGRAPHY

The Indian Point Generating Station, site of Unit No.2 is surrounded on almost all sides by high ground ranging from 600 to 1000 feet above sea level. The generating station is on the east bank of the Hudson River which runs northeast to southwest at this point but turns sharply northwest approximately two miles northeast of the plant. The west bank of the Hudson is flanked by the steep, heavily wooded slopes of the Dunderberg and West Mountains to the northwest (elevations 1086 feet and 1257 feet respectively) and Buckberg Mountain to the west-southwest (elevation 793 feet). These peaks extend to the west by other names and gradually rise to slightly higher peaks.

The general orientation of this mass of high ground is northeast to southwest. One mile northwest of the site, Dunderberg bulges to the east, and north of Dunderberg and the site, high ground reaching 800 feet forms the east bank of the Hudson as the river makes a sharp turn to the northwest. To the east of the site, peaks are generally lower than those to the north and west. Spitzenberg and Blue Mountains average about 600 feet in height and there is a weak, poorly defined series of ridges which again seem to run in a north-northeast direction. The river south of the site makes another sharp bend to the southeast and then widens as it flows past Croton and Haverstraw.

An aerial photograph showing these topographic features of the site and surrounding area is shown in Figure 2.2-1.

2.4 POPULATION AND LAND USAGE

2.4.1 GENERAL

The population and land usage within a 55-mile radius of the Indian Point site has been compiled by the Regional Economic Development Institute, Incorporated, under the direction of Dr. Edgar M. Hoover. The population and land usage has been projected to 1980 by the Institute based on estimates produced by the Regional Plan Association of New York. The report prepared by the Institute is included herein.

Dr. Hoover served on the staff of the Harvard University New York Metropolitan Region Study from 1957 to 1959. He is author of "Location of Economic Activity", and co-author of "Anatomy of a Metropolis" and has served in several federal government agencies including the President's Council of Economic Advisers.

The area surrounding the Indian Point site is generally residential with some large parks and military reservations. The majority of the area to the east of the river within 15 miles of the site is zoned for residential usage as shown on the map in Figure 2.4-3. West of the river within a fifteen-mile radius the Palisades Interstate Park and residential areas are the dominant land usage. The only agricultural areas within fifteen miles are south or northwest of the plant on the west side of the river.

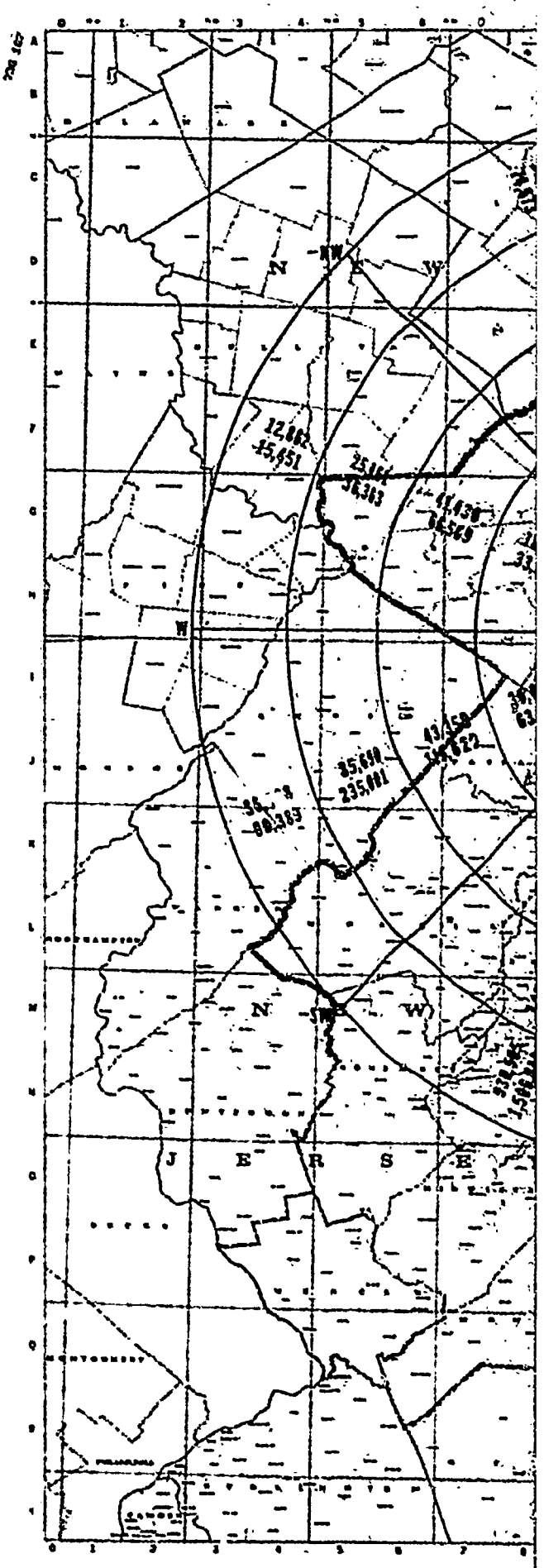
Several maps are included to illustrate the population distribution and land usage. Figure 2.4-1 and 2.4-2 show the population distribution radially by sectors out to 55 miles and 15 miles, respectively, which are based upon the report herein. Figures 2.4-3, 2.4-4, and 2.4-5 show respectively, the land usage based upon official zoning maps, areas served by public utilities, and areas served by sewage systems.

2.4.2 LOW POPULATION ZONE

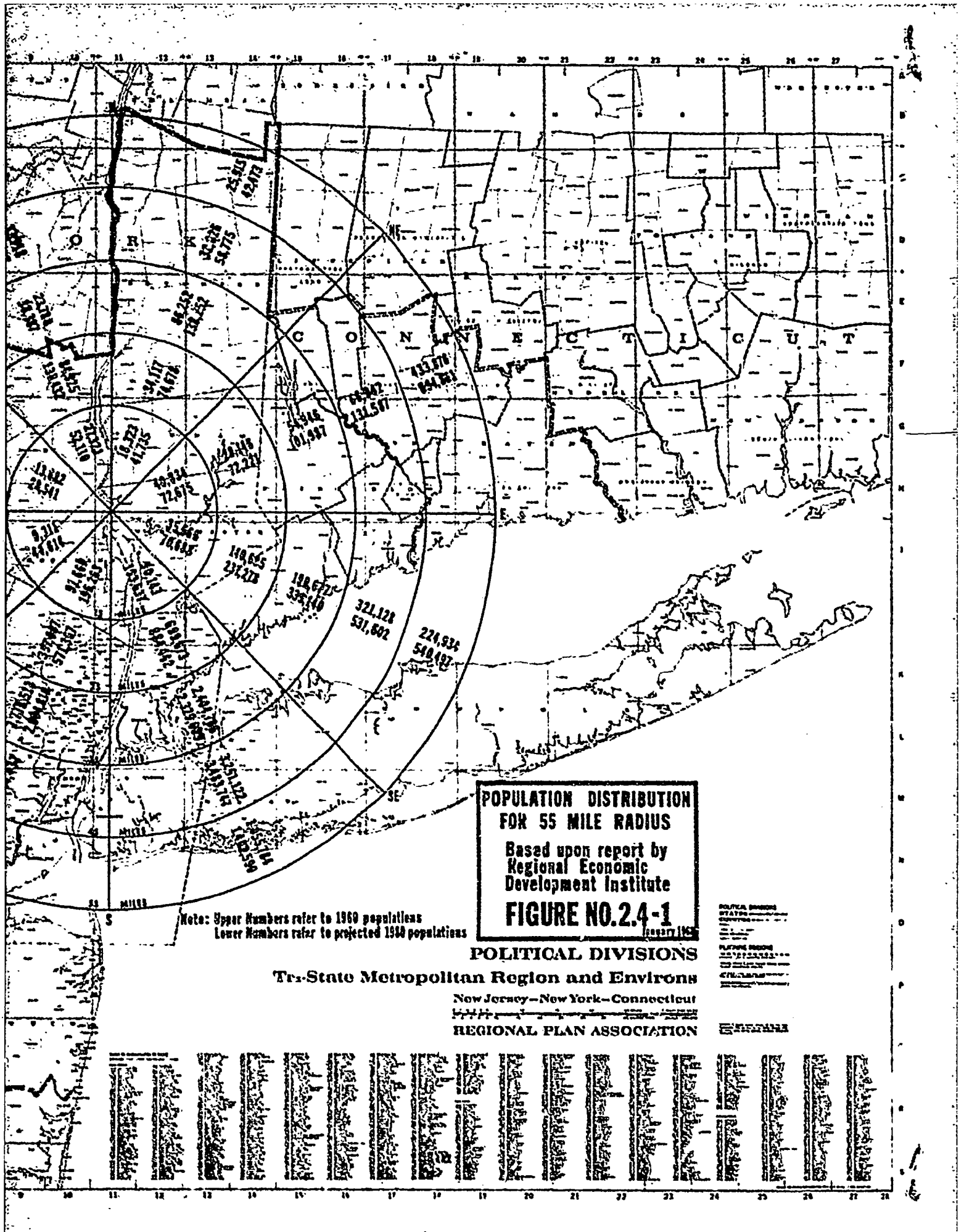
About sixty-six people reside within a 1100-meter radius of Unit No. 2, all of them to the east southeast. This distance has been used as the outer boundary of the low population zone in the analysis of a postulated fission product release. The outer boundary of the more densely populated area of Peekskill has been used as the population center distance which exceeds one and one-third times the distance from the reactor to the outer boundary of the low population zone as defined in 10 CFR 100.11.

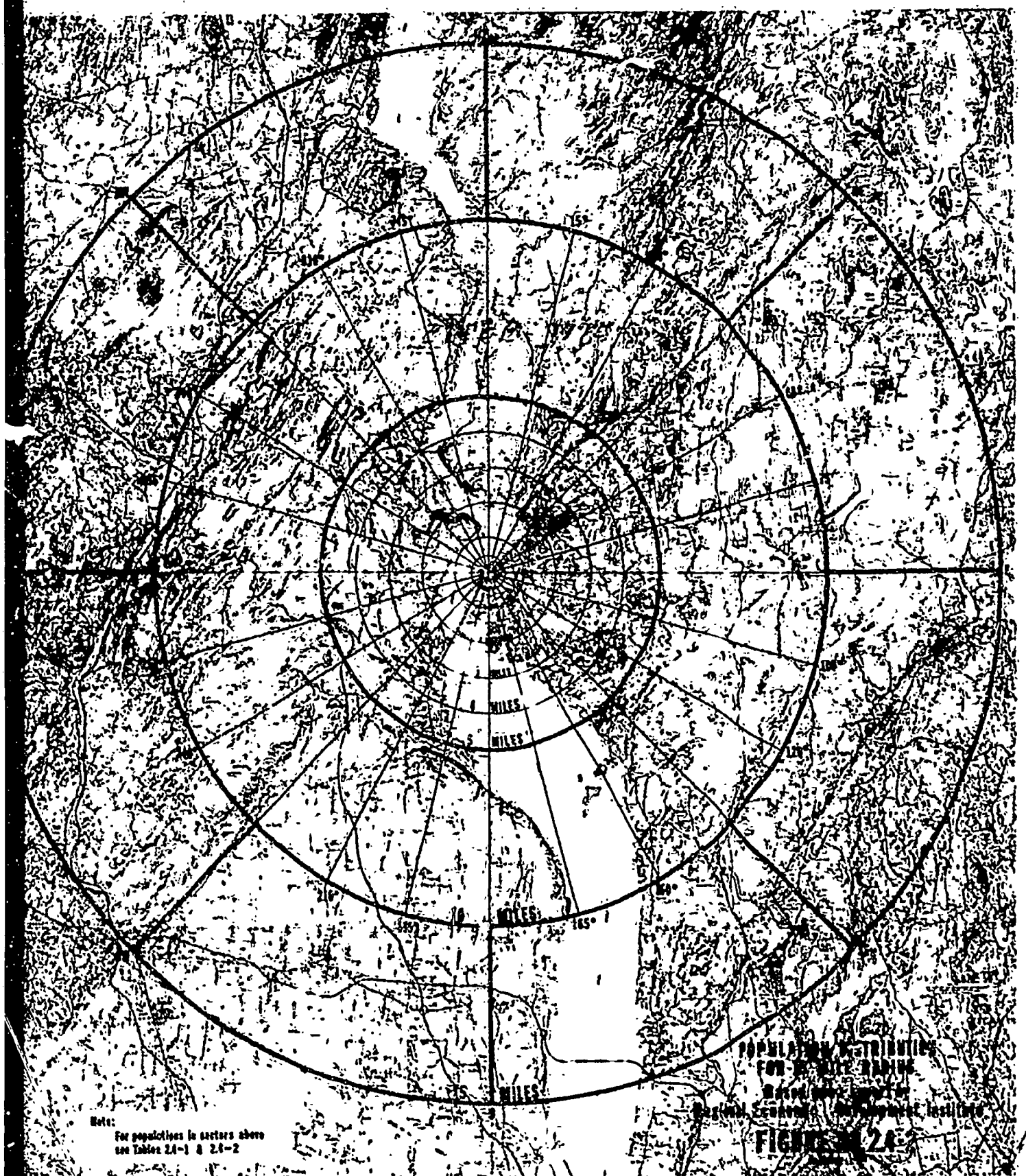
2.4.3 EXCLUSION AREA

The exclusion area for Indian Point Unit No. 2 includes plant property within a 520 meter radius of the reactor containment. An exclusion radius of 520 meters satisfies both 10 CFR 100.3 and 10 CFR 100.11.



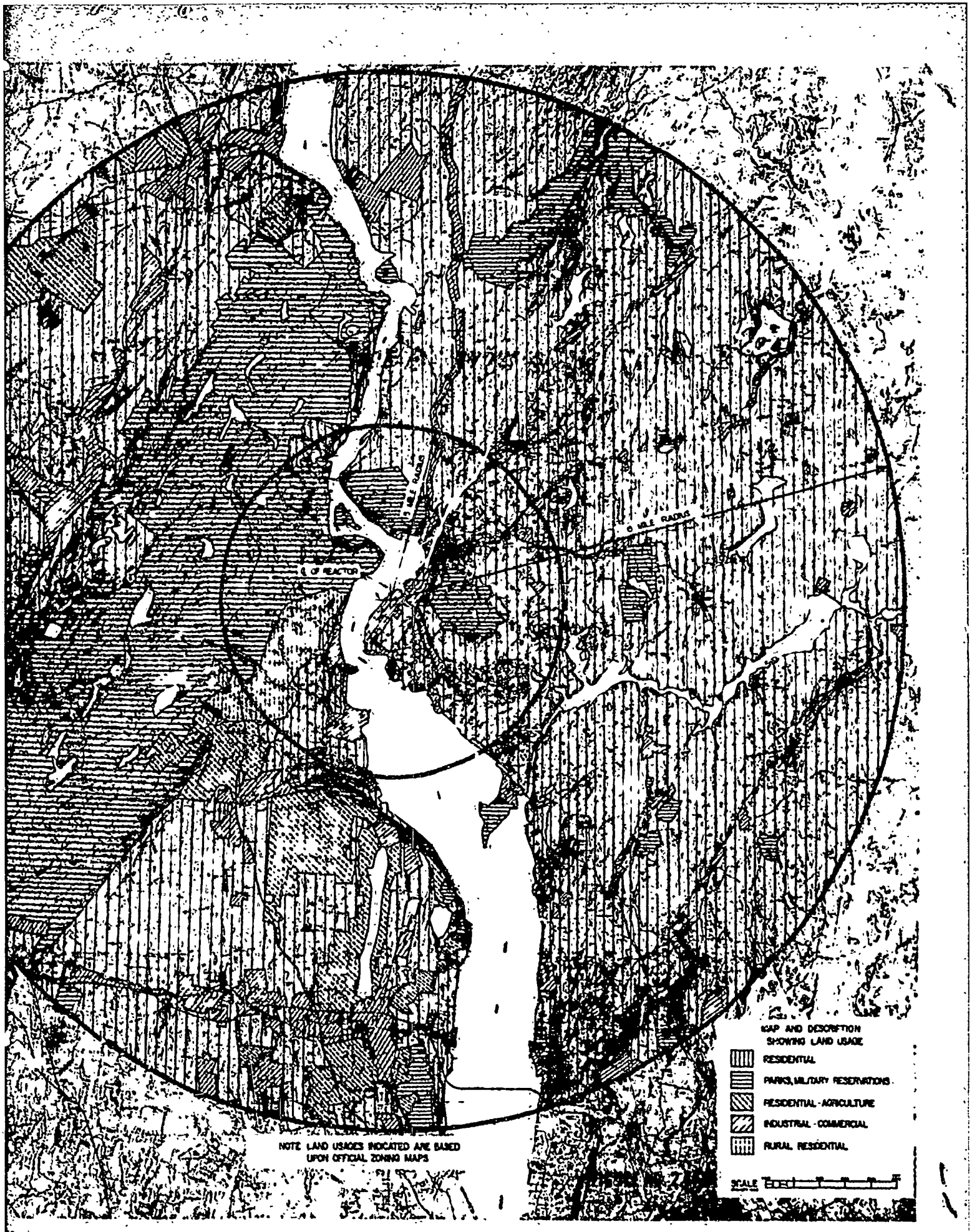
N
NW
E
W
12,862
15,451
25,161
36,383
41,430
66,569
W
35,690
235,001
43,159
N
SW
W
J
E
R
S
E


POPULATION DISTRIBUTION
FOR 55 MILE RADIUS
Based upon report by
Regional Economic
Development Institute
FIGURE NO.2.4-1
Note: Upper Numbers refer to 1960 populations
Lower Numbers refer to projected 1980 populations
POLITICAL DIVISIONS
Tri-State Metropolitan Region and Environs
New Jersey—New York—Connecticut
REGIONAL PLAN ASSOCIATION
CONNECTICUT
S
SE



Note:
For populations in sectors above see Tables 2.6-1 & 2.6-2

FIGURE 2.6-


5 MILE RADIUS
LOF REACTOR
MAP AND DESCRIPTION
SHOWING LAND USAGE
RESIDENTIAL
PARKS, MILITARY RESERVATIONS
RESIDENTIAL - AGRICULTURE
INDUSTRIAL - COMMERCIAL
RURAL RESIDENTIAL
SCALE
NOTE LAND USAGES INDICATED ARE BASED
UPON OFFICIAL ZONING MAPS

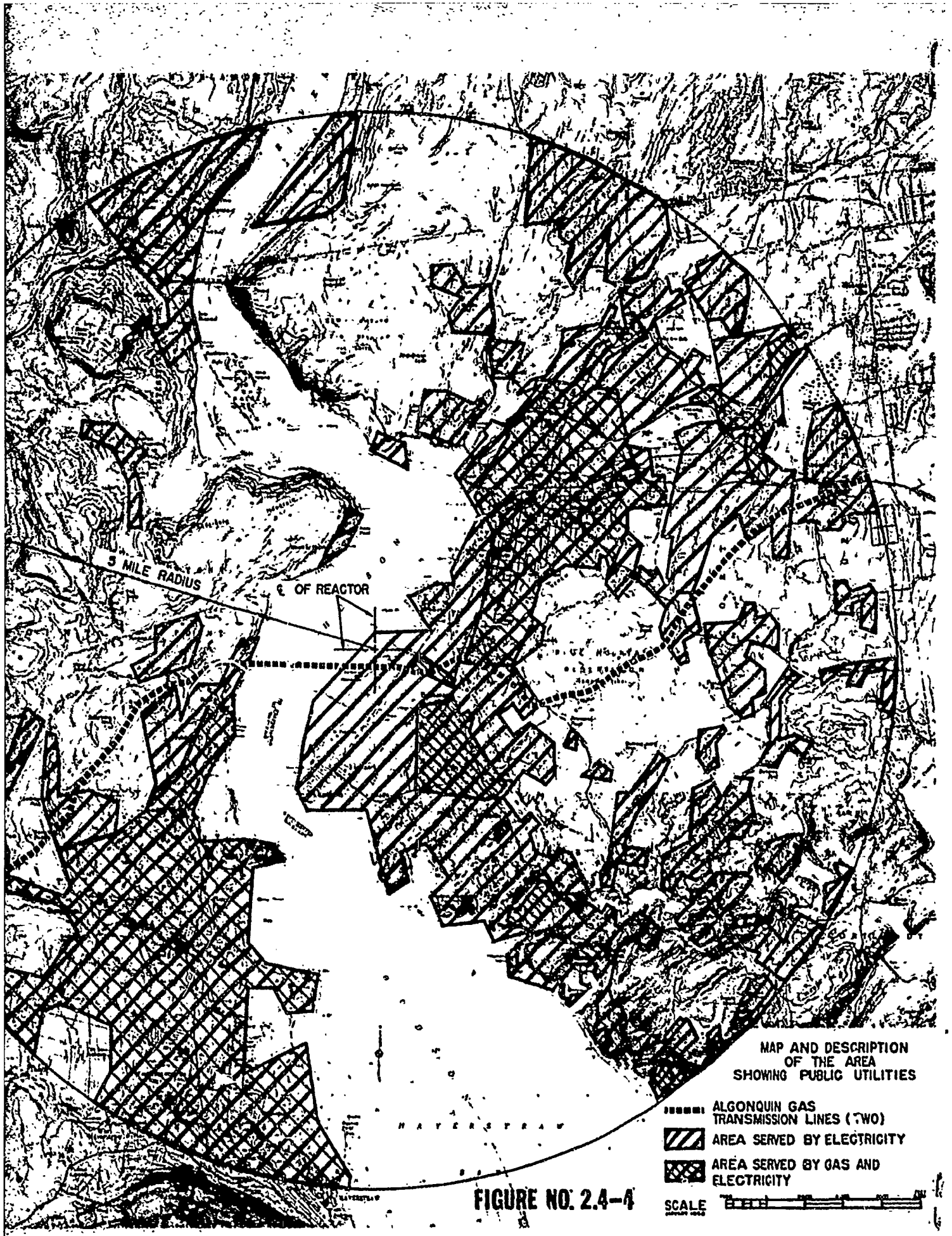
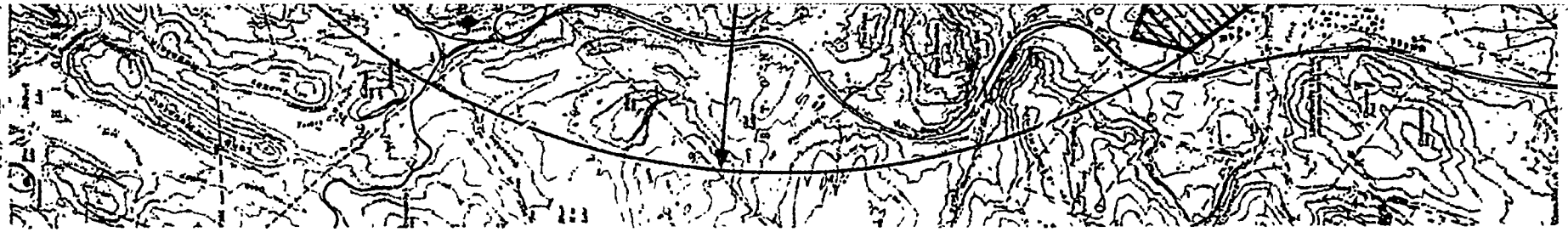


FIGURE NO. 2.4-4



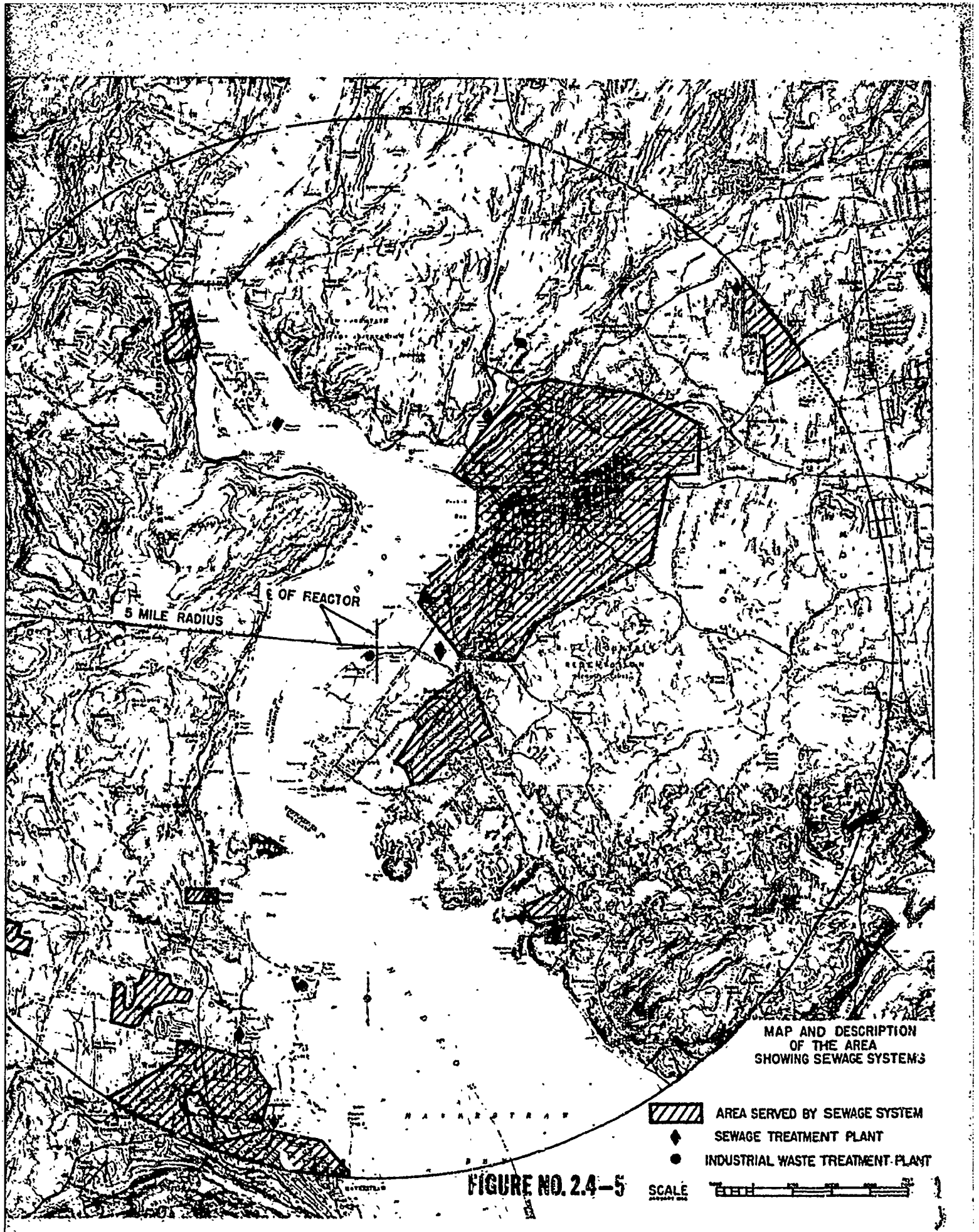


FIGURE NO. 2.4-5

REGIONAL ECONOMIC DEVELOPMENT INSTITUTE, INCORPORATED

1980 PROJECTIONS
OF
POPULATION AND LAND USE
FOR AN
AREA CIRCUMSCRIBED BY A 55-MILE RADIUS
FROM
BUCHANAN, NEW YORK

PREPARED FOR

CONSOLIDATED EDISON COMPANY OF NEW YORK, INC.

NOVEMBER 3, 1965

REGIONAL ECONOMIC DEVELOPMENT INSTITUTE, INCORPORATED

PART I

INTRODUCTION

The following two parts of this report present the results and methodology of projecting 1980 population and land use for an area circumscribed by a 55-mile radius from Buchanan, New York.

Due to the short time available to produce the projections, complete reliance was placed on the aggregate population and land use estimates produced by the Regional Plan Association of New York. These were obtained as the result of an intensive four days' perusal of their frequently revised projections, and by making extensive use of their counsel and advice.

RPA estimates for 1980 population by county were used as the limits to population growth by municipalities for the period 1960 to 1980. The 55-mile radius from Buchanan, New York, circumscribed an area which was segregated into rings-and-sectors as the following "Key to Numbering of Zones" indicates. The projections of population by municipalities constrained by RPA county estimates were then fitted to the area and totals by zones and rings-and-sectors were produced. These results appear in the following section and the methodology in the third section.

Estimates for land use in 1960 as well as projections for land use in 1980 were not available at all in county or municipal detail. Consequently, detailed land use for 1960 and projected land use in 1980 could only be controlled by average figures for land use derived from RPA estimates for some fifty

KEY TO NUMBERING OF ZONES

	Miles from Buchanan, New York				
Direction	0 - 15	15 - 25	25 - 35	35 - 45	45 - 56
N to NE	1	9	17	25	33
NE to E	2	10	18	26	34
E to SE	3	11	19	27	35
SE to S	4	12	20	28	36
S to SW	5	13	21	29	37
SW to W	6	14	22	30	38
W to NW	7	15	23	31	39
NW to N	8	16	24	32	40

municipalities, all or part of only five counties and New York City.[1]

The land use values by counties are of necessity then likely to contain error. It is believed, however, that the possible error is small and that all the values are of the correct magnitude. The results of the detailed land use estimates for 1960 and the projections for 1980 appear in the following section and the methodology appears in the third section.

Basic work sheets and data processing "print outs" and cards, including the itemization and splitting of municipalities and subsidiary projections of municipality population, have been retained by Regional Economic Development Institute, Incorporated, and are available.

This report was directed by Professor Edgar M. Hoover and produced by the research staff of the Regional Economic Development Institute, Incorporated.

PART II

RESULTS

The following tables contain 1980 projections of population for forty zones and ring-and-sector totals as well as 1980 projections of land use by county.

1980 POPULATION PROJECTIONS

Zone	Population 1960	"Extrapolative" Projection 1980	"Density" Projection 1980	"Compromise" Projection 1980	Area Square Miles
1	18,323	39,898	43,809	41,735	88.0
2	40,834	73,427	72,807	72,675	73.7
3	35,556	71,378	70,322	70,688	74.8
4	80,143	161,448	165,633	163,637	65.4
5	96,660	200,223	190,255	196,263	83.0
6	5,311	41,361	50,645	44,614	94.4
7	13,682	29,164	28,159	28,541	92.2
8	27,321	53,115	50,887	52,110	77.1
9	34,117	80,065	68,931	74,678	164.6
10	28,146	70,500	74,326	72,221	145.4
11	140,695	233,058	240,843	237,278	162.6
12	699,673	986,875	980,462	984,442	211.0
13	357,997	599,794	543,170	571,367	148.1
14	30,928	58,733	67,709	63,052	150.5
15	18,441	32,471	33,115	33,156	151.6
16	84,925	141,310	135,689	138,432	130.6
17	86,252	138,785	136,592	138,252	237.4
18	54,945	99,186	104,718	101,987	186.6
19	190,677	348,846	328,943	339,146	183.9
20	2,404,766	2,311,656	2,352,125	2,329,609	120.3
21	3,778,513	2,002,018	2,007,352	2,004,810	244.5
22	43,359	110,373	116,864	113,629	243.6
23	41,430	62,763	69,242	66,569	259.1
24	23,788	37,677	38,858	38,387	246.7
25	32,826	56,608	61,401	58,775	331.7
26	66,942	129,540	133,990	131,587	312.0
27	321,128	527,513	536,210	531,602	133.2
28	3,261,122	3,485,870	3,478,298	3,483,747	333.5
29	3,153,890	3,240,678	3,273,470	3,257,437	294.2
30	89,695	237,359	232,037	235,001	316.6
31	25,451	34,124	38,459	36,363	305.3
32	33,718	50,132	52,658	51,427	321.7
33	25,915	38,780	46,254	42,473	364.5
34	433,875	672,402	715,147	694,861	380.9
35	224,934	546,199	534,771	540,497	202.9
36	1,055,784	1,439,316	1,365,059	1,402,590	181.9
37	930,505	1,517,627	1,496,222	1,506,849	304.9
38	36,748	83,814	97,434	90,389	353.8
39	12,352	15,407	15,502	15,451	335.4
40	52,084	75,837	73,789	74,673	380.3

REGIONAL ECONOMIC DEVELOPMENT INSTITUTE, INCORPORATED

1980 "COMPROMISE" PROJECTION OF POPULATION

by Ring-and-Sector Zones

	N to NE	NE to E	E to SE	SE to S	S to SW	SW to W	W to NW	NW to N	Ring Totals
0 to 15	41,735	72,675	70,688	163,637	196,263	44,614	28,541	52,110	670,263
15 to 25	74,678	72,221	237,278	984,442	571,367	63,052	33,156	138,432	2,174,626
25 to 35	138,252	101,987	339,140	2,329,609	2,004,810	113,629	66,569	38,387	5,132,383
35 to 45	58,775	131,587	531,602	3,483,747	3,257,437	235,001	36,363	51,427	7,785,939
45 to 55	42,473	694,861	540,497	1,402,590	1,506,849	90,389	15,451	74,673	4,367,783
Sector Totals	355,913	1,073,331	1,719,205	8,364,025	7,536,726	546,685	180,080	355,029	20,130,994

8-d

REGIONAL ECONOMIC DEVELOPMENT INSTITUTE, INCORPORATED

1960 POPULATION

by Ring-and-Sector Zones

	N to NE	NE to E	E to SE	SE to S	S to SW	SW to W	W to NW	NW to N	Ring Totals
0 to 15	18,323	40,834	35,566	90,143	91,660	9,311	13,682	27,321	326,840
15 to 25	34,117	28,146	140,695	699,673	357,097	30,028	18,441	84,925	1,393,122
25 to 35	86,252	54,946	190,677	2,404,756	1,776,513	43,359	41,430	23,788	4,623,731
35 to 45	32,826	66,942	321,128	3,261,122	3,153,690	85,690	25,461	33,718	6,980,577
45 to 55	25,915	433,876	224,934	1,055,784	930,305	36,768	12,862	52,084	2,773,128
Sector Totals	197,433	624,744	913,000	7,511,488	6,311,865	205,156	111,876	221,836	16,097,398

REGIONAL ECONOMIC DEVELOPMENT INSTITUTE, INCORPORATED

TABLE 1

Counties in Con Ed Study Area				1	2	3	4	5	6	7	8	9
State	In RPA Region	Outside RPA Region	Percent In Con Ed Area	Equation 1 I 1960 Projection Sq. Miles	Averaging From I 1960 RPA Total Estimate	Best Estimate I 1960	D 1980 From RPA Population Estimate	Equation 1 I 1980 Projection Sq. Miles	Equation 2 I 1980 Projection Sq. Miles	Best Estimate I 1980 From Equations 1 & 2	Adjusted to Best Estimate I 1980 RPA Total	Square Miles Total Area
Conn.	Fairfield		100	109	131	96	1666	141	126	141	216	633
		Litchfield	52			[29]	[201]	[36]			[36]	938 (49)
		New Haven	71			[77]	[1997]	[107]			[107]	610 (433)
N. J.	Bergen		100	75	68	75	4635	90	92	92	140	233
	Essex		100	62	48	73	7812	65	59	65	99	128
	Hudson		100	31	15	31	12222-	29	18	31*	31*	45
	Middlesex		46	63	59	59	3067	97	83	97	148 (68)	913 (150)
	Morris		100	58	73	53	1530	100	87	100	153	468
	Passaic		100	49	43	49	2861	58	41	58	89	194
	Somerset		33	35	44	35	1691	55	44	55	84 (28)	307 (101)
		Sussex	100			[26]	[216]	[42]			[42]	528
	Union		100	41	41	50	5825	45	49	49	75	103
		Warren	13			[3]	[287]	[4]			[4]	361 (47)
N. Y.	Dutchess		100	61	37	55	368	82	57	82	125	816
	Nassau		100	110	113	133	5119	119	177	177	271	293
	Orange		100	63	69	63	374	85	45	85	130	829
	Putnam		100	14	24	18	471	27	28	28	43	234
	Rockland		100	27	46	34	1899	43	26	42	65	179
	Suffolk		71	133	146	133	1830	216	161	216	329 (234)	921 (654)
		Sullivan	56			[18]	[52]	[22]			[22]	986 (552)
		Ulster	88			[51]	[161]	[65]			[65]	1143 (1006)
	Westchester		100	101	99	101	2690	126	104	126	193	435
	Bronx		100	38	20	29	32143-	29*	29*	29*	29*	42
	Kings		100	68	33	49	35870-	49*	49*	49*	49*	69
	New York		100	20	11	16	68182-	16*	16*	16*	16*	22

REGIONAL ECONOMIC DEVELOPMENT INSTITUTE, INCORPORATED

TABLE 1 continued...

Counties in Con Ed Study Area				1	2	3	4	5	6		8	9
State	In RPA Region	Outside RPA Region	Percent In Con Ed Area	Equation 1 I 1980 Projection Sq. Miles	Averaging From I 1960 RPA Total Estimate	Best Estimate I 1960	D 1980 From RPA Population Estimate	Equation 1 I 1980 Projection Sq. Miles	Equation 2 I 1980 Projection Sq. Miles	Best Estimate I 1980 From Equations 1 & 2	Adjusted to Best Estimate I 1980 RPA Total	Square Miles Total Area
N. Y.	Queens		100	80	52	76	17661	53	102	76*	76*	109
	Richmond		100	20	20	20	7759	30	30	30	46	58
Pa.		Pike	65			[7]	[22]	[8]			[8]	545 (354)
	TOTAL RPA REGION			1258	1192	1248		1595	1423	1645	2408	
	CONTROL TOTALS			1248	1248	1248		2408	2408	2408	2408	
	TOTAL INTENSIVE LAND USE IN CONSOLIDATED EDISON AREA					1459					2461	

NOTES: Figures in [ ] are for counties outside RPA's Region and are only added in Column 8.

Figures with * are for counties whose population density declined or projection produced over 100 percent of land used intensively. In such cases, 1960 estimates of land use intensity were used for 1980.

Figures in ( ) in Column 8 are square miles of intensively used land in the Consolidated Edison Area for those counties which are not 100 percent within that area. Those in ( ) in Column 9 are the total square miles of the county in the Consolidated Edison Area.

REGIONAL ECONOMIC DEVELOPMENT INSTITUTE, INCORPORATED

TABLE 2

ESTIMATED LAND USE 1960 AND PROJECTED LAND USE 1980[1]

	INTENSIVE 1960 & 1980			NON-INTENSIVE 1960			NON-INTENSIVE 1980					
	1	2	3	4	5	6	7	8	9	10	11	12
	Residential	Industrial/ Commercial	Total	Institutional and Park	Public Rights of Way	Total	Community Facilities & Institutions	Parks & Recreation	Public Rights of Way	Total	Open	Grand Totals
1960												
Square Miles	1032	216	1248	696	418	1114					4062	6424
Percentage of Total Developed Land	43	9	52	29	19	48						
High	58	12		45	22							
Low	32	2		15	13							
1980												
Square Miles	2040	368	2408				876	784	682	2342	1674	6424
Percentage of Total Developed Land	43	8	51				19	16	14	49		
1960 - 1980												
Square Miles of Land to be Developed	1400	220	1620							1228		
Percentage of Total Land to be Developed			58							42		

1. The averages were derived from the data in "Table 3. The Use of Developed Land in Selected Areas of the Region." RPA Bulletin Number 100, Page 21, September, 1962. The data for square miles excludes Monmouth County from the original RPA totals.

REGIONAL ECONOMIC DEVELOPMENT INSTITUTE, INCORPORATED

TABLE 3

LAND USE BY COUNTY 1954 AND 1960 IN SQUARE MILES

Counties in Con Ed Study Area			1954			1960				
						INTENSIVE		LOW INTENSIVE		
State	In RPA Region	Outside RPA Region	Intensive	Low Intensive	Open Land	Residential	Industrial/ Commercial	Institutional and Park	Public Rights of Way	Open
Conn.	Fairfield		30	13	530	80	16	64	43	420
		Litchfield				[24]	[5]	[2]	[3]	[15]
		New Haven				[64]	[13]	[43]	[28]	[285]
N. J.	Bergen		70	10	145	62	13	19	13	126
	Essex		57	19	57	61	12	7	4	44
	Hudson		21	6	18	26	5	2	1	11
	Middlesex		49	16	248	49	10	31	20	203
	Morris		46	23	399	44	9	50	33	332
	Passaic		33	43	118	41	8	17	12	116
	Somerset		23	7	277	29	6	33	22	217
		Sussex				[22]	[4]	[60]	[40]	[402]
	Union		43	15	45	43	7	6	4	43
		Warren				[3]	[1]	[5]	[3]	[34]
N. Y.	Dutchess		39	5	772	46	9	92	61	608
	Nassau		136	38	119	114	19	78	13	69
	Orange		23	66	740	52	11	92	61	613
	Putnam		14	1	219	15	3	26	17	173
	Rockland		14	51	114	28	6	30	12	103
	Suffolk		119	79	723	110	23	95	63	630
		Sullivan				[15]	[3]	[64]	[43]	[427]
		Ulster				[42]	[9]	[115]	[76]	[764]
	Westchester		79	79	277	83	18	40	27	267

REGIONAL ECONOMIC DEVELOPMENT INSTITUTE, INCORPORATED

TABLE 3 continued...

LAND USE BY COUNTY 1954 AND 1960 IN SQUARE MILES

Counties in Con Ed Study Area			1954			1960				
						INTENSIVE		LOW INTENSIVE		
State	In RPA Region	Outside RPA Region	Intensive	Low Intensive	Open Land	Residential	Industrial/ Commercial	Institutional and Park	Public Rights of Way	Open
N. Y.	Bronx		31*	7	4	25	4	2	1	10
	Kings		34	4	31	42	7	2	2	16
	New York		20*	2	0	14	2	2	3	1
	Queens		98*	10	1	65	11	4	3	26
	Richmond		29*	8	21	17	3	5	3	30
Pa.		Pike				[6]	[1]	[42]	[28]	[377]
	TOTAL RPA REGION					1046	202	697	418	4068
	CONTROL TOTALS					1032	216	696	418	4062
	TOTAL CONSOLIDATED EDISON AREA					1221	230	1028	639	6272

NOTES: Figures with * are for 1954 land use in New York City. They were not used for 1960 and 1980 because the data for 1960 was assumed to be more discrete.

Figures in [ ] are for those counties outside RPA's Region. Th y are added in to the total for Con Ed's area.

REGIONAL ECONOMIC DEVELOPMENT INSTITUTE, INCORPORATED

TABLE 3, PART TWO

LAND USE PROJECTION BY COUNTY FOR 1980 IN SQUARE MILES

Counties in Con Ed Study Area			INTENSIVE		LOW INTENSIVE			
State	In RPA Region	Outside RPA Region	Residential	Industrial/ Commercial	Community Facilities & Institutions	Parks & Recreation	Public Rights of Way	Open
Conn.	Fairfield		183	33	92	83	71	171
		Litchfield	[30]	[6]	[3]	[3]	[2]	[5]
		New Haven	[88]	[19]	[72]	[65]	[55]	[134]
N. J.	Bergen		118	22	20	19	16	38
	Essex		83	16	6	6	5	12
	Hudson		26	5	3	3	2	6
	Middlesex		136 (58)	22 (10)	18	16	14	34
	Morris		130	23	69	63	54	129
	Passaic		75	14	23	21	18	43
	Somerset		71 (24)	13 (4)	16	15	12	30
		Sussex	[34]	[8]	[107]	[97]	[83]	[199]
	Union		63	12	6	6	5	11
		Warren	[3]	[1]	[9]	[9]	[7]	[18]
N. Y.	Dutchess		106	19	152	138	117	283
	Nassau		230	41	5	4	4	9
	Orange		110	20	154	140	119	286
	Putnam		37	6	42	38	32	79
	Rockland		56	10	25	23	19	46
	Suffolk		279 (199)	50 (35)	92	84	72	172
		Sullivan	[18]	[4]	[117]	[106]	[90]	[217]
		Ulster	[53]	[12]	[207]	[188]	[160]	[386]
	Westchester		162	31	53	48	42	99
	Bronx		25	4	3	3	2	5

REGIONAL ECONOMIC DEVELOPMENT INSTITUTE, INCORPORATED

TABLE 3, PART TWO continued...

LAND USE PROJECTION BY COUNTY FOR 1980 IN SQUARE MILES

Counties in Con Ed Study Area			INTENSIVE		LOW INTENSIVE			
State	In RPA Region	Outside RPA Region	Residential	Industrial/ Commercial	Community Facilities & Institutions	Parks & Recreation	Public Rights of Way	Open
N. Y.	Kings		42	7	4	4	4	8
	New York		14	2	1	1	1	3
	Queens		65	11	7	7	6	13
	Richmond		39	7	3	2	2	5
Pa.		Pike	[7]	[1]	[76]	[69]	[59]	[142]
	TOTAL RPA REGION		2040	368	794#	724#	617#	1482#
	CONTROL TOTALS		2040	368	876	784	682	1674
	TOTAL CONSOLIDATED EDISON AREA		2078	383	1385	1261	1073	2583

NOTES: Total RPA Region figures followed by # indicate that only the portion of the counties in Con Ed's area are included. This explains why these figures are further from the control total figures than in previous cases.

Figures in [ ] are for those counties outside RPA's Region. They are added in to the total for Con Ed's area.

PART III A

PROJECTIONS OF POPULATION, AND DISTRIBUTION BY ZONES

Our summary tables show the distribution by 40 zones (eight sectors in each of five concentric rings) of the 1960 population and of the 1980 population projected according to three different techniques. In this section we explain (1) how the projections were made, and (2) how both the 1960 and the projected 1980 populations were allocated among the 40 prescribed zones.

1. The Projections

We started from projected totals for each of the twenty-nine counties that lie wholly or partially within the 55-mile circle. All three of our projections agree in the total for each county. This starting-point was adopted because of the availability of a rather recent set of county population projections for 1980, prepared by the Regional Plan Association of New York and representing the outcome of extended and intensive study of the New York metropolitan region and its growth patterns. The RPA projections represent a careful revision and improvement of earlier projections made by Harvard University's New York Metropolitan Region Study in 1958-1959; the revisions take into account the findings of the 1960 Census and other subsequent materials. They are also tied in closely with the RPA's analysis of present and prospective land use in the area.

Consequently, we adopted the Regional Plan Association's county total for 1980, for each of the counties in which such an RPA figure is available. Our projections for individual municipalities were controlled so as to add up to the RPA total in each county.

The 55-mile circle includes some territory beyond RPA's sphere of analysis: Warren and Sussex Counties in New Jersey, and fractions of a half-dozen other counties in Connecticut, New York State, and Pennsylvania. For these additional counties and part-counties, we prepared our own projections--based on relative growth rates in 1950-1960 and 1960-1965, the projected 1960-1980 growth rates available from RPA for adjoining and roughly similar counties, and RPA's broad indications of the directions of most rapid outward spread of metropolitan growth in the next decade or two. These projections are of course crude compared with RPA's; but even fairly substantial errors in them will not be likely to distort the final results very greatly, since the population involved is only about five percent of the total within the 55-mile circle.

The next step was to disaggregate the twenty-nine counties into their 500-odd component municipalities. As simple rules for deciding how fast individual municipalities would grow relative to their counties, we used two different principles in separate projections, and then combined them in a third projection.

A. Extrapolative Projection. Our first set of projections is based on the assumption that places which grew faster than their counties in the 1950's will continue to do so, and that those which lagged behind their counties' growth in the 1950's will continue to lag in the 1960's and 1970's. Specifically, each municipality's population was first projected linearly to 1980, simply extending the 1950-1960 growth rate for another twenty years. This extrapolation was done both on a geometric basis (using the 1950-1960 percentage increase per annum) and

an arithmetic basis (using the 1950-1960 increase rate in persons per annum). The latter was adopted after inspection of the results, since it produced fewer cases that were obviously absurd[1] and also because the projected growth for the area as a whole involves a somewhat smaller percentage rate per annum between 1960 and 1980 than was registered in the 1950's.

The extrapolated figures for all the municipalities in each county were then totaled, and adjusted up or down pro rata to make the total conform to the RPA or other total already established for the county.

B. Density-based Projection. One quite obvious shortcoming of the extrapolative technique just described is that it takes no account of restraints upon growth arising from the filling-up of developable space. We sought, therefore, to develop an alternative set of projections which would incorporate the hypothesis that percentage rates of growth of individual communities slacken off with higher population densities per square mile.

Some preliminary investigations into data for selected counties were made to see if the posited inverse relation of growth rate to density is actually present to a sufficiently significant degree to make it a useful projection guide. These investigations disclosed a marked relationship of the expected sort in the 1950-1960 growth and density rate for municipalities. It appeared also that a straight-line regression relation between the logarithms of (1) the growth ratio and (2)

1. For example, take the not unrealistic case of a small, suburban community that grew from 100 in 1950 to 2,000 in 1960. Extending that rate of percentage increase would give a projected 1980 population of 800,000! The less-exuberant arithmetic extrapolation, in the same instance, would give only 5,800 for 1980.

the density of population at the beginning of the time interval provided a more appropriate formulation than a simple linear relation.

Accordingly, the data on 1950 and 1960 population and land area for all municipalities were processed so as to yield a statistically-fitted regression formula for municipalities in each county, relating rate of population growth to density of population. In fitting the equation, the data for individual municipalities were weighted according to population, giving larger places a proportionately greater influence on the formula.

These regression formulas were, in all but a few counties, associated with a high enough degree of correlation to leave no doubt about the usefulness of this approach as a guide to relative rates of expected population growth. With one unimportant exception (Sullivan County), the relationships were consistently inverse in direction as would be expected (i. e., higher density was associated with lower growth rates in any given county). In the Sullivan County case, the correlation was too small to make its size or direction significant.

The projections produced by this method for individual municipalities were then totaled for each county and adjusted to make the county totals conform to the RPA or other total already established for that county, just as was done with the first or extrapolative set of projections.

C. Compromise Projection. The two alternative sets of projections just described rest on entirely different principles, each of which (the continuity of growth differentials, and the inverse relation of growth rate to density) has demonstrable validity but falls short of complete adequacy. Consequently, it

seemed appropriate to combine the two types of projection into a third, incorporating both the continuity and the density effects.

To get this third set, labeled "compromise" projections, we took for each municipality the geometric mean between the adjusted extrapolative and the adjusted density-based projection. Then the "compromise" projections for the municipalities of each county were added up and adjusted to make the county total conform, as in the two previous cases, to the RPA or other total already established for the county.

The averaging procedure allows each of the two effects (growth-continuity and density) to exert an effect on the compromise projections. Where the extrapolative and the density-based projections were in close agreement, they reinforce each other in projecting differentiation of growth rates in different parts of a county; where the extrapolative and density-based projections give sharply differing answers, they tend to cancel one another out in terms of such differentiation, leading to compromise projections which show relatively little dissimilarity in growth rates among the parts of a county. This seems appropriate--where the two approaches we have tried give very different results, we are well advised to take both of them less seriously and have less occasion for diverging very far from the simple assumption that all parts of any given county will grow at equal rates.

The geometric mean was chosen in preference to the arithmetic as a way of still further toning-down the most extreme variations of growth rates.

It seems to us that the compromise projections are the "best" of the three so far as anyone can judge in advance. However, all three sets of projections are

presented in equal detail so that users of these reported results may make their own evaluations and decisions.

2. ALLOCATION TO RING AND SECTOR ZONES

Each of the three sets of projections by municipalities (and also the actual 1960 populations of municipalities then had to be translated into population totals for the 40 ring-and-sector zones stipulated.

A little more than half of the municipalities were found to lie wholly within one of the 40 zones. Each of the rest had to be split, with portions allocated to anywhere from two to six different zones. In view of the large number of cases involved (more than 250 municipalities,) to be split into more than 600 fractional parts, the splitting was done by inspection of a map showing the municipalities and zone boundaries, the proportional division of the municipality's area among continguous zones was estimated visually, and in most cases it was assumed that the same proportionate split would also apply to that municipality's population. Whenever feasible, however, and particularly where places of significant size were concerned, account was taken of the location of major concentrations of population within the municipality boundaries.

It was assumed that the same split-up of a municipality would apply in the case of each of the three projections, and also for the 1960 population.

The derivation of 1960 and projected population totals for each of the 40 zones was carried on in an electronic computer, which also provided totals in each case for each ring and each sector of the grid.

PART III B

PROJECTIONS OF LAND USE BY COUNTY, 1980

The basic theory of the projection technique used to disaggregate RPA gross estimates of land use is that in each county the "intensive use ratio" (I) defined as the percentage of total land area used for residential, commercial, and industrial purposes,[1] is a function of the population density (D) for that county. This relationship is demonstrated in the following Graph which plots the logarithms of I 1954 (percentage of land in intensive use in 1954) against the logarithms of D, where D is the average of 1950 and 1960 population densities for each of twenty-one counties in the RPA Region.[2]

A statistical test of the relationship between I and D was then produced by fitting a regression equation of the form $\log Y = a + b \log X$ to the scatter of points in the Graph. The fitted equation,[3] $\log I\ 1954 = -.361 + .531 \log D$ (Equation 1) has an R^2, or coefficient of determination, equal to .931. This means that 93.1 percent of the variation in the percentage of land used intensively is explained by population density. Equation 1 is the basis for disaggregating by

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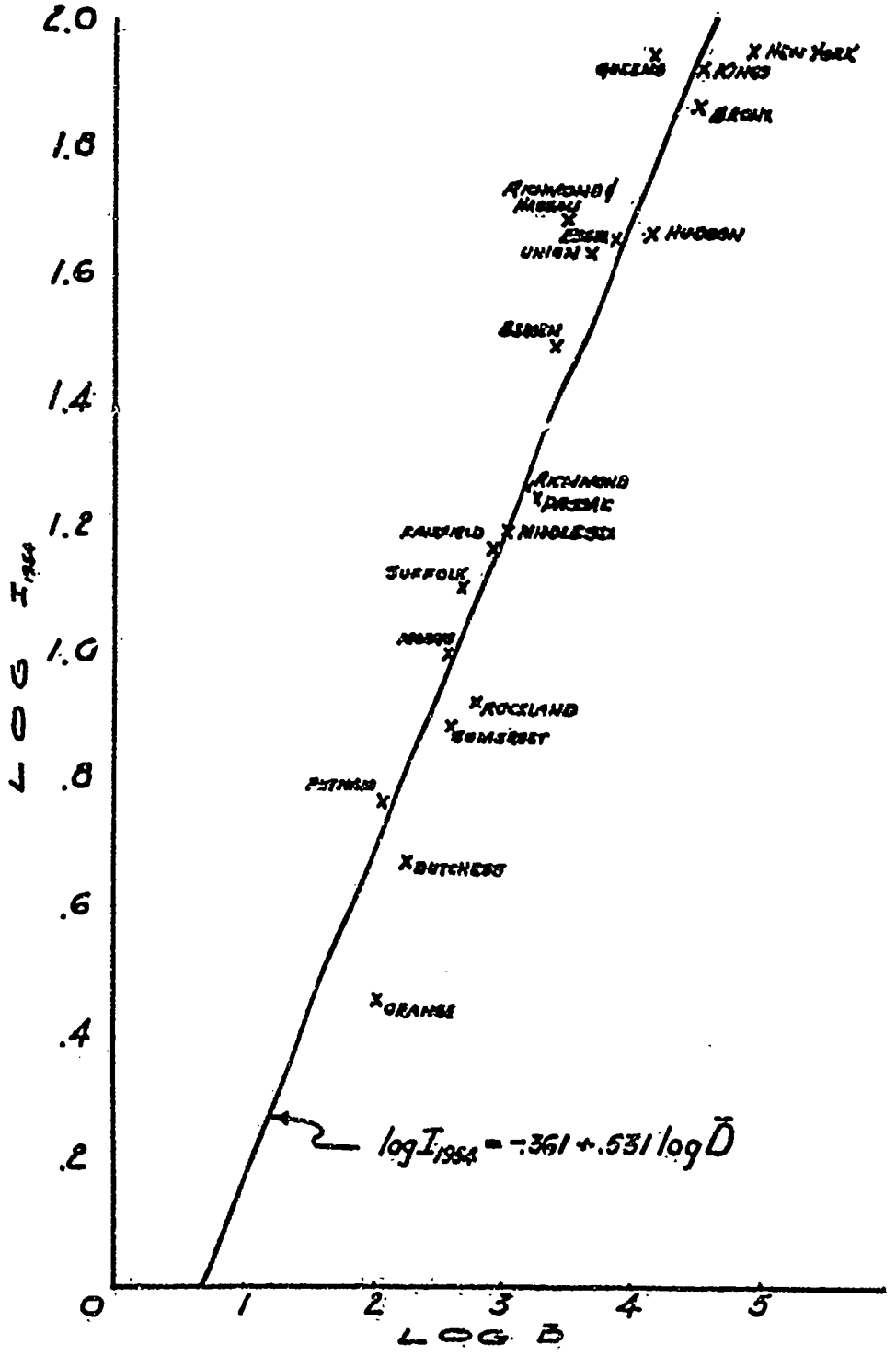
1. This definition of intensive land development is the one used by the Regional Plan Association to describe "Table 20. Land Development by County in 1954, RPA Bulletin Number 87, page 31, June, 1957.
2. Sources of Data: "Table 20. Land Development by County 1964," RPA Bulletin Number 87, page 31, June, 1957, and "Table 4. The Region's Population, 1860 to 1960, by County, RPA Bulletin Number 100, page 36 (Appendix), September, 1962.
3. An explanation of the regression technique used may be found in most general statistical research text books, such as Ferber and Verdoorn, "Research Methods in Economics and Business," New York, 1962.

GRAPH


2.0
1.8
1.6
1.4
1.2
1.0
.8
.6
.4
.2
0
1
2
3
4
5
LOG I_{1954}
LOG B
NEW YORK
BRONX
HUDSON
UNION
RICHMOND
PASSAIC
SUFFOLK
ROCKLAND
SOMERSET
BUTCHESS
ORANGE
$\log I_{1954} = -.361 + .531 \log \bar{D}$

counties RPA's gross land use estimates for its region. Land use values for counties not in RPA's Region were also obtained from Equation 1, but no control or scale factor was available.

Application of this relationship yielded the estimates of square miles of intensively used land by county in '960, in Column 1 of Table 1. A second estimate of intensive land use by county in 1960 was obtained by applying an average value of 52 percent for intensive land use to RPA estimates of 1960 "committed" land.[4] This second set of estimates (Column 2 in Table 1) permits the projection estimates from Equation 1 to be judged against a control total derived from the RPA estimates for committed land. Of the total 2, 400 square mi' imitted land in RPA estimates for 1960, 1, 248 square miles were dev. ed intensively. The 1, 258 square miles of intensively used land projected by Equation 1 is within one percent of this control total. Column 3 represents the results of an adjustment in intensively used land for those counties in Column 2 which suffered most from averaging. The adjustment was made by applying the high (low) intensity percentage for counties closer to (farther from) the core of RPA's Region.[5] This correction brought the total for the second estimate to that of the control total. The best estimate of square miles of intensively used land by counties in Column 3, Table 1, were then

---

4. The average value of 52 percent was derived from 1960 RPA estimates of committed land used intensively in selected areas of the region. See Table 2, Column 3.
5. See Table 2 for high and low values of intensive land use ratio. Also "The Region's Rings of Development," RPA Bulletin Number 100, page 6, September, 1962.

disaggregated into residential and commercial/industrial uses by applying average figures of 83 and 17 percent, respectively, which were derived from RPA aggregate estimates. The results appear in Table 3.

With this foundation, the next step was the projection of land-use intensity in 1980 by means of Equation 1. The population density for 1980 used in the projection came from RPA estimates of 1980 population by county for those counties in RPA's region.[6] (Population density for 1980 for counties not covered by RPA were obtained by extrapolating population for 1980 from that in 1950 and 1960.)[7] The results in square miles of this projection appear in Column 5 of Table 1.

Projecting by Equation 1 requires that points in the Graph far from the regression line will be on the regression line twenty years hence. Thus, an alternative form of this projecting technique embodying the same relationship was also used. These alternative-form projections, which appear in Column 6 of Table 1 were produced by

$$\log \bar{I}_{1980} = \log I_{1954} + .531 (\log D_{1980} - \log D \quad \text{(Equation 2)}$$

which uses the difference in population density between 1980 and the average of 1950-1960 to project the difference in the intensive land use ratio between 1980 and 1954. While it is felt that neither set of the projected intensive land use

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6. "Table 5. The Region's Projected Population, 1965 to 1985, by County," RPA Bulletin Number 100, page 36 (Appendix), September, 1962.
7. County and Data Book, 1962, United States Department of Commerce, Bureau of the Census, Washington, 1962.

figures are any better than could possibly be obtained from the existing data, they can be viewed with some assurance that they are of the correct magnitude.

The Regional Plan Association estimated that 2,500 more square miles of land will be developed between 1960 and 1985.[8] Assuming that this is done at an even rate over the 25-year period, about 2,000 square miles will be developed between 1960 and 1981. RPA estimates that 58 percent of this land will be developed in intensive uses.[9] Thus, 1,160 square miles of intensively developed land could be added by 1980 according to the estimate derived from RPA data. Total land used intensively in the region could reach 2,408 square miles as shown in Table 2.

With this new control total for total intensively used land in 1980, the "best" disaggregated projection from Equations 1 and 2 was adjusted by a factor of 1.529 to coincide with RPA estimates. The "best" disaggregated projected value for intensively used land in each county is given in Column 7 of Table 1. Selection of the value for this column from the values produced by Equations 1 and 2 in Columns 5 and 6, respectively, was made by taking the larger value in all cases except Hudson County and New York City. In the cases of Hudson County and New York City, Equations 1 and 2 gave smaller projected values because population is expected to decline; but, even with slight population decline, it is reasonable to assume that the percentage of land used intensively should remain

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8. "Chart 14. Extent of Land Development in the Region, 1960 and 1985." RPA Bulletin Number 100, page 20. An adjustment has been made for excluding Monmouth County in our projections.
9. Ibid.

at least as high as in 1960, so the 1960 figure is retained for 1980.

The further disaggregation of intensive land use in 1980 by county was produced by applying to the difference in land use intensity between 1960 and 1980 average values of 83 percent for residential and 17 percent for commercial/industrial uses, which were derived from RPA estimated values for the region as a whole. The resultant increment in residential and commercial/industrial usage was added to the values obtained in similar fashion for 1960; and the totals for 1980 are given in Table 3, Part Two.

TABLE 1.4-1

1960 POPULATION BY RING AND SECTOR ZONES FOR 15 MILE RADIUS

	0 to 1/2	1/2 to 1	1 to 2	2 to 3	3 to 4	4 to 5	5 to 10	10 to 15
N to 15°	0	0	0	60	60	0	1210	
15° to 30°	0	0	50	130	120	710	610	
30° to NE	0	0	280	5650	1000	890	4070	
N to NE								3490
NE to 60°	0	0	3930	7460	630	1100	2770	
60° to 75°	0	10	2050	1610	1470	1460	3690	
75° to E	0	10	270	180	140	250	5080	
NE to E								9720
E to 105°	0	300	130	50	330	90	2170	
105° to 120°	46	54	270	70	540	540	1720	
120° to SE	0	80	330	50	270	320	2270	
E to SE								25940
SE to 150°	0	320	630	310	410	300	23?40	
150° to 165°	0	50	760	1900	20	0	1010	
165° to S	0	30	350	50	0	0	8300	
SE to S								51160
S to 195°	0	160	250	0	50	300	19200	
195° to 210°	0	0	1000	20	300	3800	6300	
210° to SW	0	0	50	530	3200	1000	8500	
S to SW								47000
SW to 240°	0	0	20	350	160	50	1000	
240° to 255°	0	0	150	120	40	100	0	
255° to W	0	0	20	150	120	0	0	
SW to W								7030
W to 285°	0	0	20	30	0	0	90	
285° to 300°	0	0	40	0	0	0	1030	
300° to NW	0	0	60	50	120	0	150	
W to NW								12090
NW to 330°	0	0	20	0	40	260	100	
330° to 345°	0	20	0	50	80	3100	780	
345° to N	0	0	50	0	0	40	7880	
NW to N								14990

NOTE: The populations above refer to Fig. No. 1.4-2

TABLE 1.4-2

1980 "COMPROMISE" PROJECTION OF POPULATION BY RING AND SECTOR ZONES FOR 15 MILE RADIUS

	0 to 1/2	1/2 to 1	1 to 2	2 to 3	3 to 4	4 to 5	5 to 10	10 to 15
N to 15°	0	0	0	140	140	0	2750	
15° to 30°	0	0	110	300	270	1620	1390	
30° to NE	0	0	640	12860	2280	2030	9270	
N to NE								7940
NE to 60°	0	0	5210	13280	1120	1960	4930	
60° to 75°	0	20	3650	2860	2620	2600	6570	
75° to E	0	20	480	320	250	440	9040	
NE to E								17300
E to 105°	0	500	260	100	650	180	4310	
105° to 120°	100	100	540	140	1070	1070	3420	
120° to SE	0	160	650	100	540	640	4510	
E to SE								51500
SE to 150°	0	580	1140	560	740	540	43100	
150° to 165°	0	90	1380	3450	40	0	3290	
165° to S	0	60	640	90	0	0	15070	
SE to S								92870
S to 195°	0	340	540	0	110	640	41110	
195° to 210°	0	0	2140	40	640	8140	13490	
210° to SW	0	0	110	1130	6850	2140	18200	
S to SW								100640
SW to 240°	0	0	100	1680	770	240	4790	
240° to 255°	0	0	720	570	190	480	0	
255° to W	0	0	100	720	570	0	0	
SW to W								33680
W to 285°	0	0	40	60	0	0	190	
285° to 300°	0	0	80	0	0	0	2140	
300° to NW	0	0	130	110	260	0	310	
W to NW								25220
NW to 330°	0	0	40	0	80	490	190	
330° to 345°	0	40	0	100	150	5910	1480	
345° to N	0	0	100	0	0	80	15030	
NW to N								28420

NOTE: The populations above refer to Fig. No. 1.4-2

2.5 HYDROLOGY

The hydrological features of the Indian Point site are relevant to the analysis of radioactive liquid and gaseous discharges from the plant. During normal plant operation liquid wastes are discharged to the Hudson River through the circulating water discharge tunnel. The sources of ground water will not be susceptible to contamination from accidental ground seepage or leakage from the plant because of the permeability of the bedrock and the higher elevation of the plant relative to the river. Gaseous releases from the plant following a hypothetical accident have been studied for possible deposition of contaminants into surrounding surface water reservoirs. Therefore, the hydrological features are categorized by the Hudson River, ground water and wells, and surface water reservoirs.

Two consultants have studied the hydrology of the Indian Point site. In 1955, prior to construction of Unit No. 1, Mr. Karl R. Kennison reported the flow characteristics of the river at the site. In 1965, the firm of Metcalf & Eddy reviewed Mr. Kennison's report and further reported the ground water hydrology and surface water reservoirs. The report by Metcalf & Eddy is included in this section, appended by the Kennison report.

Flow in the Hudson River is controlled more by the tides than by the runoff from the tributary watershed. Opposite the plant the width is 4500 to 5000 feet with a depth of 55 to 75 feet less than 1000 feet offshore. Total flow past the plant during the peak tidal flow is about 80,000,000 gallons per minute about 80% of the time, and it has been estimated that about 500 feet of the shore line flow is at least 9,000,000 gallons per minute in a section 500-600 feet wide. This large flow assures adequate dilution and complete mixing of the discharges from the plant. The plant is designed and will be operated such that discharges into the river would not prevent using the river water for drinking water.

The rate in downstream flow due to runoff is as follows:

11,700,000 gpm may be expected to be exceeded 2% of the time;
4,710,000 gpm may be expected to be exceeded 60% of the time;
1,800,000 gpm may be expected to be exceeded 98% of the time.

The table of river flow for a 17-year period presented in both the Kennison and Metcalf & Eddy reports is for the net river runoff flow.

Flooding at the site is nonexistent. Flood stages are primarily the effect of tidal influence and the highest recorded water elevation in the vicinity of the site was 7.4 feet above mean sea level. This is well below the basement elevation of Unit No.2.

Within a five-mile radius of the plant only one municipal water supply utilizes ground water. Other wells are for industrial and commercial usage. The rock formations in the area and elevations of wells relative to the plant are such that accidental ground leakage or seepage percolating into the ground at Indian Point will not reach these sources of ground water but will flow to the river. This subject is discussed further in the section on Geology.

Only two reservoirs within a five-mile radius are used for municipal water supplies. The [redacted] Field Reservoir is the raw-water receiving basin for the City of Peekskill with the Catskill Aqueduct and Montrose Water District as alternate supplies. The impounding reservoir for the Stony Point water system serves the towns of Stony Point and Haverstraw, and the villages of Haverstraw and West Haverstraw. The Stony Point system is connected to the Spring Valley Water Company to provide an alternate source of supply. A third reservoir within five miles of the plant, Queensboro Lake, supplies water to a state park area only. The location of these reservoirs and those within a fifteen-mile radius are shown on Figure 2.5-1.

The City of New York's Chelsea Pumping Station is located about one mile north of Chelsea, New York, on the east bank of the Hudson River. Water will be pumped from intakes in the river at the rate of 100 million gallons per day into the city reservoir system as required to supplement the primary supply from watersheds. The pumping station is 22 miles upriver from Indian Point measured along the centerline of the river as shown on Figure 2.5.2.

Discharge of any containment to a tidal estuary will result in its distribution throughout the estuary. Factors affecting this distribution include tidal amplitude and current, river geometry, salinity distribution, and fresh water discharge. Quirk, Lawler and Matusky Engineers, Environmental Science and Engineering Consultants, of New York City, have made extensive studies of the influence of these factors and have assisted Con Edison in the study of contaminant transport in the river. A report of this study is included in this section.

This study has enabled us to determine the effect of radioactive discharges on overall river contamination, and specifically conditions at Chelsea pumping station as shown in detail in Section 14.3.2. During normal operation, discharge will not result in river concentrations that exceed MPC at the Indian Point discharge canal.

Flooding at the site has been non-existent. The highest recorded water elevation at the site was 7.4' above mean sea level during an exceptionally severe hurricane in November, 1950.

Since the river water elevation would have to reach 15'3" above mean seal level before it would seep into the lowest floor elevation of any of the Indian Point Buildings, the potential for any flooding damage at the site appears t be extremely remote.

However, in order to determine the maximum probable elevation that the Hudson River could reach, Con Edison has commissioned Quirk, Lawley and Matusky to make an in depth study of the river under various flooding conditions. A report of this study, dated April 1970, is included in this section.

Seven different flooding conditions governing the maximum water elevation at the site were investigated, including the following:

a) flooding resulting from runoff generated by a Probable Maximum Precipitation over the entire Hudson River drainage basin upstream of the site.

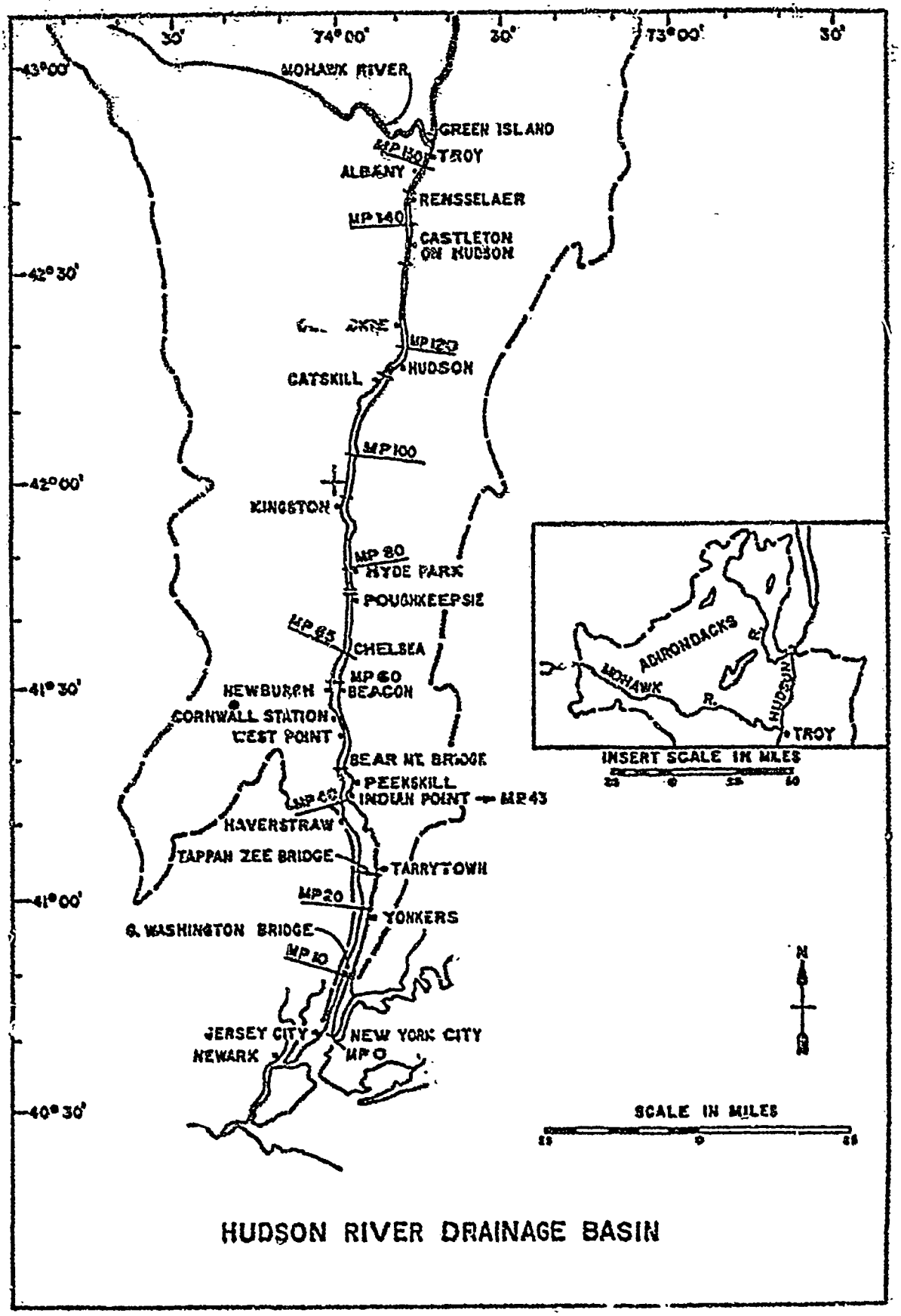
b) flooding caused by the occurance of an upstream dam failure concurrent with heavy runoff generated by standard Project Flood.

c) flooding due to the occurance of a Probable Maximum Hurricane concurrent with a spring high tide in the Hudson River.

The severest flooding condition revealed by the study is a result of the simultaneous occurance of a Standard Project Flood, a failure of the Ashokan Dam and a storm surge in New York Harbor at the mouth of the Hudson River resulting from a Standard Project Hurricane. The water level under these conditions would reach 14' above mean seal level. Local wave action due to wind effects has been determined to add one foot to the river elevation producing a maximum water elevation of 15' above mean sea level at the Indian Point Site.

Since this maximum water elevation is 3" lower than the critical elevation of 15'3" noted earlier, it is reasonable to conclude that flooding in the Hudson River will not present a hazard to the safe operation of Indian Point.

INDUSTRIAL WATER SUPPLY FROM HUDSON RIVER
INDUSTRIAL WATER SUPPLY FROM WELL
INDUSTRIAL WATER SUPPLY FROM STREAM
INDUSTRIAL WATER SUPPLY FROM POND
WATER RESERVOIR
AQUEDUCT



HUDSON RIVER DRAINAGE BASIN

FIGURE 2.5-2

HYDROLOGY OF INDIAN POINT SITE
AND SURROUNDING AREA

METCALF & EDDY ENGINEERS

OCTOBER, 1965

REPORT PREPARED BY GEORGE P. FULTON
UNDER DIRECTION OF HARRY L. KINSEL, P. E.

ACKNOWLEDGEMENTS

We acknowledge with thanks the assistance of many public officials, including the following, in furnishing data for this report:

Mr. Alfred Morgan, Chief Engineer
Palisades Interstate Park Commission

Mr. George O'Keefe, Director, Division of Environmental Sanitation, Rockland County Health Department

Mr. George Natt, Director,
Westchester County Water Agency

Mr. Michael Frimpter, U. S.
Geological Survey, Middletown, New York

INTRODUCTION

The hydrological features of the Indian Point site have been studied in three categories; the Hudson River, ground water and surface water reservoirs. Flow data and the flood history of the Hudson River in the vicinity of the Indian Point plant are discussed. Ground water sources within the area are generally used for industrial or commercial purposes with some limited residential usage on the west side of the river. The surface water reservoirs in the surrounding area that are used for water supplies and sources of alternate water supplies are also described.

HUDSON RIVER

General

The Consolidated Edison Indian Point plant is situated on the east bank of the Hudson River below Peekskill, just above Verplancks Point. In the general area of the plant, water from the Hudson River is used only for industrial cooling purposes. The nearest community utilizing the Hudson River for a public water supply at the present time is Poughkeepsie, some 30 miles upstream from the plant site.

Flow

Flow data for the Hudson River were abstracted from a previous report of Mr. K. Kennison, submitted to Consolidated Edison on November 18, 1958 (included as an appendix to the section on hydrology). Flood data were obtained from the Survey Division of the Corps of Engineers in New York City.

In the vicinity of Indian Point, the width of the Hudson River ranges from 4,500 to 5,000 feet with maximum depths of from 55 to 75 feet. Cross sectional areas of the river from a point three quarters of a mile upstream from the plant site to a mile downstream are in the order of from 165,000 to 170,000 square feet.

Flow duration records of the Hudson River for a 17-year period preceding 1930 show the following:

Rate of Flow c.f.s.	Percent of Time Exceeded
26,000	20%
15,250	40%
10,500	60%
7,000	80%
4,000	98%

It is evident that even the highest rates of flow expected will influence depth of flow in the river to only a small degree in the vicinity of the plant. This is due to the relatively high available flow section and the width of the river. River depth is affected more by the tidal influence than it can be by any anticipated flood flows.

The Hudson River is tidal as far upstream as Troy, some 100 miles from Indian Point. The elevation of the water surface in the vicinity of the plant is so responsive to the tidal cycle that average rate of flow has little effect on depth of flow or velocity of flow.

Flood History

Tide elevations vary both daily and seasonally and, in addition, can be affected by atmospheric conditions such as can exist during extreme storms or hurricanes. The atmospheric conditions can cause a surge which, added to the normal tide, establishes water elevation.

The highest water elevation at the U.S.G.S. station at Verplancks Point, one-half mile below Indian Point, was 7.4 feet above MSL (mean sea level) recorded in the year 1950. A higher surge occurred in 1960, but the normal tide stage was such that actual water elevation was somewhat less than the 1950 record. In an earlier period, before 1935, the highest recorded elevation was 4.75 ft. above MSL at Verplancks Point on August 24, 1933.

Mean water elevations at Verplancks Point are just below 1.0 (MSL). The mean range of water depth stages is about 3.0 ft.. With high runoff in the Hudson River Basin, the mean range at times averages a half a foot higher during the spring period.

The highest river elevation, recorded in 1950, was about 6.5 feet higher than average river levels, or some 5.0 feet higher than average high river stages. Considering past flood history and the fact that flood stages are primarily the effect of tidal influence, flooding of the Indian Point plant site appears to be a highly unlikely possibility.

Contamination Potential

The hazards of contamination of water supplies by discharge of water borne wastes from the Consolidated Edison Indian Point plant are almost minimal. In the reach of the Hudson River that could be affected, river water is used only for industrial cooling.

It should be mentioned that the City of New York is now in the process of constructing a river water pumping station at Chelsea in Putnam County below Poughkeepsie. The intent is to pump Hudson River water into the City system.

WELLS AND GROUND WATER

General

Within a five-mile radius of the plant the only public water supply using ground water is the Stony Point system of Utilities and Industries located in Rockland County cross the river from Indian Point. Reports on ground water resources within this five-mile radius indicate the existence of numerous other wells. These wells are for industrial and commercial usage and for individual water supplies for private residences. Residential usage, however, is almost entirely confined to the area on the west side of the Hudson River.

Ground Water Geology

Water bearing strata in the area within a five-mile radius of Indian Point can be divided into unconsolidated surface deposits and consolidated bedrock. Unconsolidated deposits cover most of the bedrock in this area and range in thickness from a few feet in the hills to several hundred feet in the larger valleys. Unconsolidated deposits range from clays, which produce only meager quantities of water, to coarse sand and gravel capable of yielding several hundred gallons per minute to a well.

The bedrock underlies the unconsolidated deposits and, where these are absent, crops out at the surface. Ground water in bedrock occurs principally in fractures and solution channels.

Thus, the water bearing characteristics are generally similar, although the rocks differ widely in mineral composition and water yield.

Bedrock in Westchester County is, for the most part, metamorphic in character and includes schist and gneiss, with smaller amounts of limestone, quartzite and slate. Small injections of granite can also be found. Only minimal yields of ground water can be obtained from bedrock formations in Westchester County.

Consolidated rocks are the chief source of water in Rockland County. Principal rock units include the following:

a) Newark Group - sandstone, shale and conglomerate.

b) Palisade Diabase - diabase with some basalt.

c) Cambrian and Ordovician Rocks - quartzite, limestone and dolomite.

d) Precambrian Rocks - granite, gneiss, with some schist and diorite.

The Newark group provides the greatest source of ground water supply in Rockland County. The other units of bedrock yield only minimal quantities, as in Westchester County.

A small area of Orange County lies within the 5-mile radius being considered. Wells in this area have been drilled in bedrock formations similar to those in Westchester County where the water yield is small.

Well Supplies

As mentioned before, the only public water supply served by wells in the 5-mile radius of Indian Point is the Stony Point System. This system serves the Villages of Haverstraw and West Haverstraw as well as portions of the Towns of Haverstraw and Stony Point. The Stony Point supply wells are located in stratified drift, an unconsolidated formation. These wells are relatively shallow, the greatest depth about 35 ft. Total yield of the wells to the system averages about 550 gpm.

Other wells in Rockland County, in the area being considered, include some wells for commercial and industrial use and many private wells serving indivi ual residences. These wells are located in bedrock for the most part and range from 100 to 300 ft. in depth. Consumption of water from wells serving private homes will vary from 100 to 1,000 gpd (gallons per day), depending on the number of persons using the supply and the facilities using water.

There are only a few wells still in use in Westchester County within the 5-mile radius. Almost all the wells within 2 to 3 miles of Indian Point have been abandoned and connections have been made to public water systems for supply. At the fringes of the area a few private wells are used for individual residences. These wells are mostly in unconsolidated deposits with depths less than 50 ft. Some wells exist in bedrock with depths varying up to several hundred feet.

A small portion of the community of Fort Montgomery in Orange County lies within 5 miles of the plant. Homes in this community are served entirely by individual private wells in bedrock. Depth of the wells vary up to several hundreds of feet.

Contamination Potential

The bedrock formation is such that it is highly unlikely that wastes percolating into the ground from the Indian Point site will reach the water bearing formations used for water supply on the west side of the river in Rockland and Orange Counties. Most of the wells in Westchester County are shallow, in unconsolidated formations with ground surface elevations considerably higher than at the plant site. This situation would preclude the possibility of contamination of the supply through ground water flow. Bedrock wells in Westchester County are similarly at higher elevations and, for the most part, are drilled in different rock formations than exists at the plant site.

SURFACE WATER RESERVOIRS

General

The major sources of water supply in the Indian Point area are lakes and surface water reservoirs. The reservoirs within a 15-mile radius of the plant site are tabulated in Tables 1-7 along with the users, capacities and distances from Indian Point. A detailed analysis of the reservoirs within 5 miles of the plant describes alternate sources of supply to those communities served by the reservoirs.

City of Peekskill-Camp Field Reservoir

The 54-million gallon Camp Field Reservoir of the City of Peekskill system, located 2.9 miles from Indian Point, is a raw-water receiving basin for the water treatment plant. Water is pumped into this basin from Peekskill Hollow Brook. For the most part, the water supply is the continuous flow of this brook. At times of low flow the supply can be supplemented by releasing water into the stream from holding reservoirs in Wicopee (Putnam County) some 11.7 miles from Indian Point or from the Catskill Aqueduct of the City of New York, located a short distance upstream from the pump intake.

The City of Peekskill system is divided into two service pressure areas. Water for the low-pressure area flows by gravity from Camp Field Reservoir through a bank of slow-sand filters

into the system. No additional storage is provided for this section of the system. Water for the high-service area flows from the reservoir through two diatomaceous earth filters by gravity and then is pumped to a pair of elevated storage tanks with a total capacity of 800,000 gallons. The high-service system serves approximately 25 percent of the Peekskill area. The remaining area, including Standard Brands and most of the other industrial consumers, is served by the low-pressure system.

Total water consumption in Peekskill averages about 5 mgd. The largest single user is Standard Brands, at an average rate of 1.5 mgd. All water is supplied from Peekskill Hollow Brook. Two connections to other systems are available for emergency conditions. One is the above-mentioned Catskill Aqueduct connection which discharges into Peekskill Hollow Brook. This flow must be processed through the two treatment facilities for use. The other emergency connection is to the Montrose Water District system which can supply between 1.0 and 1.25 mgd from the Catskill Aqueduct to the low-service section of the Peekskill system.

Since no piping is installed to bypass Camp Field Reservoir, contamination of this basin would deprive Peekskill of its normal source of supply. Installation of a bypass would involve some 800 lin. ft. of 24-in. pipe between the inlet force mains and the outlet lines to the two filter facilities. With such a

bypass, it would be possible to take water directly to the filters from Peekskill Hollow Brook after the passage of contaminated water in the event of prolonged contamination of Camp Field Reservoir. It might be necessary to accelerate flushing out of the brook and the impoundment at the pumping station in such a situation by releasing water from either the Catskill Aqueduct or the Wicopee reservoirs.

Peekskill most likely could not depend on the Montrose connection alone. This can supply less than one-half the normal demands of the low-service system even with the assumption that Standard Brands would not operate during the emergency. The high-service system has only 800,000-gallon storage, which would last less than 24 hours after shutting down the Peekskill Hollow Brook supply.

As presently arranged, the City of Peekskill would be practically deprived of a water supply with elimination of Peekskill Hollow Brook as a source. A study will soon be made under the auspices of the Westchester County Water Agency and the State of New York to determine the feasibility of connecting the Peekskill system to a proposed transmission main crossing northern Westchester County from the Delaware Aqueduct of the City of New York. This proposal could furnish an independent source of water in sufficient supply to serve all the needs of the City of Peekskill in the event of an emergency.

Palisades Interstate Park Commission - Queensboro Lake

Queensboro Lake, some 5 miles from Indian Point, 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 Queensboro Lake through stream flow or by pipe 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, an 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 1.8 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 is located some 3.5 miles from Indian Point. With contamination of this supply, the system would be left with only the wells which furnish about 45 percent of total consumption.

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 from 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.

WATER STORAGE RESERVOIRS

TABLE 1

WITHIN 15 MILE RADIUS OF INDIAN POINT

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	Fergusons Lake	Pocantico Hills Est.	40 *	13.5	28
W-13	Tarrytown Res.	Tarrytown	313	14.0	35
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 W-1)	11.6	588
W-4	Camp Field Res.	Peekskill	54	2.9	11

* Estimated

WATER STORAGE RESERVOIRS

TABLE 2

WITHIN 15 MILE RADIUS OF INDIAN POINT

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

* Estimated

WATER STORAGE RESERVOIRS

TABLE 3

WITHIN 15 MILE RADIUS OF INDIAN POINT

ORANGE COUNTY

Code	Reservoir	User	Capacity Million Gallons	Distance Miles	Surface Acres
O-11	Lusk Res.	U.S. M.A.	50 *	7.5	16
O-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
O-12	Turkey Hill La.	Palisades Int. Park	150	5.9	58
	Nawahunta La.	" " "	22	6.7	16
O-20	Silvermine La.	Palisades Int. Park	465	6.0	84
	Queensboro La.	" " "	56	5.0	37
O-16	Lake Stahahe	Palisades Int. Park	230	11.1	90
O-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
	Upper Twin La.	" " "	105	7.7	24
	Lower Twin La.	" " "	88	7.6	26
	Massawiepa La.	" " "	104	7.7	29
O-17	Lake Tiorati	Pal.Int. Pk., Tiorati & Cohasset	1,500	6.7	296
O-10	Cromwell Lake	Woodbury	80	11.2	55
O-2	Walton Lake	Chester	300	14.6	129
O-5	Lake Mombasha	Monroe	1,750	13.0	324
O-1	Echo Lake	Arden Farms	40 *	9.5	30
O-7	Or[illegible] 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
O-8&9	Tuxedo Lake	Tuxedo & Tuxedo Pk.	2,500	14.5	294
O-3	Aleck 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 POINT

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 [illegible], 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

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

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)

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

TABLE 5 (CONT'D.)

Tarrytown Reservoir

Village of Tarrytown

Glenville W D

Town of Greenburgh

Eastview

Town of Mount Pleasant

Village of North Tarrytown

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

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 of 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 Haverstraw

Village of West Haverstraw

KARL R. KENNISON
CIVIL AND HYDRAULIC ENGINEER
381 CLINTON AVE., BROOKLYN, N. Y.

Nov. 18, 1955

Mr. O. R. Milne
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 affect 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.

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

17 years.

An average rate of about 26000 cfs may be expected to be exceeded 20 % of the time
" " " " " 16250 " " " 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 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

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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 45% 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 cooling 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

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, is very 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 municipalities 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 deleterious effect.

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

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 - the avoidance of even the appearance of danger.

Very truly yours,

Karl R. Kennison
Consulting Engineer

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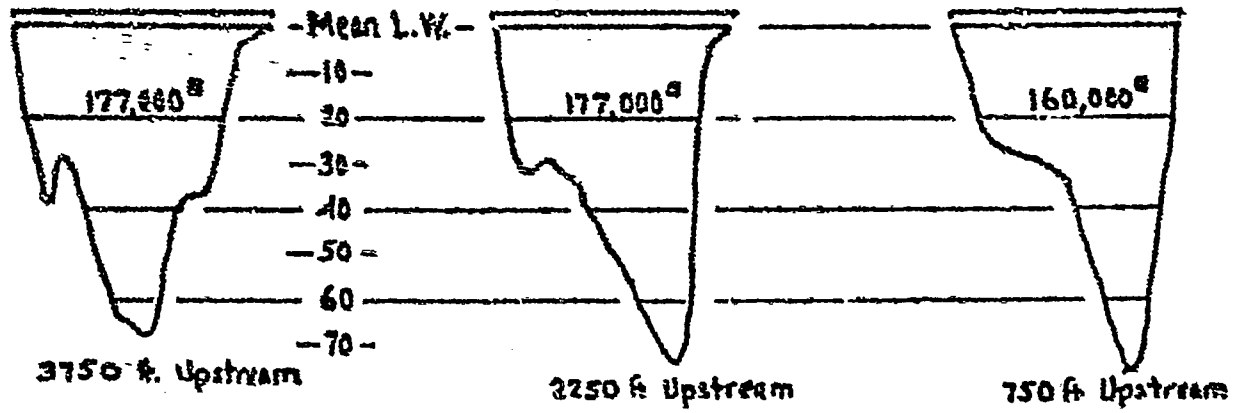
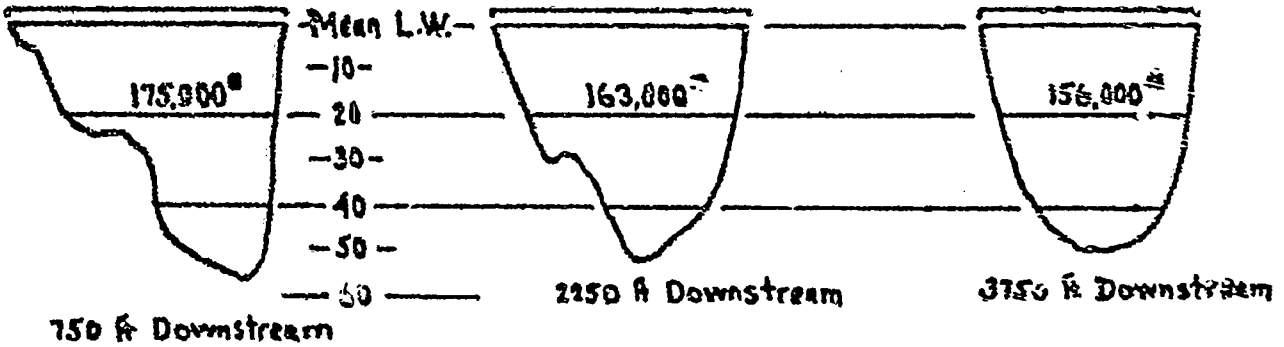
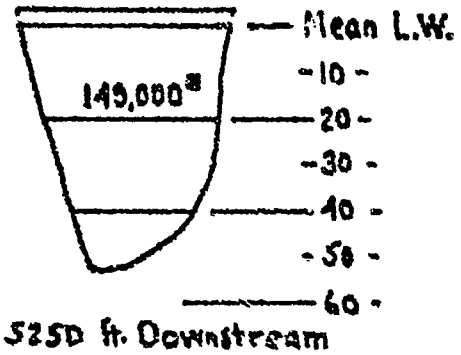
KARL R. KENNISON
CIVIL AND HYDRAULIC ENGINEER
321 CLINTON AVE., BROOKLYN, N. Y.

Sheet 1
9-13-'55







CROSS SECTIONS OF THE
HUDSON RIVER
(Looking Upstream)
Spaced 1500 ft apart past the
PROPOSED CONS. EDISON PLANT
BELOW PEEKSKILL

K.R.K.

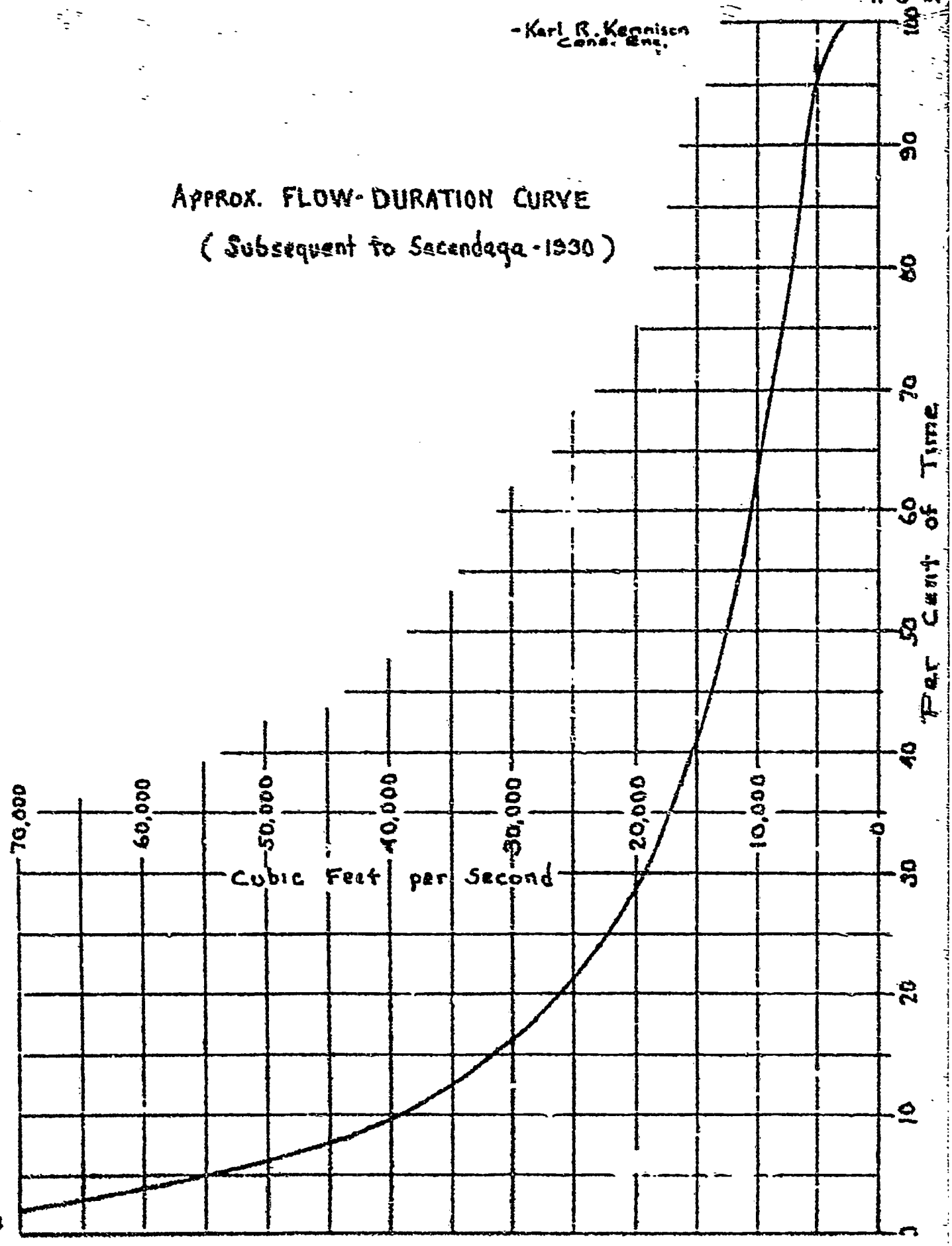
Hudson River at Location of Proposed Cons. Edison Plant below Peekskill

11-8-51

-Karl R. Kennison
Cons. Eng.

APPROX. FLOW-DURATION CURVE

(Subsequent to Secendaga - 1930)



S-33

K.R.K.

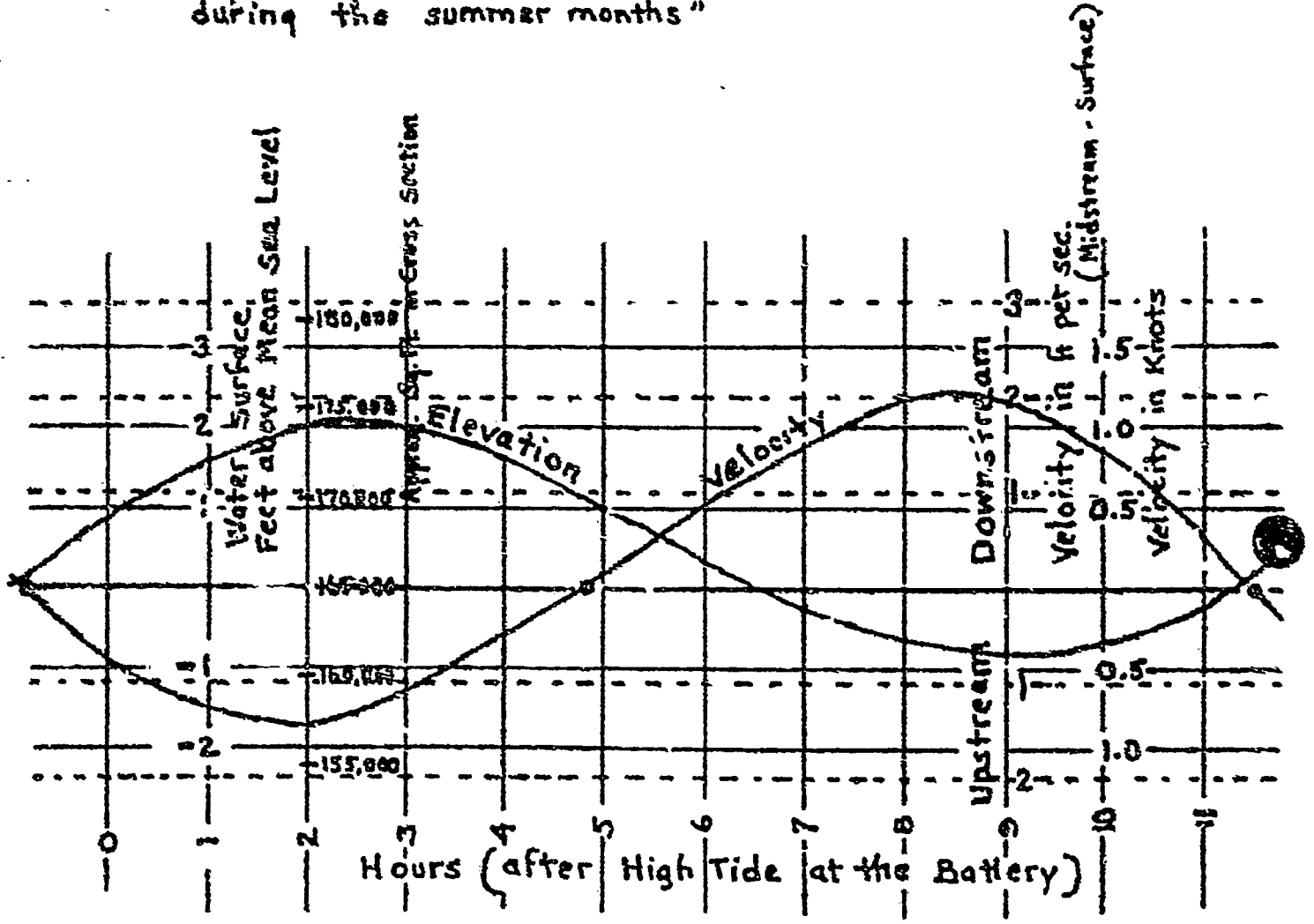
Hudson River at Location of Proposed Cons. Edison Plant below Peekskill

9-13

- Karl R. Kennison
Cons. Eng.

Data from Coast & Geodetic Survey

"Average conditions in the axis of the main channel during the summer months"

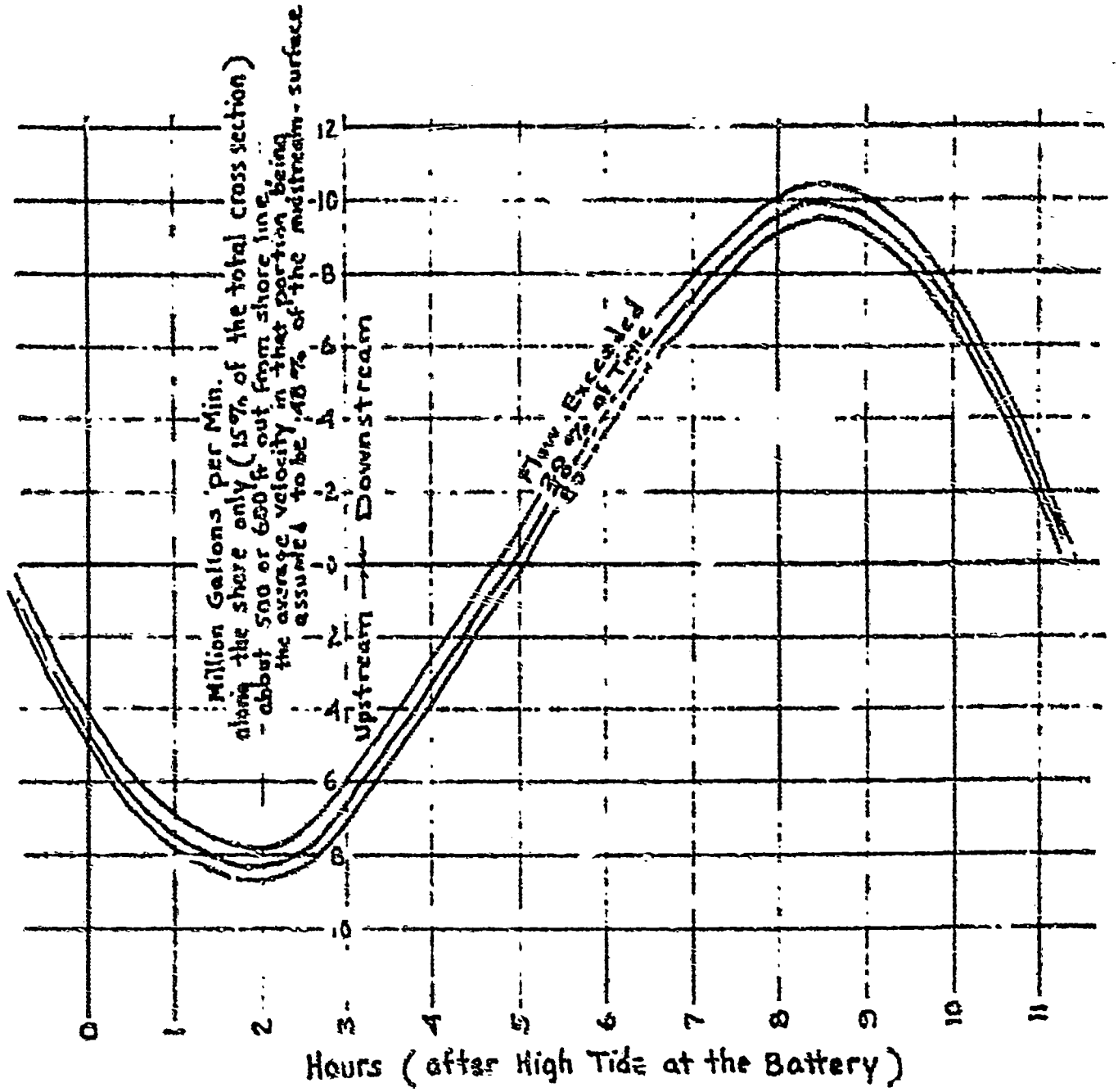


K.R.K.

Hudson River at Location of Proposed Cons. Edison Plant below Peekskill

11-8 55

- Karl R. Kennisen
Cons. Eng.



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

I. 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, Δx, in Figures 1 is written:

$$\text{Inflow} = \text{Outflow} + \text{Production} + \text{Accumulation} \quad \ldots\ldots (1)$$

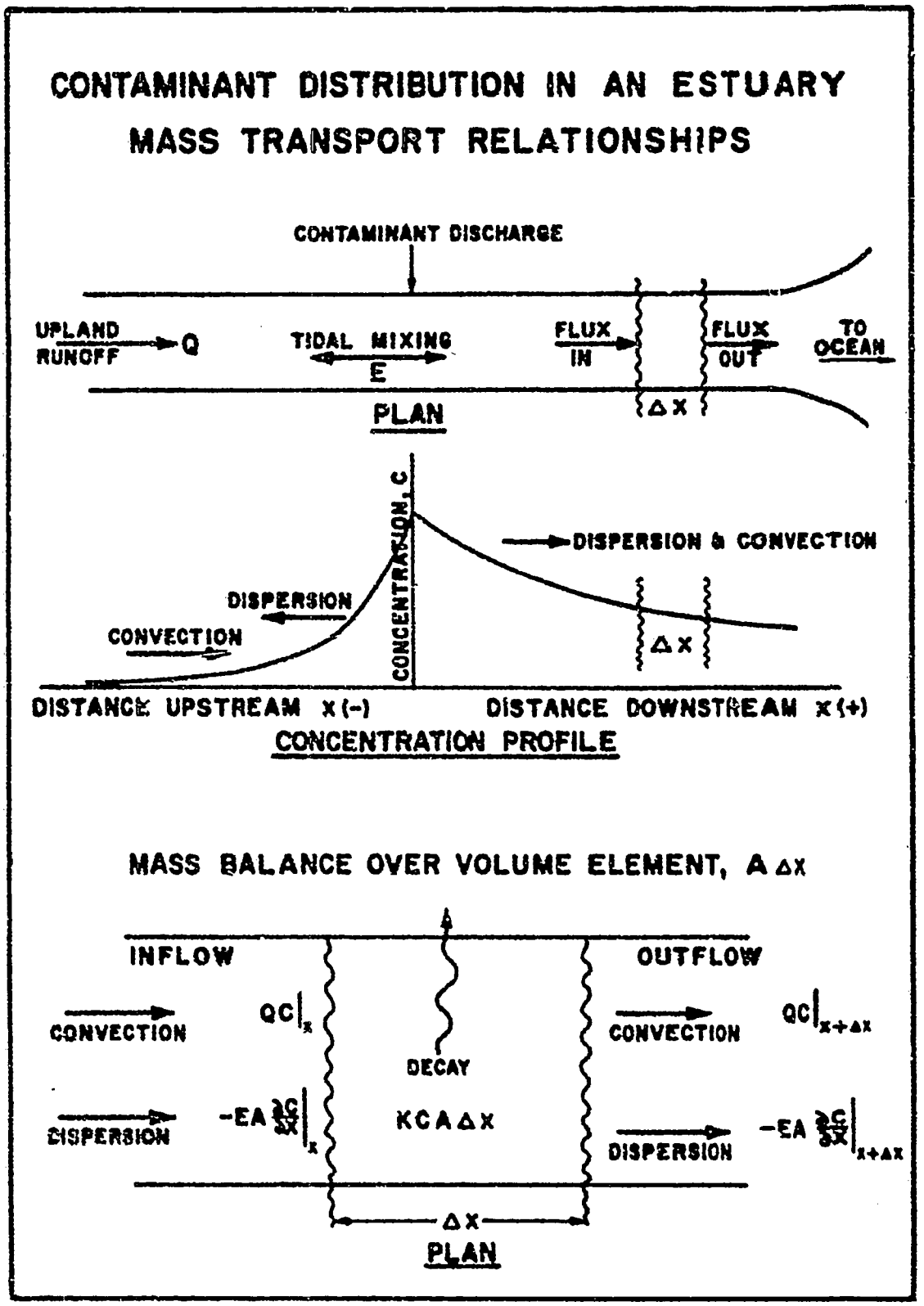

CONTAMINANT DISTRIBUTION IN AN ESTUARY
MASS TRANSPORT RELATIONSHIPS
CONTAMINANT DISCHARGE
UPLAND RUNOFF
Q
TIDAL MIXING
E
FLUX IN
FLUX OUT
TO OCEAN
ΔX
PLAN
CONCENTRATION, C
DISPERSION & CONVECTION
DISPERSION
CONVECTION
ΔX
DISTANCE UPSTREAM x(-)
DISTANCE DOWNSTREAM x(+)
CONCENTRATION PROFILE
MASS BALANCE OVER VOLUME ELEMENT, A Δx
INFLOW
OUTFLOW
CONVECTION
$QC|_x$
DECAY
CONVECTION
$QC|_{x+\Delta x}$
DISPERSION
$-EA\frac{\partial C}{\partial x}|_x$
KCAΔx
DISPERSION
$-EA\frac{\partial C}{\partial x}|_{x+\Delta x}$
ΔX
PLAN

Figure 1

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 radioactive decay, this rate of consumpiton 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 rates of inflow, outflow and production. This is equal to the time rate of change of total contamination 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:

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

The bracketed terms are average rates of change with respect to x. The limit of Equation 3, as Δx approaches zero, is as follows:

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

U is equal to Q/A and is the average fresh water velocity. Equation 4 is a linear partial differential equation in x and 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 shown 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.

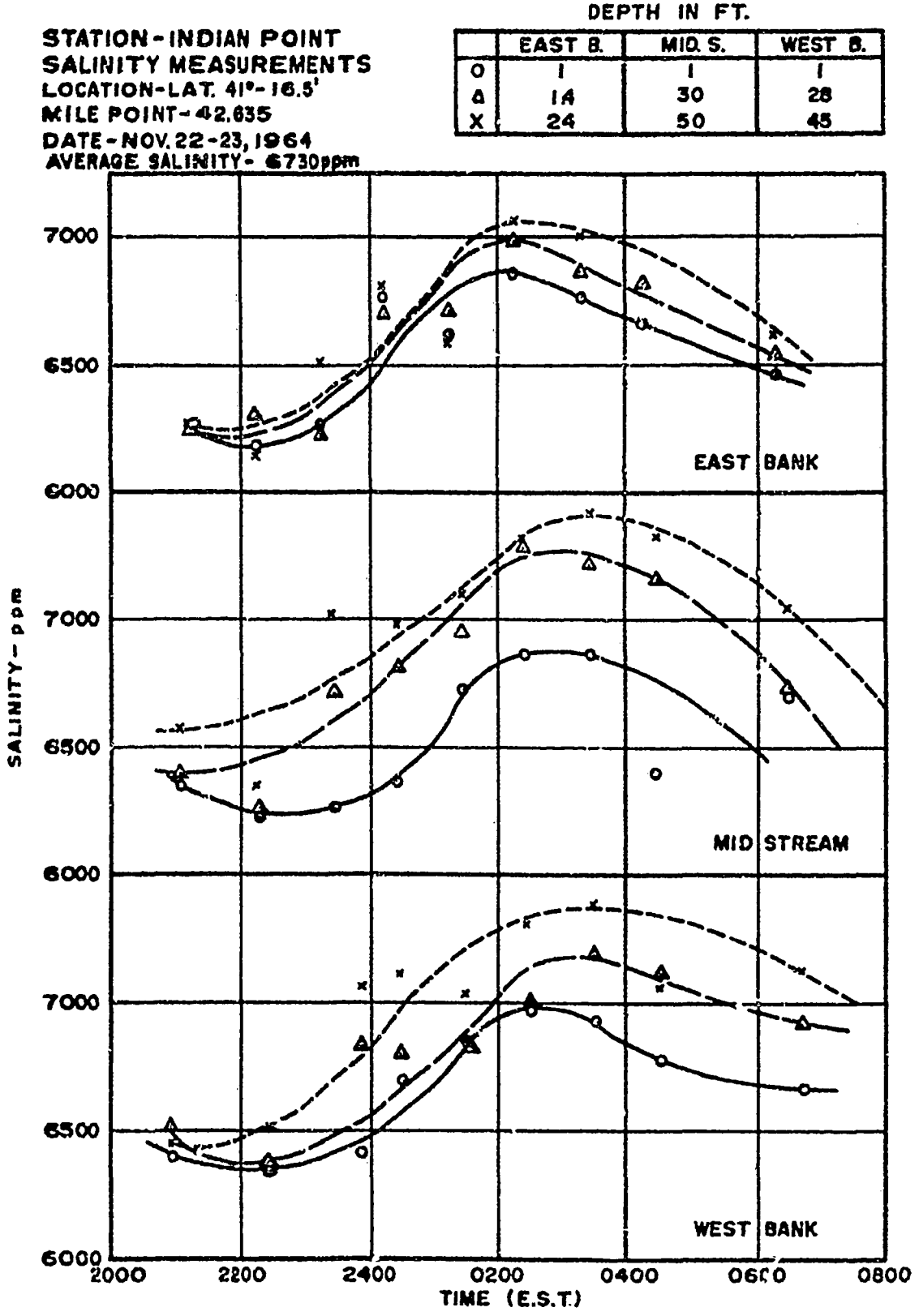
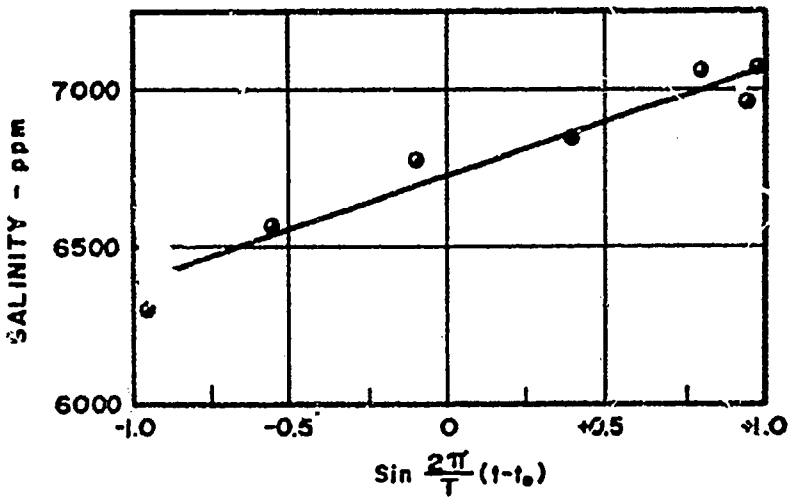

STATION - INDIAN POINT
SALINITY MEASUREMENTS
LOCATION - LAT. 41°-16.5'
MILE POINT - 42.635
DATE - NOV. 22-23, 1964
AVERAGE SALINITY - 6730ppm
DEPTH IN FT.
EAST B.
MID S.
WEST B.
O 1 1 1
Δ 14 30 28
X 24 50 45
EAST BANK
MID STREAM
WEST BANK
SALINITY - ppm
7000
6500
6000
2000
2200
2400
0200
0400
0600
0800
TIME (E.S.T.)

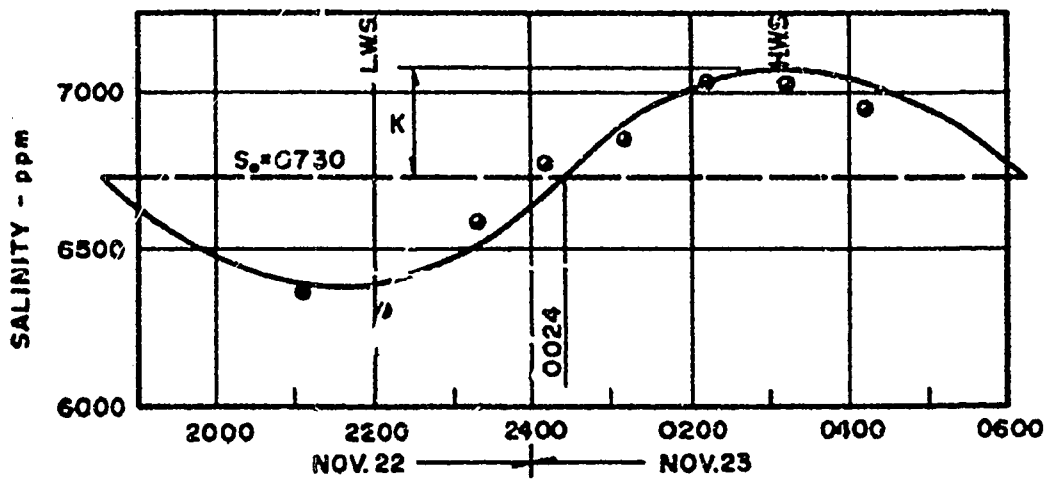
Figure 2

STATION - INDIAN POINT
LOCATION - LAT. 41°-16.5'
MILE POINT 42.635
DATE NOV. 22 - 23, 1964



S_o = 6730 ppm
t_o @ 0024 Hrs
K = 350 ppm

MEAN SECTIONAL SALINITY VS. PHASE ANGLE



MEAN SECTIONAL SALINITY VS. TIME

Figure 3

STATION - WEST POINT
SALINITY MEASUREMENTS
LOCATION - LAT. 41°-23'
[illegible] POINT 51.570
DATE NOV. 22, 1964
AVERAGE SALINITY - 4900 ppm

DEPTH IN FT.

	E. BANK	MID. S.	W. BANK
O	1	1	1
Δ	10.5	30	34
x	17.4	50	56

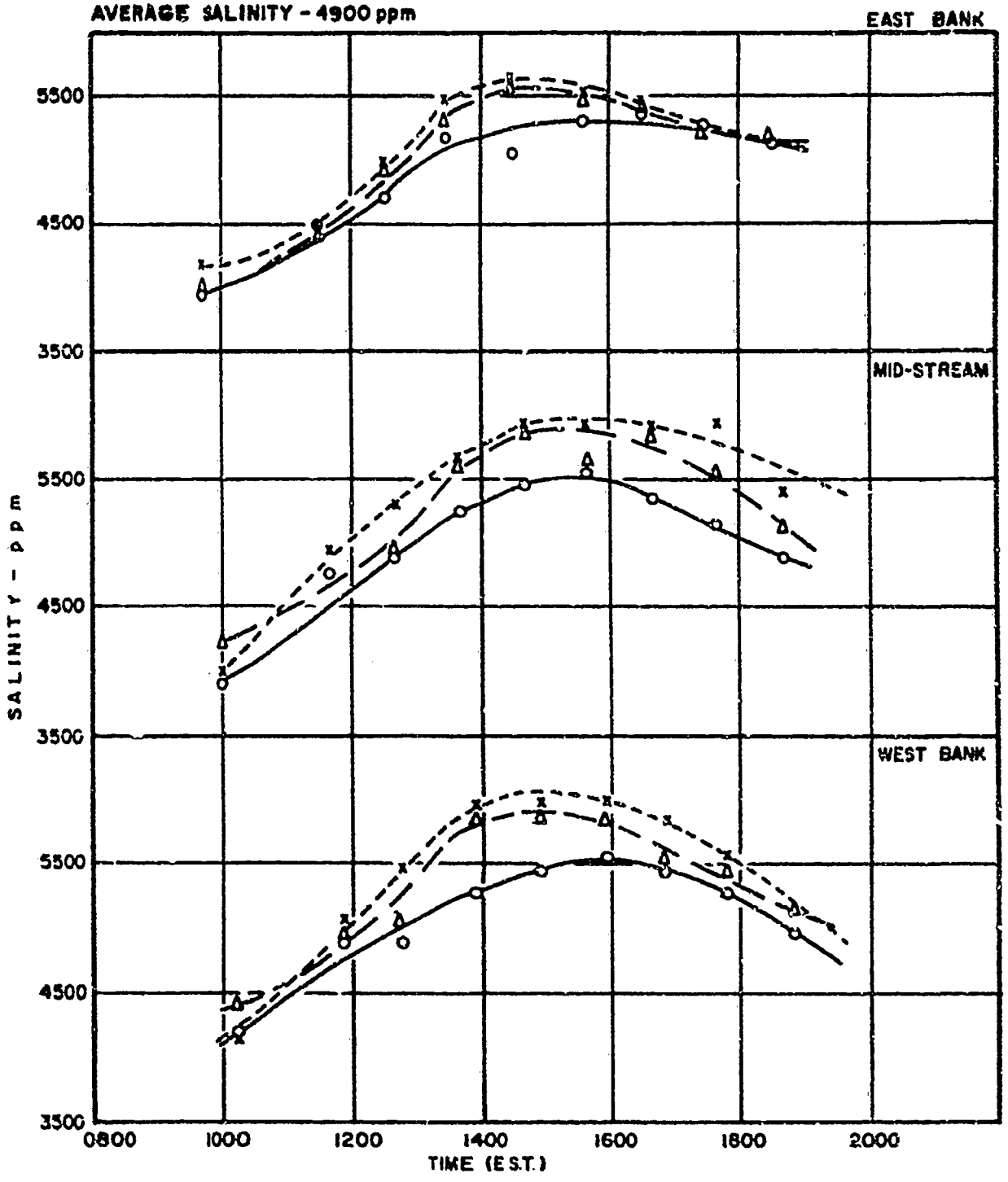
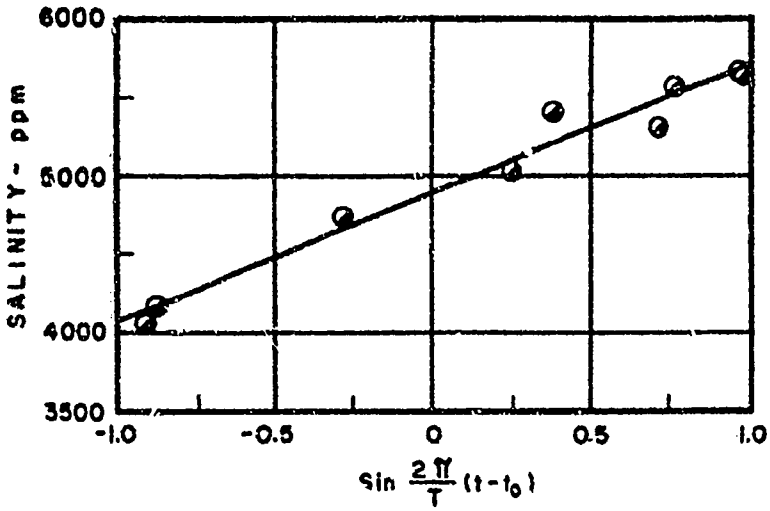


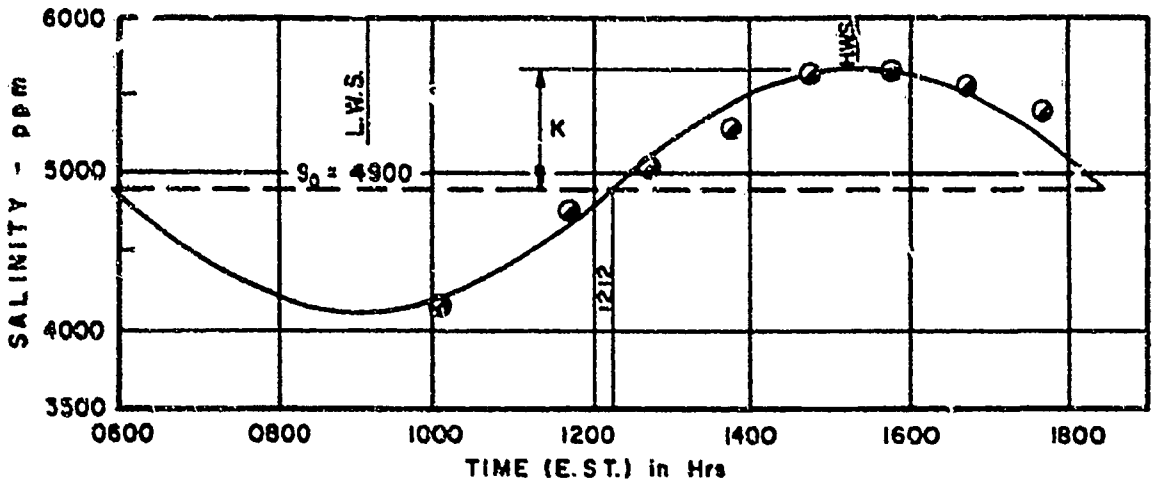
Figure 4

STATION - WEST POINT
LOCATION - LAT. 41°-23'
MILE POINT 51.570
DATE NOV. 22, 1964



S_o = 4900 ppm
t_o @ 1212 Hrs
K = 800 ppm

MEAN SECTIONAL SALINITY VS. PHASE ANGLE



MEAN SECTIONAL SALINITY VS. TIME

Figure 5

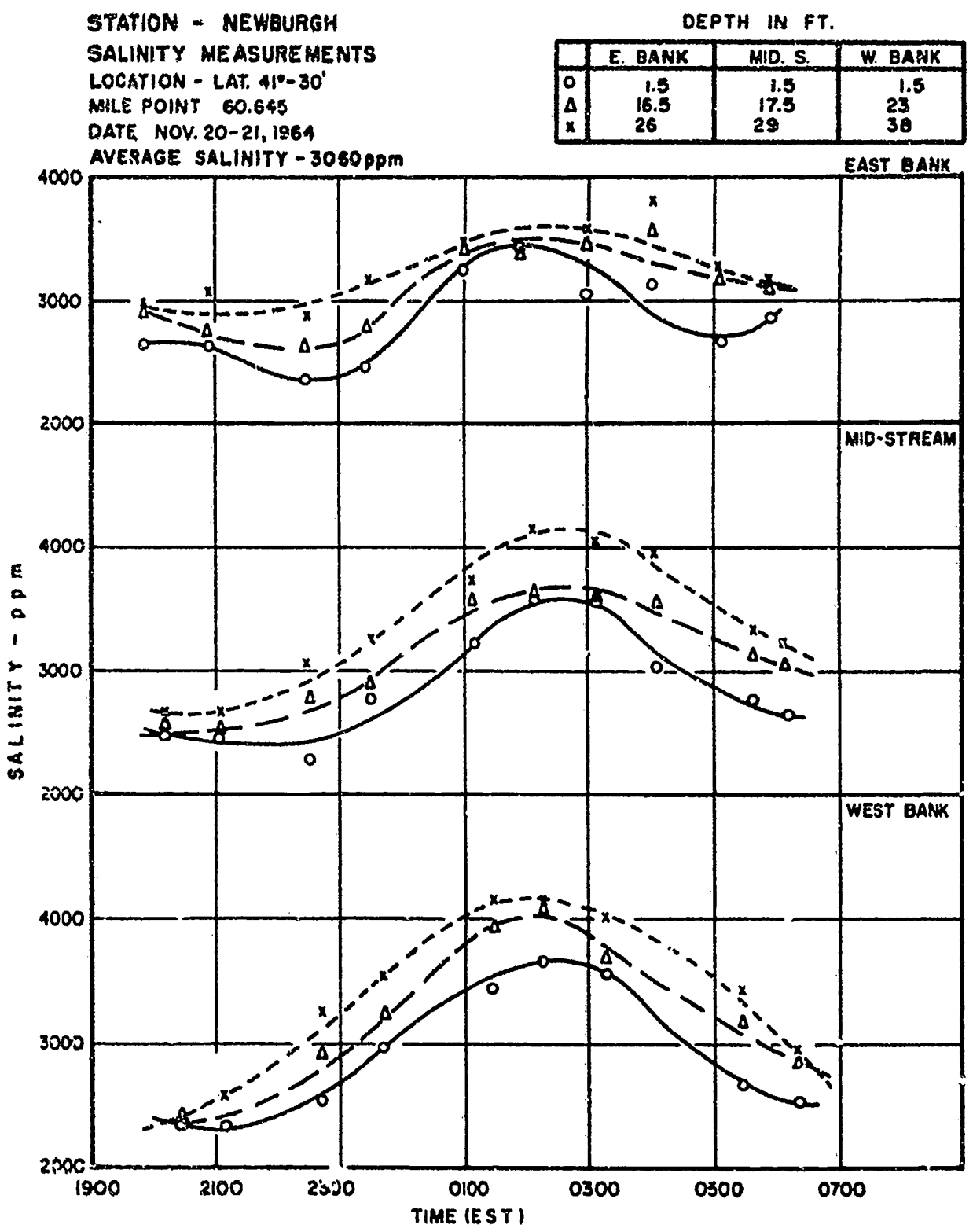
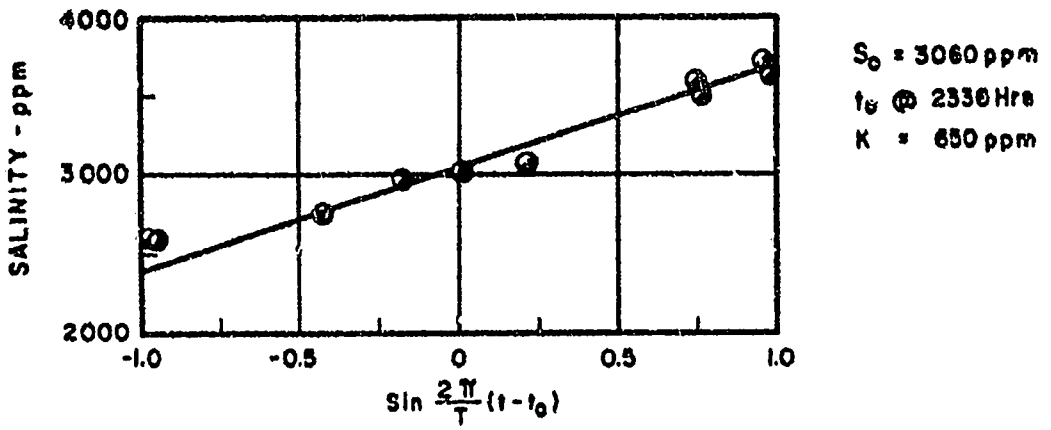
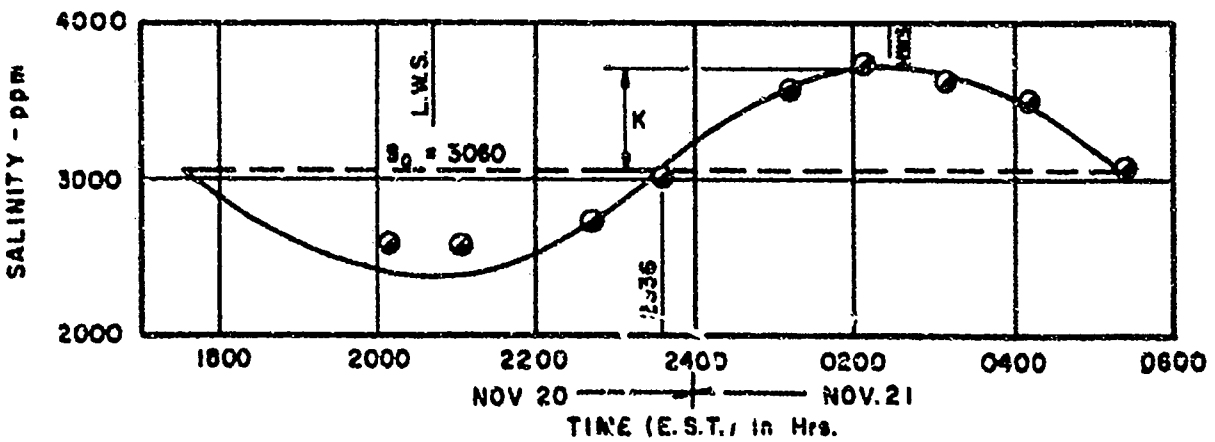

STATION - NEWBURGH
SALINITY MEASUREMENTS
LOCATION - LAT. 41°-30'
MILE POINT 60.645
DATE NOV. 20-21, 1964
AVERAGE SALINITY - 30600ppm
DEPTH IN FT.
E. BANK
MID. S.
W. BANK
O 1.5 1.5 1.5
Δ 16.5 17.5 23
x 26 29 38
EAST BANK
MID-STREAM
WEST BANK
SALINITY - ppm
4000
3000
2000
1900
2100
2300
0100
0300
0500
0700
TIME (EST)

Figure 6

STATION - NEWBURGH
LOCATION - LAT. 41°-30'
MILE POINT 60.645
DATE NOV. 20-21, 1964



MEAN SECTIONAL SALINITY VS PHASE ANGLE



MEAN SECTIONAL SALINITY VS. TIME

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 is used 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 excess of twice the volume discharged 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 decreases. 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.

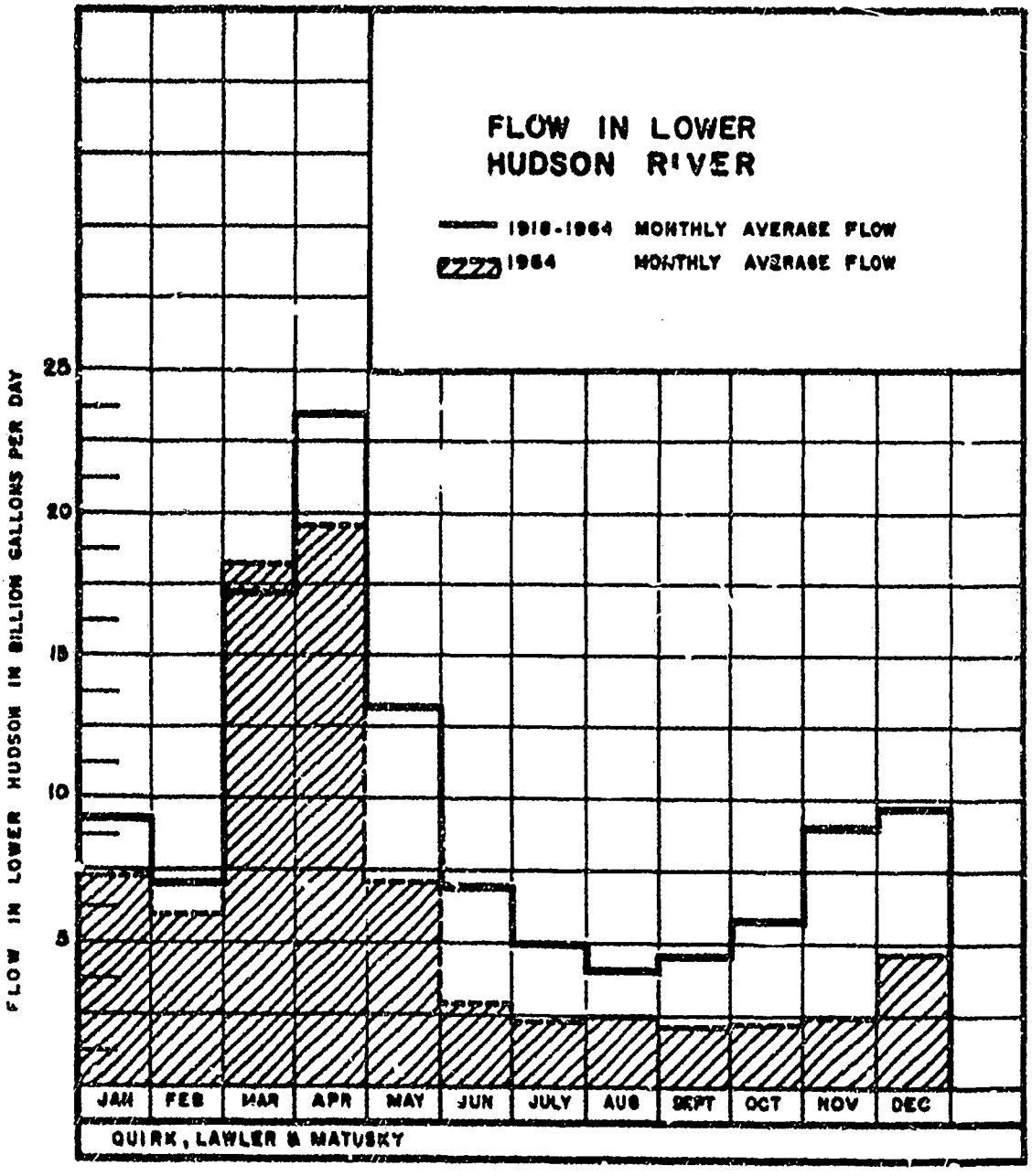

FLOW IN LOWER
HUDSON RIVER
1918-1964 MONTHLY AVERAGE FLOW
1964 MONTHLY AVERAGE FLOW
FLOW IN LOWER HUDSON IN BILLION GALLONS PER DAY
25
20
15
10
5
JAN
FEB
MAR
APR
MAY
JUN
JULY
AUG
SEPT
OCT
NOV
DEC
QUIRK, LAWLER & MATUSKY

Figure 8

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 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 known, 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

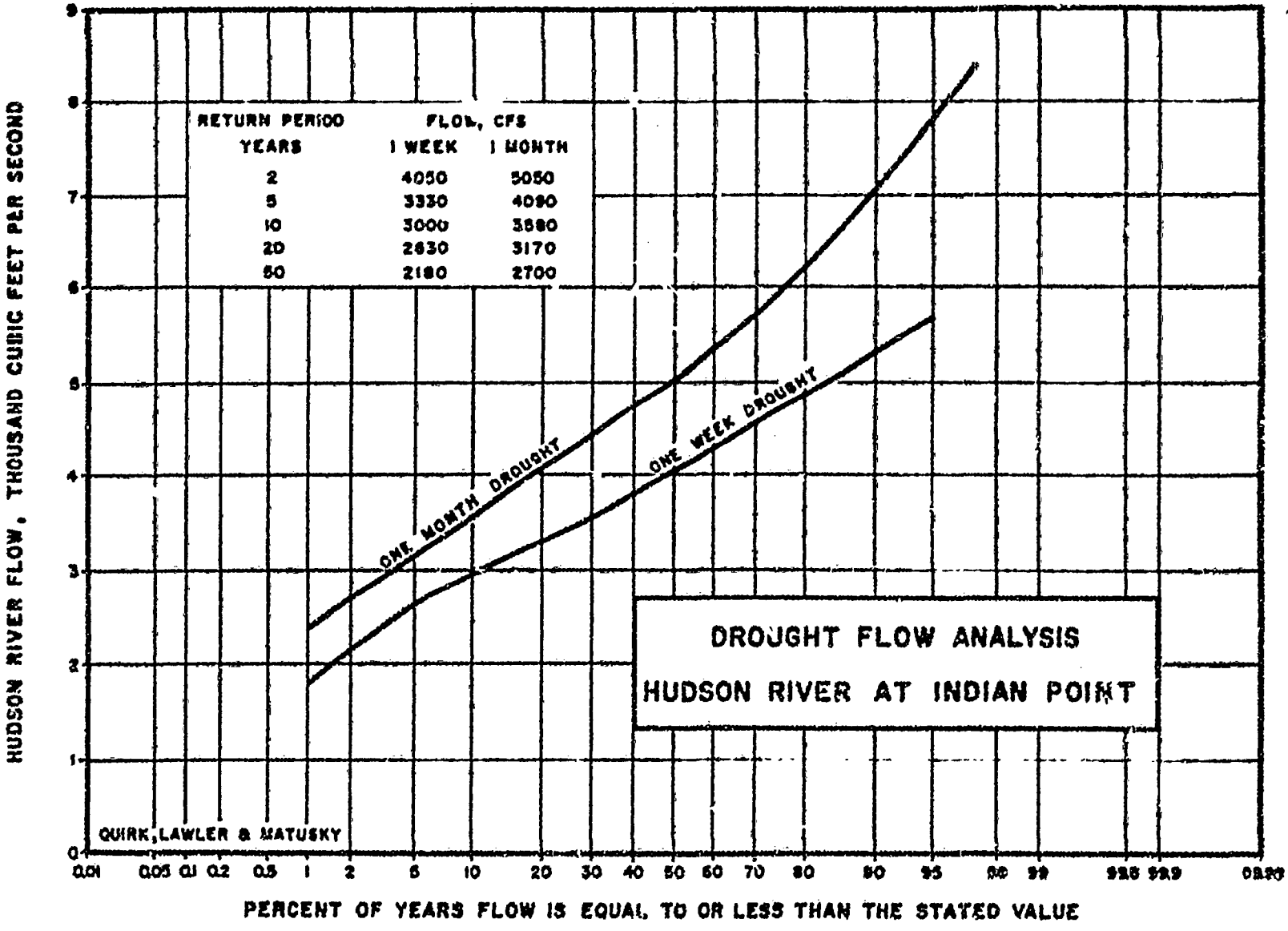
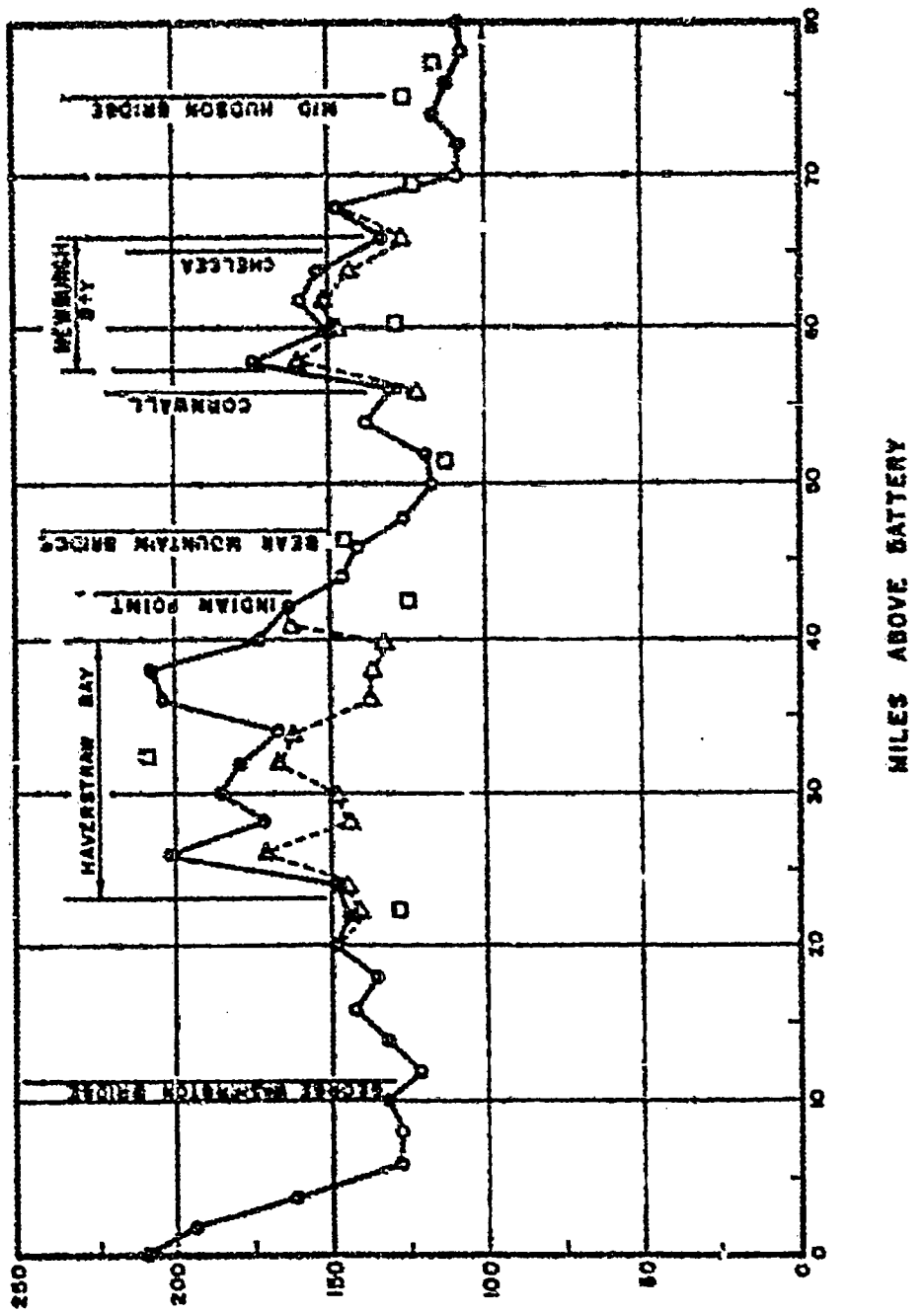

RETURN PERIOD YEARS
FLOW, CFS
1 WEEK
1 MONTH
2 4050 5050
5 3330 4090
10 3000 3580
20 2630 3170
50 2180 2700
ONE MONTH DROUGHT
ONE WEEK DROUGHT
DROUGHT FLOW ANALYSIS
HUDSON RIVER AT INDIAN POINT
QUIRK, LAWLER & MATUSKY
HUDSON RIVER FLOW, THOUSAND CUBIC FEET PER SECOND
PERCENT OF YEARS FLOW IS EQUAL TO OR LESS THAN THE STATED VALUE

FIGURE 9



HUDSON RIVER
CROSS-SECTIONAL AREA
MEAN LOW WATER

Figure 10

latter requirement poses some difficulty 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.

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

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

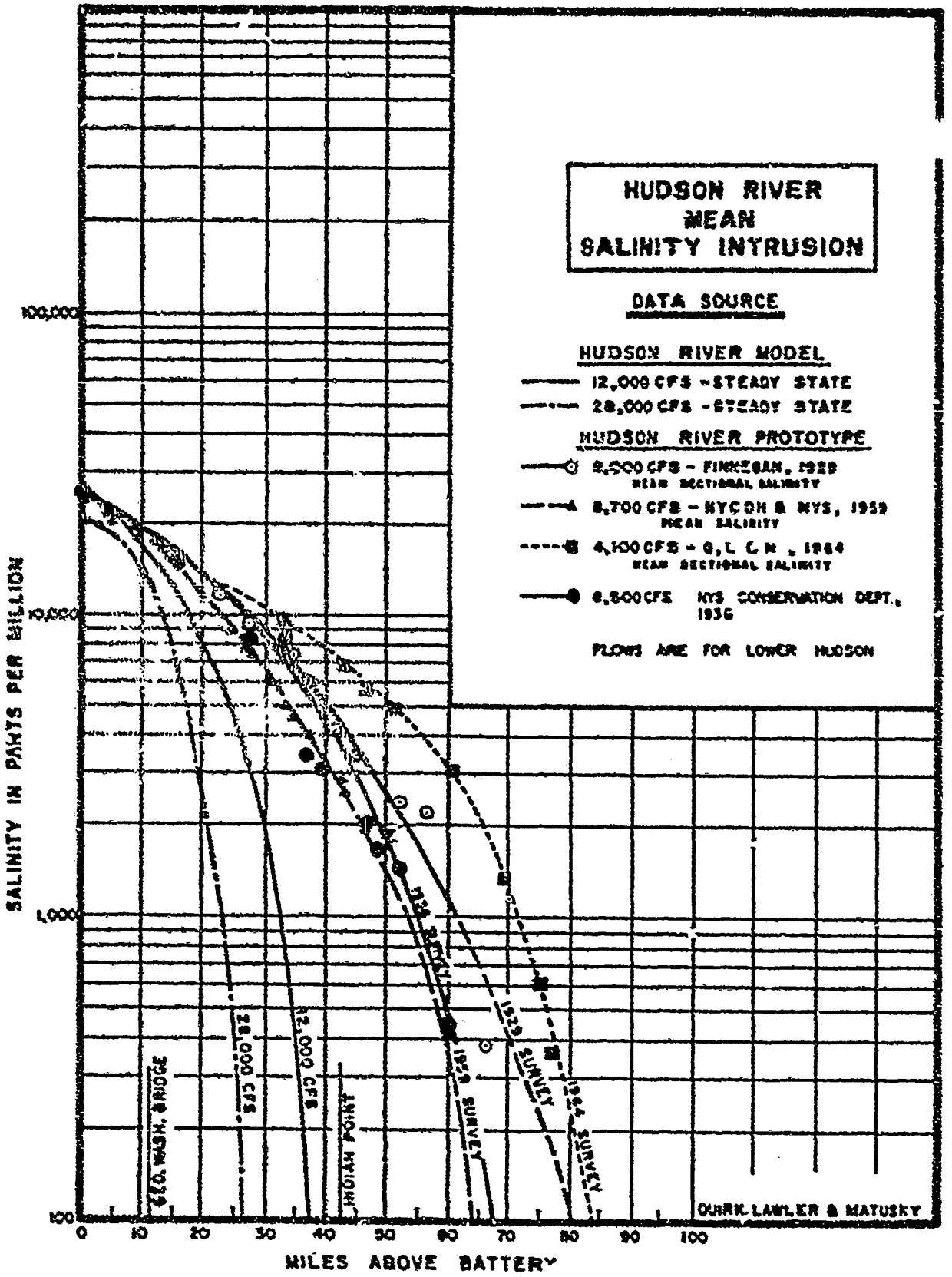

HUDSON RIVER
MEAN
SALINITY INTRUSION
DATA SOURCE
HUDSON RIVER MODEL
12,000 CFS - STEADY STATE
28,000 CFS - STEADY STATE
HUDSON RIVER PROTOTYPE
8,700 CFS - NYCDH & NYS, 1959
MEAN SALINITY
4,100 CFS - G, L & M, 1964
MEAN SECTIONAL SALINITY
8,500 CFS NYS CONSERVATION DEPT., 1936
FLOWS ARE FOR LOWER HUDSON
SALINITY IN PARTS PER MILLION
100,000
10,000
1,000
100
0
10
20
30
40
50
60
70
80
90
100
MILES ABOVE BATTERY
GEO. WASH. BRIDGE
28,000 CFS
12,000 CFS
INDIAN POINT
1959 SURVEY
1929 SURVEY
1964 SURVEY
QUIRK, LAWLER & MATUSKY

Figure 11

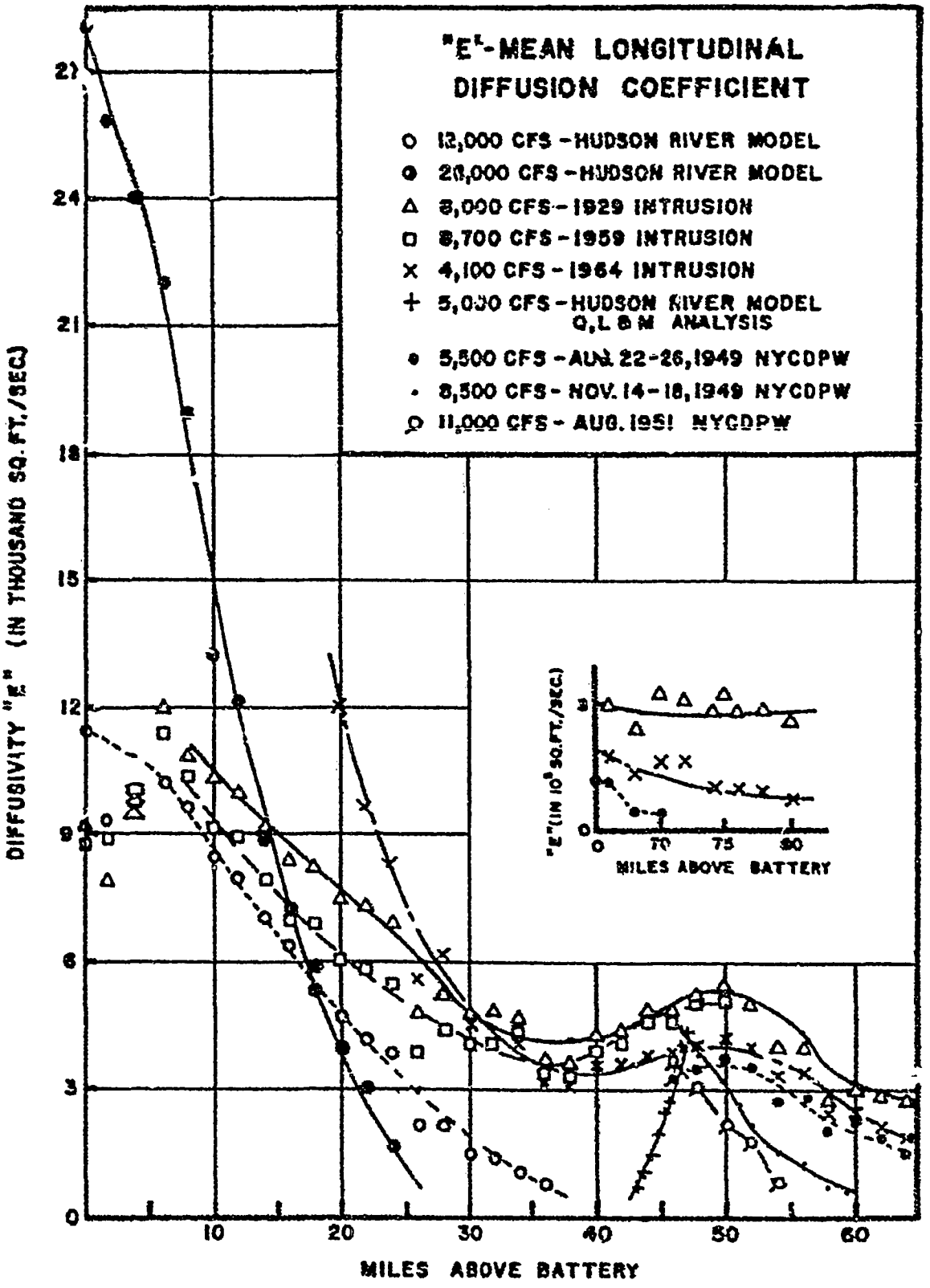

"E"-MEAN LONGITUDINAL DIFFUSION COEFFICIENT
12,000 CFS - HUDSON RIVER MODEL
20,000 CFS - HUDSON RIVER MODEL
8,000 CFS - 1929 INTRUSION
8,700 CFS - 1959 INTRUSION
4,100 CFS - 1964 INTRUSION
5,000 CFS - HUDSON RIVER MODEL Q,L & M ANALYSIS
5,500 CFS - AUG. 22-26, 1949 NYCDPW
8,500 CFS - NOV. 14-18, 1949 NYCDPW
11,000 CFS - AUG. 1951 NYCDPW
DIFFUSIVITY "E" (IN THOUSAND SQ. FT./SEC.)
MILES ABOVE BATTERY
"E" (IN 10³ SQ. FT./SEC.)
MILES ABOVE BATTERY

Figure 12

decreases. Thus, increases in fresh water flow can, under some conditions, outweight 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 points 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:

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

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

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

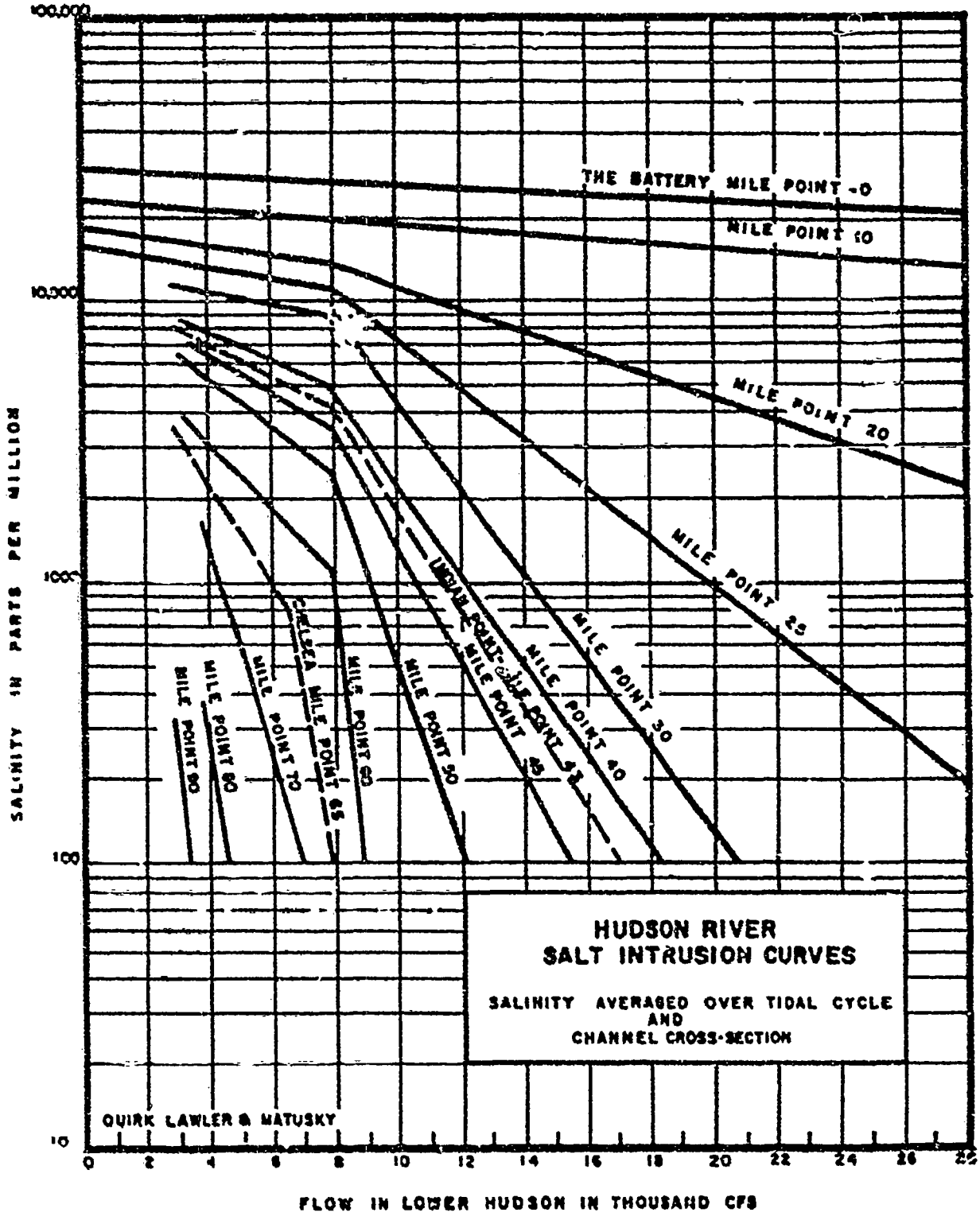

100,000
10,000
1000
100
10
SALINITY IN PARTS PER MILLION
THE BATTERY MILE POINT -0
MILE POINT 10
MILE POINT 20
MILE POINT 25
MILE POINT 30
MILE POINT 40
MILE POINT 50
MILE POINT 60
MILE POINT 65
MILE POINT 70
MILE POINT 80
MILE POINT 90
CHELSEA
INDIAN POINT
HUDSON RIVER
SALT INTRUSION CURVES
SALINITY AVERAGED OVER TIDAL CYCLE AND CHANNEL CROSS-SECTION
QUIRK LAWLER & MATUSKY
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28
FLOW IN LOWER HUDSON IN THOUSAND CFS

Figure 13

of Equation 9 to these data. For example, at a flow of 4000 cfs, the computation for average L between mile points 43 and 65 is:

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

$$= 2830 \text{ sq. ft./sec}$$

$$= 8.74 \text{ sq. mile/day}$$

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

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

$$= 4640 \text{ sq. ft./sec}$$

$$= 14.3 \text{ sq. mile/day}$$

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

EFFECT OF HUDSON RIVER FLOW
ON LONGITUDINAL DISPERSION COEFFICIENT
AVERAGE "E" BETWEEN MILE POINTS 45 & 50
AVERAGE "E" BETWEEN MILE POINTS 43 & 65
DIFFUSIVITY - E, IN MILE²/DAY
FLOW - Q, IN THOUSAND CFS
16
15
14
13
12
11
10
9
8
7
6
5
4
2
3
4
5
6
7
8
9
10

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:

$$c = C_1 e^{jx} + C_2 e^{kx} \qquad \ldots\ldots (11)$$

in which

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

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

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:

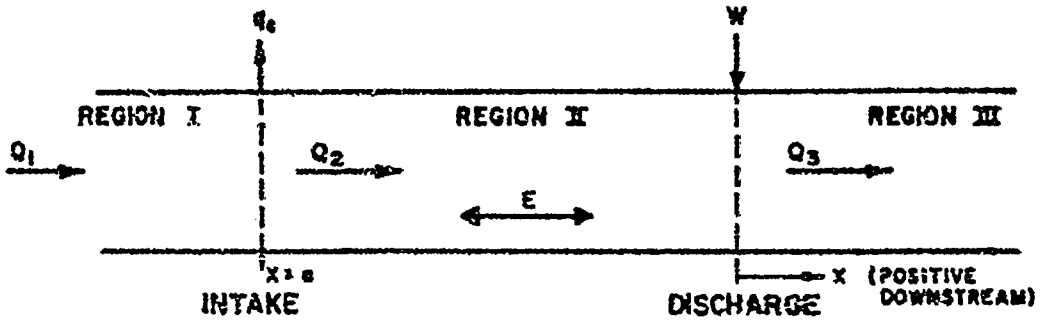
$$c_I = C_1 e^{j_1 x} + C_2 e^{k_1 x} \qquad \ldots\ldots (11a)$$

$$c_{II} = C_3 e^{j_2 x} + C_4 e^{k_2 x} \qquad \ldots\ldots (11b)$$

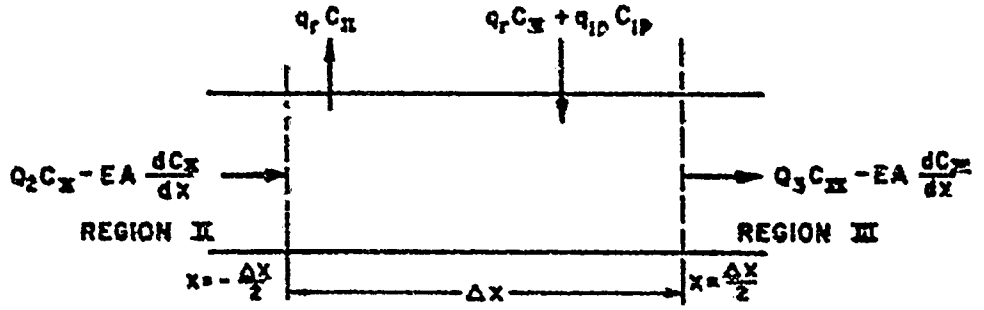
$$c_{III} = C_5 e^{j_2 x} + C_6 e^{k_2 x} \qquad \ldots\ldots (11c)$$

CONTINUOUS DISCHARGE OF RADIOACTIVE CONTAMINANT
EFFECT ON UPRIVER WATER INTAKE

1. THE THREE REGION SYSTEM



2. MATERIAL BALANCE ABOUT POINT OF DISCHARGE



3. MATERIAL BALANCE ABOUT POINT OF INTAKE

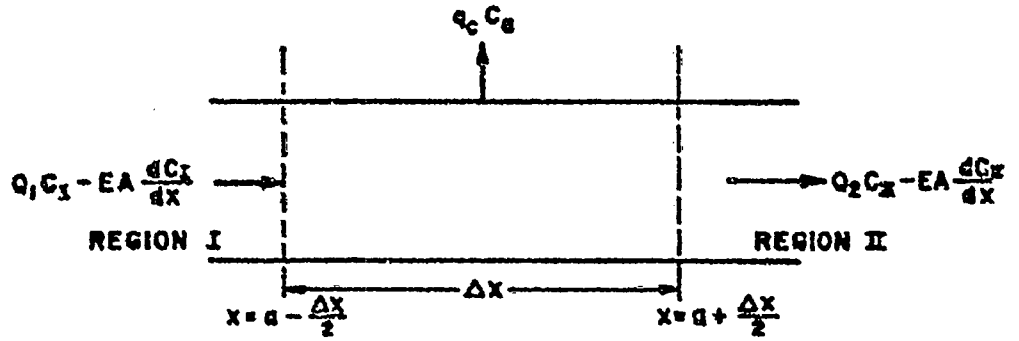


Figure 15

in which

$$\left.\begin{matrix}\lambda_1 \\ k_1\end{matrix}\right\} = \frac{U_1 \pm \sqrt{U_1^2 + 4KE}}{2E}$$

$$\left.\begin{matrix}\lambda_2 \\ k_2\end{matrix}\right\} = \frac{U_2 \pm \sqrt{U_2^2 + 4KE}}{2E}$$

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 infinitely long and the first boundary condition may be written:

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

2. In the upstream 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 derivatives 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 fourth boundary conditions:

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

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

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 $\Delta x \to 0$ yields:

$$q_{IP}\left[c_{IP} - c_{II}\right]_{x=0} = EA\left[\frac{dc_{II}}{dx} - \frac{dc_{III}}{dx}\right]_{x=0} \qquad \ldots\ldots (13)$$

In reality, virtually all of the flow from Indian Point is recirculated from the River. Therefore $q_{IP} \ldots Q_2$, 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:

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

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:

$$\left[Q_1 c_I - EA\,\frac{dc_I}{dx}\right]_{a-\frac{\Delta x}{2}} - q_c c_a - \left[Q_2 c_{II} - EA\,\frac{dc_{II}}{dx}\right]_{a+\frac{\Delta x}{2}} - K\bar{c}\,A\,\Delta x = 0 \qquad \ldots (14)$$

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

As Δx approaches zero, $c_I = c_{II} = c_a$ and Equation 14 becomes:

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

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.

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:

$$c_I = \frac{W\, e^{(j_2 - j_1)\, a + j_1 x}}{AE\,(j_1 - k_2)} \qquad \ldots\ldots (15)$$

$$c_{II} = \frac{W}{AE}\left[\frac{e^{j2\,x}}{j_2 - k_2} + \frac{(j_2 - j_1)\, e^{(j_2 - k_2)\, a + k_2 x}}{(j_2 - k_2)\,(j_1 - k_2)}\right] \qquad \ldots\ldots (16)$$

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

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:

$$C_o = \frac{W}{Q_1}\left[\frac{U_1}{U_2}\left(1 - e^{\frac{U_2 \cdot a}{E}}\right) + e^{\frac{U_2 \cdot a}{E}}\right] \qquad \ldots\ldots (18)$$

$$C_a = \frac{W}{Q_1}\, e^{\frac{U_2 \cdot a}{E}} \qquad \ldots\ldots (19)$$

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:

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

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

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

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

For the case of no drawoff at Chelsea, this reduces to $e^{\frac{U_2}{E}}$.

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:

$$c_1(x,t) = \frac{W}{2Q\sqrt{1 + \frac{4KE}{U^2}}} \left[E \times P\left[\frac{U}{2E}\left(1 + \sqrt{1 + \frac{4KE}{U^2}}\right)x\right] \cdot ERFC\left(\frac{-x}{\sqrt{4Et}} - \sqrt{\frac{U^2 + 4KE}{4E}}\, t\right) \right.$$

$$\left. - EXP\left[\frac{U}{2E}\left(1 - \sqrt{1 + \frac{4KE}{U^2}}\right)x\right] \cdot ERFC\left(\frac{-x}{\sqrt{4Et}} + \sqrt{\frac{U^2 + 4KE}{4E}}\, t\right)\right] \qquad \ldots\ldots (23)$$

The corresponding steady state solution, given by Equation 16 when $j_1 = j_2$ (no drawoff, $U_1 = U_2$), is:

$$c_1(x) = \frac{W}{Q\sqrt{1 + \frac{4KE}{U^2}}} e^{\frac{U}{2E}\left[1 + \sqrt{1 + \frac{4KE}{U^2}}\right]x} \quad \ldots\ldots (24)$$

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

$$\frac{c_1(x,t)}{c_1(x,\infty)} = \frac{1}{2}\left[\mathrm{ERFC}\left[\frac{-x}{\sqrt{4Et}} - \sqrt{\frac{U^2 + 4KE}{4E}}\,t\right] - \mathrm{EXP}\left[-\frac{U}{E}\sqrt{1 + \frac{4KE}{U^2}}\,x\right] \cdot \mathrm{ERFC}\left[\frac{-x}{\sqrt{4Et}} + \sqrt{\frac{U^2 + 4KE}{4E}}\,t\right]\right] \quad \ldots\ldots (25)$$

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

$$\frac{c_1(x,t)}{c_1(x,\infty)} = \frac{1}{2}\left[\mathrm{ERFC}\left[\frac{-x}{\sqrt{4Et}} - \sqrt{\frac{U^2 t}{4E}}\right] - \mathrm{EXP}\left[-\frac{U}{E}x\right] \cdot \mathrm{ERFC}\left[\frac{-x}{\sqrt{4Et}} + \sqrt{\frac{U^2 t}{4E}}\right]\right] \quad \ldots\ldots (26)$$

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.

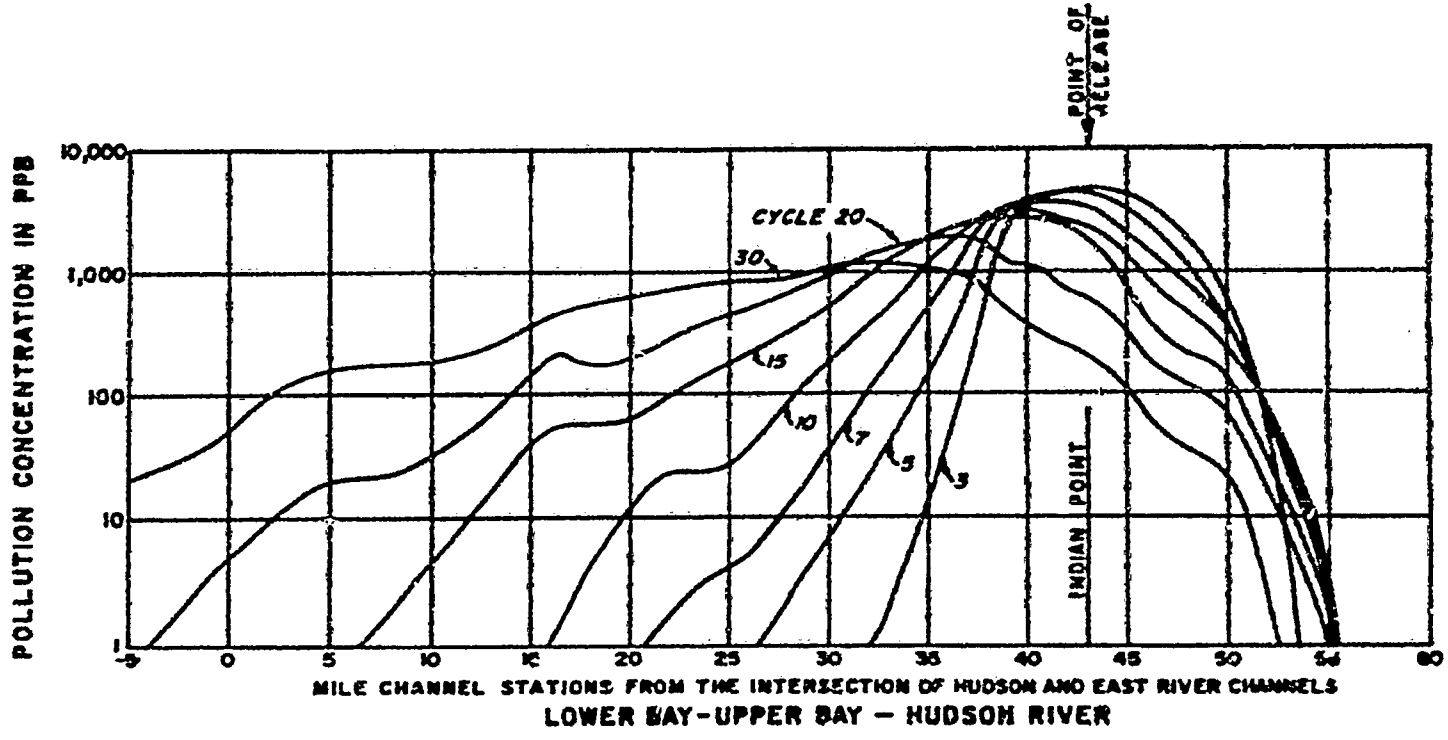
1. PREVIOUS STUDIES

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 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.



NOTE: A TOTAL OF 6 LITERS OF DYE WAS RELEASED AT A UNIFORM RATE DURING 1 TIDAL CYCLE.
THE INITIAL DYE CONCENTRATION WAS 1,000,000 PPB.
HUDSON RIVER AND RARITAN RIVER INFLOWS = 12,000 CFS AND 1770 CFS RESPECTIVELY.

INTERSTATE SANITATION COMMISSION
POLLUTION STUDIES
PLATE 30

DISTRIBUTION OF POLLUTION AT HIGH-WATER SLACK

CONSOLIDATED EDISON CO. TREATMENT PLANT
LOWER BAY-UPPER BAY — HUDSON RIVER

Figure 18

2. CONSTANT PARAMETER ANALYSIS

Dranoff 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: $C|_{t=0} = 0, \; -\infty \leq x \leq \infty$

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

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

Boundary Conditions #4: $AE\left[\left.\frac{dc}{dx}\right|_{x-\to 0} - \left.\frac{dc}{dx}\right|_{x+\to 0}\right] = f(t), \; t > 0$

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

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

in which M = Mass of contaminant released

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

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

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

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

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:

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

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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

Many persons have contributed either directly or indirectly to this report.

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.

Acknowledgment is hereby given to Messrs. Robert Forrest of the Board of Hudson River-Black River Regulating District, Robert J. O'Conner, H. Siemer and Leslie W. Waters of N.Y.C. Board of Water Resources, and Mr. Vincent V. Terenzio of N.Y.C. Board of Water Supply for the use of data and valuable discussions concerning the Sacandaga and Ashokan Dams.

We are grateful also to Messrs. 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 report. Dr. Karel Konrad of Q.L. & M. contributed materially to this study and performed many of the calculations reported herein.

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 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.

3. Quick, Lawler & Matusky 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 Suppliment 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. Flooding resulting from runoff generated by a Hudson River Probable Maximum Precipitation (PMP).
   b. Flooding caused by the occurrence of a dam failure concurrent with heavy runoff generated by a Hudson River

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 [illegible] 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 objectives 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 (#. 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. Weather Bureau for the adjacent Susquehanna Basin at Danville were used in this study. These were adopted on the basis of similarity in topography,

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 subtracted to obtain the subbasin rainfall excesses (runoff) for each subduration. The resulting rainfall and runoff distributions are shown in the upper left hand corner.

The generated 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 hydrographs 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

K·E 10 X 10 TO 1" INCH 46 1320

HUDSON RIVER AT INDIAN POINT — PROBABLE MAXIMUM PRECIPITATION OVER ENTIRE BASIN

& RELATED RUNOFF

6 HR INCREMENT	1	2	3	4	5	6	7	8	9	10	11	12	TOTAL
HR'S FRO~ BEGINNING OF PRE~	6	12	18	24	30	36	42	48	54	60	66	72	72
DEPTH OF RAINFALL (INCHES)	0·69	1·76	0·32	0·55	1·30	4·61	1·74	1·27	0·32	0·30	0·45	0·34	14·02
RUNOFF (INCHES)	0·00	0·49	0·52	0·25	1·28	4·34	1·49	0·97	0·02	0·09	0·13	0·04	9·72

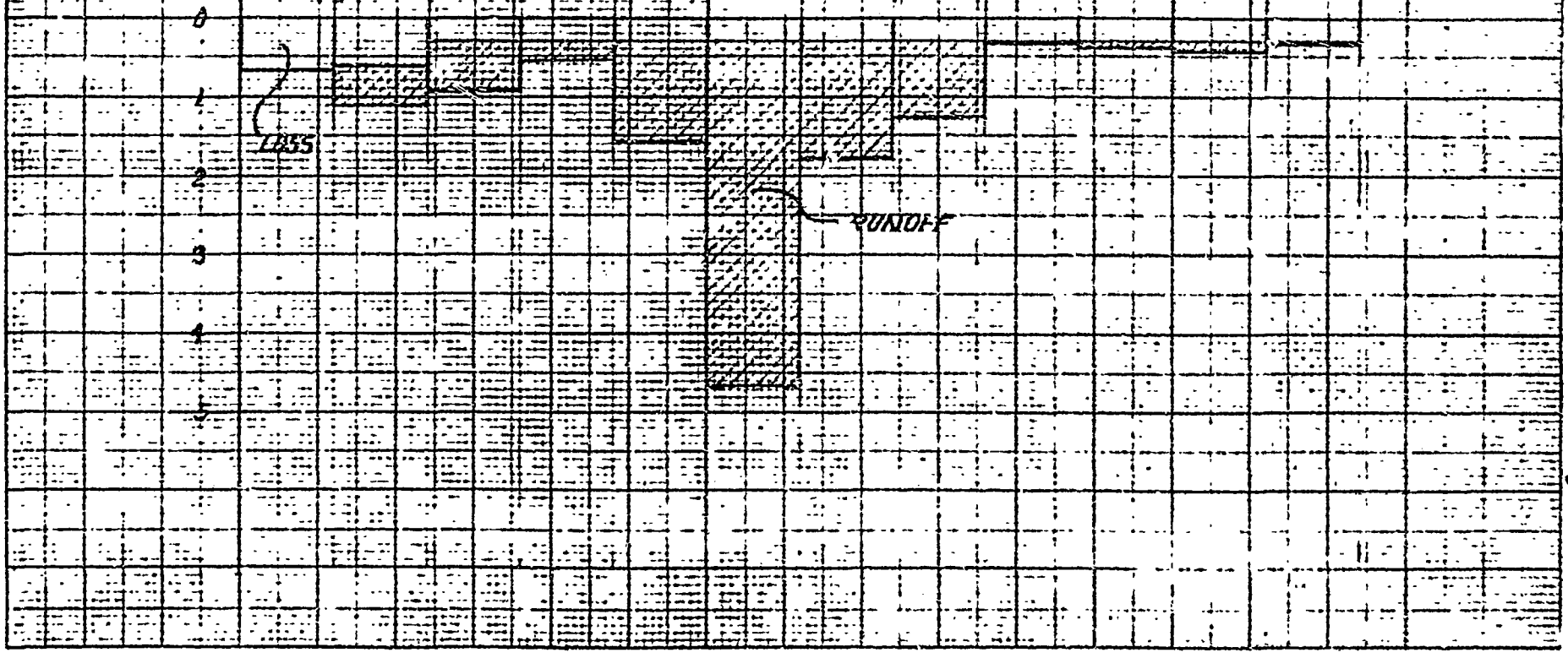
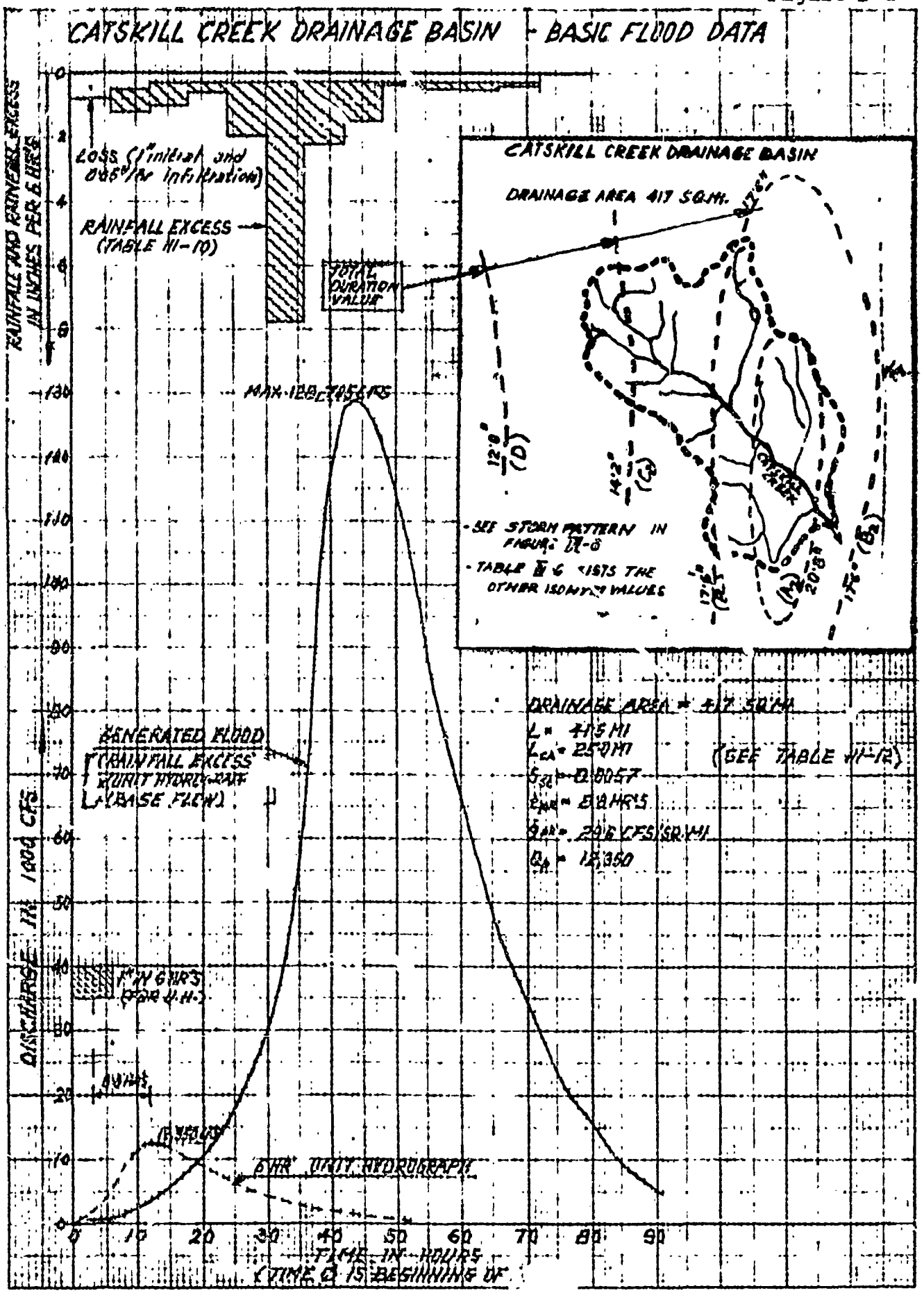


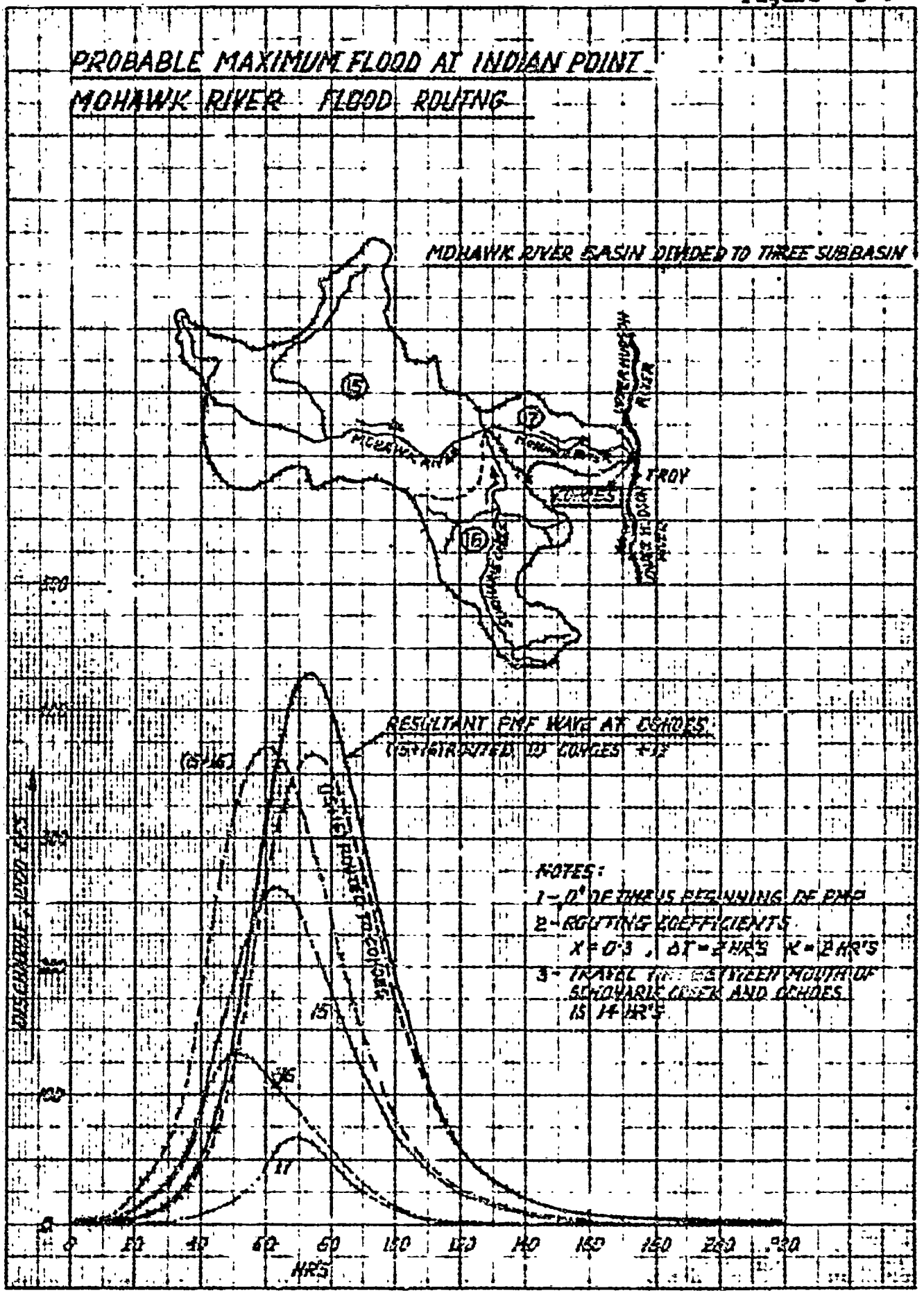
Figure S-1

Figure S-2



CATSKILL CREEK DRAINAGE BASIN - BASIC FLOOD DATA
RAINFALL AND RAINFALL EXCESS IN INCHES PER 6 HR'S
LOSS (1" initial and 0.05"/hr infiltration)
RAINFALL EXCESS (TABLE III-10)
TOTAL DURATION VALUE
CATSKILL CREEK DRAINAGE BASIN
DRAINAGE AREA 417 SQ.MI.
- SEE STORM PATTERN IN FIGURE III-8
- TABLE III-6 LISTS THE OTHER ISOHYET VALUES
MAX. 128,705 CFS
GENERATED FLOOD
(RAINFALL EXCESS x UNIT HYDROGRAPH) + (BASE FLOW)
DRAINAGE AREA = 417 SQ.MI.
L = 41.5 MI
L_{CA} = 25.0 MI
(SEE TABLE III-12)
S_{SL} = 0.0057
t_{pr} = 8.8 HR'S
q_{pr} = 29.6 CFS/SQ.MI.
Q_p = 12,350
1" IN 6 HR'S (FOR U.H.)
6 HR UNIT HYDROGRAPH
DISCHARGE IN 1000 CFS
TIME IN HOURS
(TIME Ø IS BEGINNING OF

Figure S-3


PROBABLE MAXIMUM FLOOD AT INDIAN POINT
MOHAWK RIVER FLOOD ROUTING
MOHAWK RIVER BASIN DIVIDED TO THREE SUBBASIN
(15)
(16)
(17)
TROY
COHOES
RESULTANT PMF WAVE AT COHOES
(DISTRIBUTED TO COHOES +15)
NOTES:
1 - 0' OF TIME IS BEGINNING OF PMF
2 - ROUTING COEFFICIENTS
X = 0.3 , ΔT = 2 HRS K = 2 HR'S
3 - TRAVEL TIME BETWEEN MOUTH OF SCHOHARIE CREEK AND COHOES IS 14 HR'S
HRS

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 deliberately 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. erosion of the surface of the downstream face. The specific details follow.

9. The Sacandaga Reservoir Dam, known as the Conklingville Dam, is extremely stable and has proven its ability to withstand sever

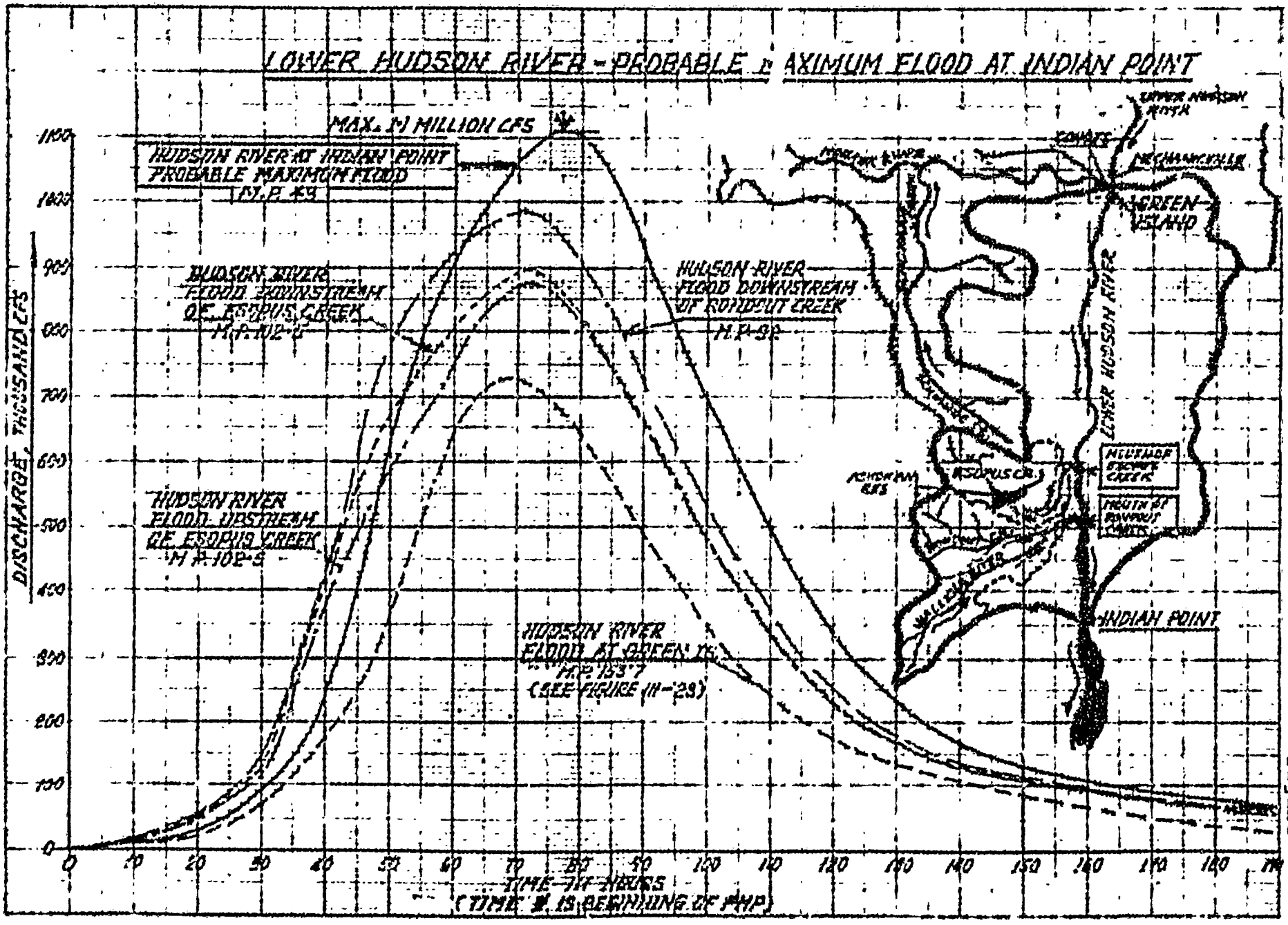

LOWER HUDSON RIVER - PROBABLE MAXIMUM FLOOD AT INDIAN POINT
MAX. M MILLION CFS
HUDSON RIVER AT INDIAN POINT
PROBABLE MAXIMUM FLOOD
M.P. 43
HUDSON RIVER
FLOOD DOWNSTREAM
OF ESOPUS CREEK
M.P. 102-S
HUDSON RIVER
FLOOD UPSTREAM
OF ESOPUS CREEK
M.P. 102-S
HUDSON RIVER
FLOOD DOWNSTREAM
OF ROUNDOUT CREEK
M.P. 92
HUDSON RIVER
FLOOD AT GREEN I.
M.P. 133.7
(SEE FIGURE II-23)
DISCHARGE, THOUSAND CFS
TIME IN HOURS
(TIME 0 IS BEGINNING OF PMF)
LOWER HUDSON RIVER
ESOPUS CR.
INDIAN POINT
GREEN ISLAND

Figure 5-4

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 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 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 outflow 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 from 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 cfs.

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 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.

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 PMF 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 maximum 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 12, 1960 and which is the storm having the greatest effect since 1821.

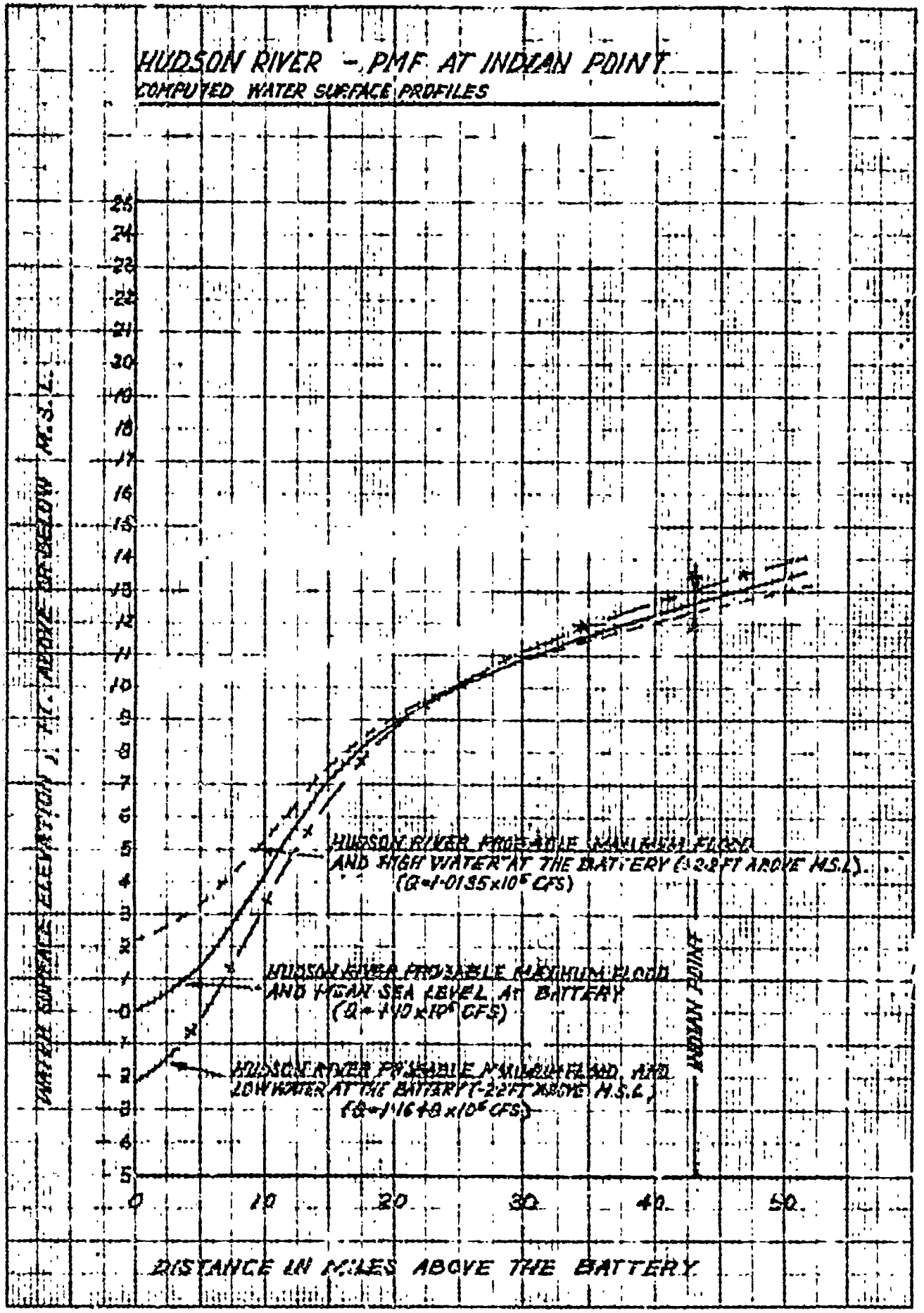
Case No. 6 considers an even more critical set of occurrences com-

TABLE 8-1

WATER SURFACE ELEVATION AT INDIAN POINT
RESULTING FROM STATED FLOW & ELEVATION CONDITIONS

Component Flow at Indian Point	Elevation at the Battery	Flow at Indian Point in Million cfs	Elevation at Indian Point (MSL Datum)	Elevation at Indian Point including local oscillatory wave height (MSL Datum)
1. Probable Max. Flood	Mean Sea Level (0.00)	1.100	12.7	13.7
2. Probable Max. Flood & Tidal Flow	High Water (+2.20)	1.014	12.4	13.4
3. Probable Max. Flood & Tidal Flow	Low Water (-2.20)	1.165	13.0	14.0
4. Standard Project Flood & Ashokan Dam Failure	Mean Sea Level (0.00)	0.705	7.2	8.2
5. Standard Project Flood	Standard Project Hurricane (+11.00)	0.550	13.0	14.0
6. Standard Project Flood & Ashokan Dam Failure	Standard Project Hurricane (+11.00)	0.705	14.0	15.0
7. Probable Max. Hurricane & Spring High Tide	Probable Max. Hurricane (+17.5)		13.5	14.5

FIGURE 5-5


HUDSON RIVER - PMF AT INDIAN POINT
COMPUTED WATER SURFACE PROFILES
WATER SURFACE ELEVATION IN FT. ABOVE OR BELOW M.S.L.
HUDSON RIVER PROBABLE MAXIMUM FLOOD AND HIGH WATER AT THE BATTERY (+2.8 FT ABOVE M.S.L.) (Q=1.0135x10^6 CFS)
HUDSON RIVER PROBABLE MAXIMUM FLOOD AND MEAN SEA LEVEL AT BATTERY (Q=1.10x10^6 CFS)
HUDSON RIVER PROBABLE MAXIMUM FLOOD AND LOW WATER AT THE BATTERY (-2.2 FT ABOVE M.S.L.) (Q=1.1648x10^6 CFS)
INDIAN POINT
DISTANCE IN MILES ABOVE THE BATTERY

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

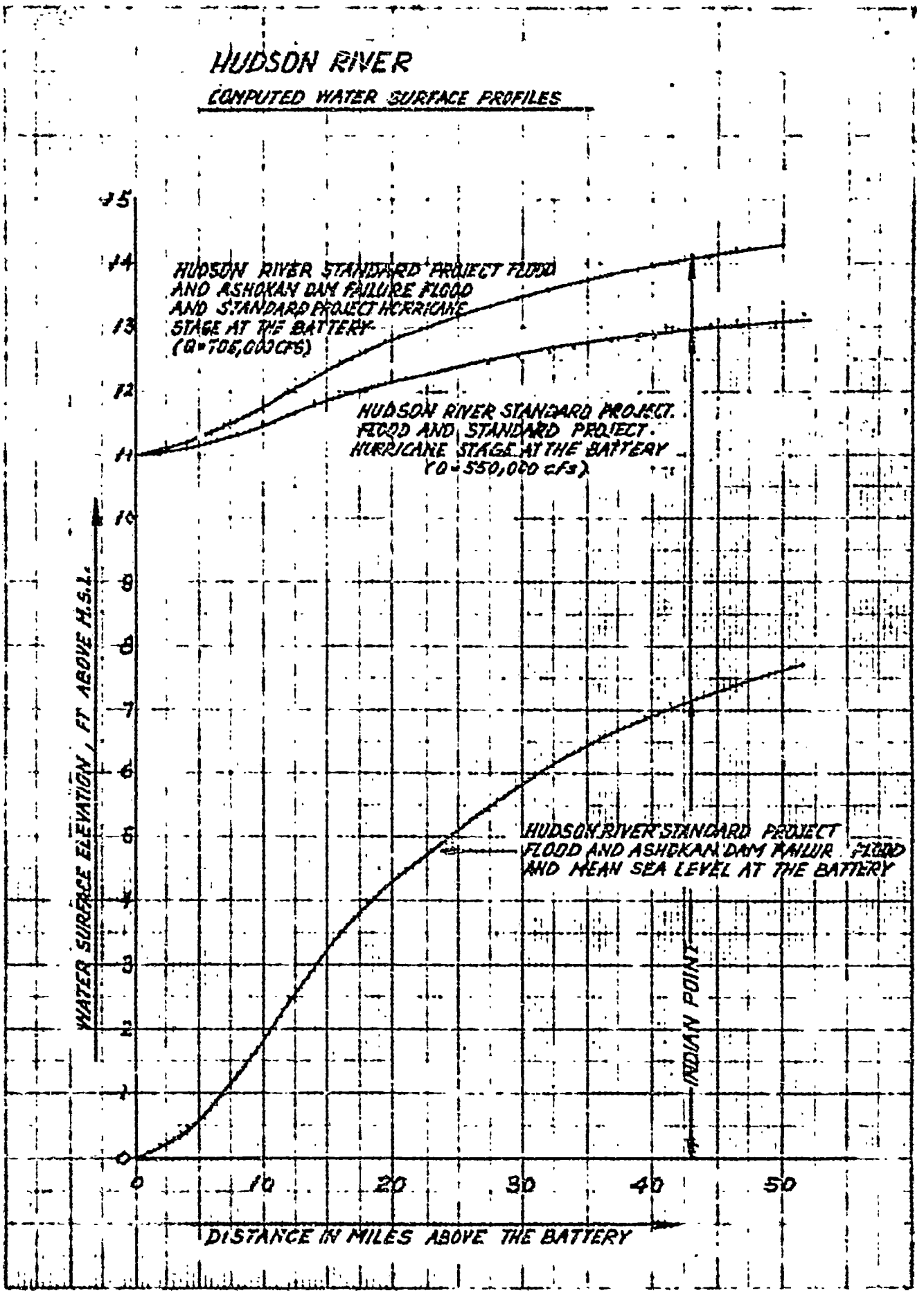
The simultaneous occurrence of these three conditions is extremely remote. Each one of these three conditions are in themselves unlikely events. The combination of all three decreases, therefore, the likelihood of occurrence significantly.

The flow profiles corresponding to cases No. 4 through 6 are depicted in Figure 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 maximum elevation at Indian Point due to flooding and wave runup is 15 feet or less.

FIGURE 8-6


HUDSON RIVER
COMPUTED WATER SURFACE PROFILES
HUDSON RIVER STANDARD PROJECT FLOOD AND ASHOKAN DAM FAILURE FLOOD AND STANDARD PROJECT HURRICANE STAGE AT THE BATTERY (Q=705,000 CFS)
HUDSON RIVER STANDARD PROJECT FLOOD AND STANDARD PROJECT HURRICANE STAGE AT THE BATTERY (Q=550,000 cfs)
HUDSON RIVER STANDARD PROJECT FLOOD AND ASHOKAN DAM FAILUR FLOOD AND MEAN SEA LEVEL AT THE BATTERY
INDIAN POINT
WATER SURFACE ELEVATION, FT ABOVE M.S.L.
DISTANCE IN MILES ABOVE THE BATTERY
0
10
20
30
40
50

I. INTRODUCTION

Consolidated Edison Company of New York, Incorporated, plans to build a third nuclear generating unit on the Hudson River 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.

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 (PSA Site safety criteria for this report requires an analysis of the area's hydrology.

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 conditions of open

seas and neglected frictional resistance in the Hudson River Channel.

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.

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 QL & 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

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 QL & M to discuss the proposed program and to delineate requirements for further refinements. At these meetings, a 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.

The report is formulated as follows:

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

II. DESCRIPTION OF HUDSON RIVER BASIN *

A. Description of the Basin

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 II-1. 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 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 the first two sections of this chapter is based on several Federal and State publications on the Hudson River

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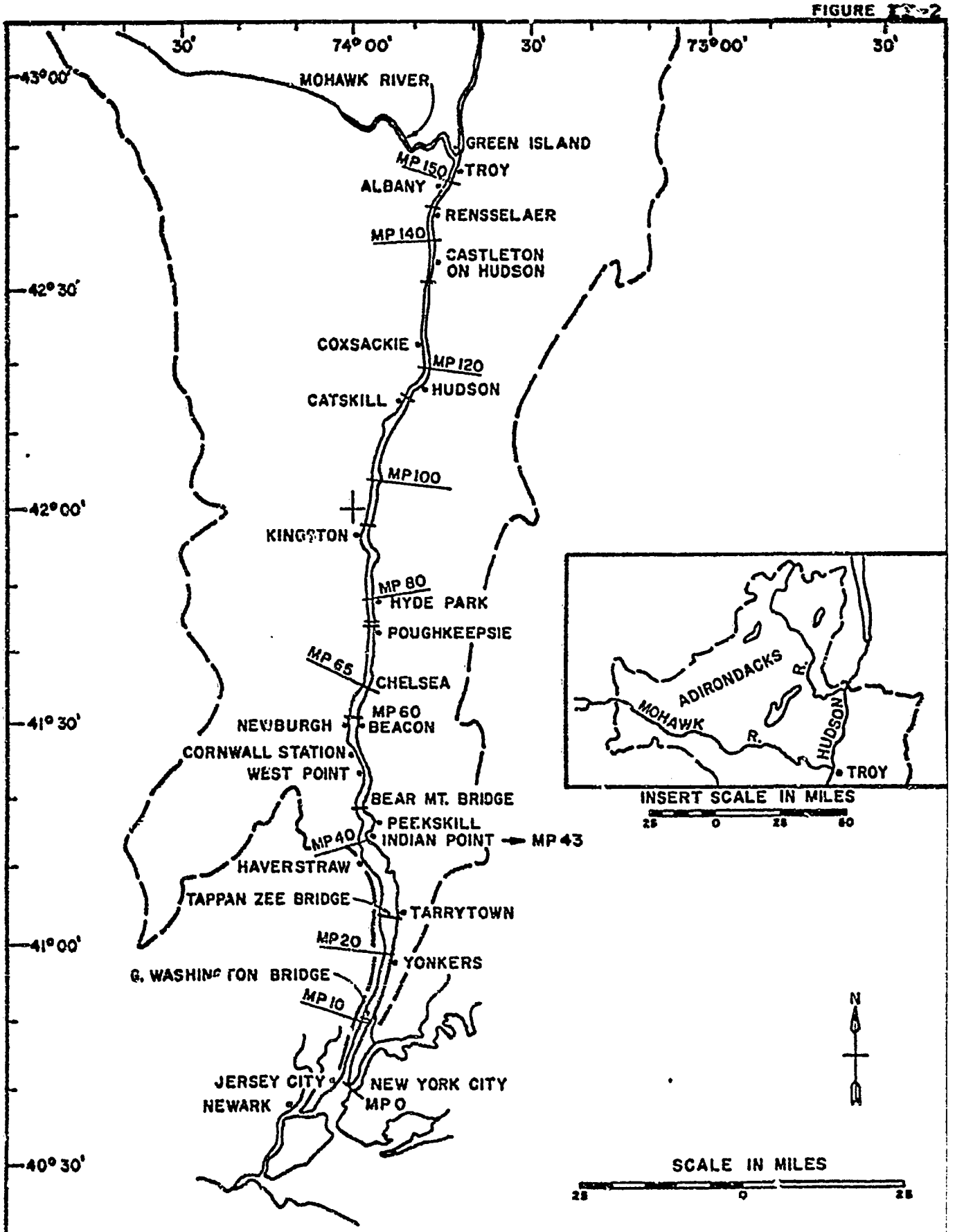
FIGURE II-2



HUDSON RIVER DRAINAGE BASIN

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 slope 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.

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

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 and 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.

The major physiographic features of the Hudson River Basin are a mountainous terrain covering 48 per cent of the basin; 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 and 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 from 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 and -42°F respectively.

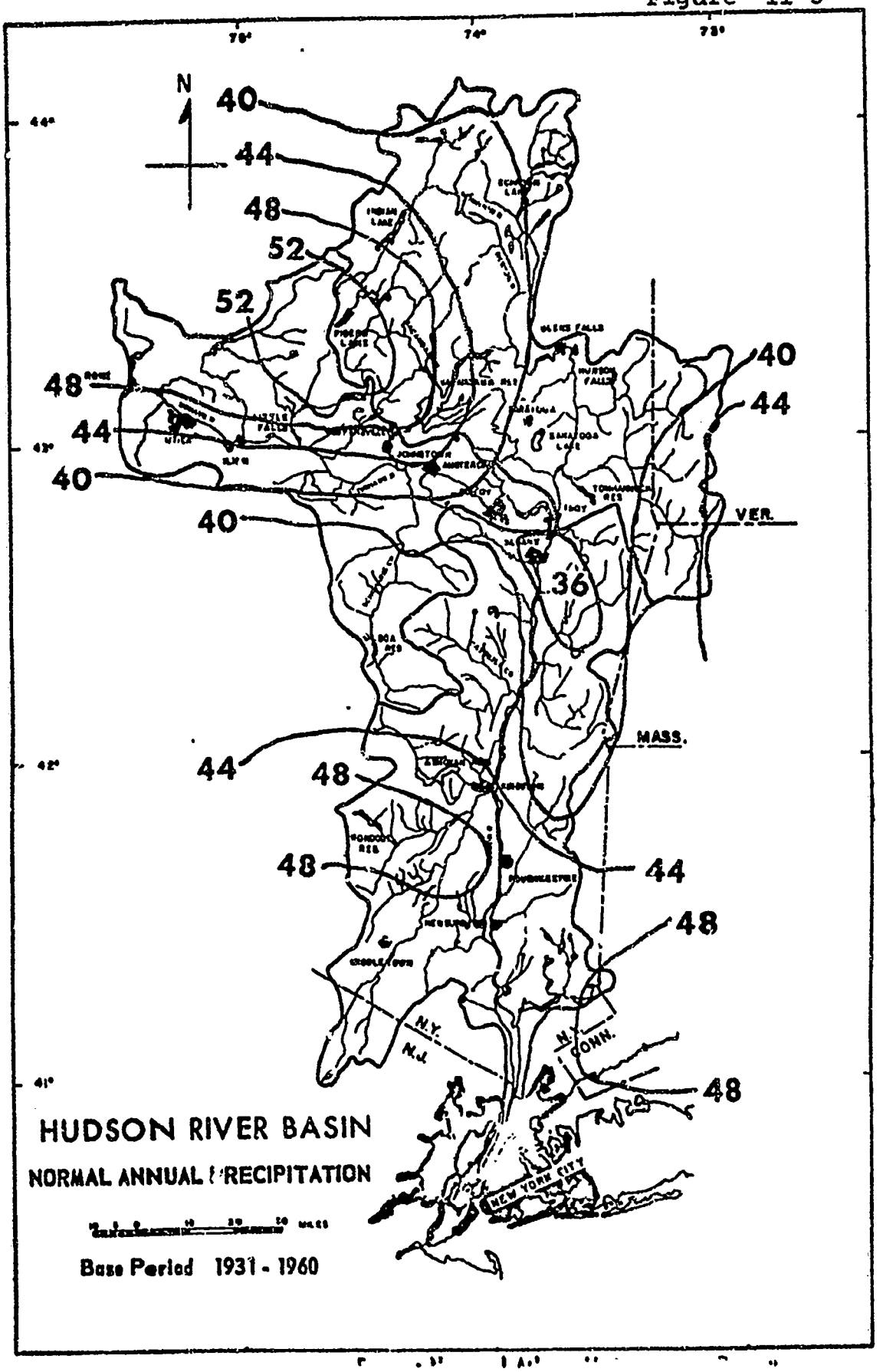
The average annual precipitation varies from 34 inches in the center of the basin to more than 50 inches in the Adirondacks. Most of the basin receives an average of about 40 inches. In general, the precipitation is distributed evenly through the year with a slight rise during the summer. Figure II-3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

Figure II-3



N
40
44
48
52
52
48
44
40
40
40
44
48
48
44
48
48
36
VER.
MASS.
N.Y.
N.J.
N.Y.
CONN.
NEW YORK CITY
HUDSON RIVER BASIN
NORMAL ANNUAL PRECIPITATION
Base Period 1931 - 1960

TABLE II-2

HUDSON RIVER BASIN CHARACTERISTICS

Subbasin	Drainage Area (sq.mi.)	Channel Length (miles)	Mean Annual Temp °F	Mean Annual precipation (inches)	Mean Annual Evaporation & Trans Precipitation	Mean Annual Runoff			Storage Usable Capacity acre - ft.
						cfs	cfs sq.mi.	in.	
Upper Hudson River Basin	4627	150	40	40	16	7400	1.60	24	863,000
Mohawk River Basin	3462	155	45	46	24	5600	1.61	22	203,900
Lower Hudson River Basin	4561	154	48	42	20	8700*	1.90	22	551,000

* Estimated

represents the statistically probable value of the ratio 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 flood.

TABLE II-3

STREAMGAGE SUMMARY AT REPRESENTATIVE STATIONS IN THE HUDSON RIVER BASIN

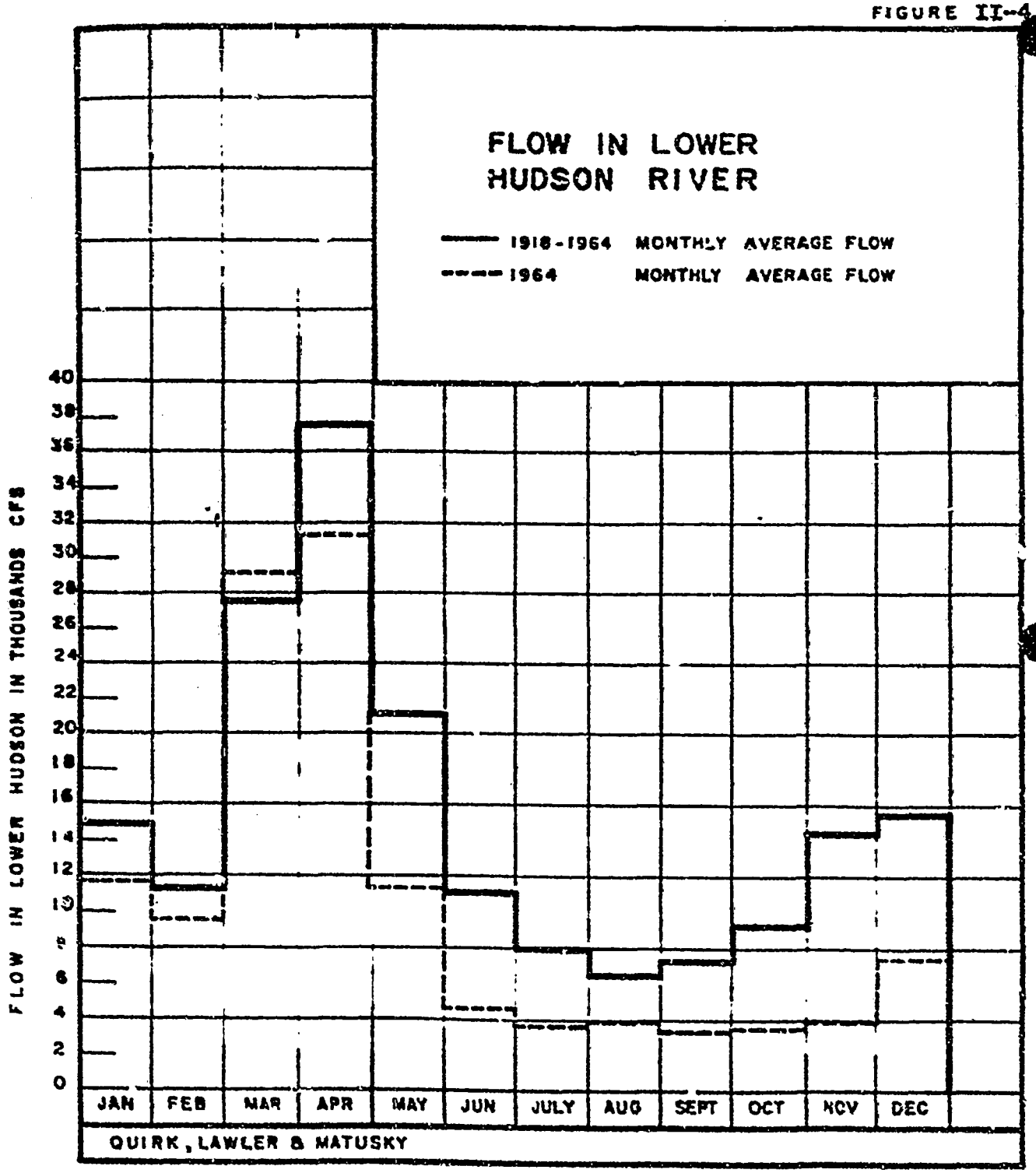
Station	Area (sq.mi.)	Mean Flow (cfs)	Mean Runoff (csm)	Mean Runoff (inches)	Years of Record
Mohawk River Basin					
Mohawk River					
near Rome	150	379	2.53	34.4	47
near Little Falls	1,348	2,708	2.01	27.1	[illegible]
at Cohoes	3,453	5,537	1.61	21.8	43
Upper Hudson River Basin					
Hudson River					
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
Kinderhook Creek at Rossman	329	454	1.38	18.8	44

TABLE II-3 (Continued)

STREAMGAGE SUMMARY AT REPRESENTATIVE STATIONS IN THE HUDSON RIVER BASIN

Station	Area (sq.mi.)	Mean Flow (cfs)	Mean Runoff (csm)	Mean Runoff (inches)	Years of Record
Catskill Creek at Oak Hill	98	126	1.29	17.5	5
Walkill River at Gardiner	711	1,045	1.47	20.0	44

FIGURE II-4


FLOW IN LOWER HUDSON RIVER
1918-1964 MONTHLY AVERAGE FLOW
1964 MONTHLY AVERAGE FLOW
FLOW IN LOWER HUDSON IN THOUSANDS CFS
40
38
36
34
32
30
28
26
24
22
20
18
16
14
12
10
8
6
4
2
0
JAN
FEB
MAR
APR
MAY
JUN
JULY
AUG
SEPT
OCT
NOV
DEC
QUIRK, LAWLER & MATUSKY

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 buil 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.

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.l.)	Peak Discharge (c.f.s.)	Peak Stage (ft.m.s.l.)	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	29.17	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.

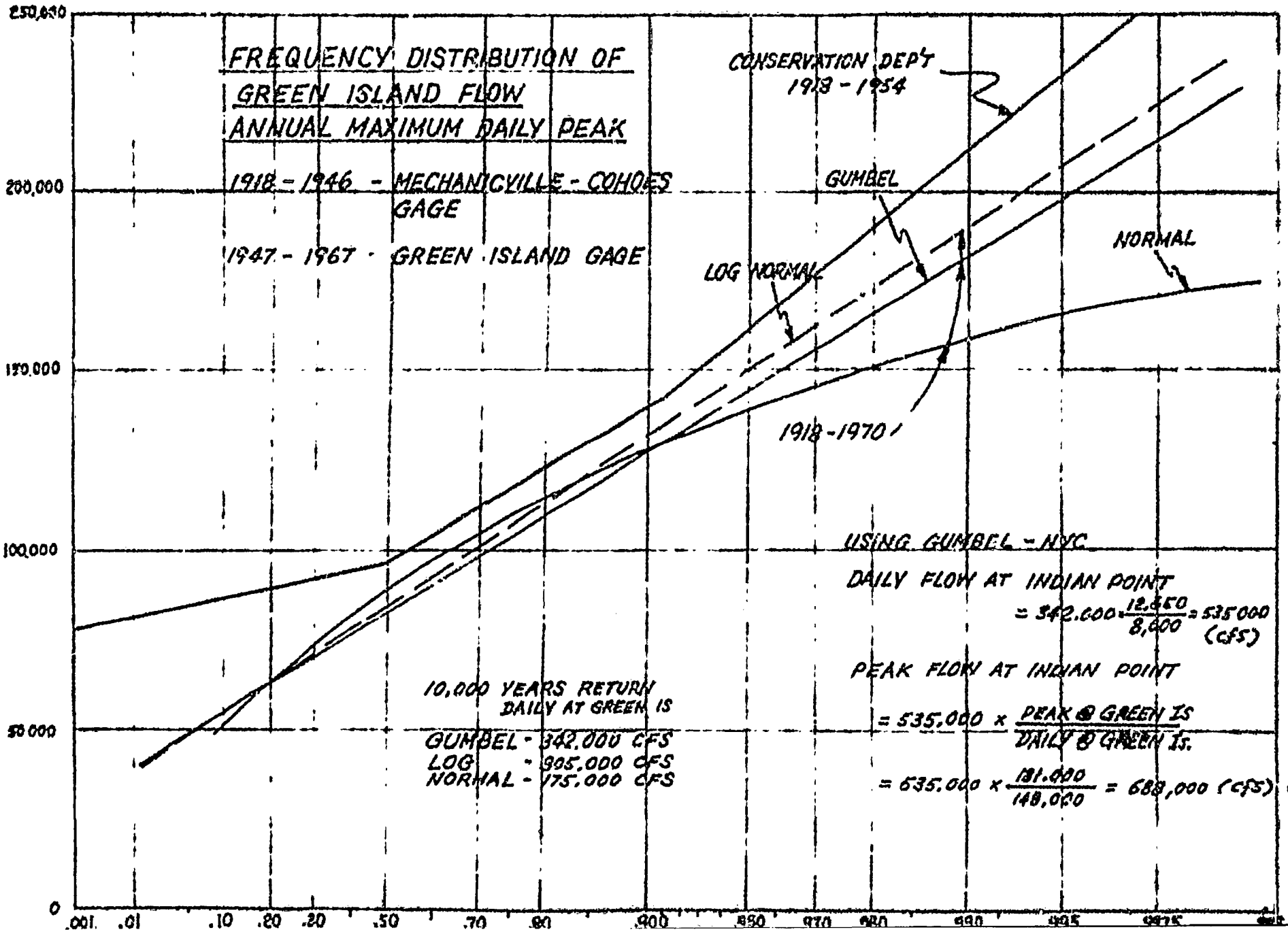

250,000
200,000
150,000
100,000
50,000
0
.001
.01
.10
.20
.30
.50
.70
.90
.950
.970
.980
.990
.995
.9975
FREQUENCY DISTRIBUTION OF
GREEN ISLAND FLOW
ANNUAL MAXIMUM DAILY PEAK
1918 - 1946 - MECHANICVILLE - COHOES GAGE
1947 - 1967 - GREEN ISLAND GAGE
CONSERVATION DEPT
1918 - 1954
GUMBEL
LOG NORMAL
NORMAL
1918 - 1970
USING GUMBEL - NYC
DAILY FLOW AT INDIAN POINT
= 342,000 × 12,500/8,000 = 535,000 (CFS)
PEAK FLOW AT INDIAN POINT
= 535,000 × PEAK @ GREEN IS / DAILY @ GREEN IS.
= 535,000 × 131,000/148,000 = 688,000 (CFS)
10,000 YEARS RETURN
DAILY AT GREEN IS
GUMBEL - 342,000 CFS
LOG - 905,000 CFS
NORMAL - 775,000 CFS

Figure II-5

TABLE II-5

MAJOR HUDSON RIVER BASIN RESERVOIR SUMMARY

No.	Reservoir	Year Placed in Service	Stream	Usable Capacity (ac-ft)	Drainage Area	Purposes	Distance from Indian 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 (Gilboa)	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 Regulation, Power	250
5	Sacandaga Reservoir	1931	Sacandaga River	760,000	1,044	River Regulation, Flood Control, Power	180
	LOWER HUDSON RIVER BASIN						
6	Ashokan Reservoir (Olive Bridge)	1915	Esopha Creek	393,000	257	New York City Water Supply	75
7	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 variation 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.

CROSS-SECTIONAL AREA
FROM MP -30 TO MP 153.7
HUDSON RIVER ESTUARY
QUIRK, LAWLER & MATUSKY ENGINEERS
CROSS-SECTIONAL AREA, THOUSAND FT²
600
500
400
300
200
100
HAVERSTRAW BAY
TAPPAN ZEE
NEWBURGH BAY
TROY DAM - MILE POINT 153.7
-30 -20 -10 0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150
MILES ABOVE BATTERY

FIGURE II-6

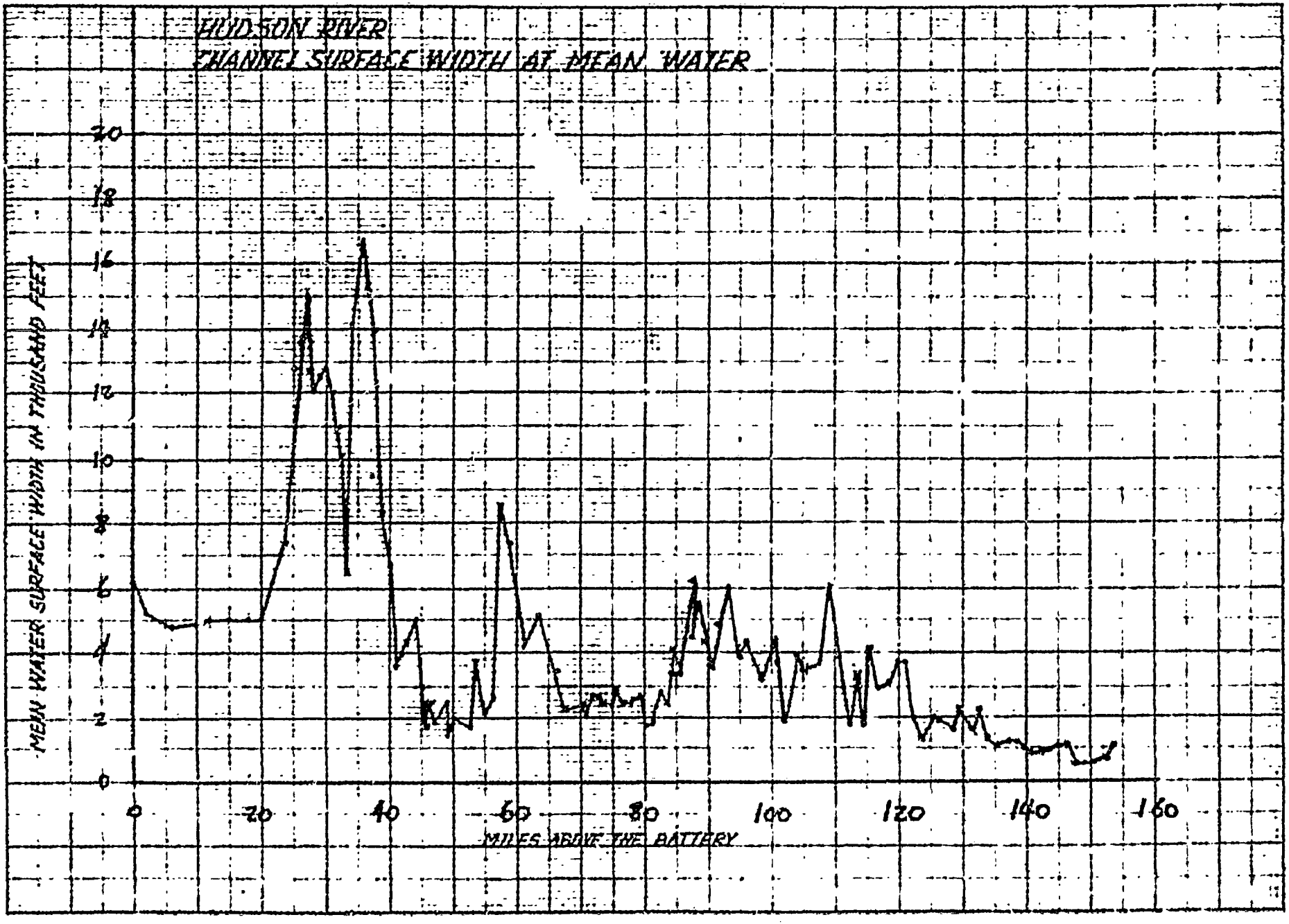

HUDSON RIVER
CHANNEL SURFACE WIDTH AT MEAN WATER
MEAN WATER SURFACE WIDTH IN THOUSAND FEET
20
18
16
14
12
10
8
6
4
2
0
0
20
40
60
80
100
120
140
160
MILES ABOVE THE BATTERY

Figure II-7

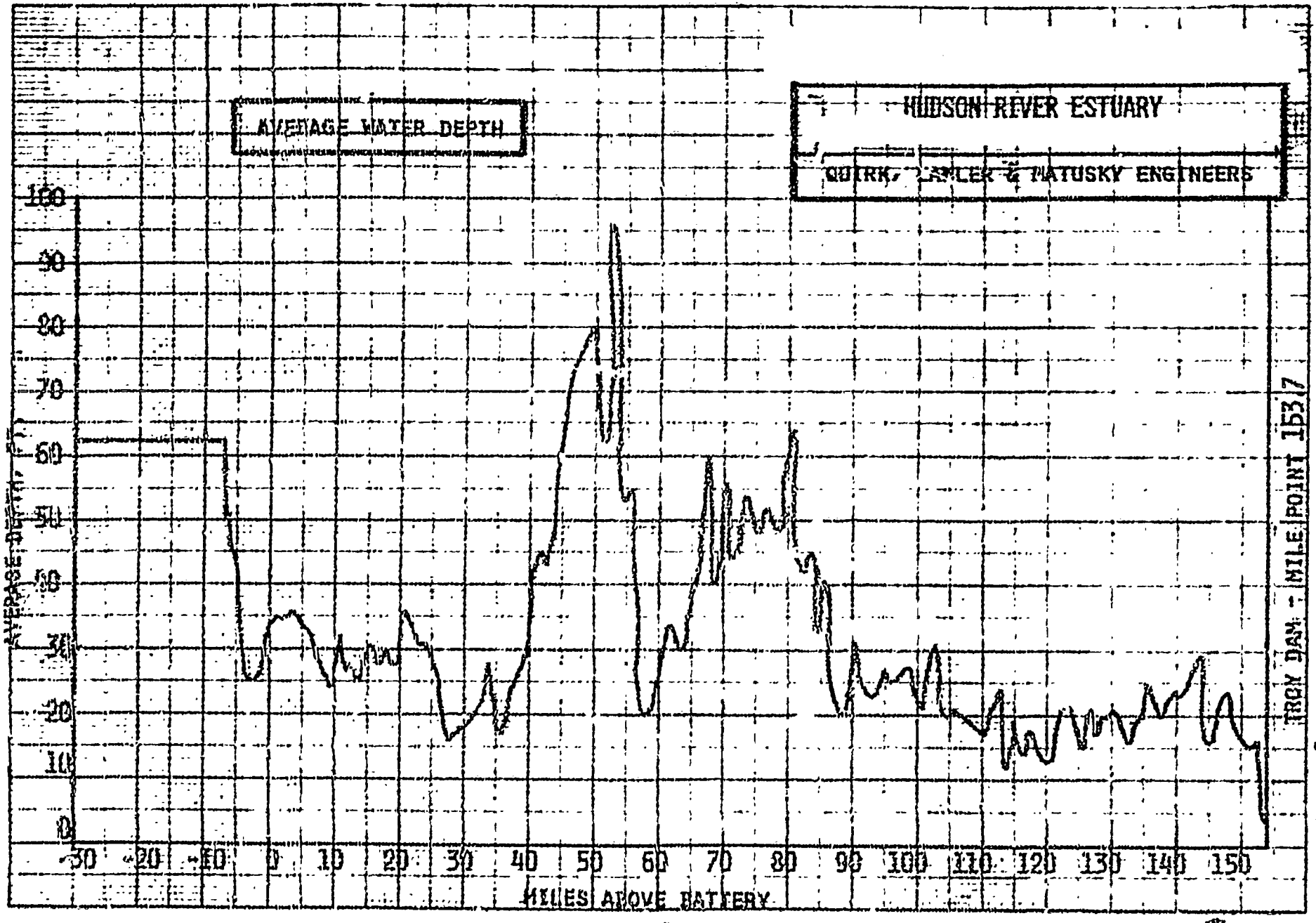

AVERAGE WATER DEPTH
HUDSON RIVER ESTUARY
QUIRK, LAWLER & MATUSKY ENGINEERS
AVERAGE DEPTH, FT
100
90
80
70
60
50
40
30
20
10
0
-30 -20 -10 0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150
MILES ABOVE BATTERY
TROY DAM - MILE POINT 153.7

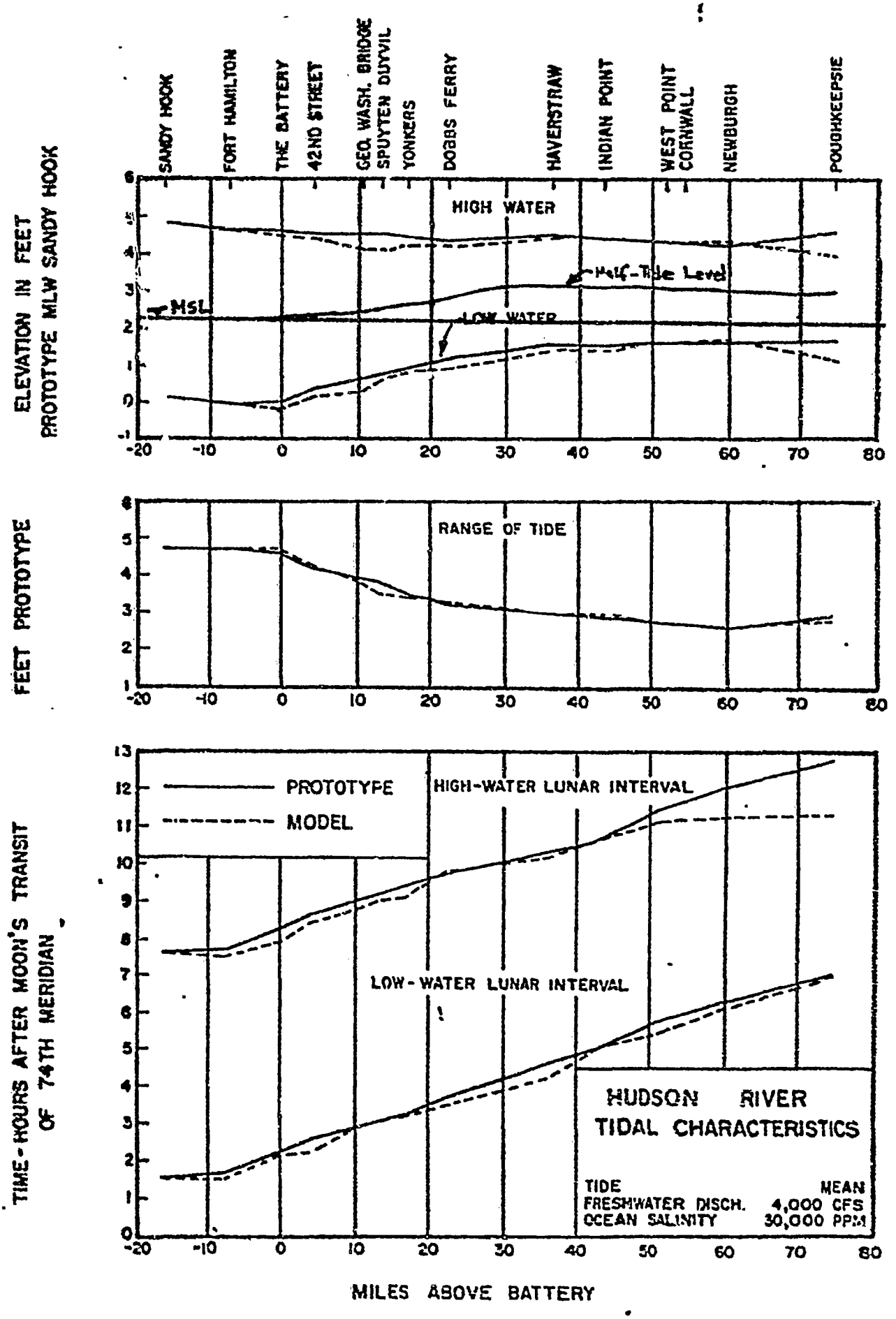
Figure II-8

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.

FIGURE II-9


SANDY HOOK
FORT HAMILTON
THE BATTERY
42ND STREET
GEO. WASH. BRIDGE
SPUYTEN DUYVIL
YONKERS
DOBBS FERRY
HAVERSTRAW
INDIAN POINT
WEST POINT
CORNWALL
NEWBURGH
POUGHKEEPSIE
ELEVATION IN FEET
PROTOTYPE MLW SANDY HOOK
HIGH WATER
Half-Tide Level
MSL
LOW WATER
FEET PROTOTYPE
RANGE OF TIDE
TIME-HOURS AFTER MOON'S TRANSIT
OF 74TH MERIDIAN
PROTOTYPE
MODEL
HIGH-WATER LUNAR INTERVAL
LOW-WATER LUNAR INTERVAL
HUDSON RIVER
TIDAL CHARACTERISTICS
TIDE MEAN
FRESHWATER DISCH. 4,000 CFS
OCEAN SALINITY 30,000 PPM
MILES ABOVE BATTERY

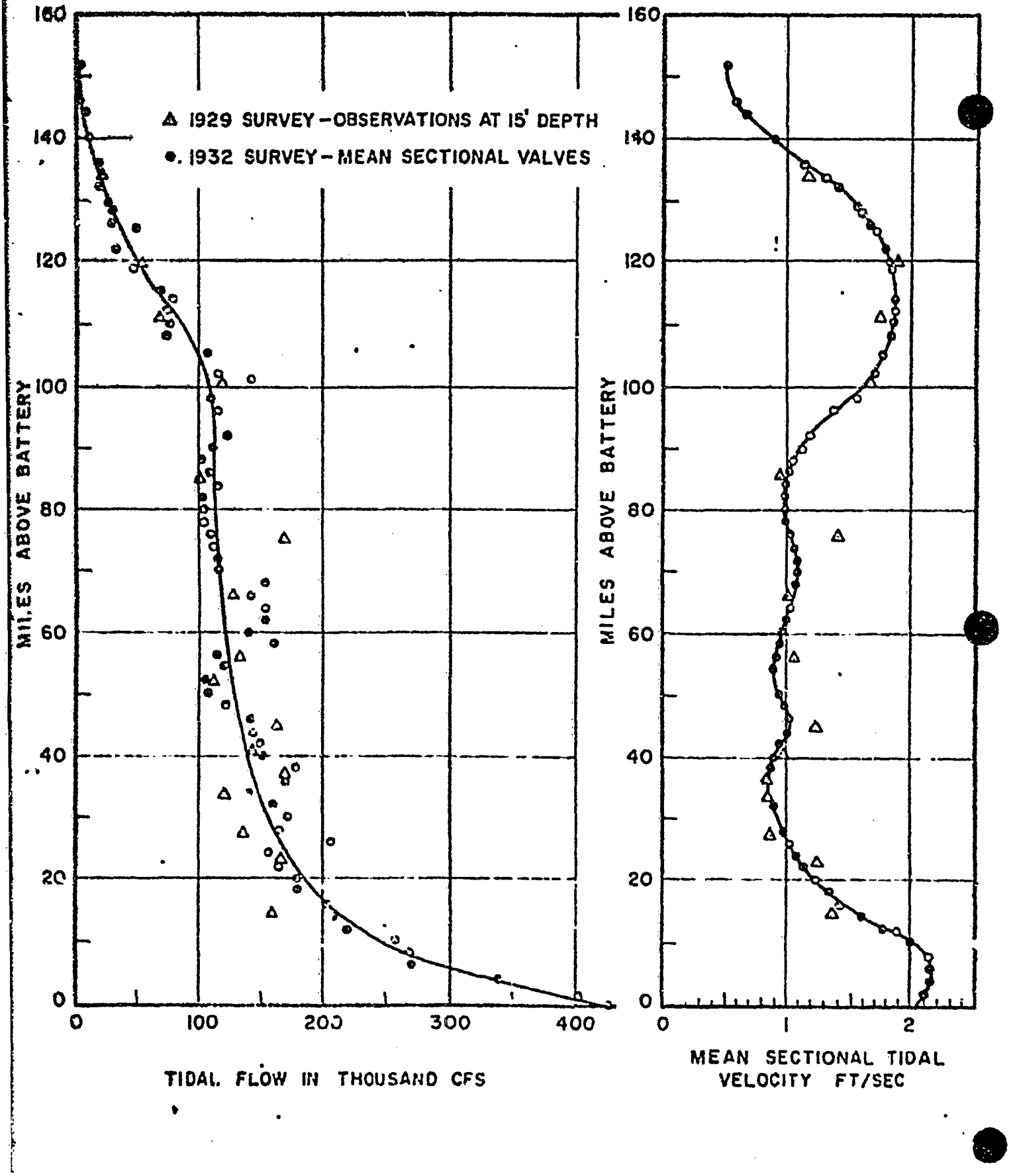
The variation in the tidal range along the river is shown in the middle figure. It will be noted that, in moving upstream, the range of tide diminishes from about 4.4 feet at the Battery to a minimum of about 2.6 feet at a point near Storm King (mile point 56). The tidal range then reaches a maximum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

FIGURE II-10



HUDSON RIVER

TIDAL FLOW AND VELOCITY

Hudson River salt intrusions at equilibrium, or near equilibrium conditions, 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 electrical measurements and 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, Vicksburg, Mississippi. Salinities shown are tidal average values over a tidal cycle and a full channel cross-section. The Vicksburg 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.

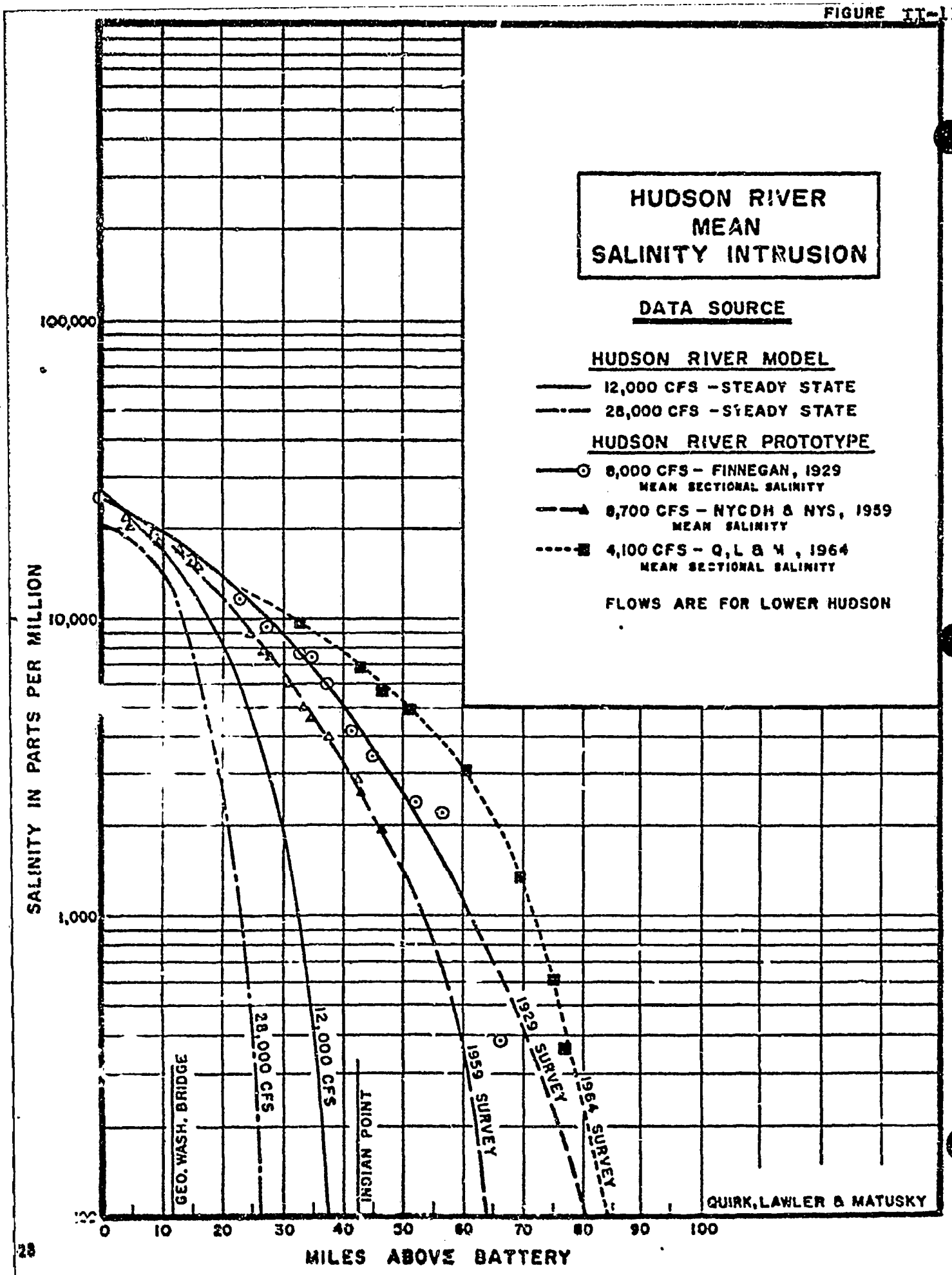
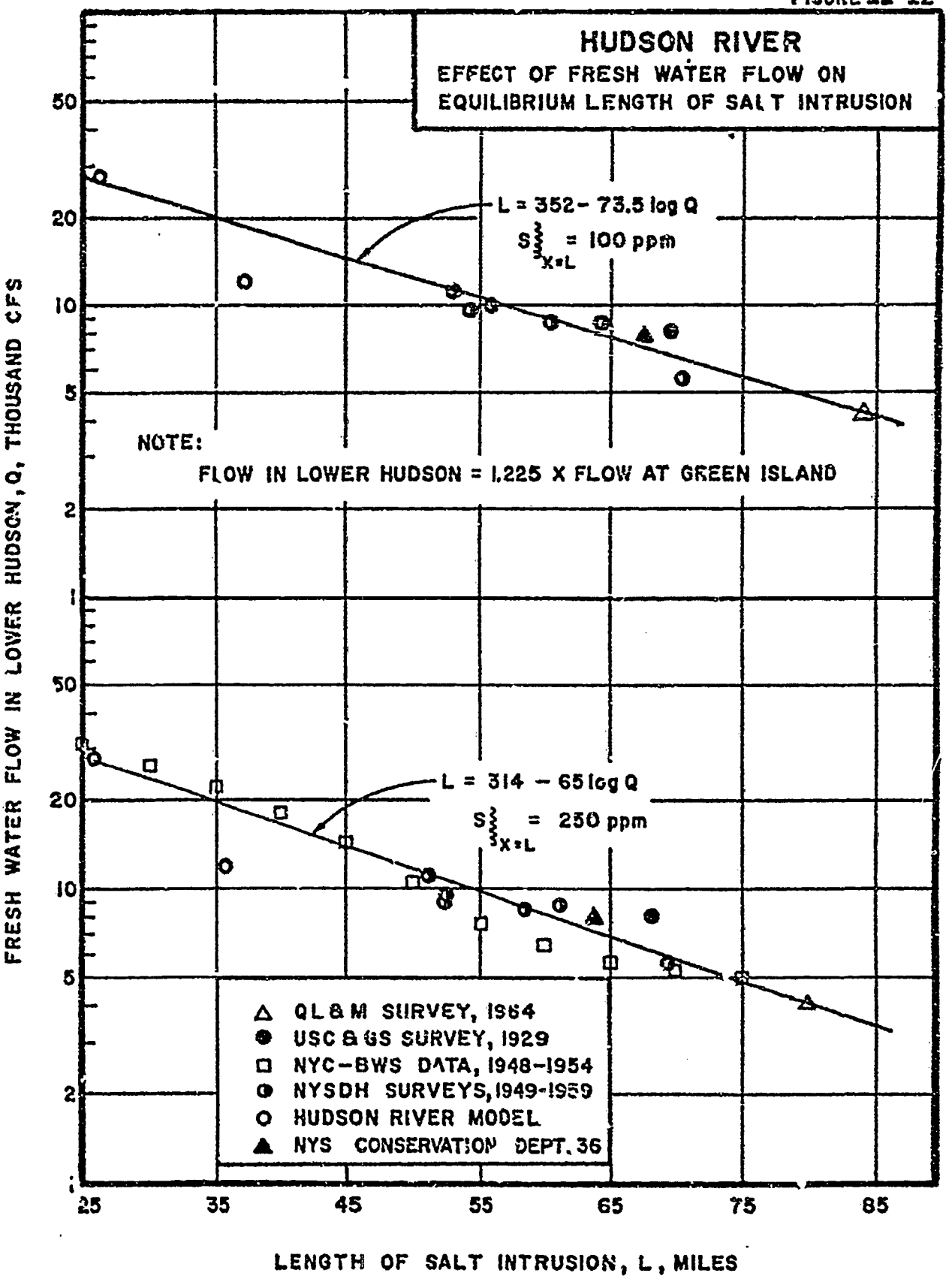

HUDSON RIVER
MEAN
SALINITY INTRUSION
DATA SOURCE
HUDSON RIVER MODEL
12,000 CFS - STEADY STATE
28,000 CFS - STEADY STATE
HUDSON RIVER PROTOTYPE
8,000 CFS - FINNEGAN, 1929
MEAN SECTIONAL SALINITY
8,700 CFS - NYCDH & NYS, 1959
MEAN SALINITY
4,100 CFS - Q,L & W, 1964
MEAN SECTIONAL SALINITY
FLOWS ARE FOR LOWER HUDSON
SALINITY IN PARTS PER MILLION
100,000
10,000
1,000
100
0
10
20
30
40
50
60
70
80
90
100
MILES ABOVE BATTERY
GEO. WASH. BRIDGE
INDIAN POINT
28,000 CFS
12,000 CFS
1959 SURVEY
1929 SURVEY
1964 SURVEY
QUIRK, LAWLER & MATUSKY

FIGURE II-12


HUDSON RIVER
EFFECT OF FRESH WATER FLOW ON
EQUILIBRIUM LENGTH OF SALT INTRUSION
L = 352 - 73.5 log Q
S X=L = 100 ppm
NOTE:
FLOW IN LOWER HUDSON = 1.225 X FLOW AT GREEN ISLAND
L = 314 - 65 log Q
S X=L = 250 ppm
△ QL&M SURVEY, 1964
● USC&GS SURVEY, 1929
□ NYC-BWS DATA, 1948-1954
◑ NYSDH SURVEYS, 1949-1959
○ HUDSON RIVER MODEL
▲ NYS CONSERVATION DEPT. 36
FRESH WATER FLOW IN LOWER HUDSON, Q, THOUSAND CFS
50
20
10
5
2
1
50
20
10
5
2
25
35
45
55
65
75
85
LENGTH OF SALT INTRUSION, L, MILES

III. HUDSON RIVER BASIN PROBABLE MAXIMUM FLOOD AT INDIAN POINT

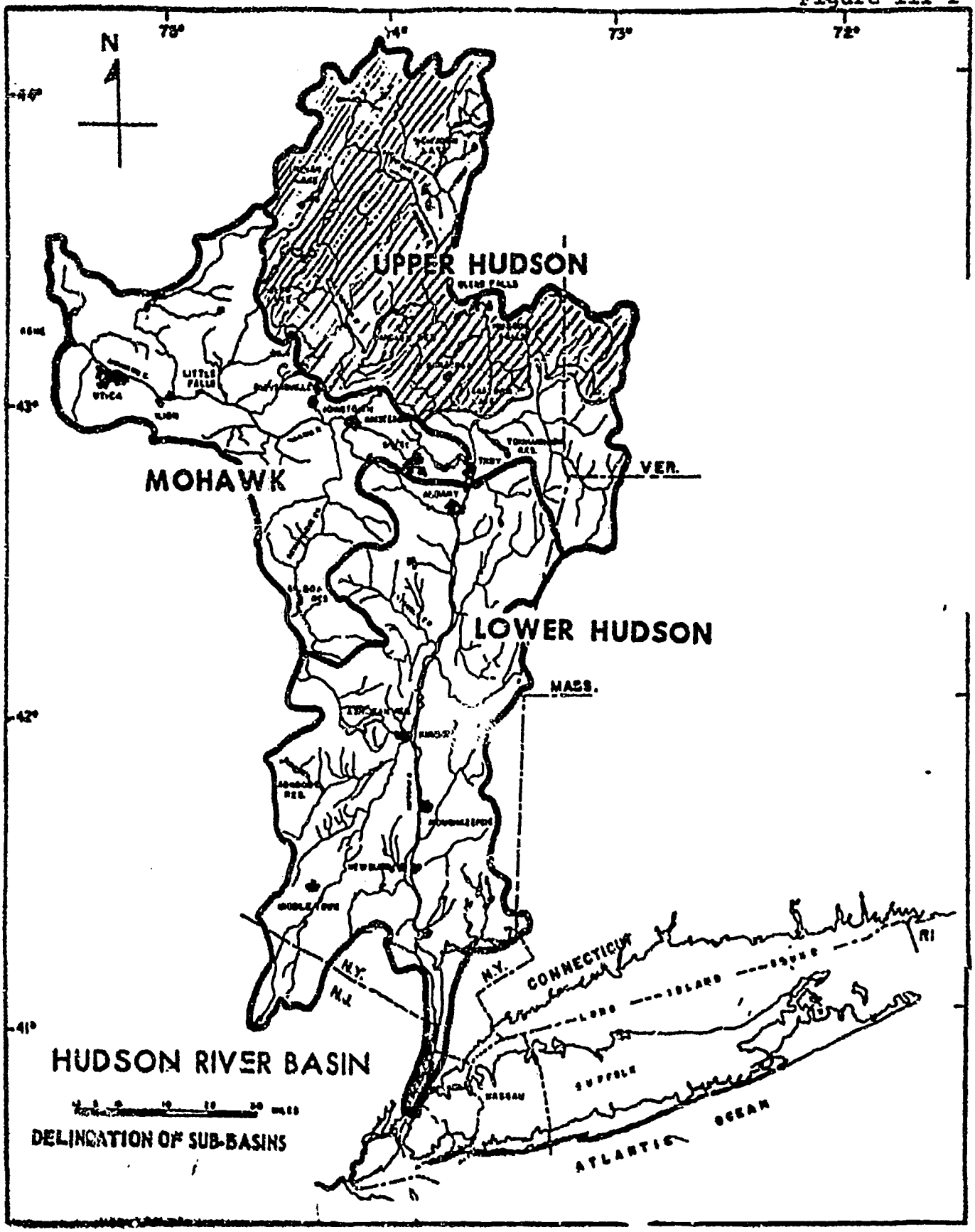
A. Introduction

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 often called "Probable 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.

Figure III-1


N
73°
74°
73°
72°
UPPER HUDSON
MOHAWK
LOWER HUDSON
VER.
MASS.
CONNECTICUT
R.I.
N.Y.
N.J.
N.Y.
LONG ISLAND
SOUND
SUFFOLK
NASSAU
ATLANTIC OCEAN
HUDSON RIVER BASIN
DELINEATION OF SUB-BASINS

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. 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 semi-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 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

TABLE III-1

COMPONENT FLOW & WATER ELEVATION
AT INDIAN POINT
February 1969 Estimates

1.	Maximum Ebb Flow	275,000 cfs
2.	Flow Caused by Dam Failure	785,000 cfs
3.	Flow Caused by Maximum Rain	397,000 cfs
4.	Total Flow at Indian Point	1,457,000 cfs
5.	Elevation at Indian Point Resulting from the Total Flow	11.7 ft (MSL Datum)

flood 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 basin 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

TABLE III-2
A SUMMARY OF THE METHODS USED TO
PREDICT PROBABLE MAXIMUM FLOOD AT INDIAN POINT

#	Method	Figure	PMF a I.P. (CES)
1	PMF vs. other Basins Drainage Area	III-2	1,300,000
2	Stillwater PMF x Ratio of D.A.	III-2	1,010,000
3	PMF vs. Hudson River Basin D.A.	III-2	530,000
4	PMF vs. Max Observed Flood	III-3&4	850,000
5	1:10,000 Yr Instant. Peak-NYSCD 1918-54	II-5	688,000
	1:10,000 Yr Instant. Peak-QL&M, log-Normal 1918-70)		(610,000)*
6	Dimensional Analysis	III-5	880,000
7	Average		880,000

* Not included in Average

TABLE III-3

COMPARISON OF DRAINAGE BASINS

Basin	(A) Drainage Area 10^3 sq. miles	(S) Available Storage cu. miles	(Q) Max. Obs. Peak 10^6 cfs	PMF 10^6 cfs	PMF / Q	Q / A
Susquehanna @ Harrisburg	24.1	.065	.740	1.75	2.36	30.8
Potomac @ Washington D.C.	11.56	.0089	.484	1.34	2.77	42.0
Delaware @ Montague	3.48	.246	.260	.785	3.02	74.5
Hudson River @ Indian Point	12.65	.515	.284	.88	3.10	22.4
Hudson River @ Stillwater	3.76	.29	.097	.30	3.10	25.8

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.

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

PROBABLE MAXIMUM FLOOD- DRAINAGE AREA RELATIONSHIP

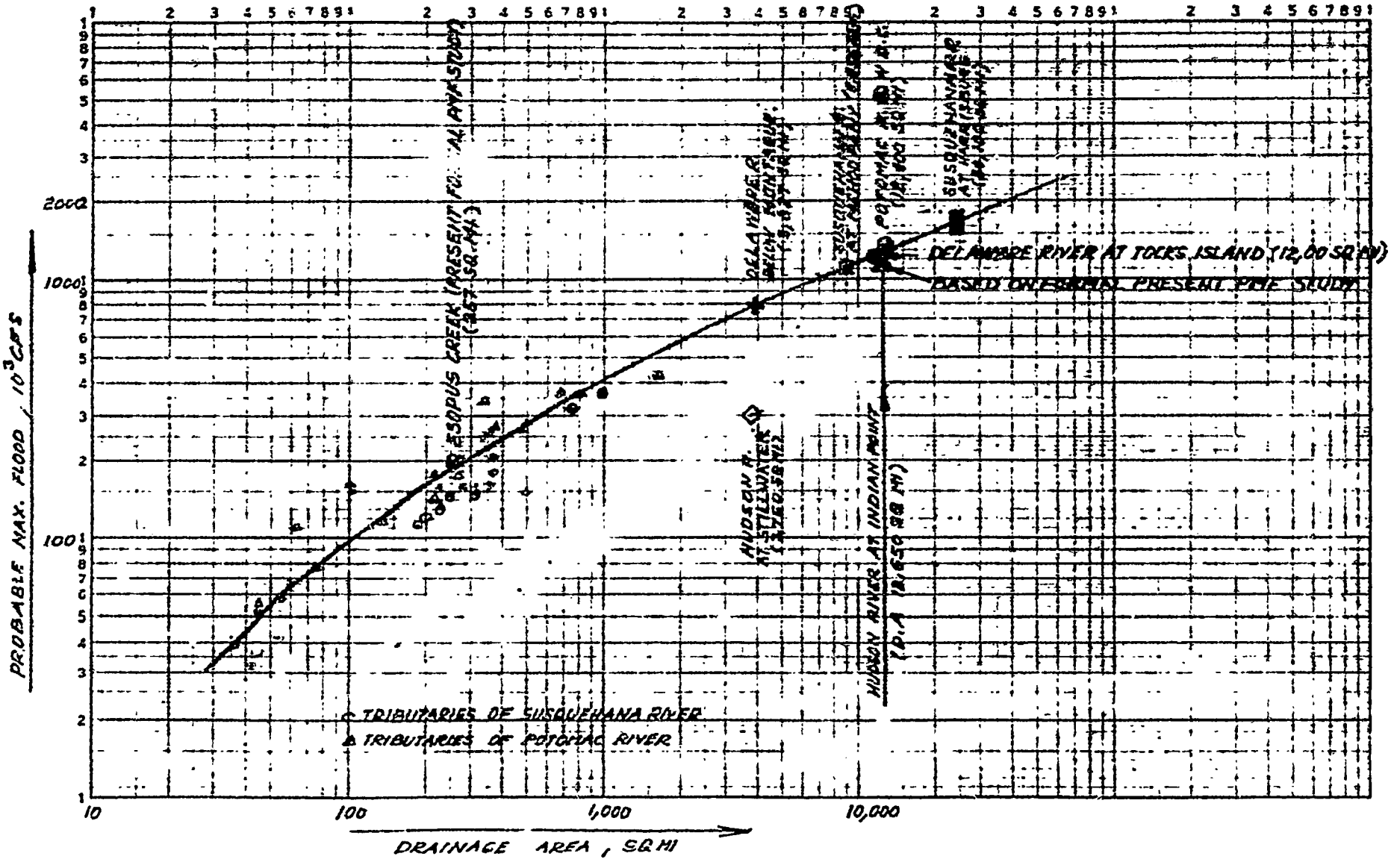


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Figure III-3

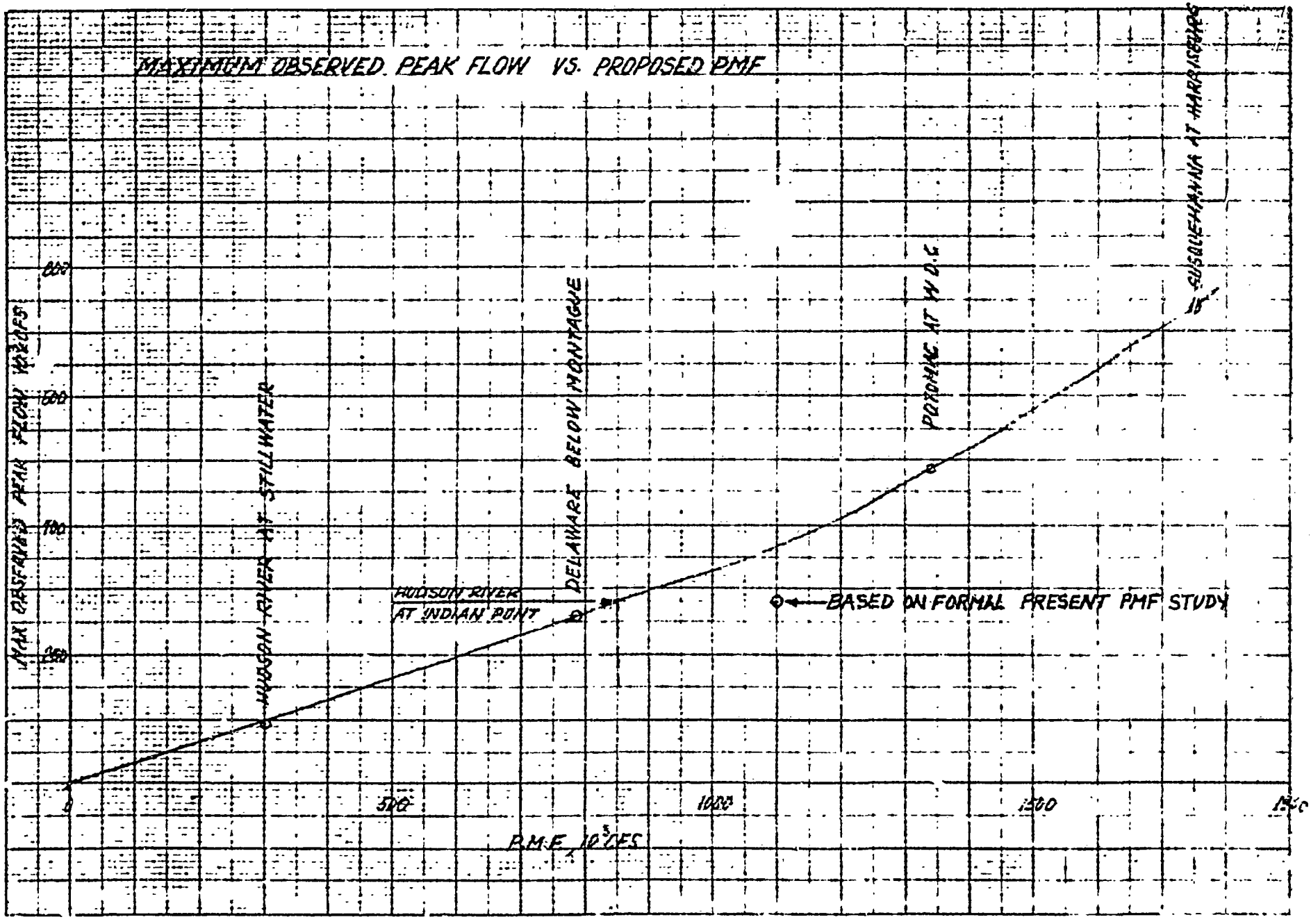

MAXIMUM OBSERVED PEAK FLOW VS. PROPOSED PMF
MAX OBSERVED PEAK FLOW 10^3 CFS
800
600
400
200
0
500
1000
1500
P.M.F. 10^3 CFS
HUDSON RIVER AT STILLWATER
DELAWARE BELOW MONTAGUE
HUDSON RIVER AT INDIAN PONT
POTOMAC AT W.D.C.
SUSQUEHANNA AT HARRISBURG
BASED ON FORMAL PRESENT PMF STUDY

Figure III-4

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.

PROBABLE MAXIMUM FLOOD - DRAINAGE BASIN CHARACTERISTICS RELATIONSHIP

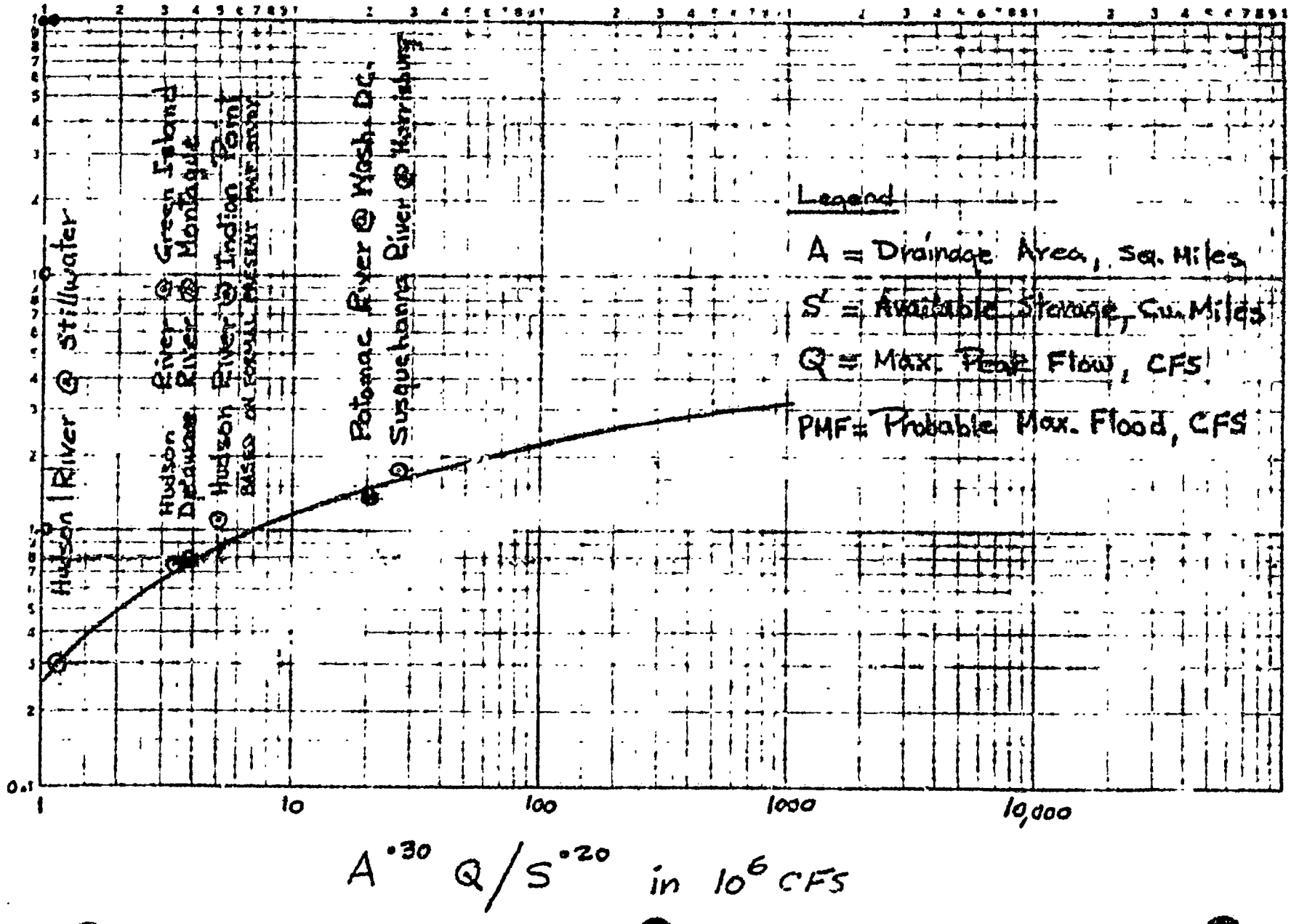


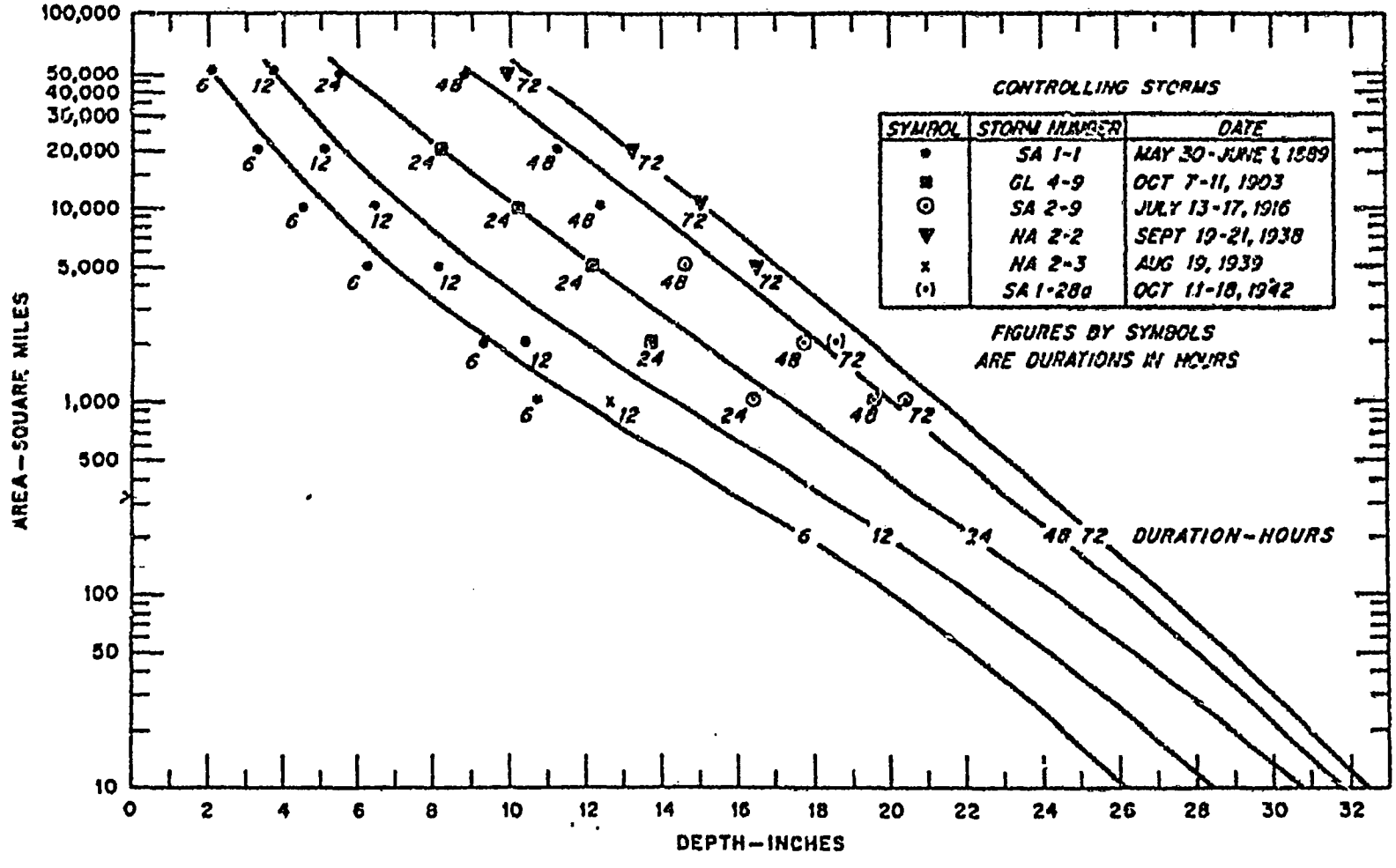
Figure III-5

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.



Enveloping depth-duration-area values of PMP for Susquehanna River Basin

Figure III-6

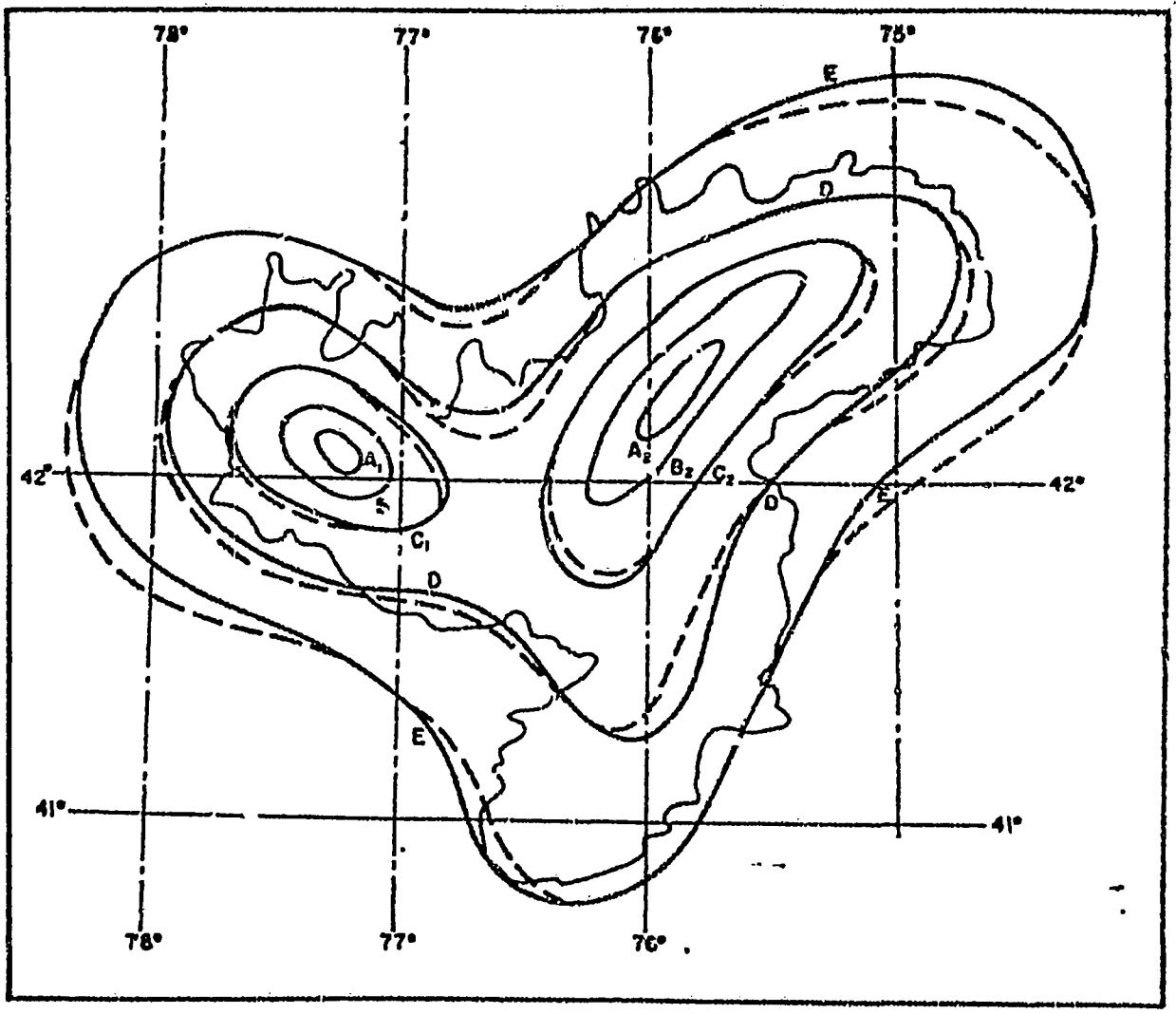
The general level of these curves has been found to be consistent with that of recent estimates for the Potomac & Delaware River Basins.[1]

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

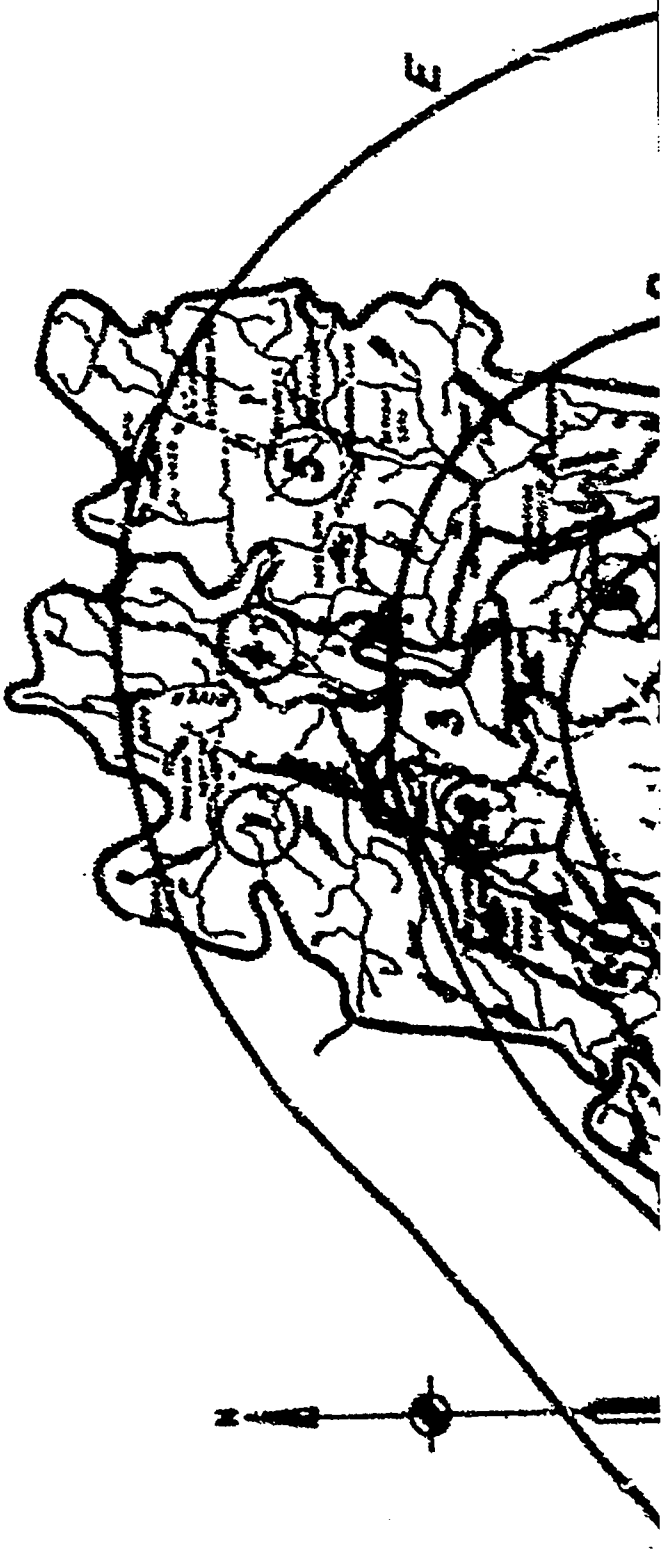
The storm selected for application to the Hudson River Basin is similar in areal distribution to that of the Susquehanna River at Danville. The Danville pattern was developed by the U.S. Weather Bureau[1] and is reproduced in Figure III-7.

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



Pattern storm for Susquehanna River at Danville, Pa.

Figure III-7



A1
9
11
10
15
17
13
16
18.1
18.2
18.3
19
NEW YORK
MASSACHUSETTS
CANADA

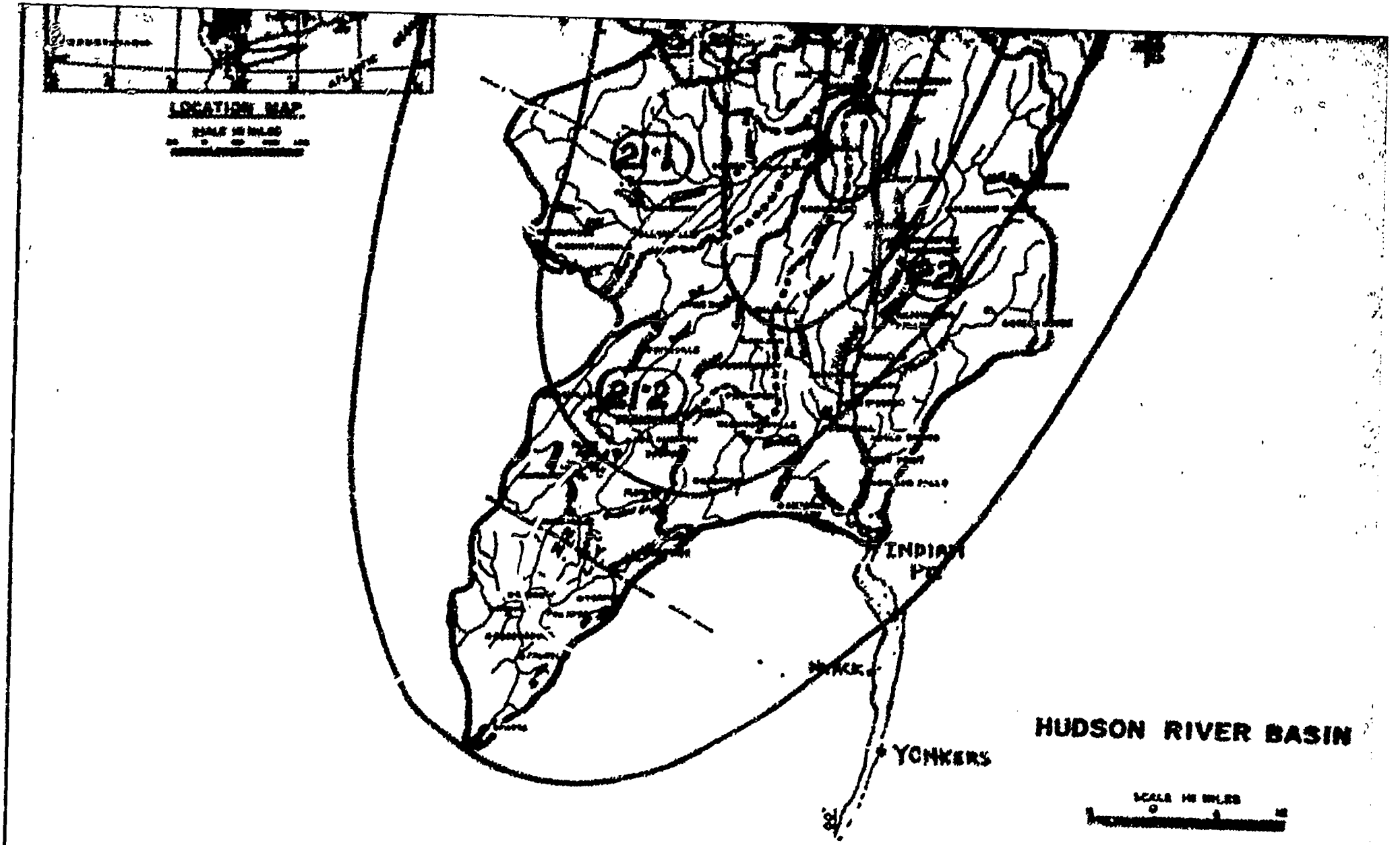


Figure III-8

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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.

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 auxiliary 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 appropriate 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 envelope 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.

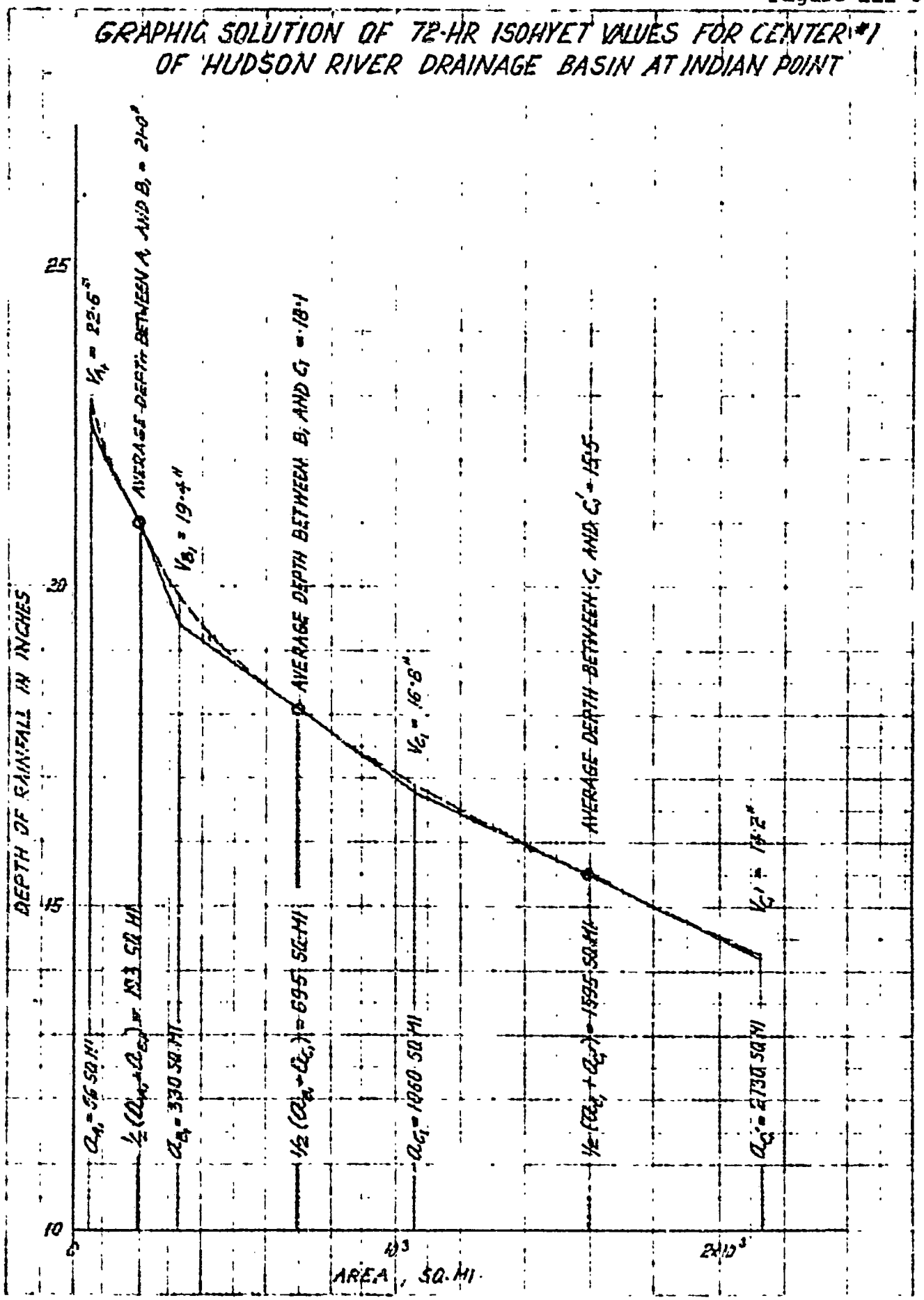
$$\frac{V_n + V_{n+1}}{2} = \frac{P_{n+1} A_{n+1} - P_n A_n}{A_{n+1} - A_n}$$

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.

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


GRAPHIC SOLUTION OF 72-HR ISOHYET VALUES FOR CENTER #1
OF HUDSON RIVER DRAINAGE BASIN AT INDIAN POINT
DEPTH OF RAINFALL IN INCHES
AREA, SQ. MI
AVERAGE DEPTH BETWEEN A, AND B, = 21.0"
AVERAGE DEPTH BETWEEN B, AND C, = 18.1
AVERAGE DEPTH BETWEEN C, AND C' = 15.5
V_A = 22.6"
V_{B_1} = 19.4"
V_{C_1} = 16.8"
$V_{C'}$ = 14.2"
a_A = 56 SQ.MI
$\frac{1}{2}(a_A + a_{B_1})$ = 193 SQ.MI
a_{B_1} = 330 SQ.MI
$\frac{1}{2}(a_{B_1} + a_{C_1})$ = 695 SQ.MI
a_{C_1} = 1060 SQ.MI
$\frac{1}{2}(a_{C_1} + a_{C'})$ = 1595 SQ.MI
$a_{C'}$ = 2130 SQ.MI
25
20
15
10
0
10^3
2×10^3

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 areas 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 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.

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. Time concentration of PMP: 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]

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

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.

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.

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.

TABLE III-5

VALUES OF AVERAGE DEPTH OVER AREAS
BOUNDED BY STATED ISOHYETS

	A_1	A_2	B_1	B_2	C_1	C_2	D	E
	AVERAGE DEPTH OVER AREAS IN INCHES							
72 hours	25.8	23.6	21.8	20.1	19.1	17.3	14.7	13.3
1st 6 hours	12.3	10.8	9.6	8.4	7.8	6.5	4.9	4.3
2nd 6 hours	2.7	2.6	2.5	2.4	2.3	2.2	1.9	1.7
3rd 6 hours	2.3	2.2	2.1	2.0	2.0	1.9	1.7	1.5
4th 6 hours	1.8	1.7	1.6	1.5	1.5	1.4	1.3	1.2
* 2nd day	4.5	4.3	4.1	3.9	3.7	3.5	3.3	3.2
V 3rd day	2.2	2.0	1.9	1.9	1.8	1.8	1.6	1.4
Total area of Isohyet (sq.mi.)	56	140	330	710	1060	2130	11020	17210

* For successive 6-hr. values use 34, 28, 21, and 17% of 2nd day values.
V For successive 6-hr. values use 29, 26, 23, and 22% of 3rd day values.

TABLE III-6

VALUES OF PATTERN STORM ISOHYETS
FOR HUDSON RIVER AT INDIAN POINT
Basin Size 12,650 Sq. Mi.

	Center 1	Center 2	A_1	A_2	B_1	B_2	C_1	C_2	D	E
	Isohyet Values (inches)									
72 hours	26.8	25.4	22.6	20.8	19.4	17.6	16.8	14.2	12.0	9.6
1st 6 hours	12.9	12.1	10.3	8.8	7.9	6.8	6.1	4.4	3.4	3.0
2nd 6 hours	2.75	2.7	2.6	2.5	2.3	2.2	2.2	2.0	1.4	1.2
3rd 6 hours	2.35	2.3	2.1	2.0	2.0	1.9	1.8	1.8	1.3	1.0
4th 6 hours	2.0	1.8	1.6	1.5	1.5	1.4	1.4	1.3	1.2	0.9
* 2nd day	4.5	4.5	4.3	3.9	3.7	3.4	3.4	3.2	3.1	3.0
V 3rd day	2.2	2.1	2.0	1.9	1.9	1.8	1.7	1.7	1.2	1.0
Total area of Isohyet (sq.mi.)	300	300	56	140	330	710	1060	2136	11020	17210

* For successive 6-hr. values use 34, 28, 21, and 17% of 2nd day values.
V For successive 6-hr. values use 29, 26, 23, and 22% of 3rd day values.

TABLE III-7

COMPUTATION OF 6HR INCREMENTS OF PROBABLE MAXIMUM PRECIPITATION ON CATSKILL CREEK BASIN (SUBBASIN 18·3)

ISOHYETS	AREA BETWEEN ISOHYETS (ONLY PLANIMETER READING)	1st 6 HR INCREMENT		2nd 6HR INCREMENT		3rd 6HR INCREMENT		4th 6HR INCREMENT		2nd DAY INCREMENT		3rd DAY INCREMENT	
		AVERAGE ISOHYET VALUE (INCHES) ②×③		AVERAGE ISOHYET VALUE (INCHES) ②×⑤		AVERAGE ISOHYET VALUE (INCHES) ②×⑦		AVERAGE ISOHYET VALUE (INCHES) ②×⑨		AVERAGE ISOHYET VALUE (INCHES) ②×⑪		AVERAGE ISOHYET VALUE (INCHES) ⑤×⑬	
①	②	③	④	⑤	⑥	⑦	⑧	⑨	⑩	⑪	⑫	⑬	⑭
D-C_2	0·45	3·90	1·365	1·70	0·595	1·55	0·543	1·25	0·438	3·15	1·102	1·45	0·505
C_2-B_2	0·95	5·60	5·320	2·10	1·990	1·85	1·759	1·35	1·282	3·30	3·135	1·75	1·663
B_2-A_2	0·75	7·80	5·850	2·35	1·762	1·95	1·462	1·45	1·088	3·65	2·740	1·85	1·388
A_2	0·80	10·90	9·500	2·60	2·080	2·20	1·760	1·70	1·360	4·30	3·420	2·00	1·600
TOTAL	2·85		22·035		6·427		5·524		4·168		10·397		5·159
AVERAGE DEPTH (IN.)			7·75		2·26		1·94		1·52		3·64		1·81

	2nd DAY		3rd DAY		
	%	IN	%	IN	
5th 6HR's	34	1·24	29	0·53	9th 6HR's
6th 6HR's	28	1·02	26	0·47	10th 6HR's
7th 6HR's	21	0·77	23	0·42	11th 6HR's
8th 6HR's	17	0·61	22	0·39	12th 6HR's

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TABLE III-8

UPPER HUDSON RIVER BASIN - PROBABLE MAXIMUM PRECIPITATION AND RUNOFF

DAY OF PMP		1st DAY				2nd DAY				3rd DAY			
6-HR INCREMENT		7th	5th	6th	8th	3rd	1st	2nd	4th	12th	10th	9th	11th
HOURS FROM BEGINNING OF PMP		6	12	18	24	30	36	42	48	54	60	66	72
SUBBASIN	NOTE	6 HR INCREMENTS, INCHES											
1	PMP	0.65	1.00	0.86	0.53	1.12	3.14	1.28	1.02	0.24	0.28	0.31	0.25
	RUNOFF	0	0.35	0.56	0.23	0.82	2.84	0.98	0.72	0	0	0.01	0
2.1	PMP	0.69	1.11	0.92	0.56	1.74	5.04	2.02	1.33	0.36	0.43	0.47	0.38
	RUNOFF	0	0.50	0.62	0.26	1.44	4.74	1.72	1.03	0.06	0.13	0.17	0.08
2.2	PMP	0.65	1.06	0.87	0.53	1.57	4.15	1.75	1.26	0.32	0.38	0.43	0.34
	RUNOFF	0	0.41	0.57	0.23	1.27	3.25	1.45	0.96	0.02	0.08	0.13	0.04
3	PMP	0.66	1.08	0.89	0.54	1.55	4.10	1.73	1.25	0.32	0.38	0.42	0.33
	RUNOFF	0	0.44	0.59	0.24	1.25	3.80	1.43	0.95	0.02	0.08	0.12	0.03
4	PMP	0.64	1.04	0.86	0.52	1.16	3.22	1.31	1.06	0.24	0.29	0.32	0.26
	RUNOFF	0	0.38	0.56	0.22	0.26	2.92	1.01	0.76	0	0	0.02	0
5	PMP	0.65	1.04	0.86	0.53	1.25	3.42	1.41	1.09	0.26	0.31	0.35	0.27
	RUNOFF	0	0.39	0.56	0.23	0.85	3.12	1.11	0.79	0	0.01	0.05	0
6	PMP	0.71	1.15	0.95	0.57	1.61	5.77	2.08	1.38	0.38	0.45	0.49	0.39
	RUNOFF	0	0.56	0.65	0.27	1.51	5.47	1.78	1.08	0.08	0.15	0.19	0.09
7	PMP	0.72	1.16	0.96	0.58	1.85	6.07	2.22	1.40	0.38	0.45	0.50	0.41
	RUNOFF	0	0.58	0.66	0.28	1.55	5.77	1.92	1.10	0.08	0.15	0.20	0.11
8	PMP	0.77	1.25	1.03	0.61	1.94	7.40	2.29	1.47	0.41	0.48	0.53	0.42
	RUNOFF	0	0.72	0.73	0.31	1.64	7.10	1.99	1.17	0.11	0.18	0.23	0.12

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TABLE III-8 (Continued)

UPPER HUDSON RIVER BASIN - PROBABLE MAXIMUM PRECIPITATATION AND RUNOFF

DAY OF PMP		1st DAY				2nd DAY				3rd DAY			
6 HR INCREMENT		7th	5th	6th	8th	3rd	1st	2nd	4th	12th	10th	9th	11th
HR'S FROM BEGINNING OF PMP		6	12	18	24	30	36	42	48	54	60	66	72
SUBBASIN	NOTE	6 HR INCREMENTS, INCHES											
9	PMP	0.68	[illegible]	[illegible]	[illegible]	[illegible]	3.35	1.63	[illegible]	0.30	[illegible]	[illegible]	[illegible]
	RUNOFF	0	0.46	[illegible]	[illegible]	[illegible]	3.65	[illegible]	[illegible]	0	[illegible]	[illegible]	[illegible]
10	PMP	0.67	1.07	0.88	0.53	1.55	3.90	1.70	1.25	0.32	0.38	0.42	0.33
	RUNOFF	0	0.44	0.58	0.23	1.35	3.60	1.40	0.95	0.02	0.08	0.12	0.03
11	PMP	0.65	0.95	0.89	0.53	[illegible]	[illegible]	[illegible]	[illegible]	0.23	0.27	0.31	0.24
	RUNOFF	0	[illegible]	0.53	0.23	[illegible]	2.79	[illegible]	0.67	0	0	0.01	0
12	PMP	0.67	1.07	0.88	0.53	1.55	3.90	1.70	1.25	0.32	0.38	0.42	0.33
	RUNOFF	0	0.44	0.58	0.23	1.25	3.60	1.40	0.95	0.02	0.08	0.12	0.03
13	PMP	0.66	1.02	0.88	0.52	1.19	3.28	1.35	[illegible]	0.25	0.30	0.33	0.26
	RUNOFF	0	0.38	0.58	0.22	[illegible]	2.98	1.05	0.75	0	0	0.03	0
14	PMP	0.67	1.07	0.88	0.53	1.55	3.90	1.70	1.25	0.32	0.38	0.42	0.33
	RUNOFF	0	0.44	0.58	0.23	1.25	3.60	1.40	0.95	0.02	0.08	0.12	0.03

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TABLE III-9

MOHAWK RIVER BASIN - PROBABLE MAXIMUM PRECIPITATION AND RUNOFF

DAY OF PMP		1st DAY				2nd DAY				3rd DAY			
6 HR INCREMENT		7th	5th	6th	8th	3rd	1st	2nd	4th	12th	10th	9th	11th
HR'S FROM BEGINNING OF PMP		6	12	18	24	30	36	42	48	54	60	66	72
SUBBASIN	NOTE	6 HR INCREMENTS, INCHES											
15	PMP	0.57	1.03	0.90	0.54	1.49	4.15	1.62	1.21	0.30	0.36	0.40	0.31
	RUNOFF	0	0.46	0.60	0.24	1.19	3.85	1.29	0.91	0	0.06	0.10	0.01
16	PMP	0.57	1.11	0.91	0.57	1.52	4.22	1.77	1.28	0.33	0.39	0.44	0.34
	RUNOFF	0	0.43	0.61	0.25	1.29	3.92	1.47	0.98	0.03	0.09	0.14	0.04
17	PMP	0.67	1.03	0.90	0.54	1.75	4.19	1.94	1.30	0.36	0.43	0.48	0.38
	RUNOFF	0	0.46	0.60	0.24	1.45	3.89	1.64	1.00	0.06	0.13	0.18	0.08

TABLE III-10

LOWER HUDSON RIVER BASIN - PROBABLE MAXIMUM PRECIPITATION AND RUNOFF

DAY OF PMP		1st DAY				2nd DAY				3rd DAY			
6 HR INCREMENT		7th	5th	6th	8th	3rd	1st	2nd	4th	12th	10th	9th	11th
HR'S FROM BEGINNING OF PMP		6	12	18	24	30	36	42	48	54	60	66	72
SUBBASIN	NOTE	6 HR'S INCREMENTS, INCHES											
18·1	PMP	0·72	1·16	0·96	0·58	1·83	6·21	2·13	1·37	0·37	0·46	0·50	0·40
	RUNOFF	0	0·58	0·66	0·38	1·53	5·91	1·83	1·07	0·07	0·16	0·20	0·10
18·2	PMP	0·67	1·08	0·93	0·54	1·60	5·28	1·77	1·27	0·33	0·39	0·44	0·34
	RUNOFF	0	0·45	0·59	0·24	1·30	5·98	1·47	0·97	0·03	0·09	0·14	0·04
18·3	PMP	0·77	1·24	1·02	0·61	1·94	7·75	2·26	1·52	0·39	0·47	0·53	0·42
	RUNOFF	0	0·71	0·72	0·31	1·67	7·45	1·96	1·22	0·02	0·17	0·23	0·12
18·4	PMP	0·67	1·08	0·88	0·54	1·63	5·39	1·91	1·28	0·33	0·40	0·45	0·35
	RUNOFF	0	0·45	0·58	0·24	1·33	5·09	1·51	0·98	0·03	0·10	0·15	0·05
18·5	PMP	0·85	1·39	1·14	0·68	2·11	8·80	2·65	1·61	0·43	0·51	0·56	0·45
	RUNOFF	0	0·94	0·84	0·39	1·81	8·55	2·35	1·31	0·15	0·21	0·26	0·23
19	PMP	0·67	1·08	0·88	0·54	1·59	4·03	1·75	1·26	0·33	0·39	0·43	0·34
	RUNOFF	0	0·45	0·58	0·24	1·29	3·73	1·45	0·96	0·03	0·09	0·13	0·04
20	PMP	0·72	1·18	0·97	0·59	1·99	5·73	2·22	1·40	0·40	0·47	0·52	0·41
	RUNOFF	0	0·60	0·67	0·29	1·60	6·43	1·92	1·30	0·10	0·17	0·22	0·11
21·1	PMP	0·73	1·18	0·98	0·59	1·72	4·08	1·89	1·37	0·35	0·42	0·47	0·37
	RUNOFF	0	0·61	0·68	0·29	1·42	3·78	1·59	1·07	0·05	0·12	0·17	0·07
21·2	PMP	0·67	1·03	0·89	0·53	1·61	4·00	1·81	1·33	0·33	0·40	0·45	0·34
	RUNOFF	0	0·46	0·59	0·23	1·31	3·70	1·51	1·03	0·03	0·10	0·15	0·04
22	PMP	0·68	1·11	0·91	0·55	1·57	4·49	1·76	1·26	0·32	0·39	0·43	0·35
	RUNOFF	0	0·46	0·60	0·24	1·35	4·19	1·43	0·93	0·02	0·08	0·12	0·04

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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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HUDSON RIVER AT INDIAN POINT — PROBABLE MAXIMUM PRECIPITATION OVER ENTIRE BASIN AND RELATED RUNOFF

6 HR INCREMENT	1	2	3	4	5	6	7	8	9	10	11	12	TOTAL
HR'S FROM BEGINNING OF PMP	6	12	18	24	30	36	42	48	54	60	66	72	72
DEPTH OF RAINFALL (INCHES)	0.69	1.70	0.32	0.55	1.58	4.64	1.79	1.27	0.32	0.39	0.43	0.34	14.02
RUNOFF (INCHES)	0.20	0.49	0.62	0.25	1.28	4.34	1.49	0.97	0.02	0.09	0.13	0.03	9.72

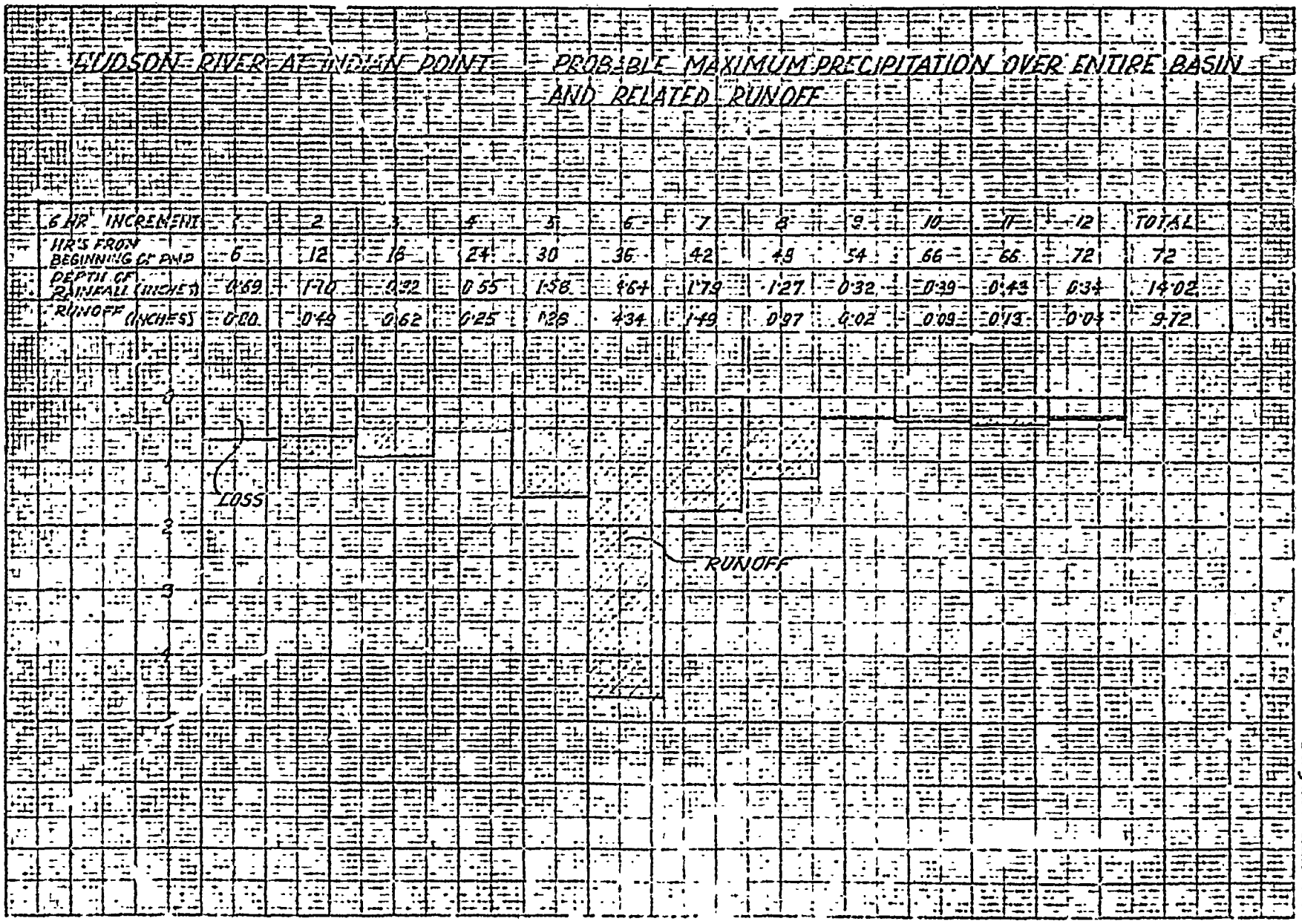


Figure III-10

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 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.

TABLE III-11

COMPUTATION OF UNIT-HYDROGRAPH LAG AND PEAK FLOW USING BASIN CHARACTERISTICS (TAYLOR AND SCHWARZ'S METHOD)

B.S.C. EQUATIONS:

$$t_{pR} = C' e^{m' t_R}$$

$$m' = 0.212 / (L L_{ca})^{0.36}$$

$$C' = 0.6 / \sqrt{S_{st}}$$

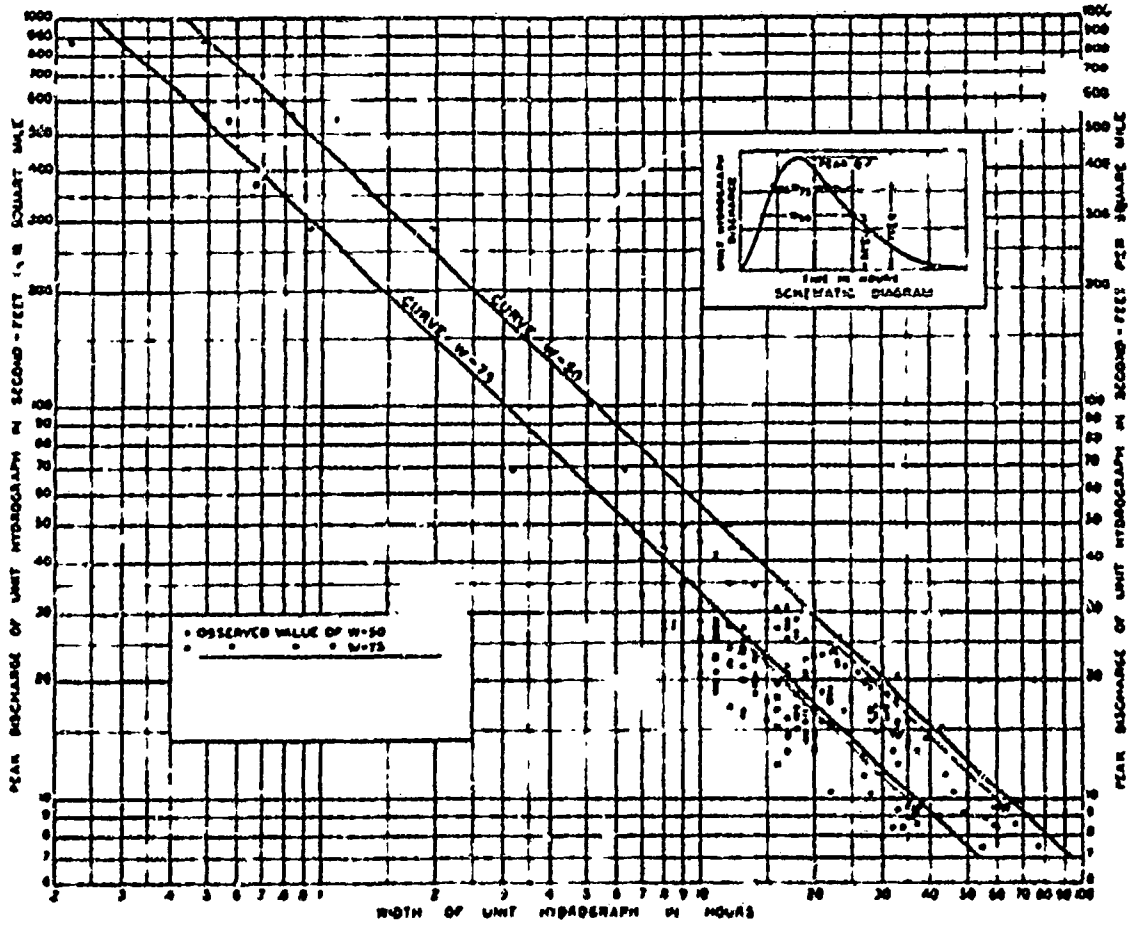
$$q_{pR} = C'' e^{m'' t_R}$$

$$m'' = 0.121\, S_{st}^{0.142} - 0.050$$

$$C'' = 382 (L L_{ca})^{-0.36}$$

Where:

- t_R - Time in hours from beginning to end of computed rainfall
- q_{pR} - Peak flow of unit hydrograph in cfs per sq mi
- t_{pR} - Lag in hours from the time the center of mass (centroid) of rainfall excess to the time of unit hydrograph peak
- L - Total length of longest watercourse, measured to basin boundary (in mi)
- L_{ca} - Distance along the main drainage channel from gage to computed center of gravity (centroid) of drainage area (in mi)
- S_{st} - Slope of uniform channel having the same length as the longest watercourse and an equal time of travel
- e - Base of natural logarithms (= 2.7183)



UNIT HYDROGRAPH PEAKS *versus* WIDTHS

Figure III-11

6HR UNIT HYDROGRAPHS CHARACTERISTICS

SUBBASIN	AREA SQ MI	L MI	L_{ca} MI	S FT/FT	t_{pR} HR	q_{pR} CFS/SQMI	NOTE
1	419	35	5	0·0040	17	10·4	
2·1	·c7	29	11·2	—	15	26·3	
2·2	73	18	8·7	0·0062	15	46·5	
3	79	—	—	—	3	95·0	
4	90	24·4	13·1	0·0083	9	38·6	
5	365	70·C	31·0	0·00059	59	2·7	
6	177	—	—	—	3	90·4	
7	130	—	—	—	3	68·6	
8	1044	67·1	34·1	—	9	27·4	
9	296	300	13·5	0·0026	13·6	38·9	
10	255	44·1	17·0	0·0016	15	12·1	
11	440	59·0	32·2	0·0020	27	13·3	
12	79	—	—	—	1	114·0	
13	713	71·5	23·0	0·00265	12·6	22·5	
14	138	—	—	—	2	91·0	
15	2170	12··	42·5	0·000428	30·7	15·7	
16	926	88·0	41·0	0·0024	13·1	18·9	
17	366	43·0	24·	0·0014	32·5	25·3	
18·	832	53·0	37·0	0·00015	46·6	20·6	
18·2	512	53·0	19·0	0·0056	8·9	30·4	
18·3	417	41·5	25·0	0·0051	6·8	29·6	
18·4	208	46·	30·0	0·0047	9·6	26·4	
19	257	27·2	12·5	0·00568	9·3	42·0	
20	165	31·0	13·0	0·0082	15·5	33·0	
21	400	47·0	22·0	0·0021	12·0	28·8	
21·2	797	101·0	54·0	0·0014	18·9	15·9	
22	912	64·0	30·0	0·00216	4··	7··	

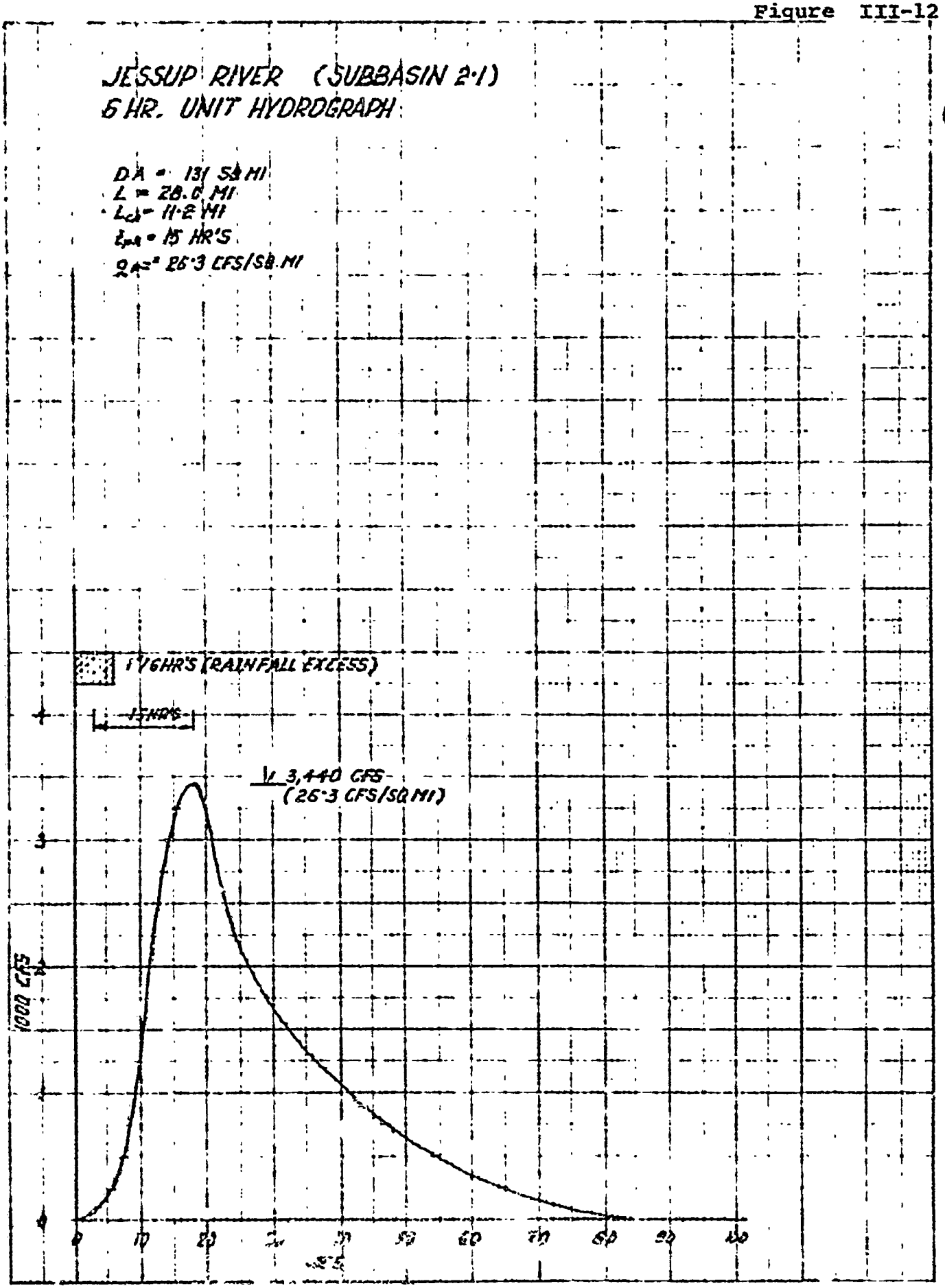

JESSUP RIVER (SUBBASIN 2-1)
6 HR. UNIT HYDROGRAPH
DA = 131 SQ MI
L = 28.6 MI
Lca = 11.2 MI
tp = 15 HR'S
QA = 26.3 CFS/SQ MI
1"/6HR'S (RAINFALL EXCESS)
15 HRS
3,440 CFS
(26.3 CFS/SQ MI)
1000 CFS
0
10
20
30
40
50
60
70
80
90
100

Figure III-I3

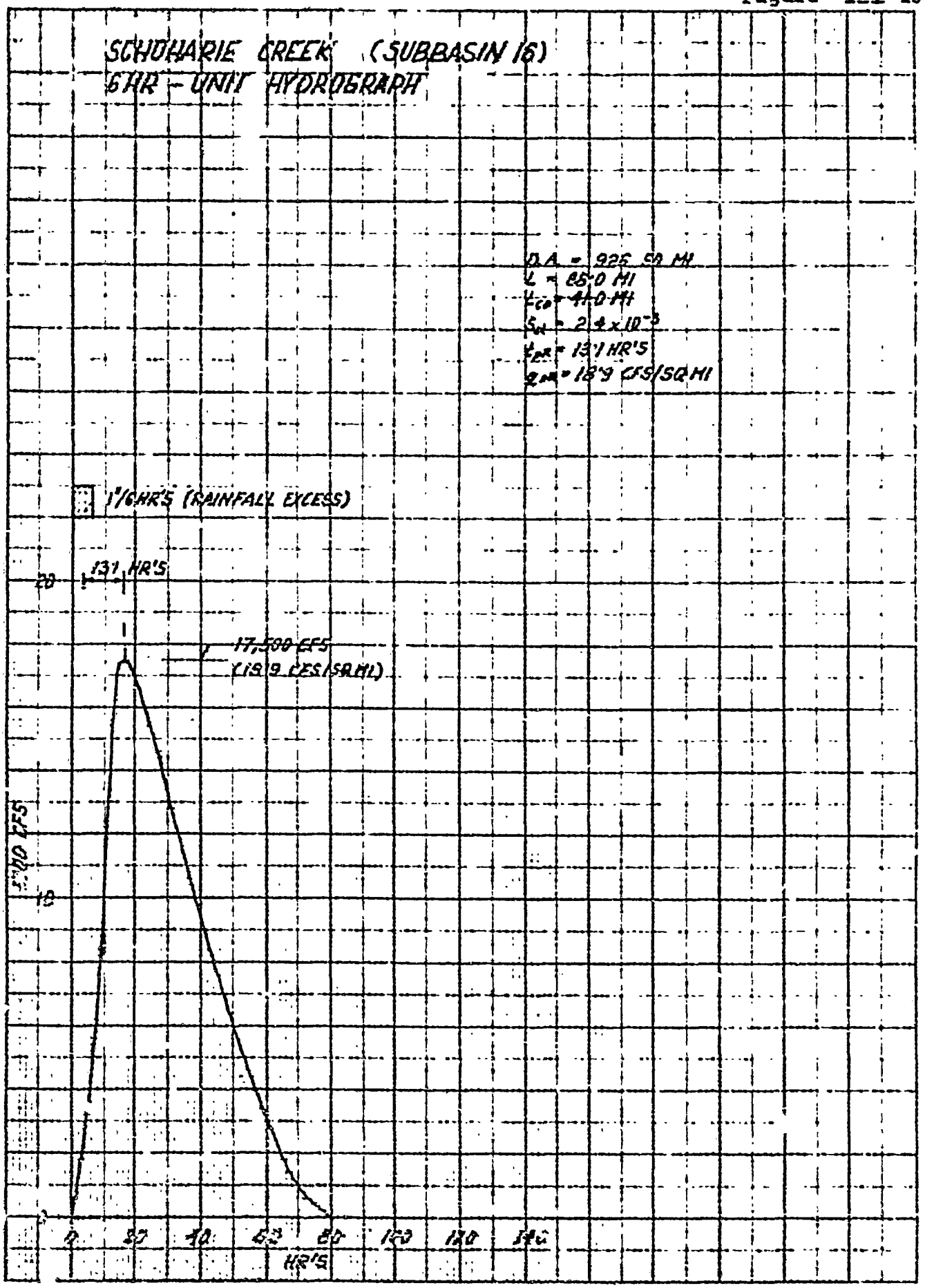
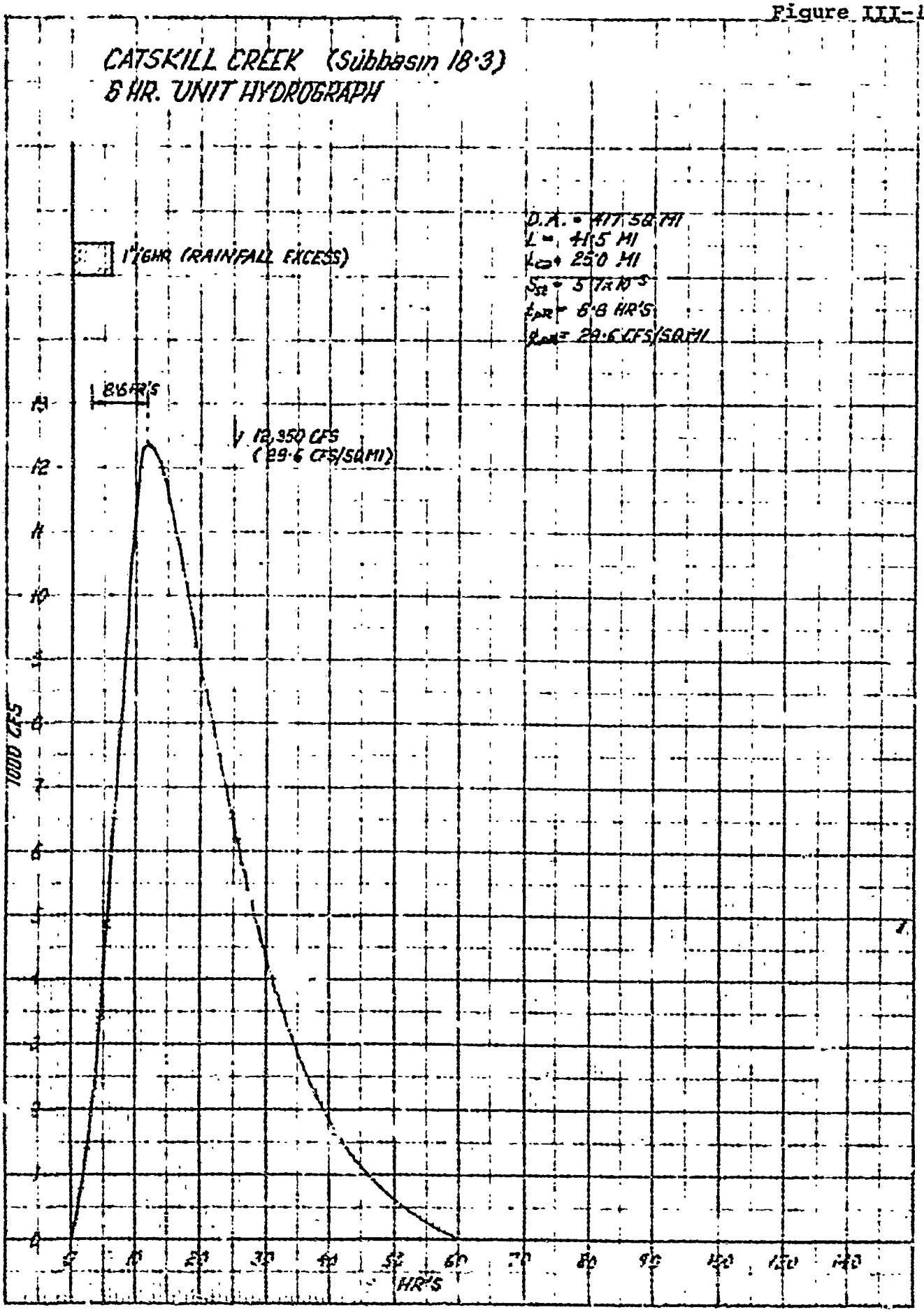


Figure III-14



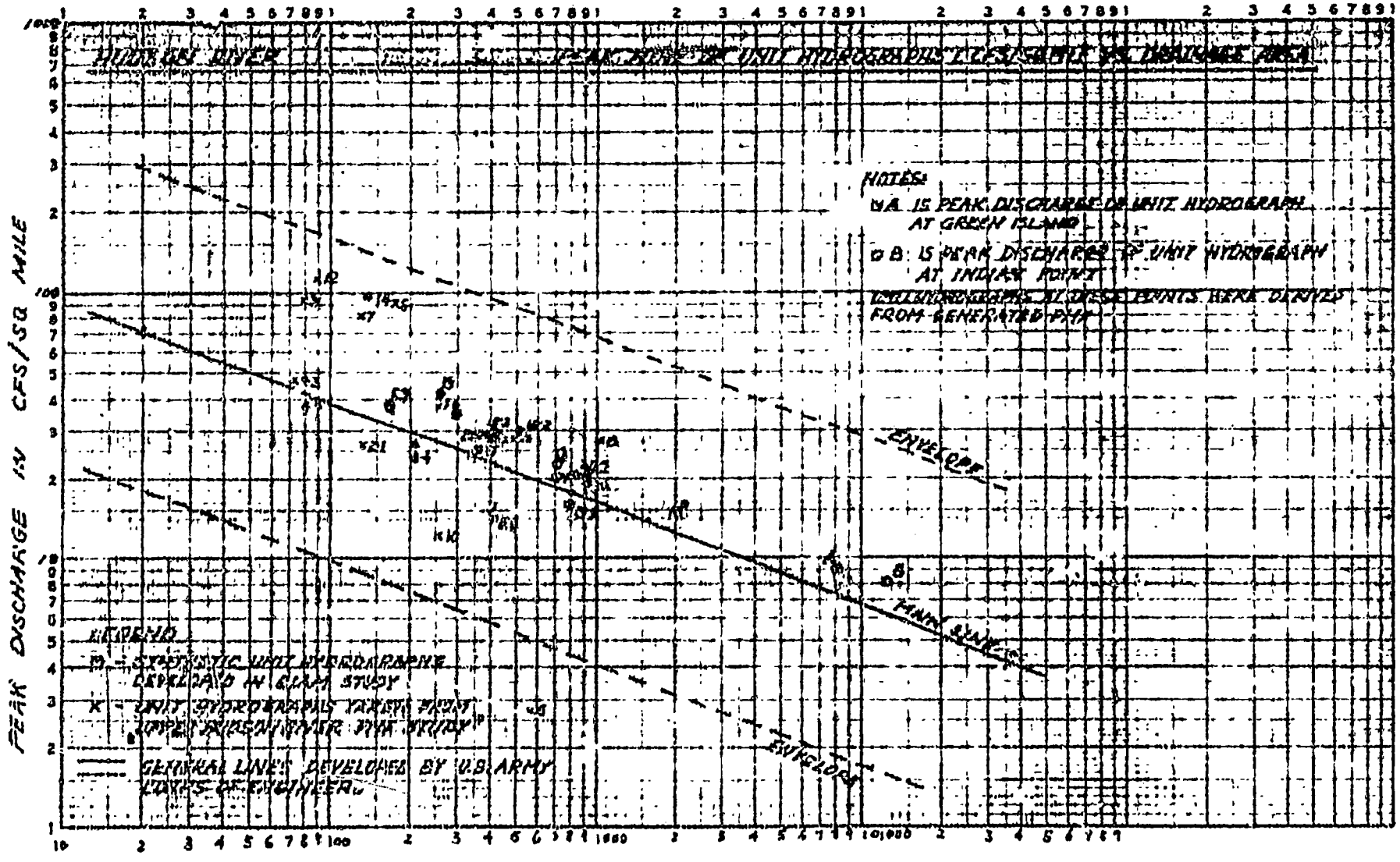


Figure III-15

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 Figures 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.

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.

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 (6×10^{-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 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.

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:

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 com-

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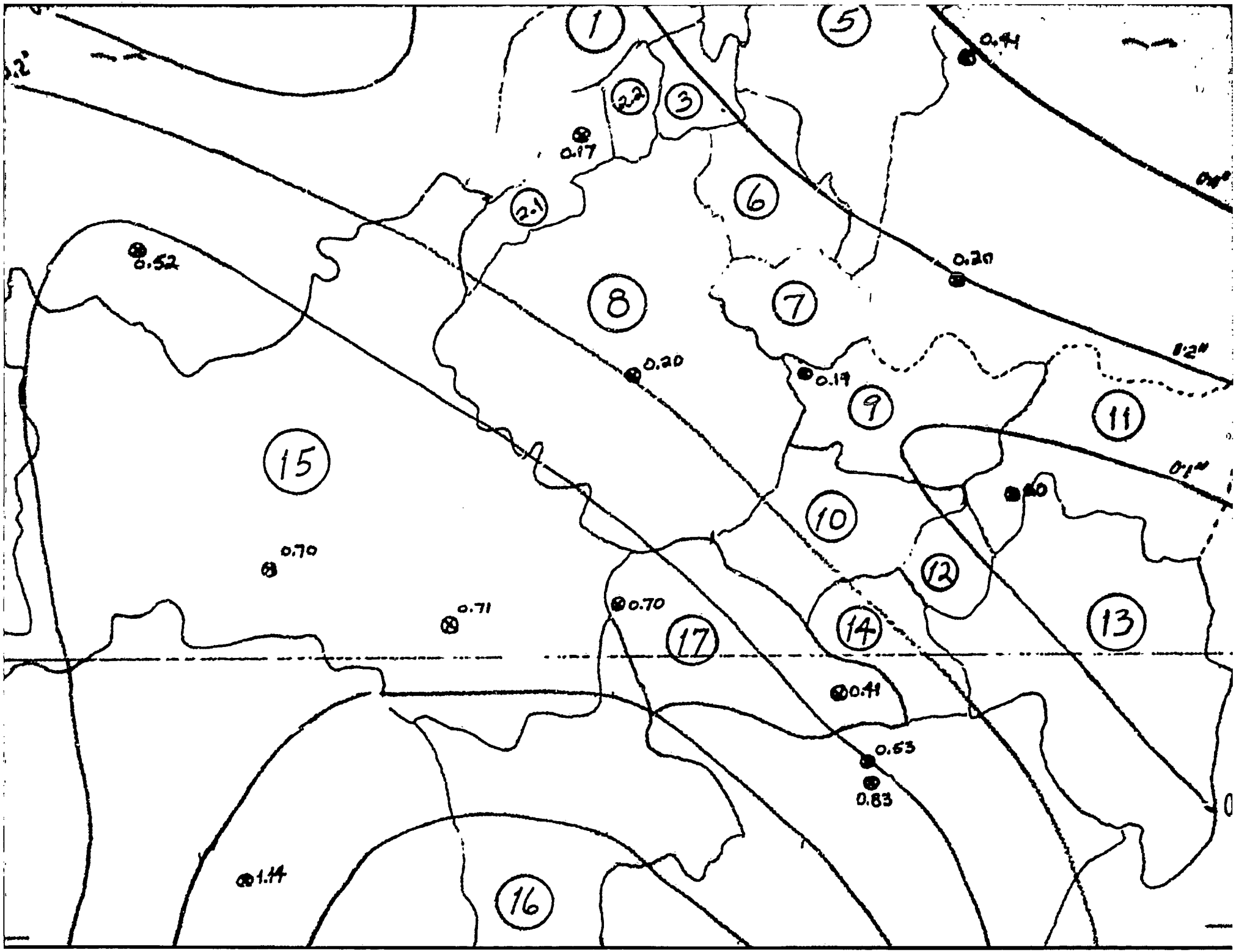
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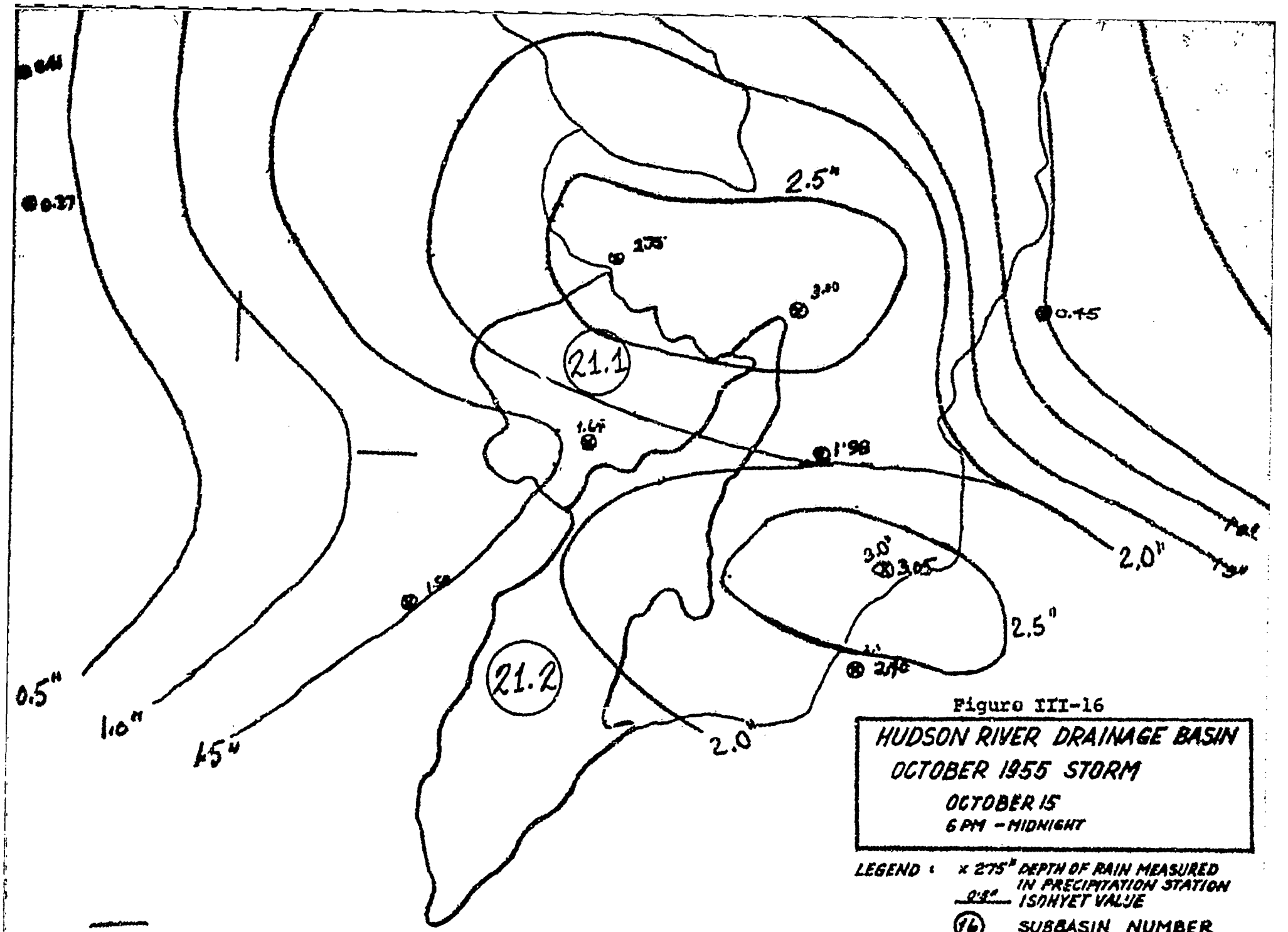

Figure III-16

HUDSON RIVER DRAINAGE BASIN
OCTOBER 1955 STORM
OCTOBER 15
6 PM - MIDNIGHT

LEGEND : x 2.75" DEPTH OF RAIN MEASURED IN PRECIPITATION STATION
0.5" ISOHYET VALUE
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TABLE III-13

SHEET 1 OF 2

UPPER HUDSON RIVER BASIN – OCTOBER 1955 STORM RAINFALL AND RUNOFF DATA

DAY OF STORM		OCTOBER 14				OCTOBER 15				OCTOBER 16			
# 6 HR INCREMENT		1ST	2ND	3RD	4TH	1ST	2ND	3RD	4TH	1ST	2ND	3RD	4TH
HOURS FROM BEGINNING OF STORM		6	12	18	24	30	36	42	48	54	60	66	72
SUB BASIN	NOTE	6 HR INCREMENTS, INCHES											
1	RAINFALL	0.0	0.0	0.0	0.08	0.058	0.05	0.246	0.22	0.26	0.218	0.06	0.01
	RUNOFF	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2.1	RAINFALL	0.0	0.0	0.0	0.09	0.038	0.10	0.188	0.16	0.40	0.441	0.42	0.23
	RUNOFF	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.097	0.12	0.0
2.2	RAINFALL	0.0	0.0	0.03	0.09	0.07	0.05	0.25	0.18	0.40	0.30	0.10	0.19
	RUNOFF	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.070	0.0	0.0
3	RAINFALL	0.0	0.0	0.03	0.08	0.087	0.09	0.30	0.20	0.40	0.214	0.05	0.16
	RUNOFF	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.031	0.0	0.0
4	RAINFALL	0.0	0.0	0.0	0.08	0.10	0.01	0.30	0.30	0.19	0.092	0.0	0.033
	RUNOFF	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
5	RAINFALL	0.0	0.0	0.30	0.08	0.163	0.10	0.478	0.316	0.28	0.092	0.08	0.07
	RUNOFF	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.187	0.0	0.0	0.0	0.0
6	RAINFALL	0.0	0.0	0.03	0.10	0.157	0.252	0.58	0.20	0.440	0.348	0.30	0.28
	RUNOFF	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.019	0.14	0.048	0.0	0.0
7	RAINFALL	0.0	0.0	0.10	0.20	0.238	0.75	0.70	0.18	0.652	0.547	0.52	0.44
	RUNOFF	0.0	0.0	0.0	0.0	0.0	0.0	0.688	0.0	0.352	0.247	0.22	0.14
8	RAINFALL	0.0	0.0	0.054	0.11	0.171	0.34	0.294	0.243	0.416	0.447	0.71	0.33
	RUNOFF	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.328	0.147	0.41	0.03

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TABLE III-13 SHEET 2 OF 2

DAY OF STORM		OCTOBER 14				OCTOBER 15				OCTOBER 16			
#6 HR INCREMENT		1st	2nd	3rd	4th	1st	2nd	3rd	4th	1st	2nd	3rd	4th
HOURS OF BEGINNING OF STORM		6	12	18	24	30	36	42	48	54	60	66	72
SUBBASIN	NOTE	6 HR INCREMENTS, INCHES											
9	RAINFALL	0.0	0.0	0.10	0.00	0.21	0.435	0.403	0.138	0.573	0.473	0.664	0.46
	RUNOFF	0.0	0.0	0.0	0.0	0.0	0.0	0.048	0.0	0.273	0.173	0.364	0.16
10	RAINFALL	0.0	0.0	0.10	0.30	0.53	0.89	0.324	0.147	0.846	0.396	0.767	0.47
	RUNOFF	0.0	0.0	0.0	0.0	0.0	0.46	0.024	0.0	0.546	0.096	0.467	0.17
11	RAINFALL	0.0	0.0	0.10	0.0	0.10	0.05	0.02	0.106	0.201	0.209	0.282	0.18
	RUNOFF	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
12	RAINFALL	0.0	0.0	0.10	0.10	0.24	0.20	0.02	0.118	0.466	0.30	0.474	0.42
	RUNOFF	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.224	0.174	0.12
13	RAINFALL	0.0	0.0	0.10	0.0	0.013	0.086	0.01	0.094	0.17	0.254	0.322	0.17
	RUNOFF	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.036
14	RAINFALL	0.0	0.0	0.247	0.20	0.624	0.463	0.02	0.23	0.674	0.370	0.460	0.40
	RUNOFF	0.0	0.0	0.0	0.0	0.0	0.224	0.0	0.0	0.374	0.170	0.160	0.10

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TABLE III-14

MOHAWK RIVER - OCTOBER 1955 STORM RAINFALL AND RUNOFF DATA

DAY OF STORM		OCTOBER 14				OCTOBER 15				OCTOBER 16			
*6 HR INCREMENT		1st	2nd	3rd	4th	1st	2nd	3rd	4th	1st	2nd	3rd	4th
HOURS FROM BEGINNING OF STORM		6	12	18	24	30	36	42	48	54	60	66	72
SUBBASIN	NOTE	6 HR INCREMENTS, INCHES											
15	RAINFALL	0.0	0.07	0.03	0.28	0.12	0.16	0.18	0.77	0.43	0.24	0.48	0.31
	RUNOFF	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.30	0.13	0.0	0.18	0.01
16	RAINFALL	0.0	0.46	1.07	1.66	0.40	1.60	1.07	1.65	0.91	0.24	0.56	0.36
	RUNOFF	0.0	0.0	0.29	1.36	0.10	1.30	0.77	1.35	0.61	0.0	0.24	0.06
17	RAINFALL	0.0	0.25	0.13	0.35	0.45	1.12	0.03	0.51	0.89	0.23	0.67	0.48
	RUNOFF	0.0	0.0	0.0	0.0	0.0	0.82	0.0	0.21	0.59	0.0	0.37	0.18

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TABLE III-15

WALLKILL RIVER AND RONDOUT CREEK BASINS - OCTOBER 1955 STORM

DAY OF STORM		OCTOBER 14				OCTOBER 15				OCTOBER 16			
#6 HR INCREMENT		1st	2nd	3rd	4th	1st	2nd	3rd	4th	1st	2nd	3rd	4th
HOURS OF BEGINNING OF STORM		6	12	18	24	30	36	42	48	54	60	66	72
SUBBASIN	NOTE	6 HR INCREMENTS, INCHES											
21.1	RAINFALL	0.0	0.343	1.50	2.441	0.308	1.70	2.26	2.01	1.01	0.32	0.219	0.256
	RUNOFF	0.0	0.0	0.643	2.141	0.008	1.40	1.96	1.71	0.71	0.02	0.019	0.0
21.2	RAINFALL	0.0	0.050	0.66	1.116	0.0927	0.691	1.93	1.55	0.318	0.091	0.176	0.115
	RUNOFF	0.0	0.0	0.0	0.116	0.0	0.391	1.63	1.25	0.018	0.0	0.0	0.0

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Figure III-17

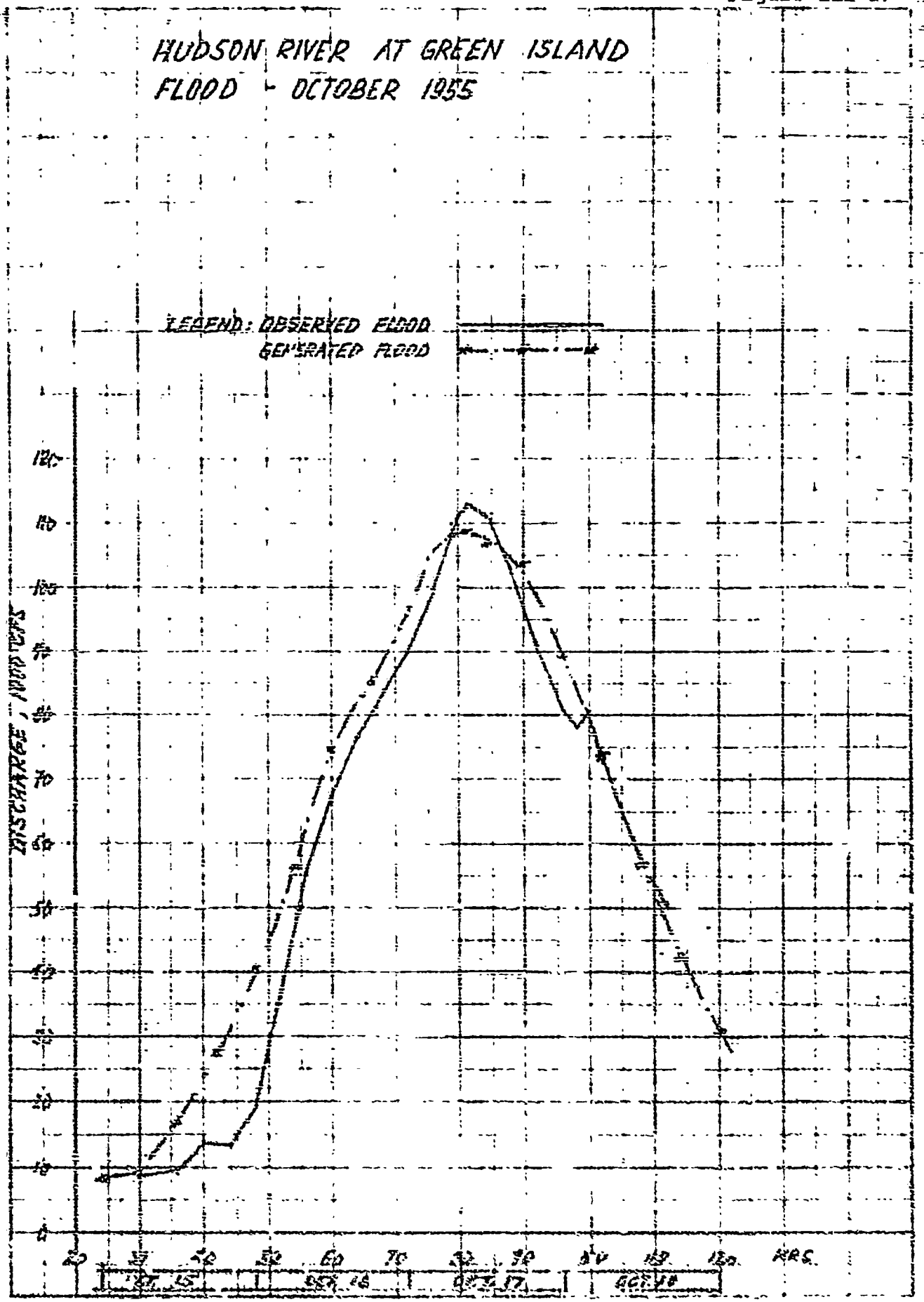


Figure III-18

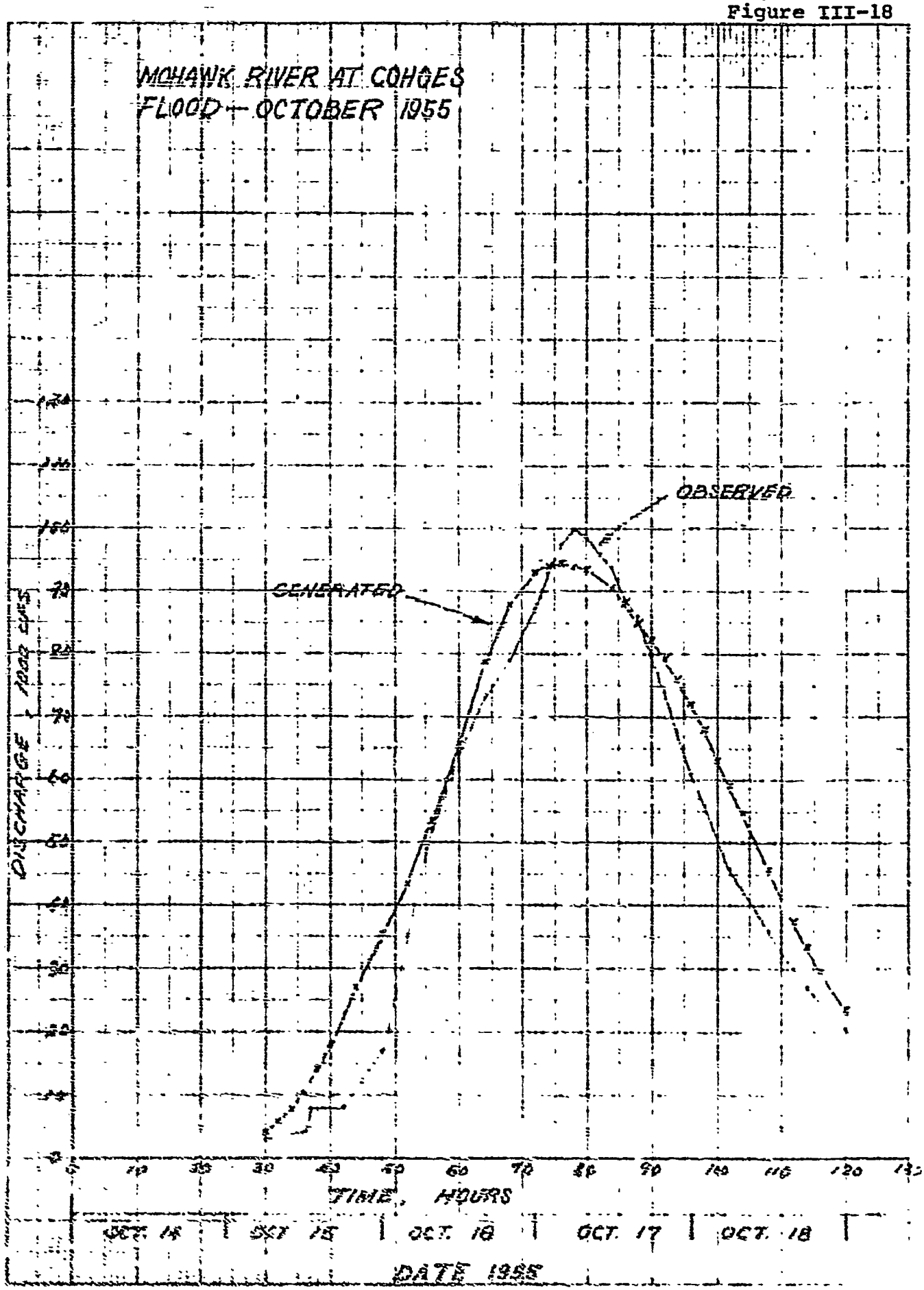
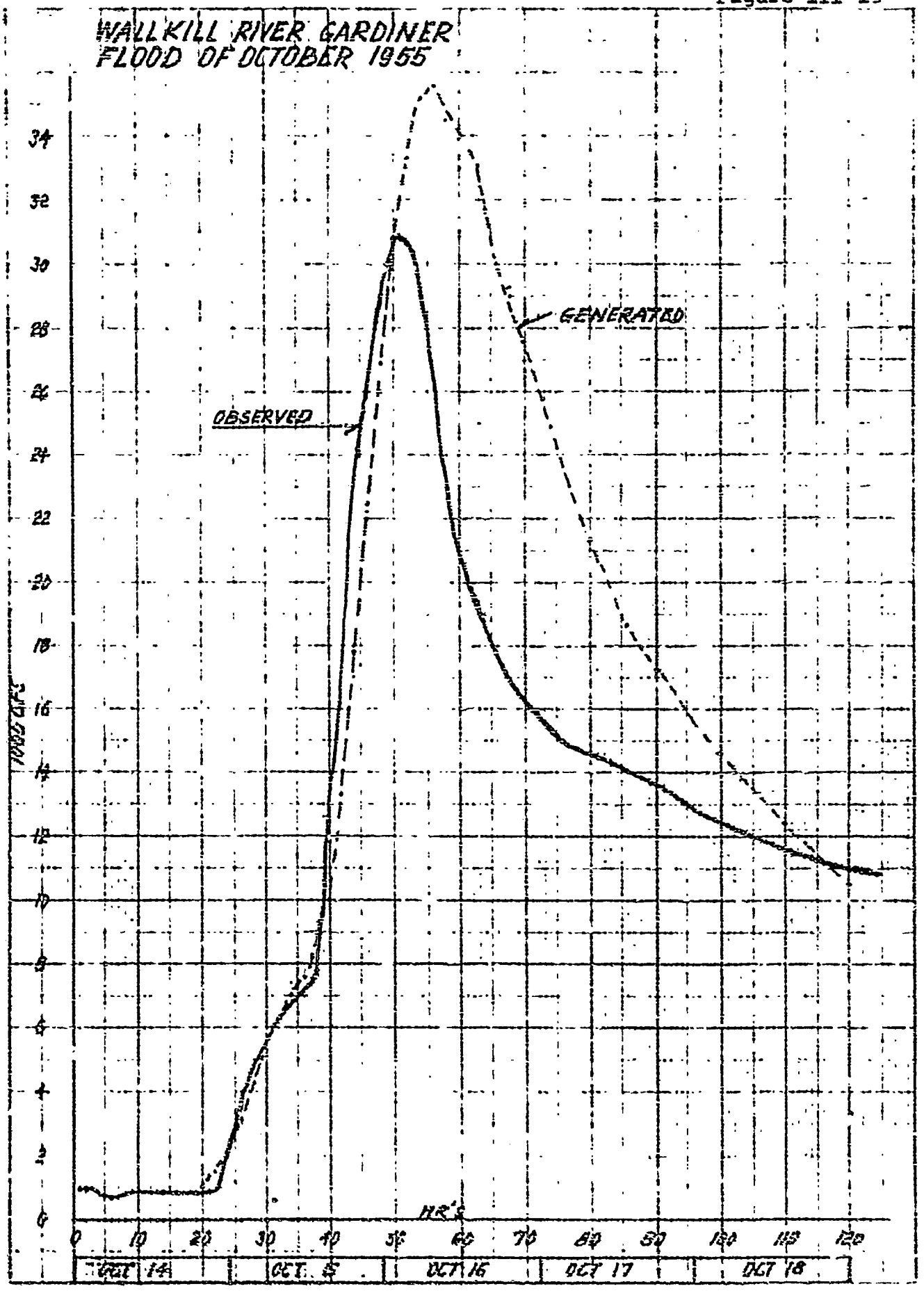


Figure III-19



puted 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 higher 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 areal distribution over large subbasins, base flow and losses estimates. The non-uniform areal 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 rainfall 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. Whenever 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

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.

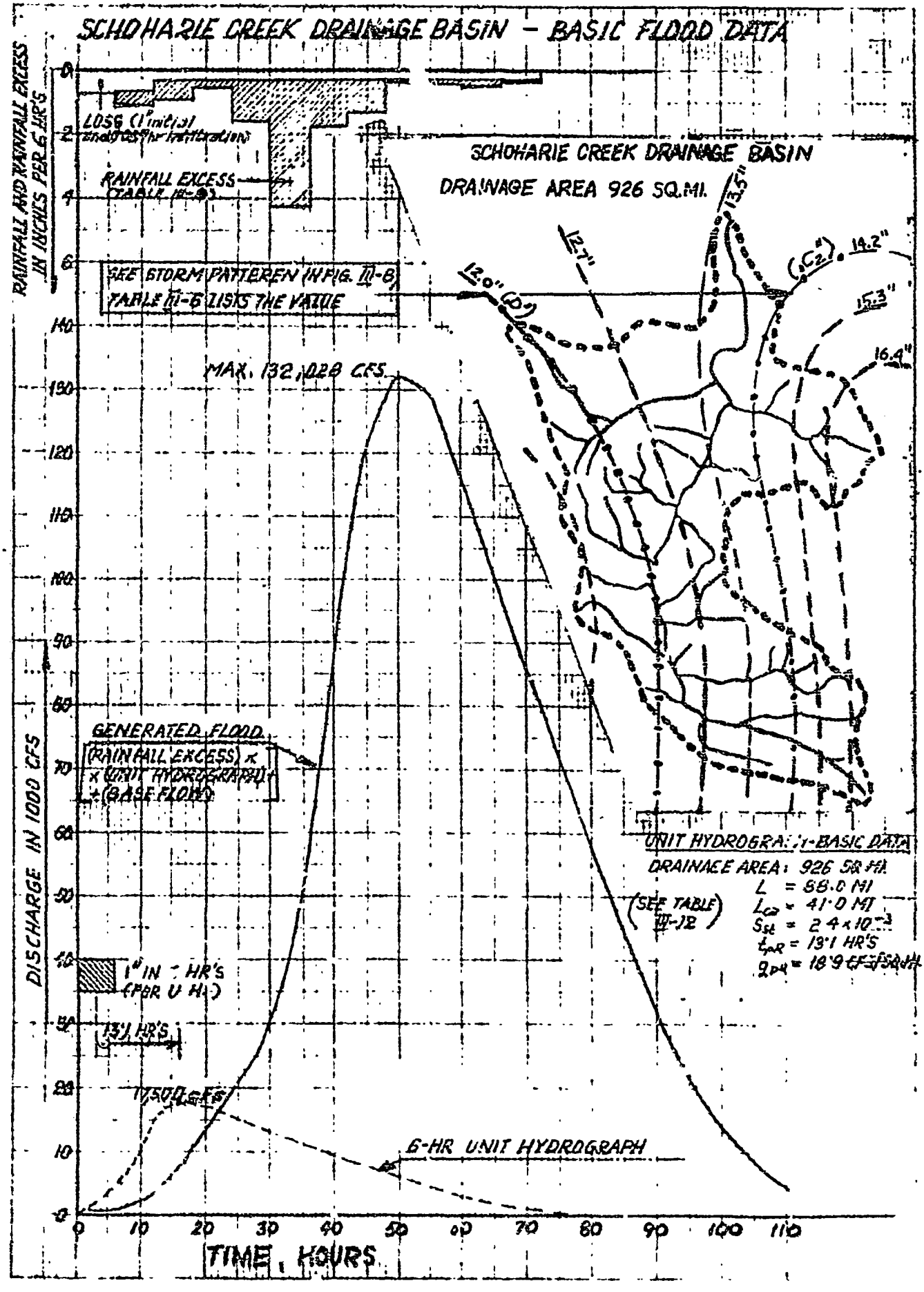
b. Subbasin Flood Hydrographs

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 interpelated 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

FIGURE III-20



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

The synthetic 6-hour unit hydrograph for this subbasin 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-13. 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 for 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 subbasins are represented by the solid curves in Figure III-21. The Mohawk River Flood downstream of its 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.

NEW YORK UNIVERSITY
ENGINEERING RESEARCH DIVISION

REPORT NO. 372.1

A MICROMETEOROLOGICAL SURVEY OF THE BUCHANAN, NY AREA

SUMMARY OF PROGRESS TO 1 DEC. 1955

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Project Director

Approved by Harold K. Work /p.p.
H. K. Work
Director, Research Division

Prepared for
Consolidated Edison Company of New York
November 1955

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1. Introduction

1.1 Description of Topography

Indian Point Park, site of the proposed power plant, is located some two miles SW of the town of Peekskill which is the most densely populated area in the immediate vicinity of the site. Indian Point, itself, is on the east bank of the Hudson River which runs NE-SW at this point but makes a sharp right angled turn some 2 miles NE of the Point (see Fig. 1). The west bank of the Hudson is flanked by the steep, heavily wooded slopes of the Dunderberg and Ramapo peaks (heights close to 1,000 ft.) which extend further to the west by other names and gradually rise to slightly higher peaks.

The general orientation of this mass of high ground is NE to SW. One mile NW of the site, Dunderberg bulges to the east, and north of Dunderberg and the site, high ground reaching 900 ft. forms the east bank of the Hudson as the river makes a sharp turn to the northwest. To the NE of the site, the narrow beds of the Canapus and Peekskill creeks lie generally in a NNE to NE orientation. To the east of the site peaks are generally lower than those to the north and west. Spitzenberg and Blue Mts average about 600 ft. in height and there is a weak, poorly defined series of ridges which again seem to run in a NNE direction. The river south of the site makes another sharp bend to the southeast and then widens as it flows past Croton and Haverstraw.

1.2 Meteorological Effects of Topography

The site then lies in a bowl surrounded on almost all side by high ground ranging from 600 to 1000 ft. Although the heights of the orographic features are relatively small when compared to classical Alpine or western United States valley studies, the topography surrounding the present

site is nevertheless pronounced enough to decisively influence the meteorology of the valley. We may expect the topography to exert its influence in the following ways.

a) The orientation of the ridges serves to channel the air flow in the valley into preferred directions. We would therefore expect the frequency distribution of wind direction to be more peaked than it would be over level terrain.

b) The ridges act as a barrier to the descent of faster moving air to ground levels and for this reason wind speeds in the valley should be lower than over level terrain.

c) The differing radiational characteristics of the valley, valley sides and plain at the mouth of the valley combined with the sheltering effect of the ridges give rise to thermally induced local air circulations. These circulations, if present, should have a well marked diurnal period and should be seasonally dependent. When present in a pure and highly developed form, the thermally induced currents have well-defined vertical branches with systematic ascending or descending currents along the valley sides and center.

d) When prevailing winds are strong and normal to the ridges some sort of quasi-stationary eddy wind system may develop in the valley. This type of wind system may be very turbulent and in a rough statistical sense may also have preferred regions of sustained positive or negative vertical currents.

e) The effect of these valley systems on diffusion rates are largely unknown. Moreover, the vertical circulation branches - if they exist - can redistribute diffusing material in the vertical in a manner which is quite inconsistent with values of the traditional vertical diffusion coefficients derived from studies over level terrain.

1.3 Objectives of Project

The objectives of the present study are to evaluate the effect of topography on the diffusion climatology of the site. This involves collection of data describing the wind distribution in the vertical and horizontal, temperature gradients and interpretation of this data in terms of diffusion from a ground and elevated source.

2. Equipment

2.1 Wind Measuring Equipment

The project has on hand now five Bendix-Friez Aerovanes, the outputs of which are recorded continuously on twin chart recorders and are the primary source of our climatological wind data. Two highly sensitive Beckman Whitley anemometers are used for low level wind determinations and for special studies. Two very sensitive bivanes developed at M.I.T. (see Fig. 2) are used to determine the three dimensional wind direction. The bivanes when used with the Beckman Whitleys form a compatible system for quantitative determination of vertical currents. The output of both the Beckman and bivane can be recorded continuously (but not regularly) on standard G-1 as Esterline Angus recorders, four of which are in our possession now.

Also available to the project are two theodolites for use in double theodolite
/pibal ascents. It is hoped that the angular readings of the theodolites when following balloons can be recorded photographically at discrete intervals, thus making possible regularly scheduled ascents despite manpower limitations.

2.2 Temperature Measuring Equipment

Temperature is measured by Type A - Brown Resistance Thermometers. When placed in specially designed wells, the time constant of these ther-

is about 3 minutes in a wind of 20 fps. The bulbs are placed in shielded, gold leafed cylinders and are aspirated at about 20 fps. The output of the resistance bulbs is recorded on a 4 channel Brown Electronik Recorder, while differences in the output of matched bulbs are recorded on a 6 channel Brown Electronik recorder. The recording cycle is 2 minutes for the four channel recorder (channels sampled at 30 second intervals) and 3 minutes for the six channel temperature difference recorder. Four of these bulbs are now in our possession. Ten others are still on order with delivery now expected by mid-December.

2.3 Smoke Generating Equipment

The Chemical Corps has kindly loaned the project an M2A1 smoke generator. The generator operates at a capacity of 50 gallons per hour and should enable us to collect smoke trajectory observations in all but the strongest wind conditions. For use with the generator, the project has acquired a 700 cfm blower (at 1.5 psi) powered by a 7-1/2 hp 220 V. motor.

3. Observation Sites

3.1 Meteorological Tower

Figure 1 is a map of the area showing the principal observation points. The underlined names are sites of more or less contiguous observations. The meteorological tower is the focal point of all observations made in the valley. The tower is a 310 ft guyed trylon type (sides about 42 inches) with an inside ladder. The base of the tower is about 110 ft above river level. Seven 10 ft booms are mounted horizontally at 50 ft intervals for support of the meteorological instruments. Aerovanes are mounted on alternate booms at heights of 210, 310 and 410 ft above river level while Brown resistance bulbs will be mounted at 50 ft intervals, (see Fig. 2). A five inch galvanized pipe runs up one corner of the tower.

The smoke generator and blower are mounted at the foot of the tower and during operations the smoke is forced up the pipe by the blower. Ports are provided at about 300, 350 and 410 ft above river level enabling us to vary the height of emission of the smoke trail as conditions warrant. T output of the meteorological instruments are recorded in the Trailer, shown in Fig. 4.

3.2 Off-Tower Anemometer Sites

The U. S. Maritime Commission has kindly granted us p ssion to use several of the Reserve Fleet ships anchored in the Hudson tony Pt. as anemometer sites. Usable observations have been continuou recorded on the Hall and Jones (see Fig. 1) since 1 Sept. The anemometer on the Hall was removed 1 Nov. for installation on the tower. The Jones anemometer will operate continuously for the remainder of the project. It has a good exposure, close to the middle of the river, and presumably removed from the slope currents of the Dunderberg peak to the west.

A Beckman-Whitley anemometer began operation 15 Nov. on the roof of a dock on the east side of the river. The Jones and Dock observations (at about 70 ft above river) will fill in the details of the flow close to the valley.

An aerovane will shortly be mounted on the roof of the main building of the Peekskill Military Academy (elevation about 200 ft). This is the approximate site of a USWB station which made regular hourly observations during 1932, 1933 and part of 1934. Reactivation of the station will enable us to get the maximum information from the old records.

3.3 Chronology of Available Observations:

In summary, pertinent data available to the project for climatological purposes as of 1 Dec. follow:

Site	Ht.(above river)	Type	Dates of Availability
Hall	70	Wind	1 Sept - 30 Oct.
Jones	70	Wind	1 Sept on
Dock	50	Wind	15 Nov. on
Tower	210	Wind	1 Nov.
"	310	Wind	1 Nov.
"	410	Wind	1 Nov.
"	T(410-T(260)	Temp. diff.	8 Dec.*
"	T(160)-T(260)	Temp. diff.	8 Dec.*
"	at 50 ft intervals	" "	10 Jan.*
Water	0	Temp.	10 Jan.*
Peekskill Military Academy	200	Wind	Jan. 1932-Sept.1934
	200	Wind	8 Dec.*

* Estimated date of installation.

4. Discussion of Data

4.1 Diurnal Variation of Wind Direction

The most striking feature of the two months of Jones data which have been analysed thus far is the diurnal variation of wind direction. At sunset there is a pronounced tendency for the wind to shift to NE. At sunrise there is again a tendency for the winds to shift to SW. On some days the shift is very abrupt and occurs just about at sunrise and sunset. On other days the shift is not so abrupt and may not take place till after midnight and then only after an hour or two of calm winds. NE winds, on occasion, persist until noon before southwesterly flow begins. On still other days, of course, the prevailing winds are so strong as to swamp any possible valley effect, and on those days there is no diurnal shift in wind direction.

A northeast wind parallels the valley orientation at the plant site and is directed down river towards the south of the Hudson. If we define the axis of the valley as a line running 040°-220°, and compute wind components along the axis of the valley and in the cross-valley direction, the diurnal variation of wind direction becomes quite apparent. Figure 5 is a plot of the median values of the up and down valley component of the wind as a function of

time of day. The down valley component reaches its peak median value at about 0600 while the up-valley component reaches its peak median value at about 1600. The peak median value of the nocturnal down wind component is 5 mph while the peak value of the daytime median wind is about 3 mph. These peak values merely reflect the fact that statistically the night time northeasterly flow occurs more frequently than does its/daytime counterpart. southwesterly

This is clearly seen in Figure 6 where the frequency distribution (calms omitted) of wind components along the axis of the valley (6a) and in the cross valley direction (6b) is shown for night (solid line) and day (dashed line). The distribution of the axial component is strongly bimodal for both night and day. At night the most frequent component is down valley (northeast) about 6 mph. A secondary mode is up-valley at about 5 mph but its frequency is about 1/3 the down valley frequency. In the daytime the principal mode becomes up-valley but a substantial down valley frequency is still observed. It is evident that two regimes are operating, one associated with the valley influence under relatively weak synoptic flow conditions, and one associated with rather strong synoptic flow which alternately swamps the day and nighttime valley influences. A third and weaker regime also operates, namely, a tendency for persistence of northeasterly winds until about noon on some days and a tendency for northeasterly winds to begin rather late at night on still other days. The latter regime is undoubtedly associated with cloudiness and other air mass radiational conditions.

The distribution of the cross-valley component (Fig. 6b) is very sharply peaked around zero at night indicating that during a great many nights the flow is substantially along the axis of the valley. In the daytime the zero peak is considerably reduced and the distribution broadens indicating a considerable cross-valley component which is generally superimposed on the up-valley daytime flow. All in all, comparison of the two

distributions indicate that night time stability conditions have the effect of channeling the flow along the axis of the valley, while in the daytime the channeling effect of the valley sides is not as evident, being partially overcome by descent of air from above the valley ridge lines. This can be clearly seen when we compare the r.m.s. wind component along the axis of the valley with the r.m.s. wind component in the cross-valley direction. At night the r.m.s. along-valley component is about twice the cross-valley r.m.s. value. During the day, the r.m.s. value of both components are about the same.

4.2. Distribution of Wind With Height and Across-Valley.

The observations discussed above were made on board the Jones anchored in midriver where presumably the down valley wind should exist in its purest form. There are two months of simultaneous data available from the Hall, anchored close to the west shoreline of the Hudson. Although not yet ready for presentation, preliminary analysis of the differences between the Jones and Hall indicate very little difference in the broad features of the flow pattern.

The project was fortunate in being able to acquire (from the U.S. Weather Bureau) a 2-3/4 year series of data made at the Peekskill Military Academy, Jan. 1932-Sept. 1934. As nearly as can be ascertained at the present time, these observations were made at a point at least 200 ft. above river level and about 2 miles NE of the site. Essentially the same diurnal variation of wind direction as discussed above was found with the old data. In addition to this, preliminary examination of tower winds for Nov. indicate that on at least some occasions the diurnal trend in wind direction is in evidence up to 400 ft. above river level. The tentative conclusion is that the down valley wind fills the breadth of the valley and on at least some occasions extends to over 400 ft. above river level.

4.3. Seasonal Dependence of Down-Valley Winds

To illustrate the seasonal dependence of night time northeasterly flow we have calculated the percent frequency that NNE to ENE winds were observed at Peekskill Military Academy, 1932-1934 as a function of time of day for the various seasons. The results are shown in Figure 7. As was to be expected, the diurnal variation of northeasterly winds reaches a maximum amplitude during summer and a minimum amplitude during winter. This result can be explained as due principally to the generally stronger synoptic flow during winter than during summer. A secondary reason is perhaps the change in cloudiness and radiational conditions from summer to winter. The dashed line plot of Fig. 7 represents the percent frequency of occurrence of both calm and NNE to ENE winds. The reason for this presentation is that calm is an arbitrary designation depending on instrumental and observer thresholds. Considering the equipment in use in 1932 we believe that a large number of the recorded night time calms were characterized by a slow drift of air from the NE. This supposition is strengthened by the fact that a similar count for the Jones Sept - Oct. data indicates a maximum night time incidence of about 70% for NNE to ENE flow.

4.4. Presentation of Climatological Data

The frequency distribution of wind speed and direction by wind speed class and by night and day for the various seasons is presented in Tables 1 thru 4. The data is from the U.S. Weather Bureau station at Peekskill Military Academy (1932-1934). Day and night were arbitrarily defined with respect to sunrise and sunset of the middle of each month. It is believed that in this form the interpretation of the frequency distribution of wind in terms of diffusion parameters is facilitated. For example, the night time class of wind from any direction with speed 1-4 mph is undoubtedly

characterised by a strong inversion. To assist in this kind of interpretation we have also available (but not presented in this report) the distribution of cloudiness for each class in Tables 1-4. Before making definite interpretations, however, we prefer to correlate the tower temperature measurements with the wind data we will shortly begin receiving from the Peekskill Military Academy.

5. Conclusions and Future Plans

5.1. Conclusions

We now have a fairly good idea of the general flow patterns at our observation points. Still unknown, is the distribution with height of the flow patterns we have discussed in the previous section. The meteorologically unique feature of the valley is, of course, the substantial night time incidence of northeasterly winds. This valley phenomenon is of great importance to our study primarily because low wind speed and presumably stable thermal stratification are associated with the night time winds. These conditions usually result in a narrow plume of relatively high concentration. However, because of the possibility - still unexplored - of sustained up or down vertical velocities it is still too early to make statements about ground concentrations.

In many ways it is fortunate that the night time winds are usually directed down valley away from Peekskill. On the other hand, it becomes of paramount importance to evaluate the effect of the bend of the river south of the site on the trajectories of sources emanating from Indian Point Park. It is also of great importance to evaluate the diffusive conditions associated with the day time up-valley winds which have a tendency to carry diffusing materials to Peekskill. This procedure must also be carried out for the occasions when night time flow associated with general weather conditions is directed up-valley.

5.2. Future Plans

Our future plans follow directly from our tentative conclusions and these are generally to evaluate trajectories and diffusive conditions under critical wind condition regimes. This will be done with the aid of our smoke installation, and as far as is consistent with horizontal visibility conditions, with quasi constant level balloon trajectories.

As winter draws on, it is apparent that our opportunity for studying the up and down valley winds will be curtailed. However, the generally stronger winter winds will afford us opportunity to examine the structure of possible eddy winds in the neighborhood of the site. These winds - if they exist at the proper height - may be instrumental in carrying smoke to the ground.

In the meanwhile we must complete our instrumental installation, iron out the bugs in the smoke system, and continue with the accumulation and analysis of climatological data.

TABLE I - Frequency distribution of wind speed for winter months (by day and night) as observed at Peekskill Military Academy, 1932-34.

	DAY				NIGHT			
	Speed (mph)				Speed (mph)			
Direction	1-4	5-10	>10	Total	1-4	5-10	>10	Total
N	.020	.031	.029	.080	.009	.019	.021	.049
NNE	.004	.003	.003	.010	.004	.004	.003	.011
NE	.048	.047	.035	.130	.077	.085	.029	.191
ENE	.006	.020	.031	.057	.006	.028	.022	.056
E	.017	.009	.001	.027	.038	.006	.001	.045
ESE	.002	.002	-	.004	.004	.002	-	.006
SE	.021	.006	.001	.028	.016	.005	-	.021
SSE	-	.005	.002	.007	.002	.004	-	.006
S	.022	.020	.003	.045	.028	.016	.001	.045
SSW	.005	.026	.011	.044	.005	.026	.011	.042
SW	.048	.051	.021	.120	.043	.045	.011	.099
WSW	.002	.015	.012	.029	.002	.001	.005	.008
W	.031	.028	.019	.078	.017	.020	.007	.044
WNW	.001	.028	.034	.063	.003	.017	.025	.045
NW	.014	.046	.103	.165	.013	.077	.094	.184
NNW	.002	.021	.034	.057	.004	.021	.035	.060
TOTAL	.243	.362	.339	.944	.271	.376	.215	.912
Calm				.056				.088
		Total		1.000		Total		1.000

TABLE II - Frequency distribution of wind speed for Spring months (by day and night) as observed at Peekskill Military Academy 1932-34.

	DAY				NIGHT			
	Speed (mph)				Speed (mph)			
Direction	1-4	5-10	>10	Total	1-4	5-10	>10	Total
N	.018	.029	.020	.067	.021	.023	.012	.056
NNE	.003	.014	.003	.020	.004	.009	.001	.014
NE	.035	.055	.007	.097	.127	.065	.010	.202
ENE	.004	.030	.026	.060	.010	.032	.015	.057
E	.012	.013	.002	.027	.041	.016	.004	.061
ESE	.002	.007	.002	.011	.004	.006	-	.010
SE	.018	.019	.003	.040	.029	.014	.002	.045
SSE	.007	.014	.006	.027	.008	.019	.002	.029
S	.036	.039	.014	.089	.040	.036	.002	.078
SSW	.012	.048	.021	.081	.010	.044	.010	.064
SW	.049	.074	.010	.133	.042	.037	.007	.086
WSW	.007	.008	.005	.020	.002	.001	.002	.005
W	.040	.026	.006	.072	.019	.011	.002	.032
WNW	.005	.028	.024	.057	.002	.010	.021	.033
NW	.022	.046	.045	.113	.012	.023	.014	.049
NNW	.002	.016	.012	.030	.002	.014	.012	.028
TOTAL	.272	.466	.206	.944	.373	.360	.116	.849
Calm				.056				.151
		Total		1.000		Total		1.000

TABLE III - Frequency distribution of wind speeds for Summer months (by day and night) as observed at Peekskill Military Academy, 1932-34.

	DAY				NIGHT			
	Speed (mph)				Speed (mph)			
Direction	1-4	5-10	>10	Total	1-4	5-10	>10	Total
N	.022	.030	.011	.063	.025	.017	.006	.048
NNE	.003	.008	-	.011	.002	.007	-	.009
NE	.064	.080	.013	.157	.200	.078	.006	.284
ENE	.008	.034	.016	.058	.010	.021	.009	.040
E	.014	.007	.001	.022	.051	.011	.001	.063
ESE	.002	.004	.001	.007	.004	.003	-	.007
SE	.017	.010	-	.027	.026	.015	.002	.043
SSE	.003	.009	.003	.015	.009	.007	.001	.017
S	.036	.040	.008	.084	.060	.031	.002	.093
SSW	.010	.060	.019	.089	.014	.030	.005	.049
SW	.069	.078	.006	.153	.042	.041	.002	.085
WSW	.006	.013	-	.019	.004	.001	-	.005
W	.026	.019	-	.045	.018	.005	-	.023
WNW	.005	.026	.008	.039	.001	.007	.001	.009
NW	.026	.063	.025	.114	.014	.041	.012	.067
NNW	.005	.014	.013	.032	.003	.009	.009	.021
TOTAL	.316	.495	.124	.935	.483	.324	.056	.863
Calm				.065				.137
		Total		1.000		Total		1.000

TABLE IV - Frequency distribution of wind speeds for FOUR months (by day and night) as observed at Peekskill Military Academy, 1932-33.

	DAY				NIGHT			
	Speed (mph)				Speed (mph)			
Direction	1-4	5-10	>10	Total	1-4	5-10	>10	Total
N	.023	.038	.022	.083	.018	.024	.008	.050
NNE	.001	-	.001	.002	.001	.003	.005	.009
NE	.049	.073	.031	.153	.102	.092	.039	.233
ENE	.005	.037	.024	.066	.008	.038	.029	.075
E	.020	.010	-	.030	.052	.004	-	.056
ESE	.004	.004	.001	.009	.002	.004	-	.006
SE	.014	.004	.002	.020	.026	-	-	.028
SSE	.001	.013	.002	.016	.003	.006	.002	.011
S	.028	.031	.010	.059	.042	.024	.010	.076
SSW	.014	.034	.015	.063	.015	.028	.011	.054
SW	.052	.070	.014	.136	.039	.032	.005	.076
WSW	.001	.008	-	.009	.001	.007	.002	.010
W	.036	.025	.004	.065	.011	.020	.005	.036
WNW	.009	.026	.026	.061	.006	.018	.016	.040
NW	.019	.044	.063	.120	.021	.055	.044	.120
NNW	-	.012	.023	.035	.005	.009	.006	.020
TOTAL	.260	.429	.238	.927	.354	.364	.182	.900
Calm				.073				.100
		Total		1.000		Total		1.000

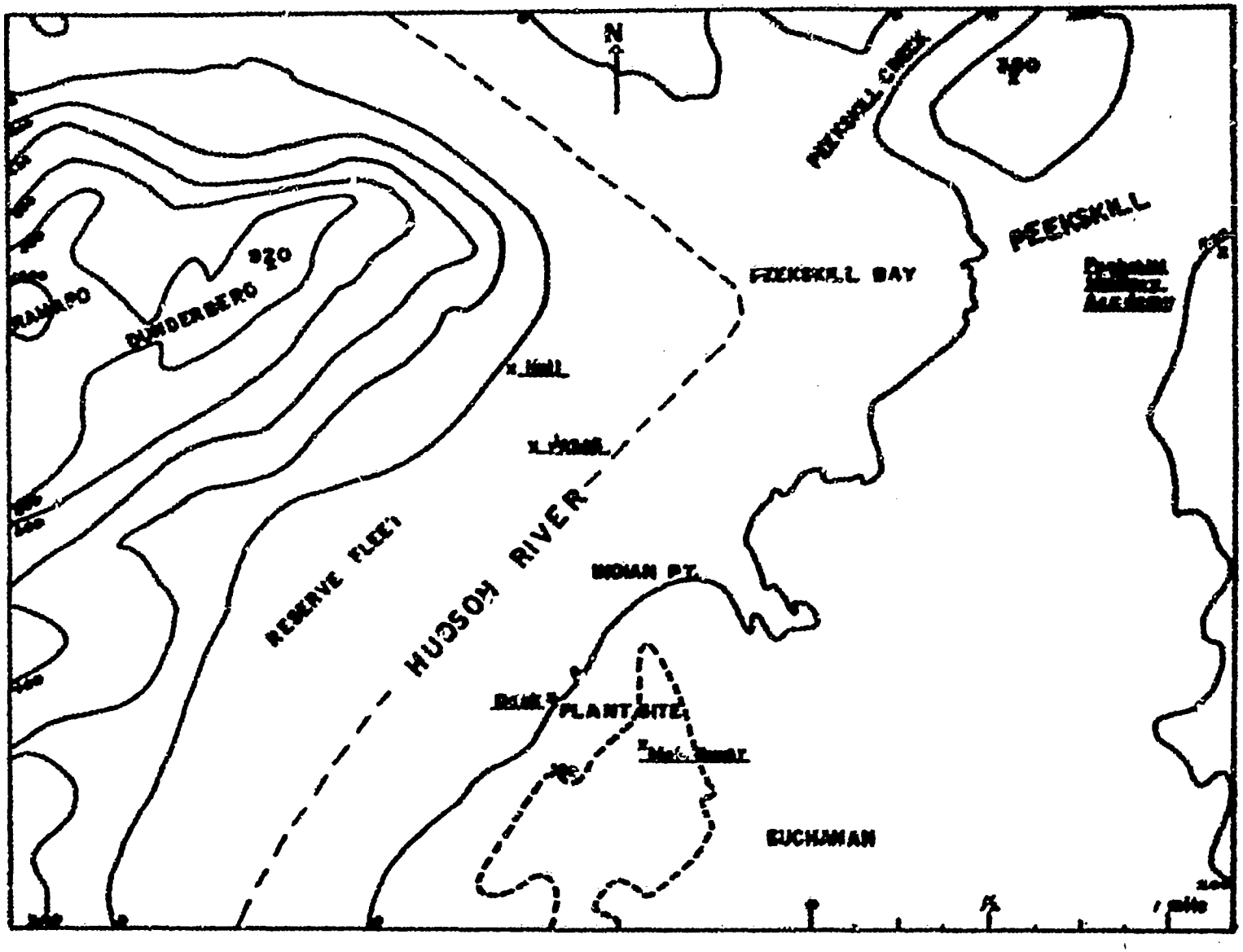


Fig. 1 Map of area surrounding meteorological tower. Underlined areas (Hall, Jones, Dock, Peekskill Military Academy) are sites of anemometer locations or sources of previous data.

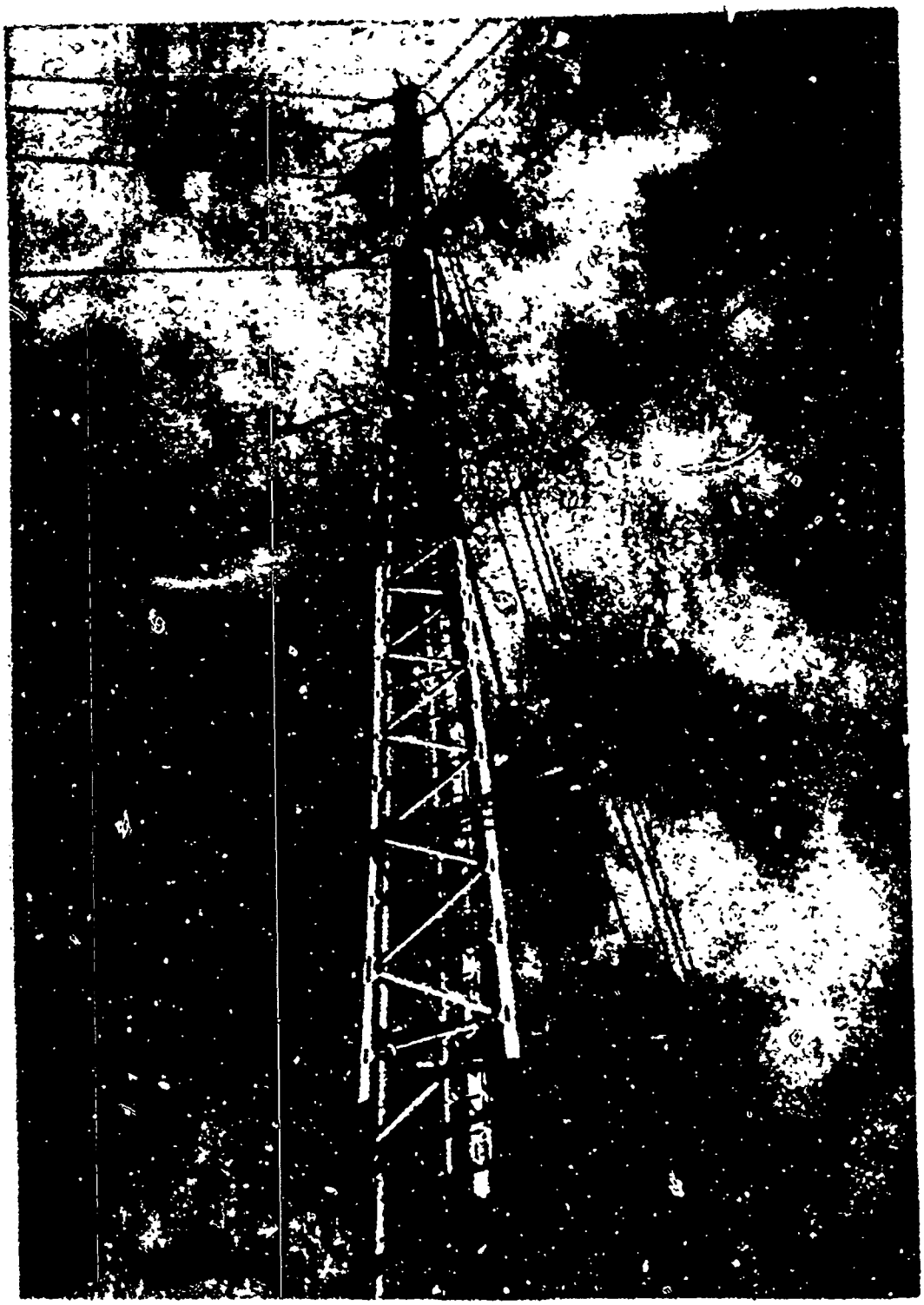


Fig. 2 Photograph of tower. Note Aerovanes at 100, 200 and 300 ft. levels.

Fig. 3 Showing bivane in Trailer office before installation.

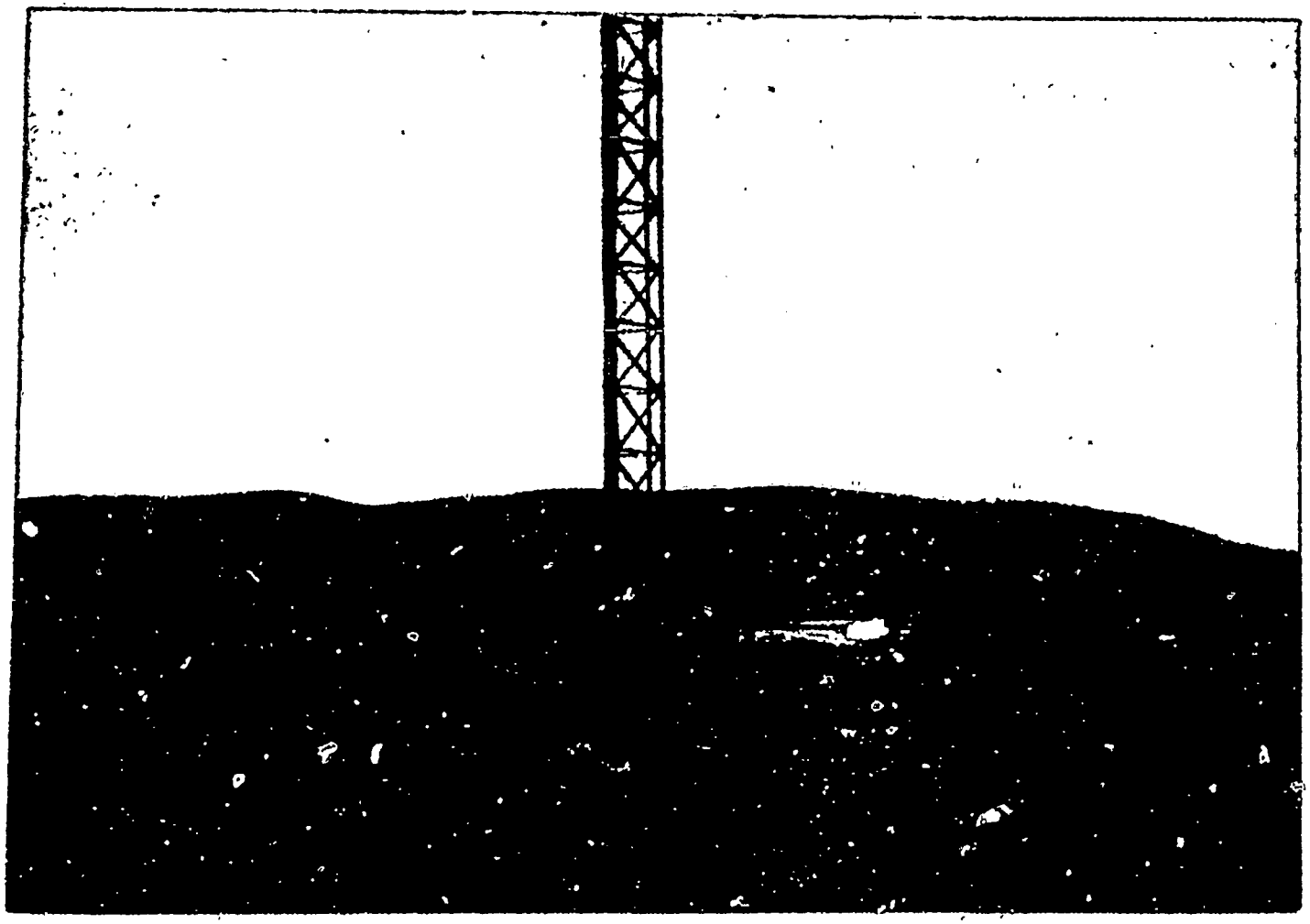


Fig. 4 Photograph showing base of tower, smoke producing apparatus, and trailer housing recorders. Note Dunderberg in background.

Fig. 5 - Diurnal Variation of wind component along axis of valley (040°= Down Valley, 220°= Up Valley), median values for indicated hours. (Data from Jones, 70 ft. above river, Sept., Oct. 1955)

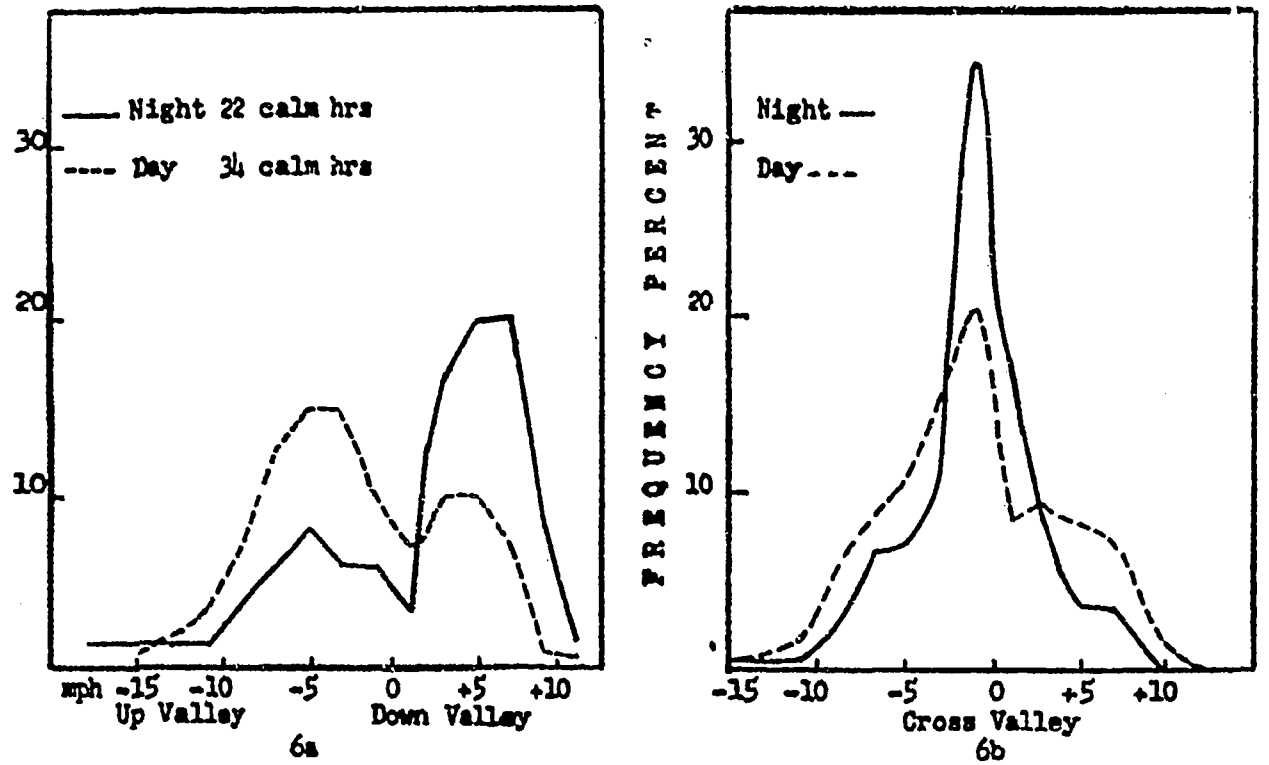


Fig. 6 - Frequency distribution of wind components along axis of valley, (040°= Down Valley, 220°= Up Valley) and in the cross valley direction (130°= +, 310°= -) by day and night (Data from Jones, Sept. Oct. 1955)

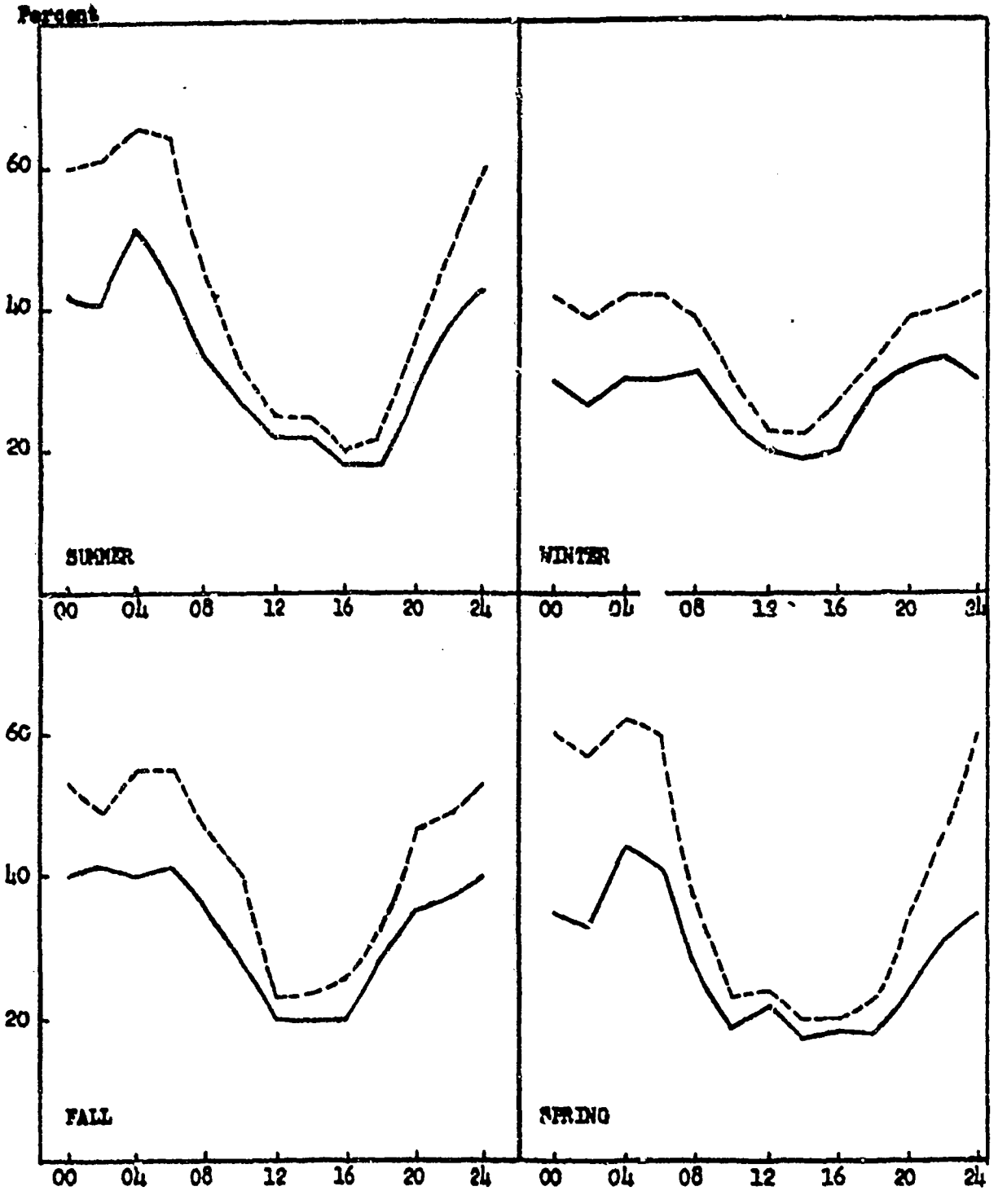


Fig. 7 - Illustrating the seasonal dependence of the diurnal variation of NErly wind direction. Solid line represents percent time wind was from NNE-ENE at the indicated hour. Dashed line represents percent time wind was either calm or from NNE-ENE. (Data from Peekskill Military Academy, Jan. 1932-Sept. 1934.)

NEW YORK UNIVERSITY

COLLEGE OF ENGINEERING
RESEARCH DIVISION
University Heights, New York 53, N.Y.

Technical Report No. 372.3

EVALUATION OF POTENTIAL RADIATION HAZARD RESULTING FROM ASSUMED RELEASE OF RADIOACTIVE WASTES TO ATMOSPHERE FROM PROPOSED BUCHANAN NUCLEAR POWER PLANT

(Section 2 & 3 only)

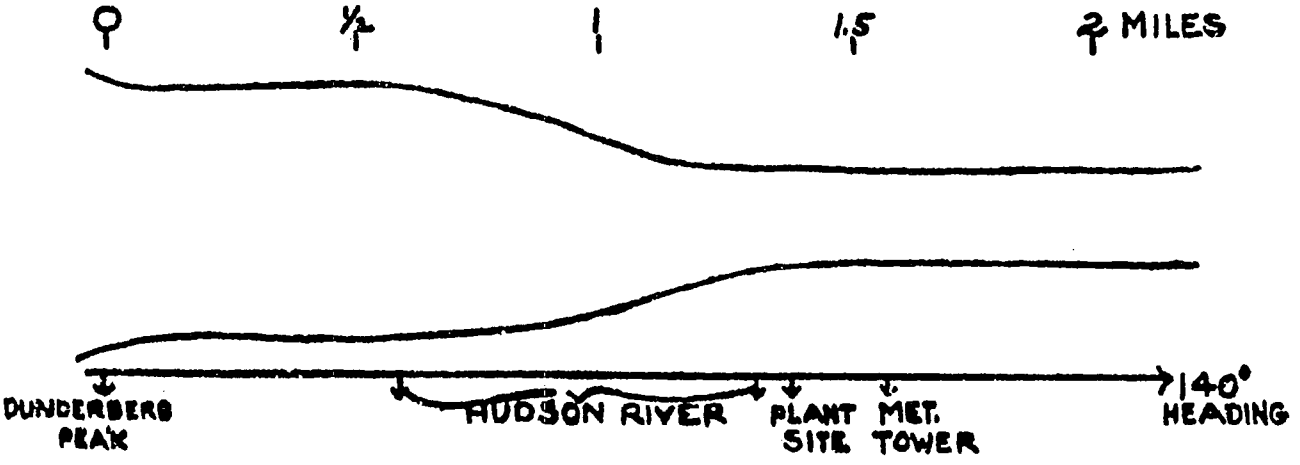
Prepared for
Consolidated Edison Co. of N. Y., Inc.
April, 1957

2. Diffusive conditions at site

2.1. Eddy wind structure in the valley

Meteorological conditions investigated were cases of daytime strong NW flow breaking over the Dunderberg peak. Data sources include instrumented aircraft runs (through cooperation of Cornell Aeronautical Laboratory), and (through the cooperation of the Meteorology Group at Brookhaven) comparison of three-dimensional wind vector distributions taken simultaneously at Buchanan and Brookhaven National Laboratory.

Unfortunately, the aircraft runs are not yet completely reduced,* but sufficient information is available to serve our purpose here. Following is a schematic sketch of the envelope of turbulent fluctuations measured by the airplane on a run at 1000 ft, starting just over the Dunderberg peak and proceeding southeast over the meteorology tower.



*The data have since been published in full in Lappe, V.O., and B. Davidson, 1960: The power spectral analysis of concurrent airplane and tower measurements of Atmospheric Turbulence, Final Report, Contract No. NOAS 58-517-d, College of Engineering, New York University.

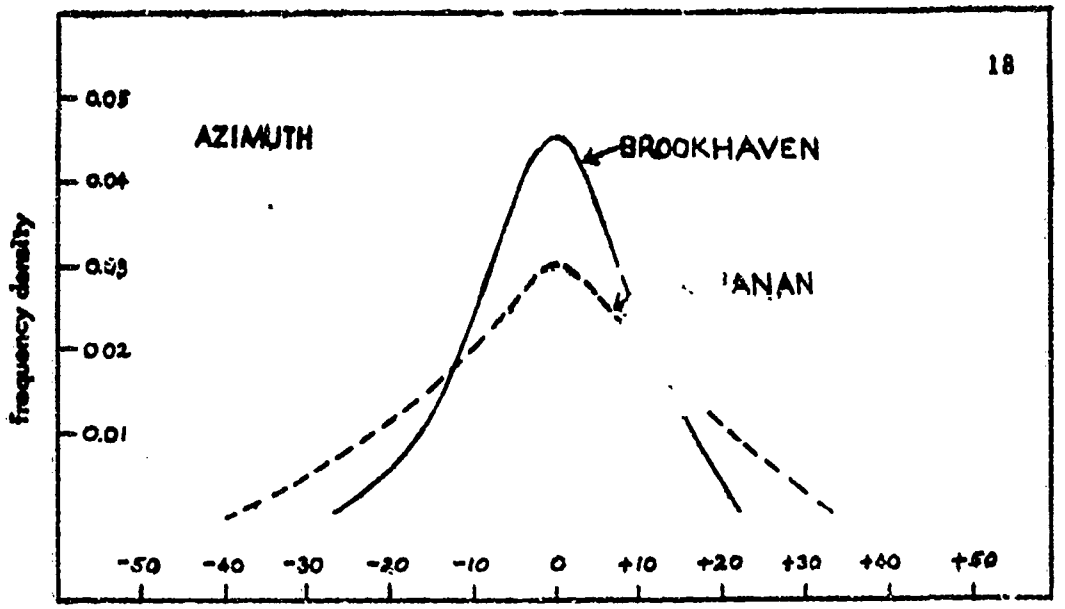
(footnote added October 1965)

The very large amount of turbulence associated with the breaking of air flow over the Dunderberg decays rather quickly downstream and the turbulent field settles down to a steady rms level at the east bank of the Hudson River. This uniform level is maintained for approximately five miles east of the site.

Comparison of an airplane run made at 400 ft elevation following the east bank of the Hudson with a run made the same day over the Brookhaven tower indicates that the rms turbulence at Buchanan is about 1.5 times greater than that observed at Brookhaven. The larger Buchanan rms value was due to the contribution of relatively strong, but highly intermittent gusts. Our conclusions are that the plant site is safely out of the very strong field of turbulence associated with the Dunderberg range, but that the site will be subject to occasional incursions of extreme gusts under strong northwest flow conditions.

This conclusion is further borne out by comparison of 10 second mean azimuth and elevation angle fluctuations at Brookhaven and Buchanan shown in fig. 2.1. The distributions cover about 3 hours of data taken under daytime strong northwest flow conditions. The azimuth angle distributions are directly comparable and indicate an rms azimuth fluctuation at Buchanan about 1.5 times that at Brookhaven. This ratio is consistent from hour to hour.

The elevation angles were measured by dissimilar instruments (ours having the faster response), but the 10 second mean should make



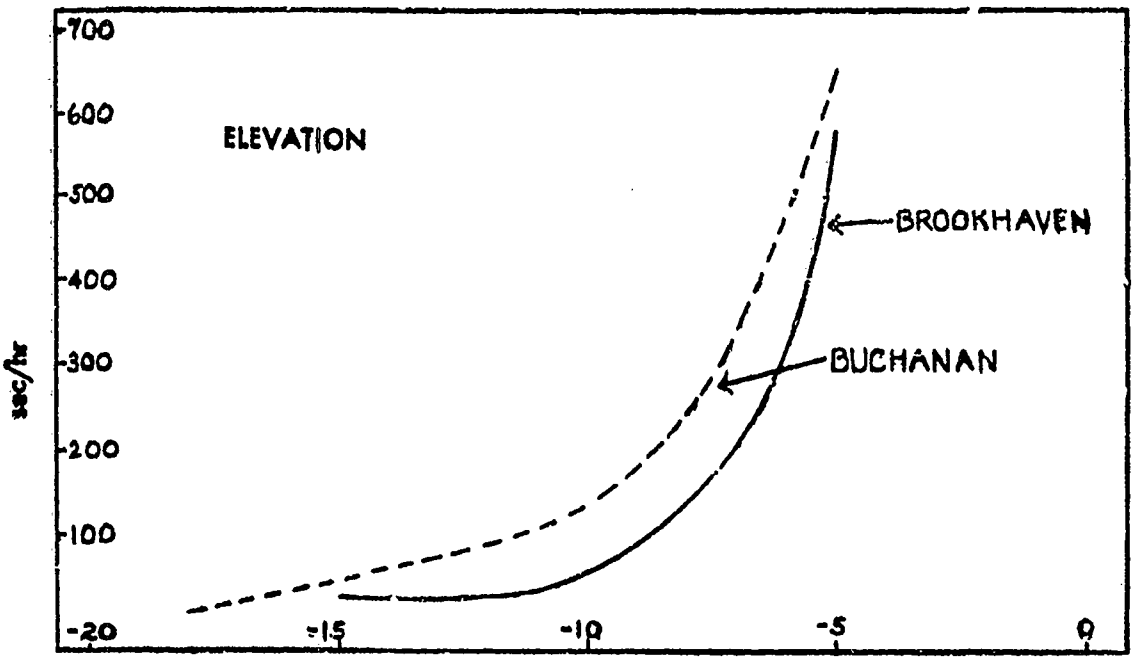


Fig. 2.1. Comparison of Brookhaven and Buchanan azimuth and elevation angle fluctuation traces.

the instrument readings comparable. There is a definite tendency toward more extreme downward directed vertical angles at Buchanan. However, this difference is not consistent from hour to hour, the major contribution to the difference coming from one hour of data; the other two hours showed but minor differences.

2.2. Diffusion coefficients

Smoke experiments were conducted at irregular intervals with a smoke generator source located 91 meters above ground. The behavior of the smoke for a distance of 1000 m from the source was documented by photographs usually made at 20 second intervals. The quantities abstracted from the film included the rate of expansion of the instantaneous plume, and a simple count of the number of times smoke was on the ground at given distances downwind. By assuming an inverse square law for concentrations, i.e., dividing smoke frequency by distance squared, we were able to estimate the point of maximum ground concentration.

For radiation calculations, it is desirable to find coefficients to fit a Sutton type concentration equation. This type of equation involves three parameters, C_y, C_z, and n. The diffusion coefficients at Buchanan were evaluated in a number of different ways. All of these approaches gave essentially the same answer. Since diffusion coefficients are fairly well known at Brookhaven, the simplest method was to use the relationship (see Ref. 8)

$$\text{Buch } C_y^2 = \frac{\left[\frac{4}{(1-n)(2-n)}\left(\frac{N}{u}\right)^n \overline{(a'^2)}^{1-n}\right]}{\left[\frac{4}{(1-n)(2-n)}\left(\frac{N}{u}\right)^n \overline{(a'^2)}^{1-n}\right]}_{\text{Brook}} \text{Buch } C_y^2 \text{ Brook} \qquad (2.1)$$

where a'^2 is the variance of the azimuth angle fluctuation, and N is the macroviscosity. Here a small angle approximation has been made for $\sin a$, and the correlation between u and a has been assumed zero.

Brookhaven fluctuation data were available for their B condition. Under identical wind speed and stability conditions, the Reynolds stress $(\overline{\rho u'w'})$ was about the same for both sites. Similar stress values under identical large scale conditions imply similar roughness (z_o) values Since

$$N = \overline{(w')}^{1/2} z_o$$

the above indicates similar N values for both sites. As a first approximation n is assumed to be 1/2 and equation (2.1) can be solved for Buch C_y^2 in terms of the known Brookhaven coefficient. The process is repeated using new values of n, until the solution converges.

A similar equation can be solved for C_z^2 (here only the variance in downward directed elevation angles was considered). Knowing the distance of maximum concentration from observation, the remaining parameter, n, can be determined from

$$d_{max} = \left(\frac{h^2}{C_z^2}\right)^{\frac{1}{2-n}}$$

For other meteorological conditions the same procedure was followed this time using the coefficients established at Buchanan for the previous case as known values. In summary we find:

Table 2.1. Summary of diffusion coefficients at Buchanan.

Condition	$C_y(m)^{n/2}$	$C_z(m)^{n/2}$	n
L_1 Lapse (light winds 1-3 m/sec)	0.60	0.48	0.20
L_2 Lapse (strong winds > 4 m/sec)	0.53	0.43	0.30
N Adiabatic to isothermal temperature gradients	0.47	0.39	0.40

There was a great deal of variability in the coefficients determined for the the A case mostly because of large aperiodic changes in the mean wind direction.

The hourly lateral vertical diffusion coefficients (C_y) in Table 2.1 are large compared to the established Brookhaven values. The primary reason for this is the large azimuth oscillations which are due to the hills and rugged country surrounding the site. On an hourly basis, therefore, smoke plumes would diffuse more quickly at Buchanan than at most other flat terrain sites. On an annual basis, however, the restrictive influence of the valley channels the flow so that the long term annual spread is probably less at the Buchanan site.

2.3. Inversion plume

A few smoke runs were made under inversion conditions. These runs were documented by photographs from below and from an aircraft flying at

3000 ft. The results indicate a half-angle expansion of the instantaneous plume of about .05 radians with wind speeds of 4 mph. With wind speeds less than 2 mph the instantaneous half-angle expansion increases to about .09 radians. Under steady wind conditions azimuth angles fluctuate with a σ of about .065 radians.

The plume holds together as a compact mass for several miles. The ratio of width to height of the plume is on the order of 5 to 10. In general the inversion plume trajectory tends to follow the river around the bends north and south of the site.

3. Climatological Data and Diffusion Classes

Tables 3.1 and 3.2 summarize the wind and temperature gradient data taken at the 300 ft tower level (410 ft above river) for the winter and summer seasons. A similar yearly summary for the 100 ft tower level (210 ft above river) is presented in Table 3.3. The temperature gradient classification is defined as follows:

For 300 ft level:

I =	Inversion class	$T_{300}-T_7 \geq 0$
N =	Isothermal-Adiabatic class	$0 > T_{300}-T_7 \geq -1.8°F$
L =	Lapse (unstable class)	$-1.8°F > T_{300}-T_7$

For 100 ft level

I =	Inversion class	$T_{150}-T_7 \geq 0$
N =	Isothermal-Adiabatic class	$0 > T_{150}-T_7 \geq 0.9°F$
L =	Lapse condition	$-0.9°F > T_{150}-T_7$

For visual purposes these data are summarized in Figs. 3.1 and 3.2. (Arrows are flying with the wind.) There is a tendency for winds to be along the axis of the valley for both the summer and winter seasons. With respect to populated areas, wind trajectories are towards Buchanan and Verplank for a substantial portion of the time. Wind trajectories towards Peekskill (the major population center in the area) are relatively infrequent.

The diurnal wind regime in the valley at low levels was discussed in some detail in NYU Report 372.1. The height variation of the diurnal wind regime in the valley is extremely complicated and quite variable. Most frequently, the 030° night-time flow extends to about 2 to 3 hundred ft above river with a slow southerly drift above the down river

TABLE 3.1. Frequency Distribution of Wind Speed and Direction at 300 Ft. Tower Level for Winter Season (Nov-April) According to Temperature Gradient Class.

I = inversion, $T_{300}-T_7 \geq 0$
N = isothermal-adiabatic, $0 > T_{300}-T_7 \geq -1.8°F$
L = lapse condition, $-1.8°F > T_{300}-T_7$

Wind Direction		Wind Speed (mph) 1 - 4	5 - 8	9-13	14-19	20-27	>27	Total
205-220	I	.0053	.0058	.0086	.0017	.0005	-	.0219
	N	.0053	.0034	.0024	.0010	.0007	.0002	.0130
	L	.0017	.0005	.0012	.0007	.0002	-	.0043
225-240	I	.0053	.0043	.0067	.0007	-	-	.0170
	N	.0010	.0017	.0034	.0007	.0010	-	.0078
	L	.0005	.0005	.0014	.0007	-	-	.0031
245-260	I	.0041	.0048	.0038	.0012	.0002	-	.0141
	N	.0005	.0007	.0034	.0077	.0019	.0002	.0144
	L	.0012	.0012	.0012	.0024	.0002	-	.0062
265-280	I	.0034	.0029	.0038	.0017	.0005	.0007	.0130
	N	.0014	.0022	.0053	.0048	.0029	.0017	.0183
	L	.0012	.0002	.0022	.0017	.0007	.0002	.0060
285-300	I	.0019	.0038	.0059	.0017	.0005	-	.0129
	N	.0010	.0038	.0168	.0176	.0089	.0014	.0489
	L	.0002	-	.0048	.0093	.0037	-	.0181
305-320	I	.0022	.0038	.0031	.0019	.0002	-	.0112
	N	.0014	.0074	.0201	.0321	.0122	.0026	.0758
	L	.0017	.0007	.0086	.0127	.0062	-	.0299
325-340	I	.0029	.0026	.0034	.0005	-	-	.0094
	N	.0007	.0034	.0173	.0206	.0103	.0034	.0557
	L	.0019	.0019	.0036	.0081	.0034	.0012	.0201
345-360	I	.0026	.0060	.0024	.0012	-	-	.0122
	N	.0026	.0065	.0216	.0168	.0185	.0031	.0691
	L	.0017	.0043	.0141	.0065	.0059	.0022	.0347

(continued on next page)

TABLE 3.1(Continued)

Wind Direction		Wind Speed (mph) 1 - 4	5 - 8	9-13	14-19	20-27	>27	Total
005-020	I	.0060	.0096	.0067	.0007	-	-	.0230
	M	.0058	.0175	.0302	.0115	.0134	.0002	.0786
	L	.0024	.0060	.0079	.0038	.0012	-	.0213
025-040	I	.0031	.0050	.0026	-	-	-	.0107
	M	.0050	.0077	.0105	.0048	.0024	-	.0304
	L	.0017	.0024	.0022	.0002	.0002	-	.0067
045-060	I	.0026	.0026	.0002	.0002	-	-	.0056
	M	.0034	.0036	.0053	.0036	.0002	-	.0161
	L	.0005	.0010	.0005	-	-	-	.0020
065-140	I	.0070	.0034	.0002	-	-	-	.0106
	M	.0103	.0080	.0048	.0012	-	-	.0243
	L	.0010	.0012	.0012	-	-	-	.0034
145-160	I	.0048	.0046	.0024	.0007	.0002	-	.0127
	M	.0026	.0053	.0024	.0014	.0002	.0005	.0124
	L	.0014	.0017	.0031	.0014	-	-	.0076
165-180	I	.0091	.0144	.0122	.0019	-	-	.0376
	M	.0098	.0089	.0091	.0079	.0029	.0017	.0403
	L	.0002	.0060	.0065	.0024	.0007	-	.0158
185-200	I	.0117	.0108	.0151	.0038	.0007	-	.0421
	M	.0070	.0086	.0115	.0038	.0007	.0005	.0321
	L	.0010	.0041	.0026	.0005	.0002	-	.0084
Calm	I							.0005
	M							.0084
	L							.0115
Total	I	.0719	.0846	.0764	.0180	.0029	.0007	.2550
	M	.0580	.0854	.1639	.1349	.0762	.0155	.5454
	L	.0182	.0316	.0611	.0506	.0230	.0036	.1996

TABLE 3.2. Frequency Distribution of Wind Speed and Direction at 300 Ft. Tower Level for Summer Season (May-October) According to Temperature Gradient Class.

I = inversion, $T_{300}-T_7 \geq 0$
N = isothermal-adiabatic $0 > T_{300}-T_7 \geq -1.8°F$
L = lapse condition $-1.8°F > T_{300}-T_7$

Wind Direction		Wind Speed (mph)						
		1 - 4	5 - 8	9-13	14-19	20-27	>27	Total
205-220	I	.0074	.0122	.0053	.0005	-	-	.0254
	N	.0023	.0038	.0058	.0015	.0003	-	.0137
	L	.0066	.0048	.0033	.0005	-	-	.0152
225-240	I	.0084	.0071	.0043	.0003	-	-	.0201
	N	.0010	.0041	.0038	-	-	-	.0089
	L	.0018	.0038	.0038	.0003	-	-	.0097
245-260	I	.0064	.0056	.0031	.0003	-	-	.0154
	N	.0013	.0018	.0033	.0005	.0003	-	.0072
	L	.0028	.0028	.0038	-	-	-	.0094
265-280	I	.0036	.0025	.0025	.0008	-	-	.0094
	N	.0019	.0005	.0031	.0033	.0010	-	.0098
	L	.0024	.0048	.0048	.0005	.0003	-	.0128
285-300	I	.0043	.0028	.0048	.0008	.0008	-	.0135
	N	.0003	.0013	.0053	.0048	.0038	-	.0155
	L	.0028	.0031	.0071	.0023	.0010	-	.0163
305-320	I	.0056	.0038	.0051	.0015	.0005	-	.0165
	N	.0010	.0028	.0064	.0046	.0031	-	.0179
	L	.0033	.0041	.0061	.0043	.0036	-	.0214
325-340	I	.0069	.0076	.0028	.0010	-	-	.0183
	N	.0008	.0015	.0074	.0038	.0018	-	.0153
	L	.0036	.0025	.0092	.0023	.0013	-	.1889
345-360	I	.0076	.0186	.0140	.0010	-	-	.0412
	N	.0013	.0031	.0195	.0064	.0023	.0005	.0861
	L	.0074	.0081	.0135	.0043	.0013	-	.0346

(continued on next page)

TABLE 3.2(Continued)

Wind Direction		Wind Speed (mph) 1 - 4	5 - 8	9-13	14-19	20-27	>27	Total
005-020	I	.0094	.0244	.0196	.0005	-	-	.0539
	N	.0025	.0086	0191	.0046	.0023	-	.0371
	L	.0053	.0122	.0099	.0028	-	-	.0303
025-040	I	.0122	.0089	.0036	.0005	-	-	.0252
	N	.0074	.0064	.0094	.0033	.0003	-	.0268
	L	.0064	.0056	.0031	.0012	-	-	.0164
045-060	I	.0043	.0015	.0003	-	-	-	.0061
	N	.0023	.0043	.0043	.0005	-	-	.0014
	L	.0018	.0015	.0020	.0010	-	-	.0063
065-140	I	.0076	.0038	.0031	-	-	-	.0145
	N	.0104	.0102	.0074	.0005	-	-	.0285
	L	.0043	.0046	.0028	.0003	-	-	.0120
145-160	I	.0079	.0038	.0071	.0010	.0003	-	.0201
	N	.0033	.0056	.0058	.0056	.0025	-	.0228
	L	.0025	.0053	.0074	.0043	.0015	-	.0210
165-180	I	.0084	.0117	.0117	.0018	-	-	.0336
	N	.0043	.0059	.0145	.0081	.0008	-	.0336
	L	.0076	.0160	.0142	.0074	.0003	-	.0455
185-200	I	.0109	.0165	.0140	.0010	-	-	.0484
	N	.0041	.0069	.0107	.0076	.0010	-	.0303
	L	.0081	.0173	.0084	.0020	.0020	-	.0378
Calm	I							.0244
	N							.0061
	L							.0920
Total	I	.1109	.1310	.1012	.0109	.0015	-	.3799
	N	.0440	.0666	.1188	.0551	.0193	.0005	.3104
	L	.0666	.0966	.0994	.0336	.0112	-	.3094

TABLE 3.3 Frequency Distribution of Wind Speed and Direction at 100 Ft. Tower Level for Entire Year According to Temperature Gradient Class.

I = inversion — $T_{150}-T_7 \geq 0$
N = isothermal-adiabatic — $0 > T_{150}-T_7 \geq -0.9°F$
L = lapse condition — $-0.9°F > T_{150}-T_7$

Wind Direction		Wind Speed (mph) 1 - 4	5 - 8	9-13	14-19	20-27	>27	Total
205-220	I	.0111	.0138	.0034	-	-	-	.0283
	N	.0044	.0031	.0014	.0001	-	-	.0090
	L	.0039	.0047	.0017	.0005	-	-	.0108
225-240	I	.0108	.0061	.0016	-	-	-	.0185
	N	.0027	.0014	.0012	.0001	-	-	.0054
	L	.0034	.0041	.0024	.0001	-	-	.0100
245-260	I	.0061	.0059	.0030	.0001	-	-	.0151
	N	.0019	.0016	.0026	.0011	.0001	-	.0073
	L	.0016	.0027	.0014	.0001	-	-	.0058
265-280	I	.0056	.0060	.0042	.0009	.0004	-	.0171
	N	.0022	.0012	.0030	.0012	.0002	-	.0078
	L	.0017	.0036	.0037	.0022	.0006	.0001	.0119
285-300	I	.0035	.0050	.0076	.0041	.0017	.0005	.0224
	N	.0017	.0015	.0067	.0052	.0022	.0001	.0194
	L	.0044	.0036	.0088	.0061	.0009	-	.0238
305-320	I	.0036	.0047	.0088	.0049	.0016	.0001	.0237
	N	.0007	.0030	.0128	.0108	.0022	-	.0295
	L	.0032	.0021	.0092	.0070	.0014	-	.0229
325-340	I	.0039	.0062	.0067	.0022	-	-	.0190
	N	.0019	.0044	.0105	.0092	.0021	.0002	.0283
	L	.0035	.0051	.0075	.0022	.0007	.0001	.0191
345-360	I	.0100	.0074	.0034	.0005	.0001	.0002	.0216
	N	.0031	.0081	.0158	.0100	.0025	.0002	.0397
	L	.0056	.0132	.0120	.0030	.0007	-	.0345

(continued on next page)

TABLE 3.3(Continued)

Wind Direction		Wind Speed (mph) 1 - 4	5 - 8	9-13	14-19	20-27	>27	Total
005-020	I	.0173	.0324	.0113	.0008	-	-	.0618
	N	.0083	.0199	.0192	.0078	-	.0001	.0553
	L	.0072	.0128	.0078	.0011	-	-	.0289
025-040	I	.0176	.0303	.0095	-	-	-	.0574
	N	.0041	.0095	.0052	.0010	-	-	.0198
	L	.0046	.0051	.0017	.0002	-	-	.0116
045-060	I	.0100	.0055	.0010	-	-	-	.0165
	N	.0021	.0049	.0027	-	-	-	.0097
	L	.0024	.0031	.0014	.0004	-	-	.0073
065-140	I	.0156	.0030	.0010	-	-	-	.0196
	N	.0106	.0052	.0007	.0001	-	-	.0166
	L	.0047	.0034	.0021	-	-	-	.0102
145-160	I	.0049	.0047	.0006	.0004	-	-	.0106
	N	.0036	.0050	.0040	.0010	.0002	-	.0138
	L	.0014	.0056	.0072	.0024	.0002	-	.0168
165-180	I	.0105	.0122	.0044	-	-	-	.0271
	N	.0059	.0082	.0118	.0012	.0006	-	.0277
	L	.0071	.0138	.0128	.0026	-	-	.0363
185-200	I	.0111	.0172	.0060	.0004	-	-	.0347
	N	.0049	.0054	.0050	.0001	.0002	-	.0156
	L	.0067	.0111	.0047	.0011	-	-	.0236
Calm	I							.0188
	N							.0052
	L							.0024
Total	I	.1414	.1603	.0725	.0143	.0036	.0006	.4115
	N	.0583	.0821	.1053	.0492	.0106	.0007	.3114
	L	.0614	.0939	.0848	.0293	.0046	.0004	.2768

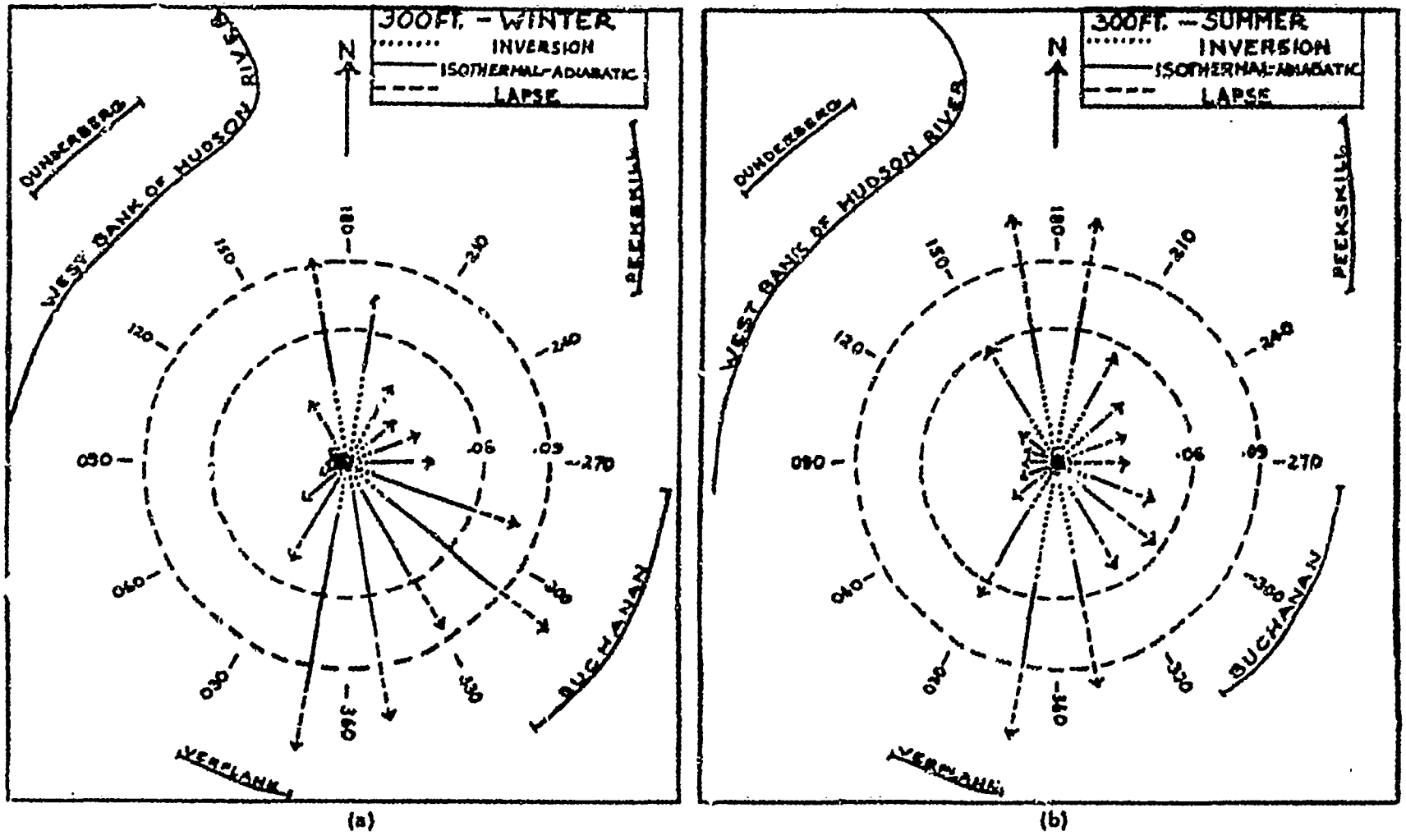
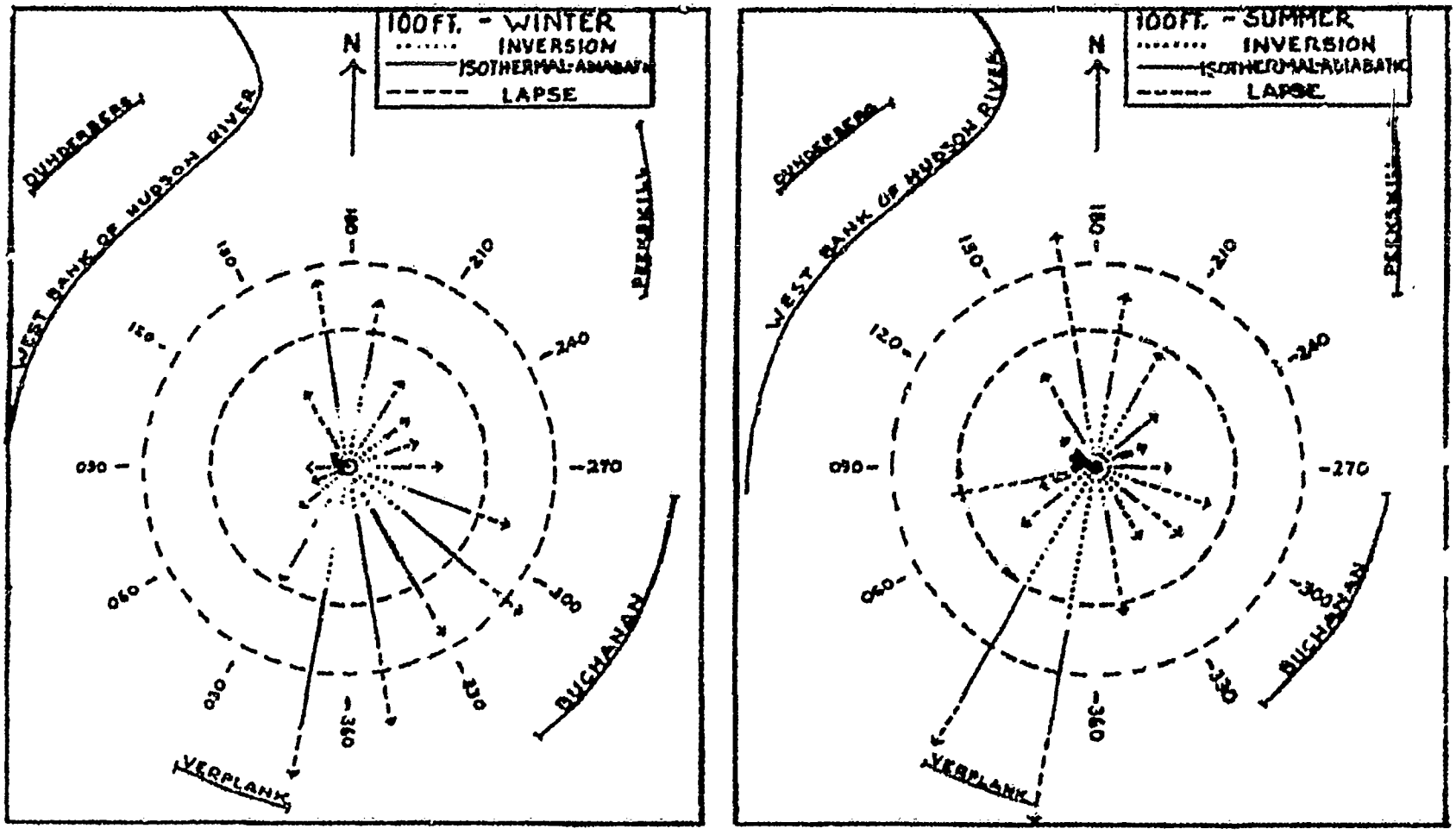


Fig. 3.1. Wind rose (300 ft) according to temperature gradient (a) winter, (b) summer.



(a) (b)

Fig. 3.2. Wind rose (100 ft) according to temperature gradient classes (a) winter, (b) summer.

flow. On occasion the 030° flow does extend to above 400 ft. (There is a lag of 2 to 4 hours in the onset of the 030 flow from 70 ft to 400 ft on these occasions.) The height to which the down valley flow extends does not appear to depend on the height or intensity of the inversion, but is probably related to very weak and unmeasurable prevailing pressure gradients.

For diffusion estimates, the following classes were defined:

L_1 - Unstable Temperature gradient, winds < 8 mph

L_2 - Unstable Temperature gradient, winds >8 mph

N - Adiabatic-Isothermal class

I - Inversion class

The percent frequency of occurrence of each of these classes follow: (300 ft data)

	Winter	Summer
L_1	5	17
L_2	15	15
N	54	30
I	26	38

A substantial portion of all hours fall into the adiabatic-isothermal class. As it happens, it is this class which is the most difficult to interpret as far as diffusion behavior is concerned. Some hours which fall into this class are characterized by relatively shallow inversions up to 50 or 100 ft with adiabatic gradients above. Under these circumstances, it is difficult to state whether or not smoke will descend to the ground. However, this will not affect the radiation calculations seriously. In view of the heated source calculations, the N class will

not enter seriously into the calculations until wind speeds of the order of 10-12 mph are attained. Study of our fluctuation traces at all levels indicate to us that smoke will descend to the ground under strong wind speed conditions.

Department of Meteorology and Oceanography

NEW YORK UNIVERSITY

COLLEGE OF ENGINEERING

RESEARCH DIVISION

SUMMARY OF CLIMATOLOGICAL DATA AT BUCHANAN, NEW YORK 1956-1957

By

Ben Davidson

Technical Report No. 372.4

Prepared for

Consolidated Edison Co. of N. Y., Inc.

March, 1958

RESEARCH DIVISION
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1956-1957

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1. Introduction

A detailed summary of climatological data collected during 1956 is contained in Technical Report No. 372.3 - Evaluation of Potential Radiation Hazard, April 1957. The tower was run on a skeleton basis during 1957. Wind observations were made at 100 and 300 feet (200 and 400 feet above river level), while temperature was observed at 7, 150, and 300 feet above ground. Because of the relative infrequency of calibration and general maintenance during 1957 the 1956 data are considered far more accurate. The 300 ft 1957 data were processed in the same manner as the 1956 data. In the present report we summarize:

(a) The effect of climatological differences between 1956 and 1957 on the radiation calculations of Report 372.3.

(b) The local wind rose as a function of height above river, and

(c) The combined 1956-1957 wind rose at 300 feet as a function of stability and wind speed.

2. Comparison of 1956-1957 data

In Table I the essential features of the 1956 and 1957 300 ft data are summarized as a function of stability class. All definitions remain the same as in the previous report. In particular, Inversion conditions (I) are defined to occur when $T_{300} - T_7 \geq 0$; Isothermal-adiabatic conditions (N) when $0 > T_{300} - T_7 \geq -1.8°F$; and Lapse conditions (L) when $T_{300} - T_7 < 1.8°F$.

Table I. Frequency of Inversion (I), Neutral (N), and Lapse (L) conditions with associated mean wind speeds, $\bar{V}$ (mph) for 1956 and 1957.

Summer	I	$\bar{V}$	N	$\bar{V}$	L	$\bar{V}$
1956	0.38	6.5	0.31	10.4	0.31	11.6
1957	0.35	6.2	0.33	12.8	0.32	9.7
Winter						
1956	0.25	7.6	0.54	12.6	0.20	8.5
1957	0.33	7.1	0.48	13.1	0.19	9.0
All seasons						
1956	0.315	6.9	0.425	11.8	0.255	10.4
1957	0.340	6.6	0.405	13.0	0.255	9.4

There are minor differences, but on the whole, the data seem compatible. There were slightly more inversion hours in 1957 than in 1956 with a slightly lower wind speed. The yearly frequency for each temperature gradient condition does not vary more than 10 percent whilst the mean wind speed for each class is also within 10 percent of the 1956 figure. Almost all of the radiation calculations are inversely proportional to the mean wind speed or to the harmonic mean. There is not too great a difference between the two years and for this reason the total integrated dosage for the area should not vary too greatly, say within 10 to 20 percent,

which is well within the range of uncertainty of the original calculations.

The areal distribution of radiation contained in Figs. 1.1 and 1,2 of the earlier report depends in the mean on the distribution of wind direction. Fig. 1 is a comparison of the annual distribution of wind direction for 1956 and 1957. Again the differences are not great; the 1957 distribution seems a bit more peaked than the 1956 data. This may be due in part to systematic individual differences in reading the charts. Whatever the cause, the differences in the distribution are well within the limits of accuracy of the initial calculations.

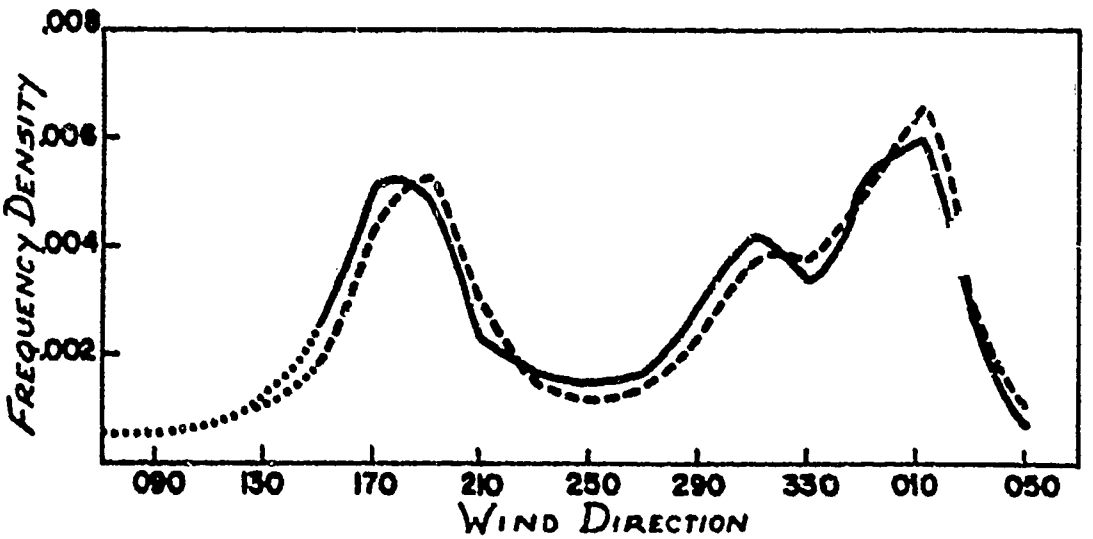


Fig. 1. Comparison of wind direction distribution for all stability classes (1956, solid line; 1957, dashed line).

3. Variation of wind direction with height

Some idea of the variation of wind direction with height may be gained from the 100 and 300 ft summer wind rose (Figs. 3.1 and 3.2 of the original report). To supplement this information, we compare in Fig. 2 the distribution of wind direction for the 1956 summer season at 400 ft (300 ft tower level), 200 ft (100 ft tower level) and 70 ft above river. The 70 ft data were obtained from an anemometer mounted on the "Jones", a ship anchored in mid-river. The ship site is about 0.8 mile northwest of the tower (see map in Report 372.1). It is evident that there are systematic differences in the three distributions. The most obvious is the build-up of southerly winds with height. The Jones distribution is flat from 150° to 250°, while the 100 and 300 ft tower

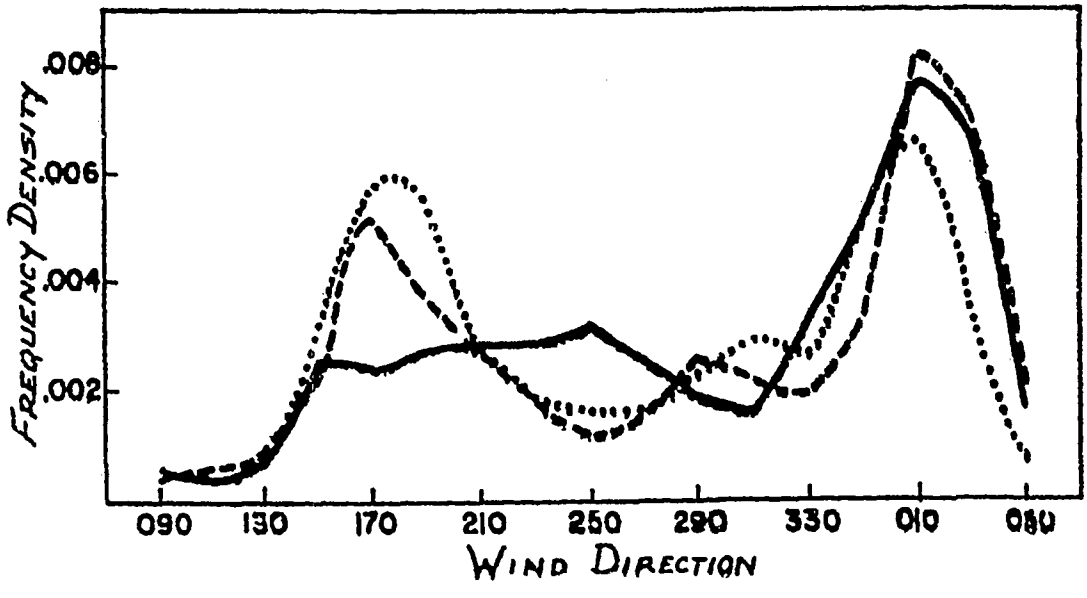


Fig. 2. Comparison of wind direction distribution for all stability classes, summer 1956. (Jones, solid line; 100 ft tower, dashed line; 300 ft tower, dotted line).

distributions peak fairly well at 170°. On the down valley side of the distribution (about 020°), The Jones and 100 ft tower level distributions are fairly well matched. The 300 ft tower level distribution does not reach nearly the same frequency at 030° as do the other two distributions. Some of the essential differences in the two distributions are summarized in the following table.

Percent time indicated wind direction ranges were observed at

Direction Range	Jones	100 ft Tower	300 ft Tower
340-040	38	37	30
360-040	28	30	19
160-320	16	23	27
160-200	10	18	22

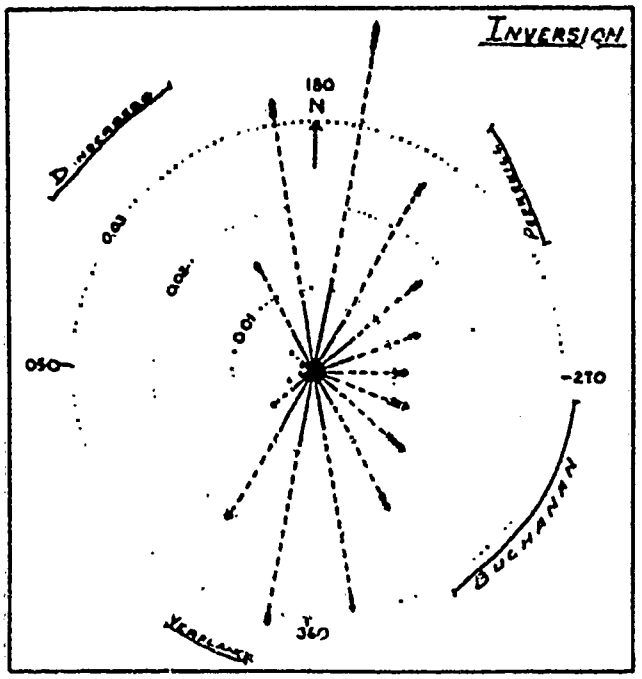
Part of the difference between the distributions can be explained by the tendency for light southerly winds to be observed at the 300 ft tower level when the nocturnal NNE winds have set in at the Jones and 100 ft tower locations. The remainder appears to be a daytime phenomenon and indicates that The Jones distribution is affected by the proximity of the valley walls in a rather complicated fashion.

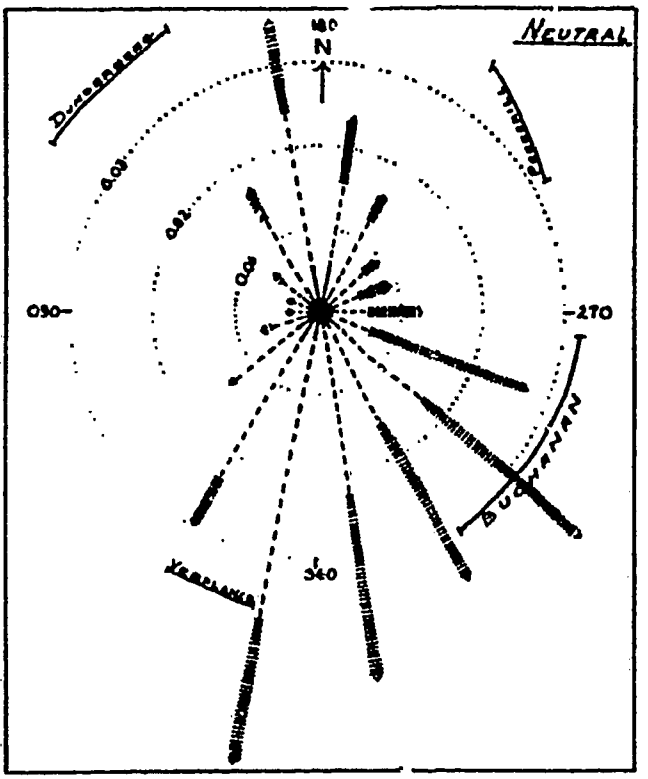
4. Wind rose presentation

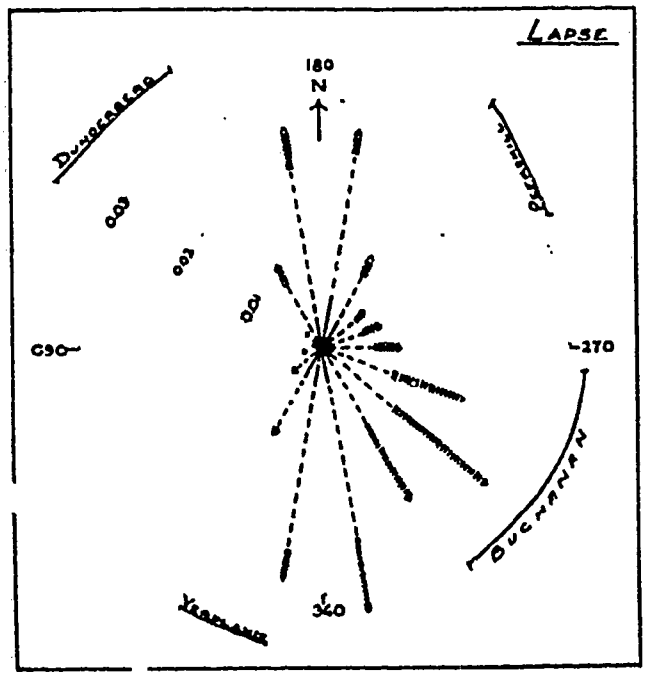
In Fig. 3 we present wind roses based upon two years of data for inversion, neutral, and lapse conditions at the 300 ft level. The bars here are flying with the wind and pointing to the indicated meteorological wind direction. The length of the bar is proportional to the average frequency of occurrence per year of the appropriate wind direction and stability condition. For convenience in interpretation we indicate the general location of populated areas surrounding the site.

An interesting feature of the wind rose is the elongation along the axis of the valley during inversion hours. Wind trajectories towards Peekskill, the most densely populated area near the site, are relatively infrequent during neutral and lapse conditions. There is a sizeable frequency of 210° winds during inversion hours. This trajectory would just about brush the northern outskirts of Peekskill, but it is probable that terrain effects would tend to curve the trajectory so that it follows the river. In general, the inversion wind rose shows a high frequency of up and down valley wind directions.

During lapse and neutral conditions, the wind rose indicates a substantial frequency of northwest winds which are the prevailing winds over flat land in this area. Under these temperature gradient conditions, one may expect effluent concentrations on the ground. There are a substantial number of wind trajectories toward the villages of Buchanan, Montrose and Verplank during neutral and lapse conditions, and towards the village of Verplank during inversion conditions.







Legend: — less than 5 mph
--- 5-13 mph
''' > 13 mph

Fig. 3. Wind rose at 300 ft tower level for inversion, neutral and lapse conditions, based upon 1956-1957 data. (Bars are flying with the wind). Calm: Inversion .026; Neutral .0107; Lapse .0052.

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Davidson, B., and J. Halitsky, 1955: A micrometeorological survey of the Buchanan, N.Y. area. - Summary of progress to 1 December 1955. Technical Report No. 372.1, Research Division, New York University, College of Engineering.

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TECHNICAL PAPER NO. 15

Maximum Station Precipitation for 1, 2, 3, 6, 12, and 24 Hours

Part X: New York

DIVISION OF HYDROLOGIC SERVICES
HYDROMETEOROLOGICAL SECTION

In cooperation with
CORPS OF ENGINEERS, U. S. ARMY

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R-12

Station: BEAR MOUNTAIN, NEW YORK

Drainage basin: HUDSON County: ORANGE

Lat. 41° 19'N Long. 74° 00'W Elev.(ft.) 1301

Period of record: 1941-1950

Month		Duration (hours)					
		1	2	3	6	12	24
Jan.	Amt.	0.38	0.60	0.85	1.29	1.49	1.51
	Date	7/1946	31/1942	31/1942	1/1945	31/1942	5-6/1949
Feb.	Amt.	0.26	0.41	0.56	0.80	1.12	1.42
	Date	14/1944	14/1944	14/1944	14-15/1944	20-21/1947	20-21/1947
Mar.	Amt.	0.35[a]	0.57	0.[illegible]	0.99	1.19	1.41
	Date	21/1948	3/1942	3/1942	3/1942	6-7/1944	2-3/1947
Apr.	Amt.	0.61	0.89	1.06	1.51	1.71	2.08
	Date	30/1947	30/1947	1/1948	1/1948	1/1948	18-19/1949
May	Amt.	0.70	1.21	1.35	1.77	2.51	2.87
	Date	0/1949	20/1949	30/1948	27/1946	27/1946	27/1946
Jun.	Amt.	0.67	0.8[illegible]	0.88	1.01	1.50	1.82
	Date	21/1945	21/1945	21/1945	23/1942	2/1946	1-2/1946
Jul.	Amt.	1.57	1.72	1.85	2.47	2.74	3.[illegible]
	Date	2[illegible]/1945	22/1946	22/1946	22-23/1945	22-23/1945	18-19/1945
Aug.	Amt.	1.2[illegible]	1.44	1.71	1.93	2.30	2.47
	Date	26/1947	16/1942	16/1942	16/1942	9/1942	24-25/1945
Sep.	Amt.	0.81	1.21	1.71	2.08	2.2[illegible]	2.80
	Date	30/1946	24/1946	24/1946	24/1946	24/1946	26-27/1942
Oct.	Amt.	0.59	0.86	1.03	1.53[b]	2.83	3.95
	Date	10/1950	26/1943	26/1942	26/1942	26-27/1943	26-27/1943
Nov.	Amt.	1.18	1.97	2.22	3.14	3.65	3.65
	Date	8/1947	8/1947	8/1947	8/1947	8/1947	8/1947
Dec.	Amt.	0.63	1.17	1.48	1.99	2.09	3.33
	Date	25/1945	25/1945	25/1945	25/1945	25-26/1945	30-31/1948
Annual	Amt.	1.57	1.97	2.22	3.14	3.65	3.98
	Date	7/20/45	11/8/47	11/8/[illegible]	11/8/47	11/8/47	7/18-19/45

[a]Also 23/1949. [b]Also 26/1943.

U. S. DEPARTMENT OF COMMERCE, WEATHER BUREAU

Station, Bear Mountain County, Orange State, New York

Latitude 41.19 Longitude, 74.00 Elevation, 1300 feet.

Data, Precipitation, Monthly and Annuals

Year.	January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Annual.
1939	3.81	3.62	3.16	5.90	1.30	5.32	3.04	3.36	3.04	4.20	1.69	3.39	41.83
1940	5.58	3.87	5.72	6.68	6.53	2.12	3.68	4.05	2.82	3.38	4.48	3.37	53.78
1941	2.77	2.67	2.22	2.00	1.79	4.[illegible]	6.31	3.33	0.25	2.35	3.18	4.47	36.00
1942	3.98	1.[illegible]	5.67	0.92	3.20	3.80	5.79	5.51	4.44	3.61	4.79	4.62	48.18
1943	2.36	1.19	2.00	3.47	4.56	3.80	3.73	2.56	2.99	12.64	4.18	± 1.01	44.49
1944	1.93	2.05	5.60	5.30	2.54	3.06	2.03	2.42	5.99	2.12	5.09	2.37	40.50
1945	2.97	2.46	1.79	3.79	7.18	4.28	16.87	4.73	5.36	2.13	6.53	4.46	62.55
1946	1.79	1.65	2.97	1.97	8.91	3.11	8.10	4.93	6.24	2.13	1.03	2.18	45.31
1947	2.85	3.39	3.48	4.76	9.49	6.55	7.38	2.78	± 1.90	2.69	8.51	3.68	57.46
1948	3.05	1.21	3.29	5.28	7.30	4.84	3.52	2.75	0.68	1.92	4.90	6.14	44.89
1949	3.08	2.27	1.88	5.47	6.53	0.96	3.45	2.94	5.60	2.52	1.91	2.79	41.40
1950	2.81	4.46	3.[illegible]	2.97	6.02	3.77	5.16	2.94	2.26	2.45	5.39	6.24	47.87
1951	4.60	4.14	8.40	2.94	4.11	3.87	5.07	5.13	2.06	5.13	2.55	5.96	59.02
1952	4.53	3.22	5.46	8.54	5.29	3.93	5.13	8.13	5.01	0.52	4.48	5.84	62.07
1953	6.75	1.89	5.71	4.72									
Sums													
Means													

REMARKS

U. S. DEPARTMENT OF COMMERCE
WEATHER BUREAU
NATIONAL WEATHER RECORDS CENTER

JOB NO. 6729

SURFACE WIND SPEEDS VERSUS
DIRECTION WHEN SOME FORM OF
PRECIPITATION IS OCCURRING
(ANNUAL AND MONTHLY)

STATION: BEAR MOUNTAIN, NEW YORK

PERIOD: JANUARY 1944 - DECEMBER 1948

Sponsored by: Consolidated Edison Company of New York, Inc.

DATE OCTOBER 28, 1965

FEDERAL BUILDING
ASHEVILLE, N.C.

Book 2 of 2

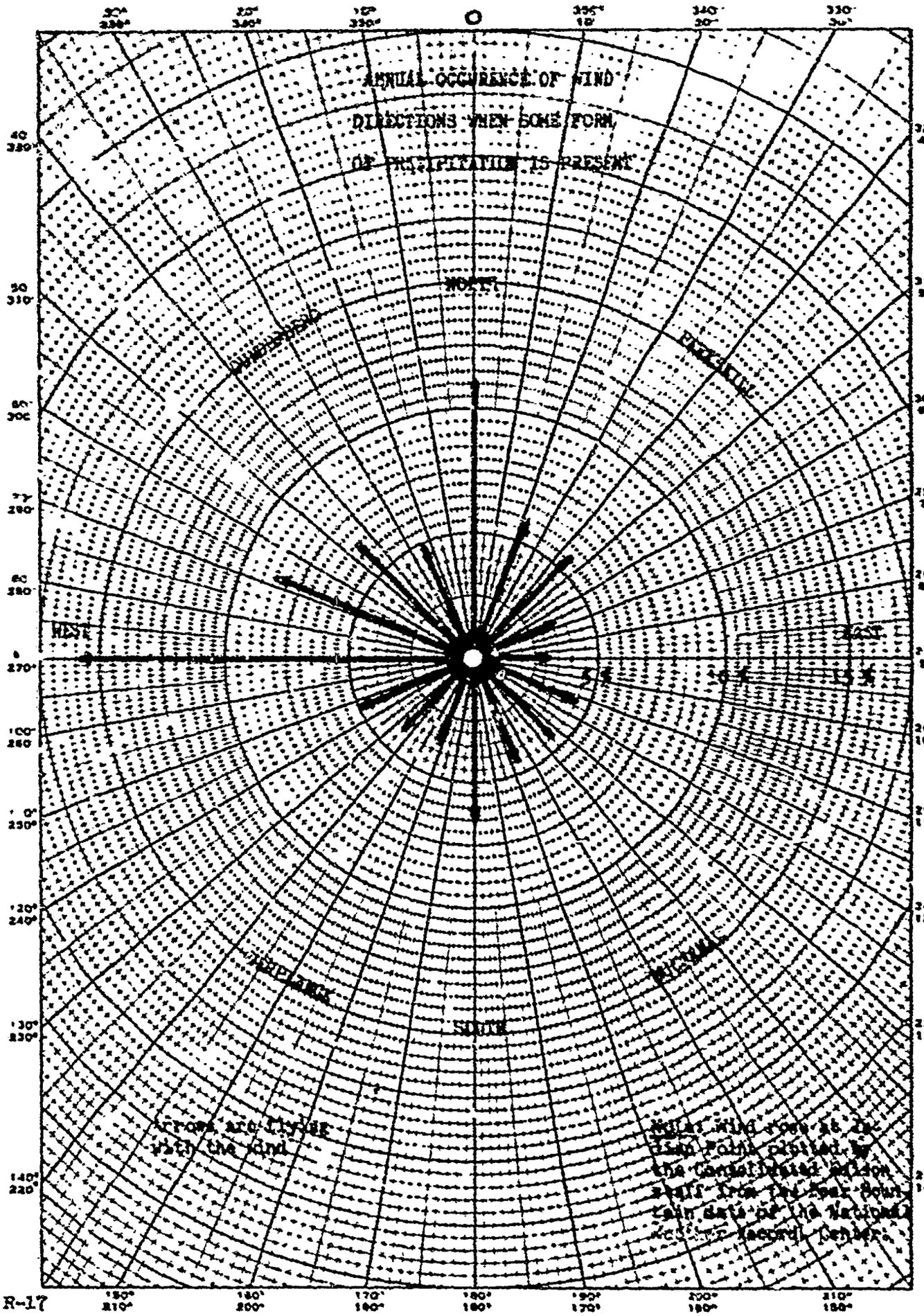
USCOMM-WB-ASHEVILLE

SURFACE WIND SPEEDS VERSUS DIRECTION
WHEN SOME FORM OF PRECIPITATION IS OCCURRING

BEAR MOUNTAIN, NEW YORK — ANNUAL — JANUARY 1944 - DECEMBER 1948

Speed / Dir.		MILES PER HOUR 1-3	4-7	8-12	13-18	19-24	25-31	32-38	39-46	47 Over		Total		
N		16	61	106	71	54	41	17	1	1		368		
NNE		4	28	58	57	24	14	0	1	4		190		
NE		7	24	72	79	29	10	0	0	0		221		
ENE		7	45	109	74	23	12	1	0	0		271		
E		17	103	314	316	101	20	1	0	0		872		
ESE		8	65	150	183	48	10	4	0	0		468		
SE		8	67	147	104	16	13	3	0	0		358		
SSE		1	30	97	82	36	16	3	4	0		269		
S		19	78	181	162	98	43	17	8	3		609		
SSW		2	27	82	86	69	40	5	11	0		322		
SW		8	31	93	97	51	27	6	2	1		316		
WSW		4	21	51	53	35	1.	8	6	1		196		
W		7	32	34	41	26	16	9	7	4		175		
WNW		4	19	45	46	66	43	2[illegible]		3		259		
NW		9	33	67	52	42	35	12	10	1		261		
NNW		15	43	65	71	49	22	5	0	0		270		
CALM	55											55		
TOTAL	55	136	707	1671	1574	767	379	116	57	18		5,480		

5,480 occurrences out of a possible 43,848.


ANNUAL OCCURENCE OF WIND
DIRECTIONS WHEN SOME FORM
OF PRECIPITATION IS PRESENT
NORTH
WEST
EAST
SOUTH
Arrows are flying
with the wind

U. S. DEPARTMENT OF COMMERCE
WEATHER BUREAU
NATIONAL WEATHER RECORDS CENTER

JOB NO. 6729

OCCURRENCE OF WIND SPEED
AND DIRECTION DURING THUNDERSTORMS
(ANNUAL ONLY)

STATION: BEAR MOUNTAIN, NEW YORK

PERIOD: JANUARY 1944 - DECEMBER 1948

Sponsored by: Consolidated Edison Company of New York, Inc.

DATE OCTOBER 28, 1965

FEDERAL BUILDING
ASHEVILLE, N.C.

Book 1 of 2

USCOMM-WB-ASHEVILLE

JOB NO. 6729

OCCURRENCE OF WIND SPEED AND DIRECTION DURING THUNDERSTORMS

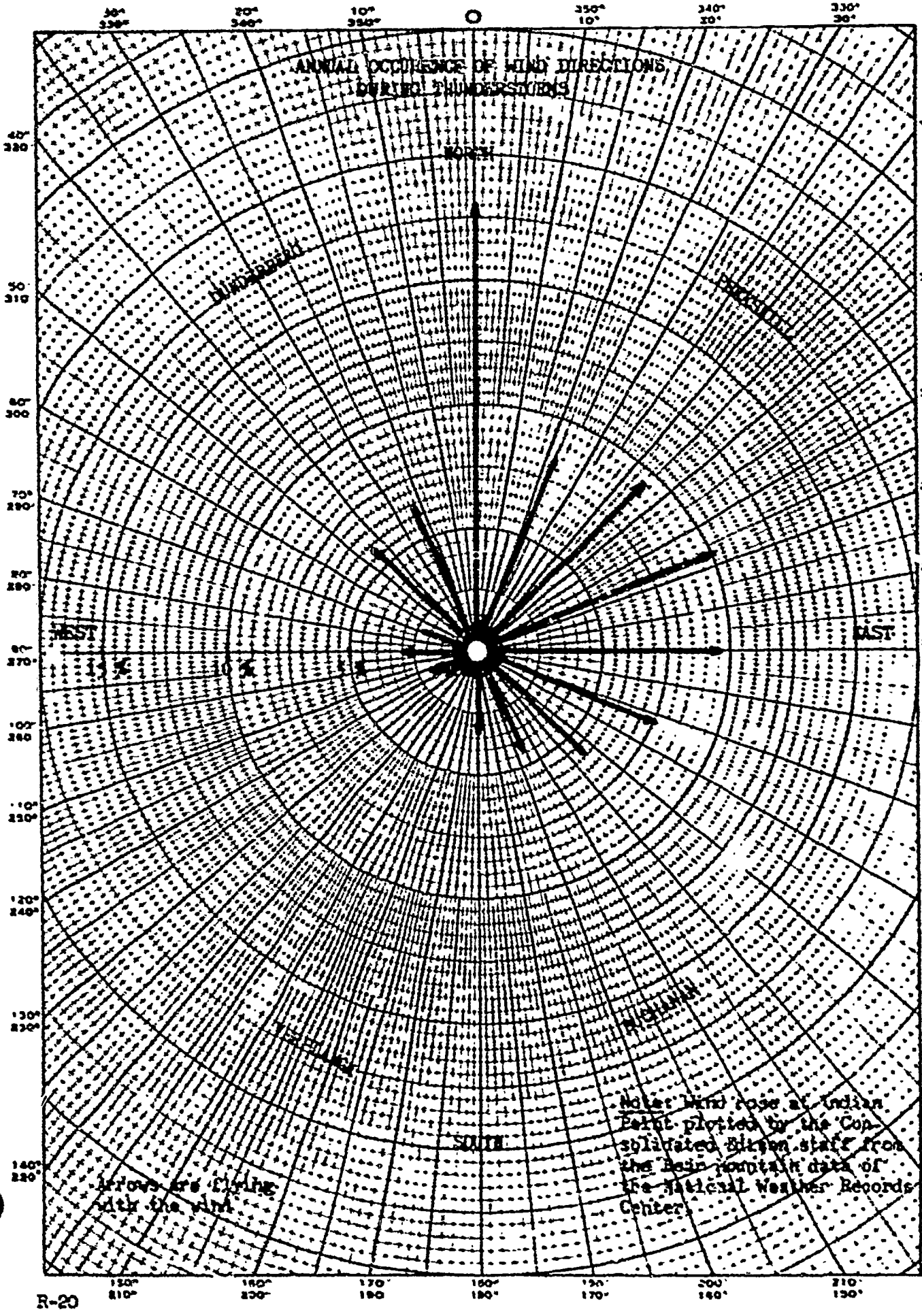
BEAR MOUNTAIN, NEW YORK

ANNUAL 1944 - 1948

MILES PER HOUR

Speed / Dir		1-3	4-7	8-12	13-18	19-24	25-31	32-38	39-46	47 Over		Total		
N		2	5	10	6	2	1	1				27		
NNE			2	2	4	1						9		
NE		2	1	3	4	2	1					13		
ENE			3	7	4	2						16		
E			8	10	7							25		
ESE			5	11	3							19		
SE		2	10	16	16	3						47		
SSE			1	26	19	5						51		
S		4	24	57	47	11	1					144		
SSW			4	23	26	12	1	1				67		
SW		3	6	22	24	11	4	1	1			72		
WSW		1	8	12	34	15	5	4		1		80		
W		3	11	12	27	10	6	3	1	4		77		
WNW			3	18	11	10	12	2	2	3		61		
NW			6	17	4	9	6	2	2	1		47		
NNW			5	12	4	8	1	2	2			34		
CALM	7											7		
TOTAL	7	17	102	258	240	101	38	16	8	9		796		

There were no thunderstorms observed for the months November, December, January & February during this period (1944-1948).


ANNUAL OCCURRENCE OF WIND DIRECTIONS
DURING THUNDERSTORMS
NORTH
NORTHEAST
EAST
SOUTHEAST
SOUTH
SOUTHWEST
WEST
NORTHWEST
15 %
10 %
5 %
Arrows are flying with the wind.
Water wind rose at Indian Point plotted by the Consolidated Edison staff from the Basin Mountain data of the National Weather Records Center.

2.7 GEOLOGY

The geological characteristics at the location of Unit No. 2 are practically identical to the existing Unit No. 1 and proposed Unit No. 3. The Indian Point site consists of a fine-grained phyllite, a schist, and limestone with bedrock lying very close to the surface. Unit No. 2 will be located on limestone, which is hard although jointed. The consulting geologist has stated that the bedrock is sufficiently sound to support any loads which may be anitcipated up to 50 tons per square foot. This far exceeds any foundation load that will be superimposed on the site. The limestone is not cavernous. The intensely jointed condition of the limestone shows that residual stresses within the rock formation have been dissipated, almost eliminating the possibility of local changes in rock formation. The jointed limestone formation is very permeable and all ground water will flow from the hills toward the river making it impossible for drainage from the plant to go anywhere except the river.

The Indian Point site and surrounding area were studied in 1955 by Sidney Paige, Consulting Geologist, prior to construction of Unit No. 1. His report is included in this section. In 1965, Thomas W. Fluhr, P. E. and Engineering Geologist, reviewed the geology of the site and made additional borings at the location of Unit No. 2. Also included is his report, which verifies Mr. Paige's conclusions and confirms that the geologic features of Unit No. 2 are the same as Unit No. 1.

MEMORANDUM

ON

GEOLOGIC FEATURES

OF

INDIAN POINT NUCLEAR POWER PLANT SITE

by

Thomas W. Fluhr, P.E.
Engineering Geologist

Consolidated Edison Company of New York, Inc.

New York, N. Y.

October 18, 1965

INTRODUCTION

The Consolidated Edison Company owns a large tract of land in the Village of Buchanan, N. Y. An electric generating station powered by nuclear energy occupies part of the tract.

It is proposed to install a second nuclear-powered unit adjacent to the existing one.

The site has been inspected, records of previous subsurface investigations studied, and three additional test borings are now being made.

The geologic features of the site make it well-adapted to construction of either a conventional or nuclear-powered plant.

DISCUSSION

The site is located on the east bank of the Hudson River about 35 miles north of New York City. It lies close to good highways and a railroad. It can also be served by water transportation. A natural-gas transmission line crosses the property.

The proposed second unit will be located immediately adjacent to the existing Unit No. 1, and to the northeast of it. The topographic and geologic features of the two unit sites are virtually identical.

The Topography.

Along this part of the shore of the Hudson River,

rocky cliffs rise from the river to elevations 80-90 feet above mean sea level. At the site proper, there is a break in the cliffs, forming a basin in which the existing Unit No. 1 is located. To the northeast, the basin is partially rimmed by hills and ridges, reaching to approximate elevation 80-90. To the east and south, the basin is bounded by similar hills and ridges extending to elevations 100-140. A half-mile to the northeast, the topography becomes rugged, with numerous small steep hills and ridges reaching approximate elevations 70-100. The rugged and irregular topography lies east of the highway known as Broadway, and marks a different geologic province from the remainder.

The proposed site for Unit No. 2 lies to the northeast of existing Unit No. 1. At this place the ground is level, about 60 feet above sea level, and is covered with fill. It looks as if it has been leveled artificially. Bedrock lies very close to the existing surface.

The Geology.

The property as a whole is underlain by three geologic rock formations. They are:

(a) A Phyllite or Schist. This is the uppermost formation of sedimentary origin. In some places the rock is a fine-grained phyllite resembling the phyllite of the Hudson River series. In other places it is a muscovite schist resembling the Manhattan schist found

typically in New York City.

(b) Beneath the phyllite or schist, and apparently conformable with it, is a limestone. Most of this is coarsely crystalline white or gray limestone, with some dolomitic and silicious bands, and a few quartz veins. Its original bedding structure has been obscured in part by shearing and jointing.

(c) The schist, in the easterly part of the tract, about a half-mile east of the Hudson River, has been intruded by basic igneous rocks, known as the Cortlandt Series.

The geologic age of the limestone and phyllite or schist has been a subject of dispute among geologists for nearly a hundred years. Some class the formations as Cambro-Ordovician; others as pre-Cambrian. For present purposes the geologic age is immaterial.

The entire tract was not studied, but outcrops were mapped in the area immediately surrounding the power unit sites, and a geologic map prepared, a print of which is attached.

The geologic map shows that existing Unit No. 1 and proposed Unit No. 2 will be located within the limestone belt.

Foundation Conditions.

The top of bedrock is high over most of the site,

dropping off rapidly in elevation on approach to the river margin.

The limestone has a well-defined layered structure, believed to be original bedding, which strikes N-S to N E and dips easterly at 45 to 65 degrees. This layered structure is marked by shear planes and, rarely, thin shaly layers.

The notable feature of the limestone is its extremely jointed condition. A major joint system extends at about right angles to the bedding structure, but, in addition, there are also many irregular joints. The jointing has an intensity which might almost be described as brecciation. The joints are open, but few display decay. This limestone formation is not cavernous.

The limestone is hard, because of its jointed condition, it is my opinion that its supporting value for foundation purposes should be held to no more than 50 tons per square foot.

The rock cuts existing on both sides of Unit No. 1 show that the rock can be excavated on steep slopes, and that it will remain unsupported without excessive weathering or raveling. A rock slope of 1 horizontal to 4 vertical is judged to be reasonable.

Possible Contamination.

In the case of nuclear plants, a question is sometimes

raised in regard to the possibility of contaminating water supplies if a nuclear accident should occur.

Nuclear Unit No. 2 will be situated very close to the Hudson River. It will be located on a bed of jointed limestone which is very permeable. When test borings are made into the limestone there is no return of drill water; it all runs away into the limestone.

The two nuclear units will lie in a basin ringed to north and east by hills. In the hills the ground water table is at a high elevation. The ground water will flow from the hills toward the basin, and into the river. There is no possibility of out flow from the plant, which is at a low elevation, working against the flow of underground water toward the river.

North and east of the plant are hills and ridges of phyllite and schist. These rock formations are very impermeable. East of Broadway, the schist has been intruded by a still more impermeable igneous rock formation, the Cortlandt Series.

All these factors make it an impossibility for any drainage from the plant to go anywhere except into the Hudson River. No problem of contamination of water supplies exists.

Rock Stress and Earthquake Hazard.

The highly folded and jointed character of the lime-

stone indicates that, at one time, it was subjected to high stress of orogenic origin. The fact that it now is intensely jointed shows that any such stresses have been dissipated. Additional stresses cannot accumulate in a rock formation as jointed as this one.

Earthquake shocks have been reported in Westchester County in 1872, 1874, 1878, 1916, and 1926, but none have done appreciable damage. This region is not noted for seismic activity. It appears to be one of the more stable areas of the Earth's crust. The possibility of earthquake damage to this plant is regarded as remote.

FINAL STATEMENT

The nuclear reactor with its containment vessel, the heat-exchangers, and the condensers and generating units will be founded on bedrock. The bedrock is sufficiently sound to support any loads which may be anticipated.

In my opinion, no hazard to existing water supplies from nuclear accident is possible.

It is my opinion that no unrelieved stresses exist in this rock formation, which might offer a hazard.

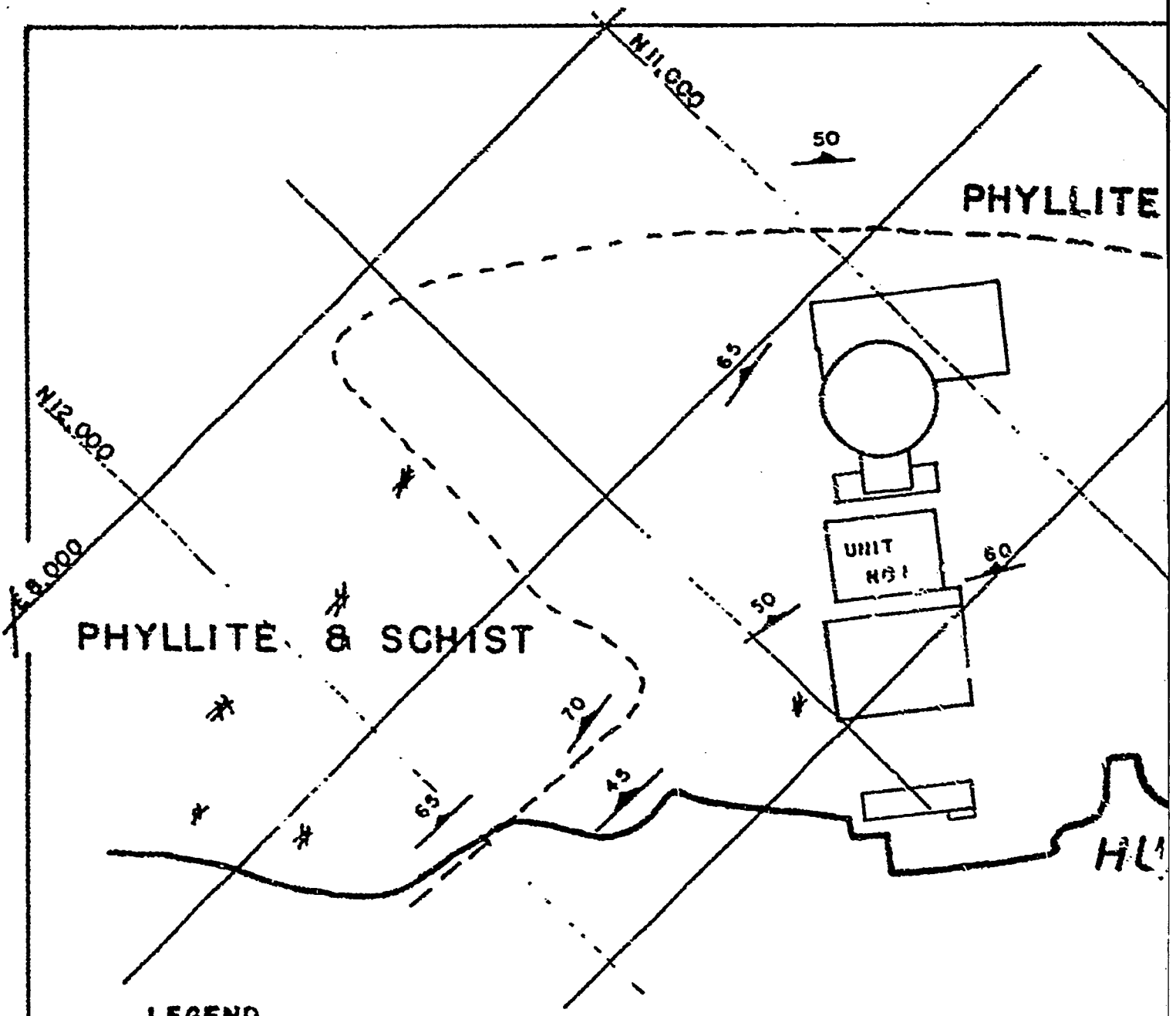
The record of earthquake tremors in this region renders the possibility of damage from earthquakes very remote.

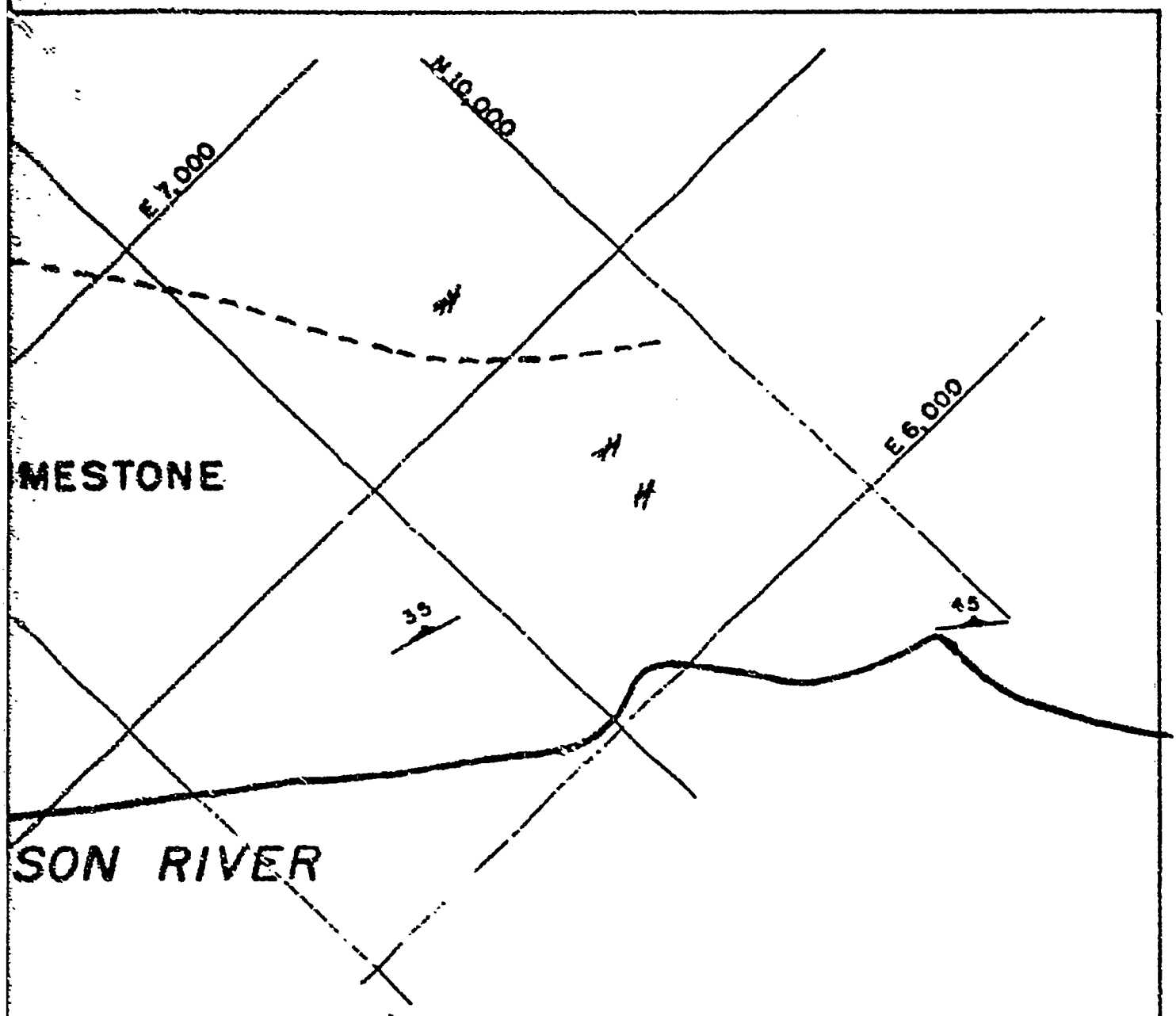
There are no geologic faults of magnitude extending

through the site nor close to it.

In view of these considerations, I regard the site as being well-adapted for construction of the proposed second nuclear-powered generating unit.

Thomas W. Fluhr P.E.
Engineer/Geologist


N11,000
50
PHYLLITE
63
N12,000
E9,000
UNIT
HG1
60
50
PHYLLITE & SCHIST
70
63
43
HU
LEGEND
45
OUTCROP SHOWING STRIKE & DIP
OUTCROP-NOT MEASURED
BASED ON CON ED DWG. 159920
Thomas W. Fluker, P.E.
ENGINEERING GEOLOGIST.



CONSOLIDATED EDISON CO. OF NEW YORK, INC.

DIAN POINT NUCLEAR POWER PLANT SITE

GEOLOGIC MAP

OCTOBER 16, 1965

A GEOLOGIC REPORT

on the

INDIAN POINT POWER HOUSE SITE

of the

CONSOLIDATED EDISON COMPANY, INC.

NEW YORK, NEW YORK

Sidney Paige,

Consulting Geologist

INTRODUCTION

The proposed Indian Point Power house site of the Consolidated Edison Company, Inc. of New York City is located approximately two miles southwest of the city of Peekskill, Westchester County, New York, on the east bank of the Hudson River. The site is part of a tract of 348 acres owned by the Consolidated Edison Company. (See figure 1 in folder).

The report that follows is divided into three parts:

I Regional geology and physiography
II Geology surrounding the site
III Engineering considerations at the site

REGIONAL GEOLOGY AND PHYSIOGRAPHY

The region we propose to briefly describe as a setting for the more detailed account of the geology surrounding the Power house site, embraces a number of physiographic provinces, namely, (1) the Atlantic Coastal Plain, with its submerged portion the continental shelf, (2) the Piedmont Plateau, (3) the New Jersey Highlands, (4) the Triassic Lowlands of New Jersey. (See physiography of the Eastern United States, Nevin M. Fenneman, McGraw-Hill Book Company, Inc., New York, 1938).

The Atlantic Coastal Plain

The Atlantic Coastal Plain is well developed along the New Jersey coast. It owes its plain-like features in part to the gently seaward dipping Cretaceous and Tertiary sandstones, shales and marls that underlie its surface; and to the fact that these beds have been only recently raised above the sea and eroded to a gently seaward dipping plain. This plain bevels the outcropping edges of successive sedimentary strata. North of the latitude of Long Island the plain is submerged beneath the waters of the Atlantic Ocean, and becomes a part of the Continental Shelf that borders the entire eastern United States from Maine to Florida.

We are not as much concerned with the sedimentary rocks of the Atlantic Coastal Plain Province, since these are not present in our immediate area of interest, as we are with the surfaces of erosion (pene-plains) that lie beneath them, and on which they were originally deposited. This relationship will be referred to later.

The Piedmont Plateau Province

The Piedmont Plateau Province, meaning the plateau at the foot of the mountains, extends from the inner boundary of the Coastal Plain at the "fall line" to the foot of the mountains, which rise sharply above it at most places along the atlantic seaboard. In the south these mountains are known as the Blue Ridge Mountains. To the

north and in New Jersey they are known as the Reading Prong of the New England Province, or more commonly as the New Jersey Highlands.

The Piedmont Plateau is everywhere an uneven surface of erosion, a surface cut for the most part across complexly folded and faulted rocks of both pre-Cambrian and post-Cambrian age, sedimentary and igneous rocks alike. A striking characteristic of these rocks is the high degree of metamorphism which they generally exhibit. This is a result of their long and complex history which included deep burial, intense compression and deformation, and the emplacement of numerous masses of granite, diorite, and even more basic representatives of such intrusives. The structural and stratigraphic complexities of these rocks, and the attempts, successful or otherwise, to unravel their long history have given rise to many current controversies. With one of these we will be concerned below, in assigning an age to the rock beneath the Power house site which lies within the Piedmont Province, near the southern border of the New Jersey Highlands. Geologic reports covering the Piedmont of the Atlantic seaboard are voluminous.

There is a notable exception to the statement that the Piedmont Plateau is everywhere underlain by metamorphic rocks and their related plutonic masses. At a number of places, thick sequences of Triassic sediments have been down-faulted into the underlying pre-Cambrian basement and thus have been

preserved from erosion. Such an area is that underlain by the Newark Series of New Jersey. This assemblage of conglomerates, sandstones and shales with their intercalated beds of basaltic lava, and the well known intrusive basaltic sill of the "Palisades", constitutes the Piedmont Plateau in a large area west of the Hudson River. The northern tip of this basin of deposition very closely approaches our area of interest on the opposite side of the river near Stony Point. (See figure 2).

The Blue Ridge-Reading Prong (New Jersey Highlands) Province

The Blue Ridge-Reading Prong comprises a number of sub-parallel mountain ridges extending from Georgia to northern New England where the Taconic and Green Mountains may be regarded as their most northern extension. This belt of mountains is actually the eastern border of the next physiographic province to the west, the "Ridge and Valley Province" of the Appalachian Highlands. With this latter province we are only slightly concerned. It includes in part, the lowland northwest of the New Jersey Highlands. In it are rocks generally quite free from appreciable metamorphism. This characteristic will be referred to again in discussing the geology of our immediate area of interest.

That part of the Blue Ridge-New Jersey Highlands Province which concerns us is the New Jersey Highlands portion adjacent to the Hudson River. Here the northeast

trending ridges are underlain by complexly folded granitoid gneisses and schists, intricately involved with grano-dioritic intrusives. Prevailing dips in this entire region are steep towards the southeast. Because of forest and grass cover, and the presence of glacial deposits of sand, gravel and clay, it seems doubtful whether details of geologic structure and statigraphy will ever be accurately resolved and mapped. The bulk of the Highland rocks are pre-Cambrian in age, but those with which we are particularly concerned are in-faulted and in-folded strata of Cambro-Ordovician age.

With this brief introduction, we proceed to a more detailed description of the geology more closely associated with the Power house site area.

GEOLOGY OF THE AREA SURROUNDING THE POWER HOUSE SITE

The late Professor Charles P. Berkey of Columbia University, published the only detailed geologic report and map of the area in which the Power house site is located. We refer to a bulletin of the Albany, New York State Museum, entitled "Geology of the West Point Quadrangle, New York". It appeared in 1919. Since that time, although much work has been done in the surrounding area, and many controversies have arisen regarding the age of the rocks and their detailed structure, this is still the only paper embracing this region in such detail.

The report contains a geologic map on a scale of I : 62500. In a northwest-southeast direction the map embraces the entire width of the New Jersey Highlands, some 17 miles. In fact, the main body of the map lies within the New Jersey Highlands. Its generally mountainous character is quite evident. Only in the southern part of the area, in an irregular east-west strip roughly four miles wide does the Piedmont Plateau appear within the mapped area. It is with this southern part of the area for the most part that we are concerned. Figure 2 is a generalized redrawing of this southern portion of Berkey's geologic map, with a small addition at the south to include Stony Point. The map shows the principal formation boundaries, sedimentary and igneous, appearing on Berkey's map, and also includes the principal fault lines as

determined by Berkey. We have added structural axes of folding to the map, and our correlation of the rocks east and west of the river is different from that of Berkey's.

Of particular interest on Berkey's original map are the sedimentary sequences designated Cambro-Ordovician in age, as contrasted with those other sedimentary sequences designated as of "doubtful" age, and a third sequence designated as of pre-Cambrian age (Grenville). We are concerned here only with the first two of these sequences. Most Geologists who have studied this area, including Dr. Berkey, are agreed that the sedimentary sequence of Poughquag Quartzite, 600' thick; Wappinger Limestone about 2000' thick; and Hudson River Phyllite 2000' or more thick; outcropping just north of Peekskill is of Cambro-Ordovician age. They believe them to be a metamorphosed phase of the quartzite, dolomite, and shales that lie to the north and west beyond the Highlands, where they appear as normal unmetamorphosed strata only slightly deformed.

The strata at Tomkins Cove, west of the river, well exposed in the large quarry at the New York Trap Rock Company, were likewise assigned a Cambro-Ordovician age. On the other hand, the sequence of dolomites and schists appearing south and southeast of Peekskill and at Indian Point on the east side of the river were correlated with the Inwood Limestone and Manhattan Schist of Manhattan Island. On Berkey's map their age is designated "doubtful".

Thus the dolomites on the east and west sides of the river well exposed in huge quarries, were regarded as of possibly very different age.

It is important to record here that the geologic map of New York State, published in 1901, and described in bulletin 56 entitled State Geologic Map 1901 by F.J.H. Merrill, assigns a Paleozoic age to the dolomites on the east and west sides of the river; in fact to all the dolomites of the Peekskill region.[1.]

The observations that follow, support the viewpoint of those earlier workers; that the Inwood Limestone and Manhattan Schists are the metamorphosed equivalent of the Wappinger Limestone and Hudson River Shale.

The quarry at Tomkins Cove is a roughly quadrangular opening 3000' or so in length, and some 1500' wide. The excavation is cut on several benches to a depth of several hundred feet. The dolomites are for the most part thin to medium bedded. They strike on the average N 35° E , and dip steeply to the southeast. Dips range from vertical to 70° southeast, or at somewhat lower angles on the east side of closely appressed folds. Beds vary in color from creamy white, through gray and blue; no attempt was made, however, to map distinct strata.

The west wall of the quarry is limited by phyllites or mica schists representing the Hudson River Shales. These schists likewise dip about 70° southeastward beneath the dolomites, and since they are stratigraphically higher

1. Paper entitled "Structural and Petrographic Studies in Dutchess County, N.Y. By - Robert Balk
Geological Society of America 1936 Vol 47 pp 685-774

than the dolomites, their relationship indicates that the beds here are overturned to the northwest. (See cross-section A-B on figure 2).

The notable structural features observed in the quarry are the prevailing steep southeastward dip, the occurrence of closely appressed nearly isoclinal folds along the west wall, and along the southern face, and the fact that dips are steepest on the western limb of small folds. All of this indicates overturning toward the northwest. Thus observations strongly indicate that the dolomites, originally laid down nearly horizontally, now appear in a tightly compressed fold overturned to the northwest.

The dolomites are succeeded to the east by phyllites or mica schists. These may be observed in the railroad cut at Stony Point. Beyond these schists in the railroad cut, there appears to the east intrusive igneous rock of the Cortlandt Complex.

To repeat, one cannot escape the conclusion, therefore, that Wappinger dolomite at this place is metamorphosed, thrown into a closely appressed over-turned anticline striking about N 35° E , and appears to plunge northeastward beneath the Hudson River.

About the center of the southern wall of the quarry, the dolomites are so intricately fractured that they are unsuitable as quarry rock. We suggest that this condition was caused by the violent and intense deformation of the dolomites near the axis of the over-turned closely folded anticline.

The eastern quarry, on the east bank of the Hudson north of Verplanck is somewhat smaller than the quarry at Tomkins Cove. Here, the northern wall of the quarry is a shear cliff in which the structure of the dolomites is clearly set forth. The quarry now contains a lake, but the use of a boat facilitated a close examination of all the walls of the quarry. In the north wall, beds of dolomite dip steeply southeast, about 70°. No over-turned folds were observed in this face. Along the southern face of the quarry, however, deformed beds may be seen accompanied by small faults. The heavy shooting of these banks necessary to supply rock for processing, so fractured the rock that the original joint patterns of these rocks is almost completely obscured.

The eastern face of the quarry is limited by the appearance of mica schists dipping steeply southeastward and striking with the long extension of the quarry, N 35° E; a strike similar to that in the quarry at Tomkins Cove. This strike in the dolomites of the eastern quarry would, if minor folding is excluded, continue across the river southwest directly into the Tomkins Cove quarry.

It thus appears to the writer as virtually established that the dolomites of the two quarries are identical, as are the phyllites or mica schists which stratigraphically overlie them.

The suggestion might be offered by someone that a north-south trending fault beneath the water of the Hudson

separates these dolomites, and that they are of different ages. This view is opposed by the results of geophysical work in connection with the construction of the Nyack Bridge. It was discovered there, that Triassic strata (Newark Series) crosses the river in unbroken beds dipping 6° westward. (Personal communication--J.L.Worzel, Columbia University)

One of the considerations that in the past has confused the issue and made it difficult to correlate the dolomites and schists on the opposite sides of the river, and to correlate the beds that appear north and south of Peekskill, has been the different degrees of metamorphism displayed by these sequences. The writer feels that there is no valid basis for this confusion. The age of rocks should rarely, if ever, be determained by the degree of metamorphism they display. Degree of metamorphism is a result of factors independent of the age of the rock, to wit: depth of burial, crustal deformation, and the presence or absence of large intrusive bodies.

It may be shown here that these considerations can explain all differences between the dolomites and the phyllites north of Peekskill, generally accepted as of Cambro-Ordovician age, and the crystalline dolomites and mica schists south of Peekskill, by some thought of as pre-Cambrian in age, and designated in Berkey's report as Inwood Limestone and Manhattan Schist of "doubtful" age.

To clarify this discussion, figure 2 has been prepared. On it are the boundaries of the sedimentary sequences, the

igneous rock of the Cortlandt Complex, and the faults which were mapped by Berkey. Details of the granitoid gneisses and granites are omitted. There have been added, lines indicating the position of structural axes of two folds, and the hypothetical outcrop of boundaries between schist and dolomite beneath the river. A dotted boundary of the Cortlandt Complex beneath the Hudson is also drawn to clarify the picture.

Two diagramatic cross-sections are offered. The one from Tomkins Cove eastward across the river through the eastern quarry to the Cortlandt Complex; the other from the Cambro-Ordovician strata north of Peekskill, southeast across a ridge of granitoid gneiss to and across Manhattan Schist(?) to the Cortlandt Complex. The strong faulting that affects the folding is symbolized in these cross-sections.

ENGINEERING CONSIDERATIONS AT THE SITE

The site of the Power house is shown in figure 1 and figure 2, the first a map of the property on a scale of 1" = 200', on which accurate positions and elevations can be readily determained. The foundations of the structure will be cut into the low hillside bordering the river to meet design requirements.

Alluvial or glacial overburden at the site is shallow, hence dolomite lies very near the surface. Depth to bedrock is shown on figure 3. Bedrock is a dolomite (Cambro-Ordovician in age, in the opinion of the writer) disposed in steeply dipping beds inclined generally 70° to the southeast. The beds strike about 35° to the northeast and are a continuation towards the northeast of the beds well exposed in the eastern quarry of the New York Trap Rock Company north of Verplanck. Structurally the site lies on the eastern limb of an overturned anticline, very near to the intrusive contact of the Cortlandt Complex.

Four diamond drill holes have, at this writing, been drilled as shown in figure 3 (in pocket). "Thin sections" have been cut from the drill cores, one near the top of each hole, and one near the bottom. Examination of these sections with a petrographic microscope gave the results recorded below.

DESCRIPTIONS OF THIN SECTIONS

K Tomkins Cove quarry--near southwest corner

Fine grained dark gray dolomite with thin bedding. Breakage seems due to ordinary compaction of sediments rather than tectonic folding, as jointing is perpendicular to bedding, and normally gives cubic blocks. Grains are slightly rounded, approximately .01 cm. in diameter, and do not have sutured contacts. This indicates that recrystallization has not occurred. The general dusty appearance of the section suggests clays and other impurities such as carbon.

N-1 Tomkins Cove quarry--west wall

Grains average .015 cm. in diameter, with sutured contacts. The rock is a fine grained dolomite or marble. Sutured contacts along with a few shards of muscovite or sericite indicate recrystallization of the rock.

N-2 Tomkins Cove quarry--west wall

Blue gray dolomite, fractured and rehealed by both calcite and cold water quartz. Breakage here is of a tectonic nature rather than jointing. Fractures bear no angular relationship to one another. The rock appears similar to specimen K except for the fracturing and rehealing.

N-3 Tomkins Cove quarry--west wall

Finely bedded blue gray dolomite with good bedding plane cleavage, and jointing normal to this plane. These joints have been filled by calcite. The impression is, that this is the same rock as specimen K, with recrystallization just beginning; perhaps 20% of the rock has interlocking crystal boundaries. Occasional grains of quartz make up 0.5% of the slide.

Drill Hole G-8 9'-14'

Dolomite or marble, blue gray with a slight suggestion of faint bedding. The rock is completely recrystallized with sutured contacts between grains. Quartz, muscovite, and a bit of potash feldspar make up perhaps 2% of the slide; an indication of the recrystallization of original argillaceous material. Grain size averages approximately .02 cm. for the calcite grains.

Drill Hole G-8 85'-90'

Blue gray dolomite or marble as specimen above. This slide shows more fracturing with indication of solution along the breaks, and partial rehealing. Specimen is recrystallized with minor quartz, potash feldspar, and muscovite as before. Grain size averages .02 cm.

Drill Hole H-8 20'-25'

Light colored recrystallized dolomite with fractures showing solution and iron staining. Grain size averages .03 to .04 cm. in diameter. Quartz, potash feldspar, and muscovite are again present to an extent of about 1%. Reflected light indicates that at least part of the small opaque matter is pyrite.

Drill Hole H-8 56'-61'

Light colored recrystallized dolomite as above, with fewer fractures but still slight traces of solution and iron staining. Grain size remains about .03 to .04 cm. in diameter, and quartz, potash feldspar, and muscovite make up 2% of the slide. Reflected light again shows opaque matter to be pyrite.

Drill Hole H-8 81-86'

Blue gray dolomite with quartz filling joint plane 1/8 inch wide, partly open forming small vugs. Rock is completely recrystallized, with interlocking grains. Quartz, potash feldspar, and muscovite again make up 2% of the rock, with less than 0.5% opaque pyrite. Grain size averages about .02 cm. in diameter.

These thin sections indicate that the primary difference between the rocks on the east and west sides of the river is grain size, due to recrystallization of the calcite. The evident higher grade metamorphism of the dolomites on the east as contrasted with those on the west, may reasonably be explained by the nearby presence of the massive basic intrusive of the Cortlandt Complex.

The bedrock is fractured as are most rocks near the surface. Such fractures generally decrease as depth is attained. The percentage of recovery of core in the diamond drill holes, and the fracturing of the core indicate the prevailing conditions. Hole G-6 drilled to elevation -21', and G-8 drilled to elevation -16' have good recovery of core. G-10 drilled to elevation -54' has a generally poor recovery. H-8 drilled to elevation -28' had satisfactory recovery. Practically all the core recovered indicates the presence of hard dolomite, fractured to varying degrees.

The ground water-table as recorded by the diamond drill holes stood at elevation 55' in hole G-6; elevation 47' in hole G-8; elevation 38' in hole G-10; and elevation 49' in hole H-8. As might be expected, the lower reading in hole G-10 could be the result of more intense fracturing.

The bearing strength of the foundation rock far exceeds any load that will be superimposed upon it. In fact, the rock properly grouted could support any engineering structure that man might build. On the other hand, the fractured condition of the rock will call for a grouting program which can be undertaken after the foundations are opened up to grade. The desireability of "line drilling" during excavation to reduce "shooting" to a minimum is indicated.

A question always arises in evaluating the suitability of limestone or dolomite foundations, as to whether caverns exist beneath the surface. At this site, judging by observations in two very large nearby quarries, one may feel very sure that caverns do not exist. This conclusion will be doubly checked when the grouting program mentioned below is carried out after the foundations are opened up to grade.

The writer understands that it is not part of his responsibility to discuss ground water at length; the subject of "Hydrology" is reported on elsewhere.

Never the less, ground water is part of a foundation, an important part in many places, and in this case unusually so. Therefore, although the writer has made no systematic study of the water-table, nor of the flow of water beneath it, it is desireable to point out the nature of the problem, and to call attention to certain steps that should be undertaken in the treatment and design of the foundation.

The problem involves how best to prevent the contamination of ground water in the event of leakage from the plant, by accident or otherwise. The ground water-table is indicated on the diamond drill hole logs at the time of drilling. The water-table varies in position with the rainfall; falling in drought, and rising in wet seasons. The ground water beneath the hillside on which the plant will be built flows downward and outward to the Hudson

River. Beyond the crest of the hill to the east, some of the water will flow eastward to the low drainage area in that direction. The rocks to the east, schists and intrusive, are less permeable than the dolomites. Flow to the west, to the Hudson, through the more permeable dolomites will carry the bulk of the water beneath the plant.

The writer suggests in this connection that the position of the ground water-table be determained as accurately as possible in all directions from the plant, for a distance of half a mile. Engineers of the company can undertake this work by determining the position of the water in dug or driven wells throughout the area.

It is desireable to seal off from the ground water, that part of the plant from which contamination might arise. This can be done by thorough pressure grouting of the block of ground beneath the plant, and a block of ground equally deep surrounding that part of the plant where leakage might be dangerous. This second block should extend for at least 100' beyond the plant. That is, cut off the flow of ground water to or from the ground surrounding or beneath the plant. The shape and dimensions of this block of pressure grouted foundation can best be determined after final design of the plant is established, and the foundations have been opened up to grade.

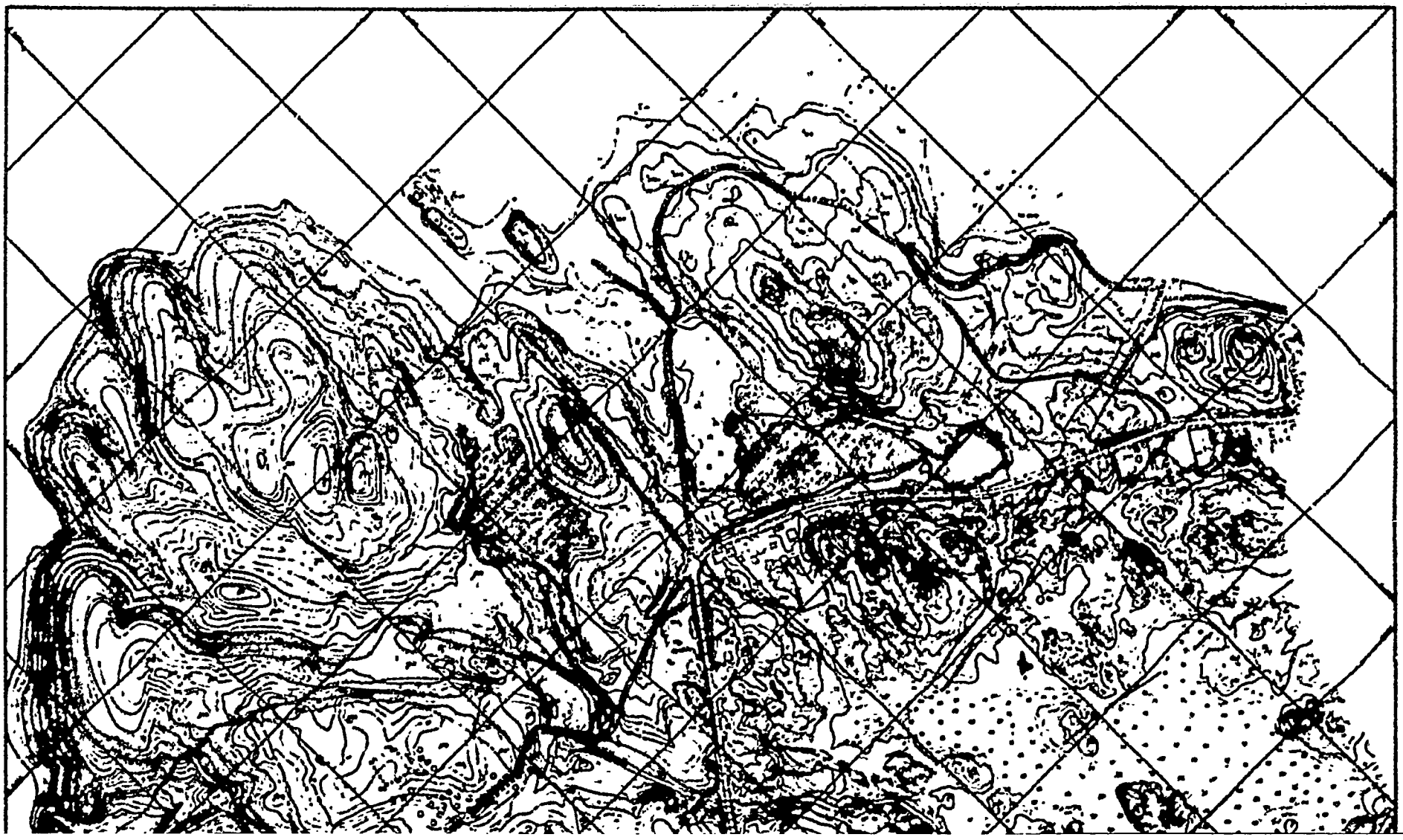
There is another important aspect of this problem on which attention should be focused. It will be valuable to know in advance the radioactivity of the muds and sands along the shore of the river, of the water in nearby streams, and of water in domestic wells. This data should be carefully recorded for future reference, as a yardstick by which to study possible future contamination, or claims of contamination.

I have discussed this matter with Mr. Eugene Simpson a member of the United States Geological Survey, and presently studying for his doctorate at Columbia University.

Sidney Paige

Sidney Paige

Consulting Geologist
October 12, 1955



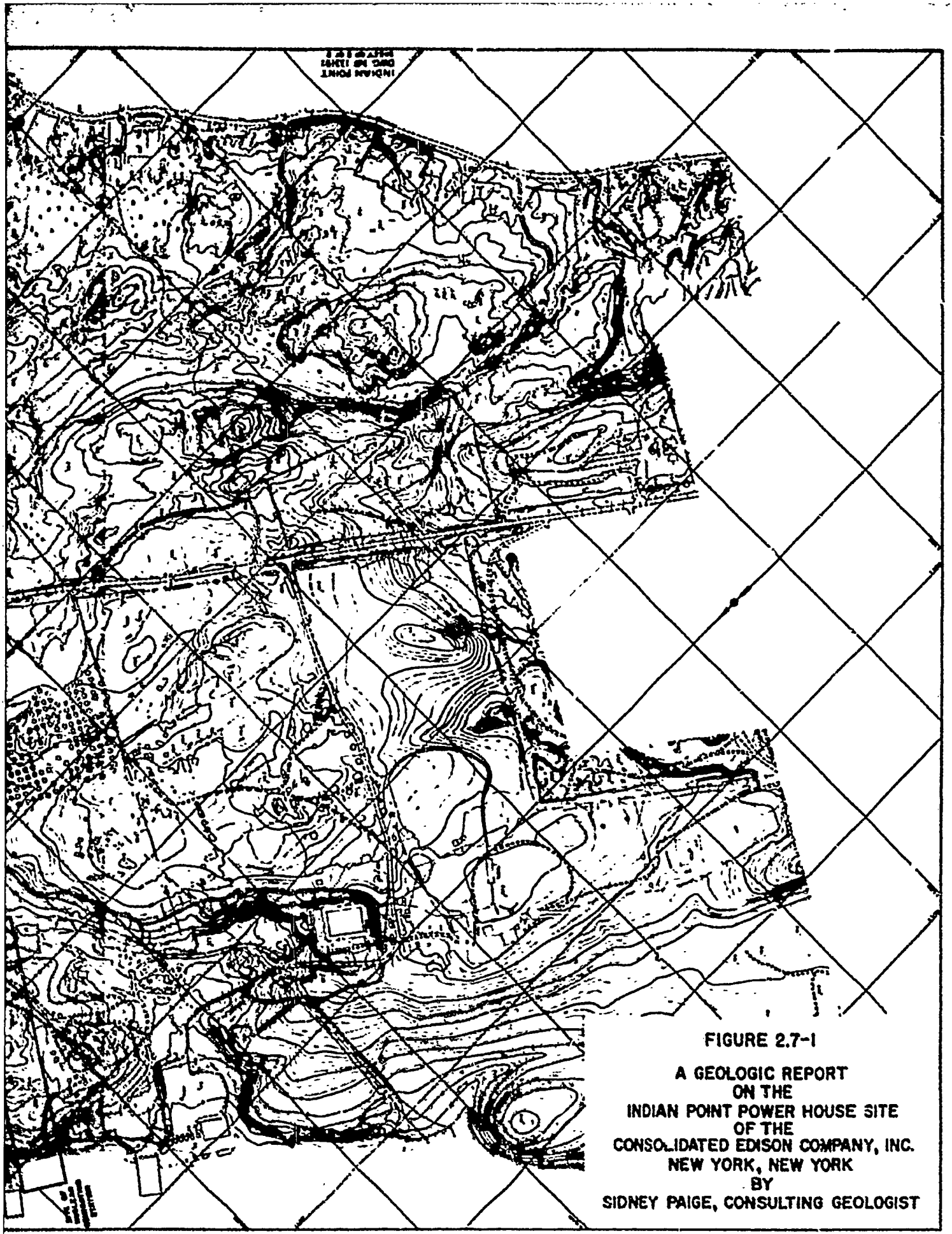


FIGURE 2.7-1

A GEOLOGIC REPORT
ON THE
INDIAN POINT POWER HOUSE SITE
OF THE
CONSOLIDATED EDISON COMPANY, INC.
NEW YORK, NEW YORK
BY
SIDNEY PAIGE, CONSULTING GEOLOGIST

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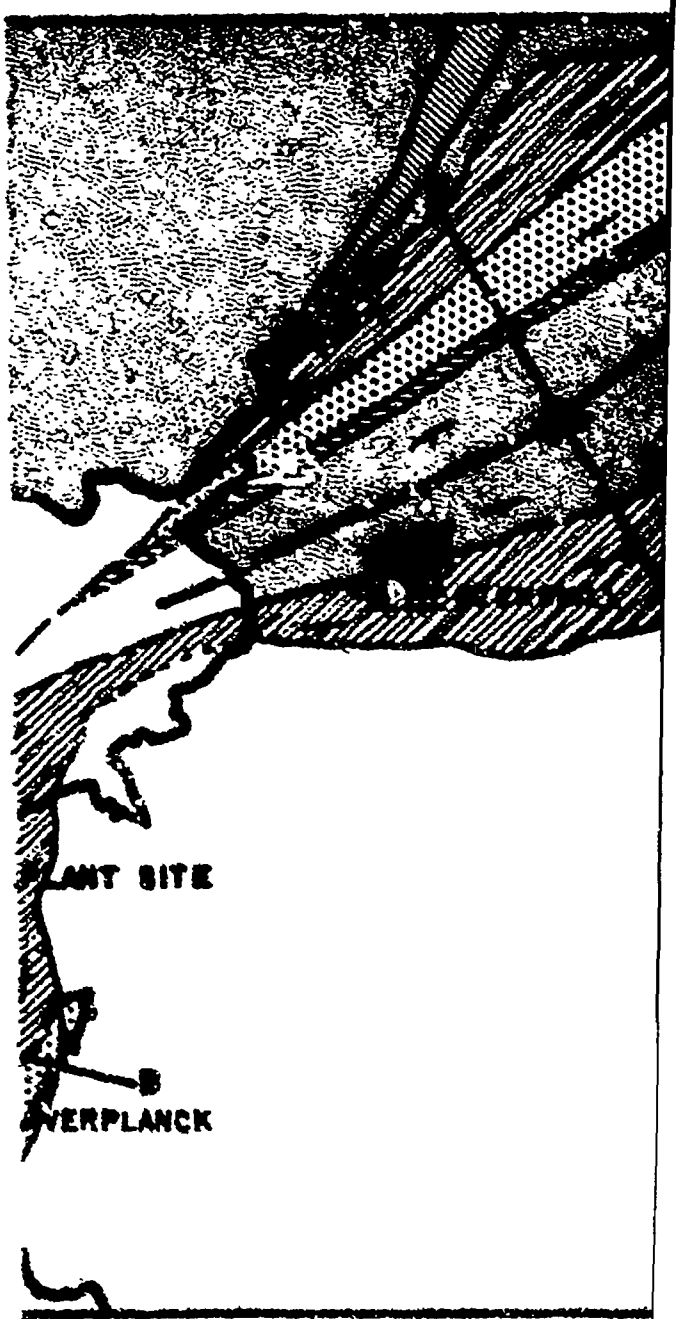


FORMATIONS SOUTH OF PEEKSKILL, AND EAST OF THE HUDSON RIVER, REGARDED BY BERKEY AS OF "DOUBTFUL" AGE AND MAPPED AS INWOOD LIMESTONE AND MANHATTAN SCHIST, ARE HERE MAPPED AS CAMBRO-ORDOVICIAN IN AGE.

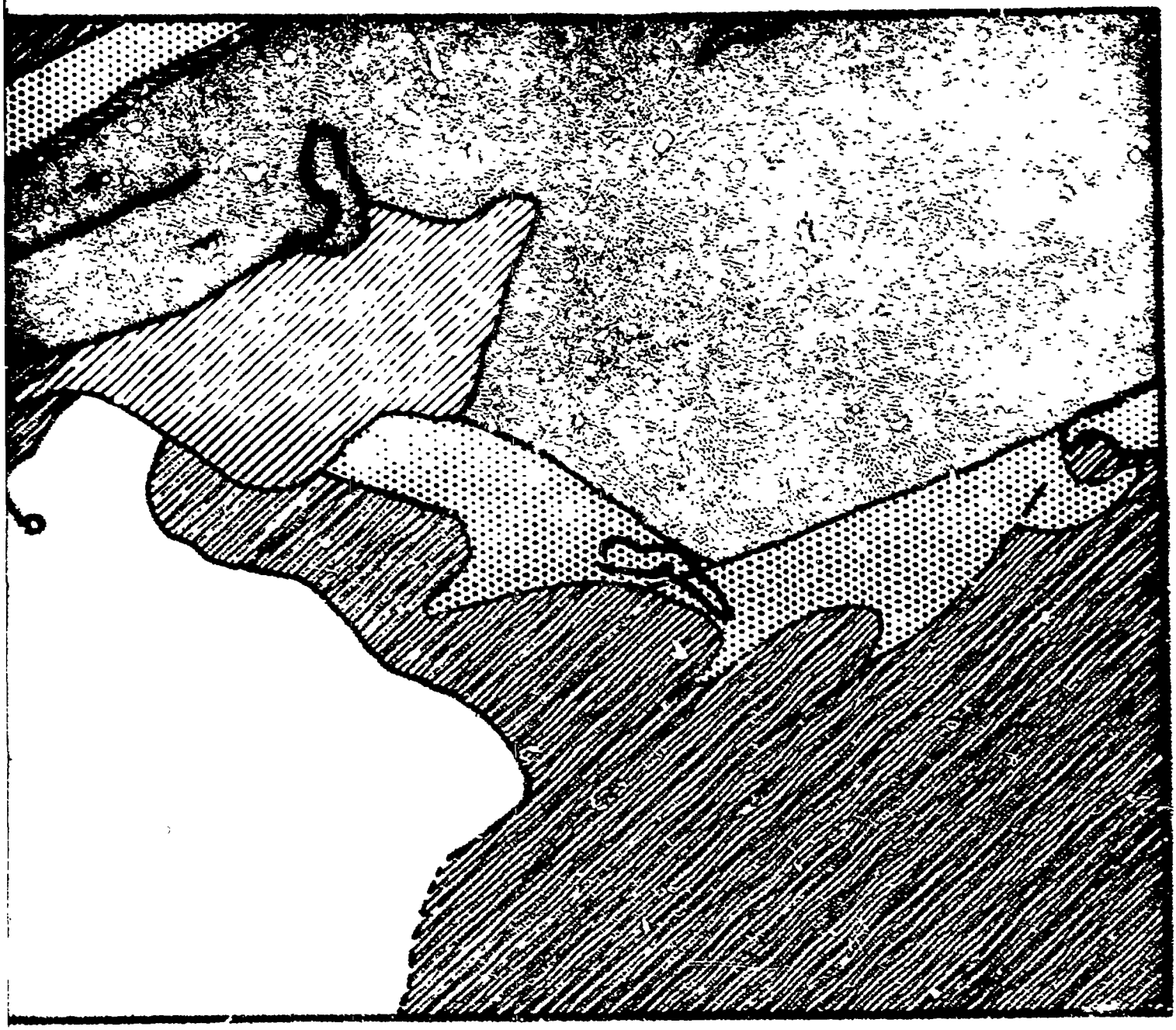
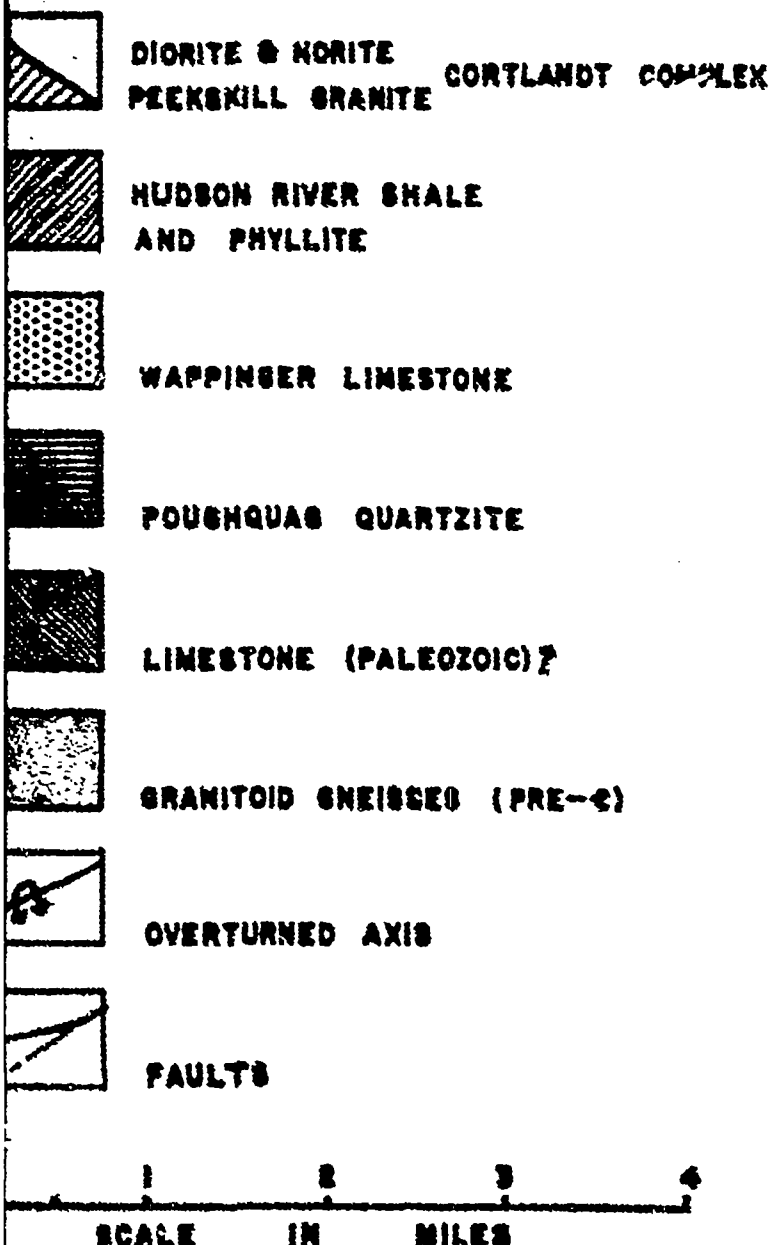
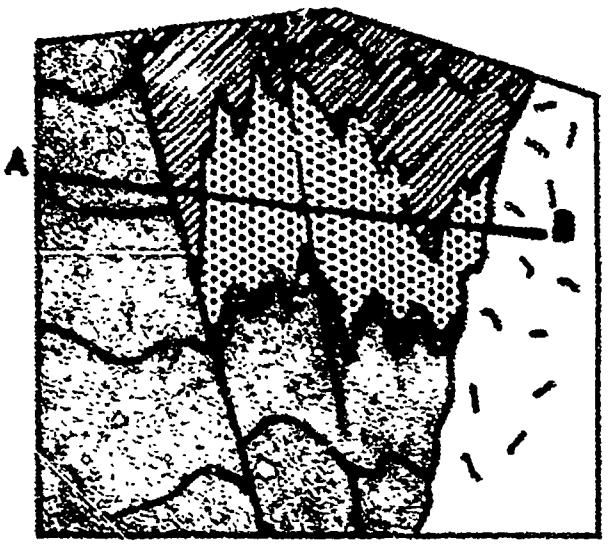


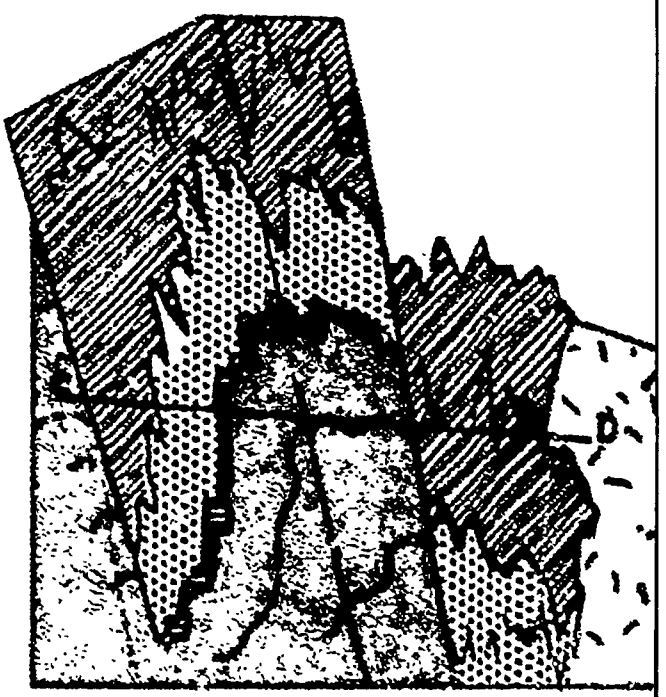
FIGURE 2.7-2

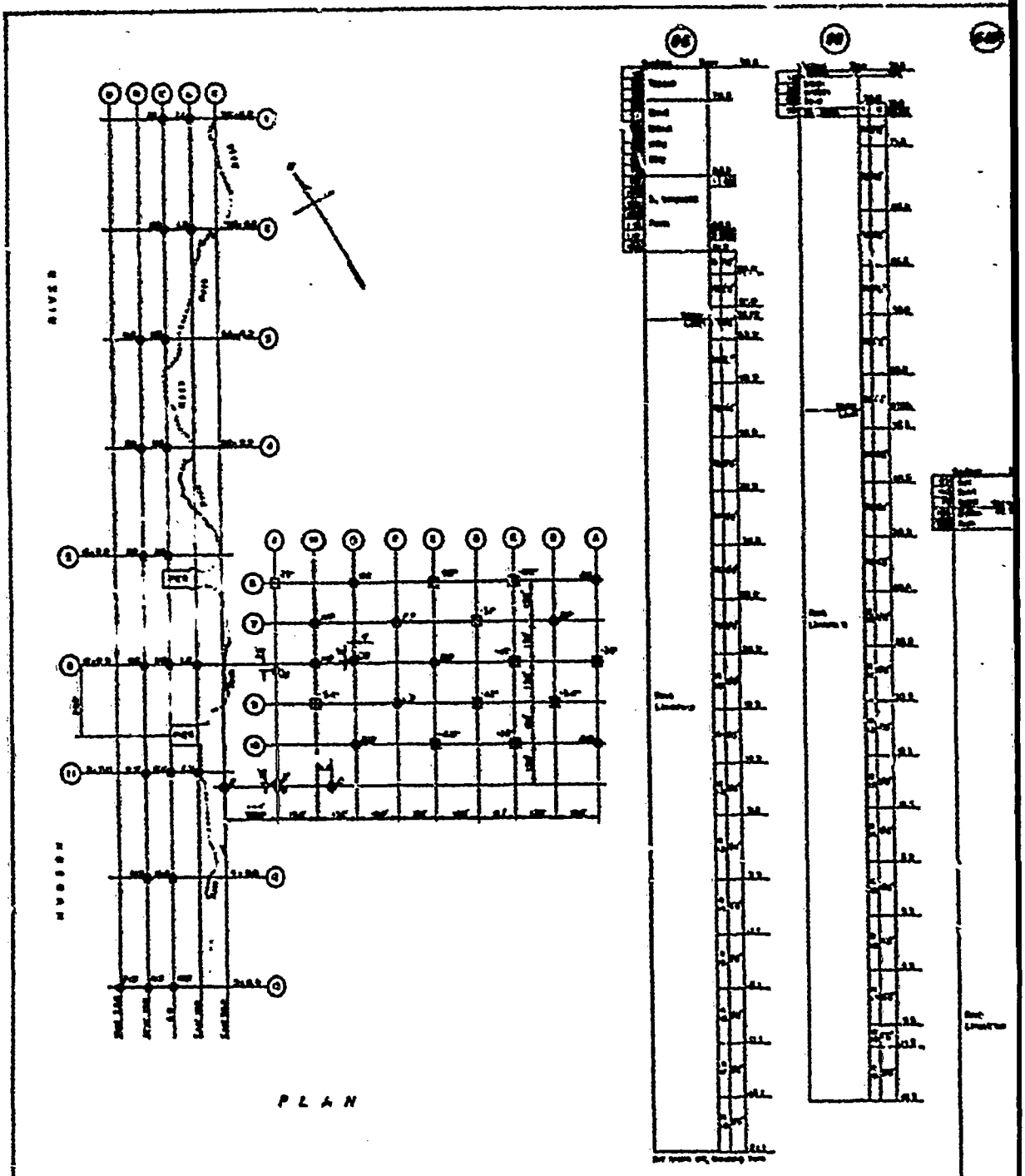
A GEOLOGIC REPORT OF THE INDIAN POINT POWER HOUSE SITE OF THE CONSOLIDATED EDISON COMPANY, INC. NEW YORK, NEW YORK

BY SIDNEY PAIGE, CONSULTING GEOLOGIST





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RIVER
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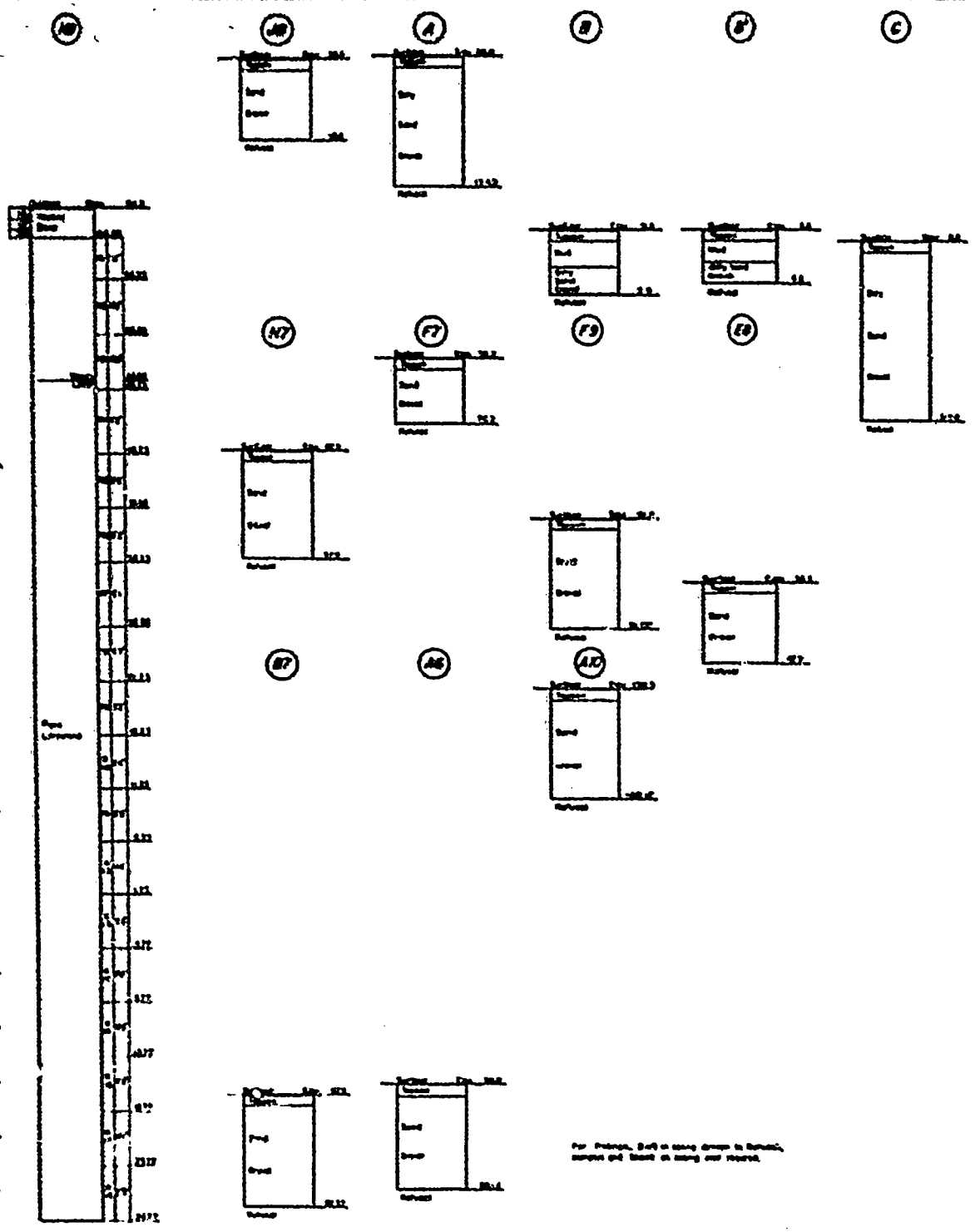
SECTIONS



OF BORINGS

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WARREN GEORGE
150 LIBERTY STREET NEW YORK 6, N.Y.

FIGURE 2.7-3

A GEOLOGIC REPORT
OF THE
INDIAN POINT POWER HOUSE SITE
OF THE
CONSOLIDATED EDISON COMPANY, INC.
NEW YORK, NEW YORK
BY SIDNEY PAIGE, CONSULTING GEOLOGIST

2.8 SEISMOLOGY

The Indian Point site is located in one of the safest seismological areas. The 1958 Uniform Building Code seismic probability map classifies this area in Zone 1, which is a zone where only slight earthquake damage can be expected, with intensities limited to Modified Mercalli readings of V or VI. Table 2.8-1 shows the earthquakes recorded in the area during the last 95 years, and Table 2.8-2 describes the Modified Mercalli Intensity Scale of 1931.

The Reverend Joseph Lynch, S. J., Director of the Fordham University Seismic Observatory has stated". . . . . that the probability of a serious shock occurring in this area for the next several hundred years is practically nil. The area therefore would certainly seem to be as safe as any area at present known." Captain Elliott B. Roberts, Chief of the Geophysics Division of the Department of Commerce, substantially agrees with the conclusions of Rev. Lynch.

Present experience indicates that where future major activity is to be expected minor seismic activity is continuous. For instance, in California, the Dominican Republic, Japan, etc., minor shocks are constantly being recorded. In the Indian Point area there has been no continuous activity in the time that the Fordham Seismic Observatory has been recording these data, over 30 years.

The faults in the area are pre-glacial and have been inactive for over 10,000 years.

The range of expected horizontal acceleration of ground motion for earthquakes of intensity VI on the Mercalli scale, the highest recorded in this area, is 60-100 cm/sec^2 near the epicenter, or about 0.1 g. At a distance of 100 miles from the epicenter, the acceleration drops 50%. These are average accelerations based on data obtained from (1) accelerographs operated in California by the Coast &

Geodetic Survey, (2) instruments located on different geological formations, and (3) earthquakes of various intensities.

Zero period have been assumed for the horizontal accelerations since the knowledge of periods involved during an earthquake are scant. However, it has been fairly well established that, irrespective of periods involved, a building constructed to withstand a horizontal force 1/10 of its weight has successfully withstood ground accelerations greater that 1/5 g in major quakes.

Rev. Lynch has stated that the "estimated maximum ground acceleration of 0.03 g is reasonably conservative for the area." This has been established as the basis for design of the plant. Rev. Lynch states further that the "safety factor for a horizontal stress of 0.1 g is therefore, ... more than adequate." For earthquakes having a horizontal acceleration of 0.1 g and a vertical acceleration of 0.05 g acting simultaneously at zero period, the plant is designed to have no loss of function of systems important to safeguards, although in some cases, the stresses may reach or slightly exceed yields.

In 1955, prior to the construction of Indian Point Unit No. 1, Rev. Lynch appraised the earthquake risk in the Indian Point area and stated that the area was "as safe as any area at present known." Captain Elliott B. Roberts, Chief of the Geophysics Division of the Department of Commerce, concurred with Rev. Lynch's appraisal. In 1965, in connection with Indian Point Unit No. 2 and again in 1967 in connection with Unit No. 3 Rev. Lynch was again asked for his appraisal of the site with regard to earthquake risks. He re-affirmed the validity of his Report of 1955, stating that the seismic situation in the area has remained unchanged, and will for many centuries. The seismological history of the site between 1955 and 1967 has been supplied by Mr. James Dorman of the Lamont Geological Observatory of Columbia University.

TABLE 2.8-1

LIST OF EARTHQUAKES

DATE	HOUR	LOCALITY	N. LAT.	W.LONG.	SQ.MI.	INTENSITY MODIFIED MERCALLI SCALE
July 11, 1872	5:25	Near New York City	40.9	73.8	100	VI
Dec. 10, 1874	22:25	Westchester, N. Y.	40.9	73.8	2000	VI
Oct. 4, 1878	2:30	Hudson River, N. Y.	41.5	74.0	600	V
Aug. 10, 1884	14:07	New York City Vicinity	40.6	74.0	31,000	VI
Mar. 9, 1893	0:30	Near New York City	40.6	74.0	Local	VI
June 8, 1916	16:15	Near New York City	41.0	73.8	Local	V
Apr. 11, 1926	22:30	New Rochelle, N.Y.	40.9	73.9	150	V
Sept. 3, 1951	-	Rockland County, N. Y. Bernardsville fault-eastern New Jersey across Southeast N. Y. to New England	-	-	5500	V (Peekskill-IV)
Dec. 7, 1951		Poughkeepsie, N. Y.	-	-		light shock
Aug. 16, 1953		Bergen County, N. J.	-	-		IV
1957		West Central New Jersey (not felt in N. Y. State)	-	-		VI
Nov. 22, 1967		Southern End of Kensico Reservoir				V or VI

TABLE 2.8-2

MODIFIED MERCALLI INTENSITY SCALE OF 1931
(Abridged)

I. Not felt except by a very few under especially favorable circumstances. (I Rossi-Forel Scale)

II. Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing. (I to II Rossi-Forel Scale)

III. Felt quite noticeably indoors, especially on upper floors of buildings, but many people do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibration like passing truck. Duration estimated. (III Rossi-Forel Scale)

IV. During the day felt indoors by many, outdoors by few. At night some awakened. Dishes, windows, doors disturbed; walls made creaking sound. Sensation like heavy truck striking building. Standing motor cars rocked noticeably. (IV to V Rossi-Forel Scale)

V. Felt by nearly everyone; many awakened. Some dishes, windows, etc., broken; a few instances of cracked plaster; unstable objects overturned. Disturbances of trees, poles, and other tall objects sometimes noticed. Pendulum clocks may stop. (V to VI Rossi-Forel Scale)

VI. Felt by all; many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight. (VI to VII Rossi-Forel Scale)

VII. Everybody runs outdoors. Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable in poorly built or badly designed structures; some chimneys broken. Noticed by persons driving motor cars. (VIII Rossi-Forel Scale)

VIII. Damage slight in specially designed structures; considerable in ordinary substantial buildings with partial collapse; great in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Disturbed persons driving motor cars. (VIII+ to IX Rossi-Forel Scale)

IX. Damage considerable in specially designed structures; well designed frame structures thrown out of plumb; great in substantial buildings with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken. (IX+ Rossi-Forel Scale).

X. Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations; ground badly cracked. Rails bent. Landslides considerable from river banks and steep slopes. Shifted sand and mud. Water splashed (slopped) over banks.

TABLE 2.8-2 (Cont'd)

XI.	Few, if any (masonry), structures remain standing. Bridges destroyed. Broad fissures in ground. Underground pipe lines completely out of service. Earth slumps and land slips in soft ground. Rails bent greatly.
XII.	Damage total. Waves seen on ground surfaces. Lines of sight and level distorted. Objects thrown upward into the air.

2.9 ENVIRONMENTAL RADIOACTIVITY

Surveys of radioactivity by Con Edison in the vicinity of Indian Point Station were begun in 1958, about four years before Unit No. 1 began operation, and have continued as a regular program which is reported semi-annually to the AEC Docket 50-3. The measurements have shown that operation of Indian Point Unit No. 1 for over five years has had no significant effect on the plant environment.

The reports contain tabulations and charts of radioactivity in soil, water, vegetation, marine life, and air particulate samples collected during the nine year period. Each report lists the sampling points for the reporting period and locates these on a map.

The present environmental program includes continuous sampling of atmospheric dust, Hudson River water, and measurement of the gross gamma background on the Indian Point site. Surface water from a small lake on the site and drinking water from the Indian Point tap and nearby reservoirs are sampled weekly. Monthly samples are taken of Hudson River water upstream and downstream from the site, vegetation on the site, marine life from the river, and water from the Indian Point well. Vegetation and soil samples are taken regularly during the growing season and gamma radiation surveys on roads in the station vicinity are made annually.

The results of the environmental surveys conducted by Consolidated Edison have been verified by the Bureau of Radiological Health Services of the New York State Health Department.[1][2] The most recent report has concluded the following:

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(1) Consolidated Edison Indian Point Reactor, Post Operational Survey August, 1965, Division of Environmental Health Services, New York State Department of Health, Hollis S. Ingraham, M. D., Commissioner.

(2) Consolidated Edison Indian Point Reactor, Environmental and Post Operational Survey July, 1966, Division of Environmental Health Services, New York State Department of Health, Hollis S. Ingraham, M. D. Commissioner.

"The summary of environmental discharges indicates that the plant has been operating within the allowable limits."

"No increase in radioactivity was observed in this period which can be attributed to the operation of the Indian Point Reactor." (2)

This has also been confirmed by Dr. Merril Eisenbud, Director of Environmental Radiation Laboratory, Institute of Environmental Medicine, New York University Medical Center, who has found that the levels of environmental radioactivity are associated with natural background and fallout from nuclear weapons tests.(3)

In a study of the radioactivity in the Hudson River (4) Mr. Sherwood Davies, Director, Bureau of Radiological Health Service, New York State Department of Health, et. al., have concluded that the discharges from Indian Point Unit No. 1 "are a minute fraction of the federal limits." Additional conclusions are as follows:

"All evidence indicates that fallout contributes much more radioactivity that other man-made sources. The Health Department, in its responsibility for radiological health in the environment, has monitored intensively in the area of any nuclear plant before and after operation. None of these studies has shown any increase in radioactivity attributable to operation of an industrial plant."

In a statement for the record at the public hearing for construction permit for Indian Point Unit No. 2, Hollis S. Ingraham, M. D., Commissioner, New York State Department of Health said:

"The Department has monitored environmental radioactivity in this vicinity since 1958, with modifications in the program as indicated. It has collected and analyzed air, milk, water, vegetation and other environmental samples and has carried out gamma background surveys. The company has kept the

Department fully informed of environmental releases from Unit No. 1. There has been mutual cooperation in establishing a satisfactory environmental surveillance program."

"Fallout from nuclear weapons testing late in 1962 was the prime cause of the activity increases observed throughout the state in air, fallout, water and milk. Our review of the surveillance program data and actual environmental releases reported by the company indicates no measurable increase in the background radioacitivity levels attributable to the operation of the Indian Point plant."

---

(3) Ecological Survey of the Hudson River: Progress Report No. 2, submitted to Division of Radiological Health, USPHS, Contract PH 86-95. Neg. 141, December 1966, by New York University Medical Center Institute of Environmental Medicine.

(4) Sherwood Davies, Bureau of Radiological Health Service, New York State Department of Health and Frank Cosolito and Merril Eisenbud, Institute of Environmental Medicine, New York University, Radioactivity in the Hudson River, Symposium on Hudson River Ecology held October 4-5, 1966, Hudson River Valley Commission of New York.

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1 November 1965

Mr. William J. Cahill, Jr.
Chief Nuclear Engineer
The Consolidated Edison Company
Of New York
Four Irving Place
New York, New York 10003

Dear Mr. Cahill:

This is in reply to your request that I supply you with a statement summarizing the present status of environmental radioactivity in the vicinity of Indian Point.

The radioactive background at Indian Point was measured by both the Company and the State Department of Health prior to August 1962 when Unit #1 went into operation, and has been under continuous surveillance since then by the health physics group employed by the Indian Point Station, and the New York State Department of Health. In addition, beginning in the summer of 1964 this Institute has undertaken ecological studies of the Hudson River under a contract with the United States Public Health Service, and detailed radiological measurements have been included as part of this research.

The measurements of all three groups have shown that operation of Indian Point Unit #1 for more than three years has had no perceptible effect on the environs of the plant. Minor perturbations of the radioactive background have occurred during the past few years, but these have been due entirely to radioactive fallout from nuclear weapons tests.

The investigations referred to above have been reported in the following documents:

1. New York State Department of Health, Division of Environmental Health Services, POST OPERATIONAL SURVEY OF CONSOLIDATED EDISON INDIAN POINT REACTOR, August 1965.

2. New York University Institute of Environmental Medicine, RADIOECOLOGICAL SURVEY OF THE HUDSON RIVER, Report to the Division of Radiological Health, U. S. Public Health Service, 1965.

3. Consolidated Edison Company, Semi-Annual Reports to the Atomic Energy Commission.

Very truly yours,

Merril Eisenbud
Professor

ME:cc

2.9 ENVIRONMENTAL MONITORING PROGRAM

2.9.1 GENERAL

2.9.2 SURVEY PROGRAMS

Fallout (Rainwater)
Air Particulate and Organic Iodide
Reservoir Water
Hudson River Water
Lake Water
Well Water
Lake Aquatic Vegetation
Hudson River Aquatic Vegetation
Hudson River Bottom Sediments
Hudson River Fish
Vegetation
Soil
Direct Gamma (Spot Readings)
Direct Gamma (Continuous)

2.9.3 SUMMARY

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Figure 2.9-1 Indian Point Station Environmental Sampling Site Locations

Figure 2.9-2 Scale Plot Plan of the Site Showing the Location of the Facility, Indian Point Station, Buchanan, New York

2.9 ENVIRONMENTAL MONITORING PROGRAM

2.9.1 GENERAL

A program to determine the environmental radioactivity in the vicinity of Indian Point Station was instituted in 1958 four years prior to the initial operation of Indian Point Unit No. 1. The purpose of this survey was to determine the natural background radioactivity and to show the variations in the activities that may be expected from natural sources, fallout from bomb tests and other sources in the vicinity. This program has been continued to the present so that changes in the environment resulting from station operations, could be accounted for. The results of these surveys are reported semi-annually to the Atomic Energy Commission under Docket No. 50-3. A history of the Indian Point Nuclear Generating Station Environmental Monitoring Program is reported in Environmental Monitoring in the vicinity of the Indian Point Nuclear Generating Station 1958-1969 which is included in the section.

In addition, the New York State Department of Health has conducted surveys throughout the State of New York since 1955, including extensive surveys in the vicinity of the Indian Point Station since 1958. In 1965 and 1966, they reported the findings in the vicinity of the Indian Point Station in two special reports. Since that time, their reporting has been on a statewide basis in quarterly bulletins and in annual reports.

In 1964, the New York University Medical Center began a research program on the ecology of the Hudson River. The New York University studies include the biology of the Hudson River, the distribution and abundance of fish in the river, pesticides and radioecological studies. The results of this program, supported by the United States Public Health Service, the New York State Department of Health and the Consolided Edison Company, have been submitted in several progress reports.

The various studies mentioned above included measurements of radioactivity in fresh water, river water, river bottom sediments, fish, aquatic vegetation, soil, vegetation and air in the vicinity of the Indian Point Station. The results of these monitoring programs have shown that the operation of the Indian Point Station has had no deleterious effects on the environment.

2.9.2 SURVEY PROGRAMS

The survey of environmental radioactivity in the vicinity of Indian Point Station provides an indication of the integrity of the in-plant radiation monitoring instrumentation and can reveal any buildup of long lived radionuclides.

By determining the activity of filterable air particulate, rainwater, vegetation, soil, lake water, lake aquatic vegetation, well water and above-ground gamma fields, an indirect monitoring of discharges to the atmosphere is provided by the environmental survey program.

The effect of liquid effluents on the Hudson River is monitored by measuring the activity of the cooling water inlet to the station against all liquid discharges from the plant, activity analysis of river bottom sediments and river aquatic vegetation. Table 2.9-1 indicates the routine frequency of sample collections, types of measurements made and equipment used for the measurements. Maps delineating sample locations and thermoluminescent dosimeter locations are given in Figures 2.9-1 and 2.9-2 respectively.

A detailed description of the media sampled in the environmental monitoring program is given below:

Fallout (Rainwater)

A plastic container with a twelve-inch diameter opening is used as a collector of filterable and non-filterable materials deposited by rainwater at two locations. One location is on-site and the other is about fifteen miles south at Eastview. Samples are collected at the end of each month and analyzed for gross beta activity and tritium content. In the event of sample evaporation, 1/2 liter of distilled water is added to and mixed with the sediment in the bottle to provide a representative sample for analysis.

Air Particulate and Organic Iodide

Concentration of radioactive particles in the air is measured weekly from three stations on site and in addition, one located south of the plant at Trap Rock and one located north of the plant at Standards Brands, Inc.

Air sampling is also done at four selected off-site points. A mobile monitor is stationed at one of the four selected locations for a one-week interval. After this duration, the air monitor is moved to another location.

Membrane filters, 0.8 micron mesh, precede charcoal impregnated filters. The former is assayed for gross beta activity and the latter for radio-iodine.

Reservoir Water

Drinking water is sampled monthly from three area reservoirs: New Croton Reservoir, Camp Field Reservoir and Camp Smith Reservoir. The water is analyzed for gross beta activity and tritium content.

Hudson River Water

Continuous flow samples of the condenser inlet cooling water and discharge water are regulated to fill two fifty gallon drums. A sample from each drum is taken weekly and analyzed for gross beta activity. In addition, a sample from each drum is taken and stored so that at each month's end, a tritium analysis may be made on the composite sample.

Lake Water

Off-shore water grab samples are taken monthly from three area lakes: Indian Point Lake, Trap Rock Lake and Lake Meahagh. The water is analyzed for gross beta activity and tritium content.

Well Water

Grab samples are taken monthly from two deep wells. One is on site and the other is on private property in Verplanck. The water is analyzed for gross beta activity and tritium content.

Lake Aquatic Vegetation

During the Spring, Summer and Fall, samples of lake aquatic vegetation are taken along the shores of Indian Point Lake, Trap Rock Lake and Lake Meahagh. Samples are taken once during these seasons and analyzed for gross beta activity. A gamma spectrum is also made on these samples.

Hudson River Aquatic Vegetation

During the Spring, Summer and Fall samples of river aquatic vegetation are taken at the mouth of the discharge canal, Peekskill Bay, Tomkins Cove, Lovett plant of Orange and Rockland Utilities and off Verplanck. The samples are taken once during these seasons and analyzed for gross beta activity. A gamma spectrum is also made on these samples.

Hudson River Bottom Sediment

During the Spring, Summer and Fall, samples of river bottom sediment are taken at the same locations as the Hudson River Aquatic Vegetation. The samples are taken once during these seasons and analyzed for gross beta activity. A gamma spectrum is also made on these samples.

Hudson River Fish

Fish are caught monthly where available near the site and analyzed for gross beta activity and a gamma spectrum is also made. The bone as well as the flesh is ashed and counted 48 hours after collection to allow for decay of the radon daughters.

Vegetation

During the Spring, Summer and Fall, samples of vegetation are collected in the normal downwind direction and at one point north of the site. The samples are taken once during these seasons in a 100 square foot area at several locations and are analyzed for gross beta activity. Gamma spectral analysis is also performed on these samples.

Soil

Samples of soil are taken once a year at the same locations as the vegetation samples. The soil samples are dried for gamma spectral analysis immediately after collection. Gross beta analysis of the dried soil is made 48 hours after collection to allow for decay of the radon daughters.

Direct Gamma (Spot Readings)

Once a year, spot readings of background gross gamma radiation are made within a five-mile radius of the Indian Point Station and along the principal roads. The readings are made at approximately one-tenth mile intervals with the measuring device hand-held about three feet above the ground.

Direct Gamma (Continuous)

At eleven locations on site and at selected locations off site, the background gross gamma radiation is continuously monitored. The measuring devices consist of ionization chambers or film badges or thermoluminescent dosimeters. The measuring device is removed at monthly intervals and the amount of absorbed background radioactivity recorded.

2.9.3 SUMMARY

The environmental monitoring program conducted by Consolidated Edison supplies sufficient data to determine the compliance of Indian Point Unit Nos. 1, 2 and 3 with the requirements of 10CFR20. The environmental survey

program which monitors air, water, sediments, terrestial and aquatic vegetation, soil and selected aquatic biota provides an indication of the cumulative amounts of radioactivity in the environment.

Results of the environmental monitoring program are reported on a semi-annual basis to the Atomic Energy Commission, who are thereby advised of the short and long-term trends in the environment. In addition, discharges of radioactive liquids and gases are reported to the Atomic Energy Commission and the New York State Department of Health.

In the event that Consolidated Edison detects increases in the background radiation levels, the Company will notify the New York State Department of Health, Bureau of Radiological Health. The Department of Health would increase its surveillance program in the vicinity of Indian Point Station as the need requires, and Consolidated Edison would increase the frequency of collections at the existing sample locations.

It should also be noted that the results of the research program on the Hudson River by the Institute of Environmental Medicine of New York University's Medical Center, which is concerned both with evaluation of the total ecology and of radionuclides in relation to ecology, are reported on an annual basis to the New York State Department of Health, the Department of the Interior, the United States Public Health Service, the Consolidated Edison Company, the Hudson River Valley Commission, the New York State Conservation Department, the New York City Environmental Protection Agency, the New York State Power Development Association and other interested research organizations. Although this is not a monitoring program, the results of this research are germane since they bear upon the manner in which radionuclides behave in the river ecosystem.

To conclude, although the design of Indian Point Unit No. 3 and administrative controls will be such that liquid and gaseous effluents will be released in accordance with the requirements of 10CFR20, the environmental monitoring program of the Consolidated Edison Company provides a redundant

means of insuring that the operation of this facility does not pose any undue risk to the health and safety of the public. The New York State and New York University programs provide an independent means of verifying the facility's compliance with 10CFR20.

	Media	Type	Sampling Frequency	
1	Fallout	Continuous	Monthly	Open coll
2	Air Particulate and Organic Iodide	Continuous at 1 CFM	Weekly	Two bran (0.8 size a ch
3	Reservoir Water	Grab	Monthly	
4	Hudson River Water	Continuous	Weekly	Cont regu 50 g Repr samp a we
5	Lake Water	Grab	Monthly	1 li fro

TABLE 2.9-1

AN POINT STATION ENVIRONMENTAL SURVEY

d of ction	Locations	Analysis	Minimum Sensitivities	Measurement Instrumentation	Remarks
type rain	Point 1 and at Eastview (15 miles south of the site)	Gross beta and tritium	1 picocurie per liter for gross beta	Gas flow, window-less proportional counter for gross beta Nuclear Measure-ment Corp. Type PC 3A Type PC 11A Type PC 11T	Measurement made 48 hrs. after collec-tion to allow for decay of radon daugh-ters
			3000 picocuries per liter for tritium		
l mem-lters ron ceding l filter	Points 1, 2, 3, 4 and 5. In addition off-site at points in Peekskill, Buchanan, and Verplanck for one week periods consecutively.	Gross beta and gamma spectrum	0.1 picocuries per cubic meter for gross beta	Same as 1 for gross beta	Measurements made soon after collec-tion and 48 hrs. later to allow for de-cay of radon daughters
			2 picocuries per cubic meter for I-131	Gamma spectrum with 3" x 3" NaI crystal with 400 channel analyzer Radiation Instru-ments Development Laboratories Model 3412 Gamma Spectrometer	
	Points 6, 7 and 8	Same as 1	Same as 1	Same as 1	Same as 1
as flow l to fill drums. ative kan once d drums	Hudson River in-let pipe into the plant, and at plant dis-charge canal. Points 9 and 10	Same as 1 and trium on monthly com-posite	Same as 1	Same as 1	Same as 1
ample shore	Points 11, 12, and 13	Same as 1	Same as 1	Same as 1	Same as 1

	Media	Type	Sampling Frequency	Me Co
6	Well Water	Grab	Monthly	From d pumps
7	Lake Aquatic Vegetation	Grab	Once each in Spring, Summer and Fall	Along shore
8	Hudson River Aquatic Vegetation	Grab	Same as 7	Along
9	Hudson River Bottom Sediment	Grab	Same as 7	Same a
10	Hudson River Fish	Catch	Monthly	Same a
11	Vegetation	Grab	Same as 7	Grab s within area
12	Soil	Grab	1 per year	Grab s in dia 2" dee

TABLE 2.9-1 (Cont'd)

M POINT STATION ENVIRONMENTAL SURVEY

of tion	Locations	Analysis	Minimum Sensitivities	Measurement Instrumentation	Remarks
well	Points 6, 14 and Verplanck	Same as 1	Same as 1	Same as 1	Same as 1
lake	Same as 5	Same as 2	1 picocurie per gram for gross beta 2 picocuries per gram for I-131	Same as 2	Dry weight for spectrum soon after collection. Sample ashed and counted 48 hrs. after collection for gross beta
r shore	Points 10, 15, 16, 17, and 22. At mouth of discharge canal, Peekskill Bay, Tompkins Cove off Verplanck, and at the Lovett plant of Orange and Rockland Utilities.	Same as 2	Same as 7	Same as 2	Same as 7
	Same as 8	Same as 2	Same as 7	Same as 2	Mud dried for both measurements
	Where available near site	Same as 2	Same as 7	Same as 2	Sample ashed and counted 48 hrs. after collection for gross beta and gamma spectrum taken
les $0 ft^2$	Points 6, 18, 19, 20 and 21	Same as 2	Same as 7	Same as 2	
les 2" r by	Same as 11	Same as 2	Same as 7	Same as 2	Soil dried for spectrum and measured soon after collection. Gross beta of dried soil made 48 hrs. after collection

	Media	Type	Sampling Frequency	M C
13	Direct Gamma	Spot Readings	Once a year	
14	Direct Gamma	Continuous	Monthly	

TABLE 2.9-1 (Cont'd)

IAN POINT STATION ENVIRONMENTAL SURVEY

od of action	Locations	Analysis	Minimum Sensitivities	Measurement Instrumentation	Remarks
	Along principal roads within a 5 mile radius of plant	Gross gamma background	2.2×10^6 counts per min. in a Cesium-137 field of 1 mr/hr. Minimum sensitivity 1 ur/hr.	Franklin Systems Inc. Model 15-2	Instrument readings in counts per minute measured at approximately 1/10 mile intervals. Readings converted to microrem per hr
	Selected locations in Buchanan, Verplanck, Montrose, Peekskill, and at a number of points on-site at the plant perimater	Same as 13	1 mr/hr	Victoreen Ionization Chamber Model 239 0-10 Mr or Film badges or TLD-Thermoluminescent dosimeters	

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1 November 1965

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This is in reply to your request that I supply you with a statement summarizing the present status of environmental radioactivity in the vicinity of Indian Point.

The radioactive background at Indian Point was measured by both the Company and the State Department of Health prior to August 1962 when Unit #1 went into operation, and has been under continuous surveillance since then by the health physics group employed by the Indian Point Station, and the New York State Department of Health. In addition, beginning in the summer of 1964 this Institute has undertaken ecological studies of the Hudson River under a contract with the United States Public Health Service, and detailed radiological measurements have been included as part of this research.

The measurements of all three groups have shown that operation of Indian Point Unit #1 for more than three years has had no perceptible effect on the environs of the plant. Minor perturbations of the radioactive background have occurred during the past few years, but these have been due entirely to radioactive fallout from nuclear weapons tests.

The investigations referred to above have been reported in the following documents:

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ENVIRONMENTAL
MONITORING
PROGRAM
REPORT

INDIAN POINT STATION

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1.0 HISTORY OF THE INDIAN POINT NUCLEAR GENERATING STATION ENVIRONMENTAL MONITORING PROGRAM

1.1 Introduction

The Atomic Engergy Commission charges private nuclear reactor operators with the responsibility of insuring that no hazardous condition which could endanger the health and safety of the public will result from the operation of a reactor. Relevant to this determination, this responsibility is considered to be satisfactory discharged if it is demonstrated that the operation of the reactor has not increased the level of radioactivity of the environment of exposed people in the vicinity of the plant beyond specified limits. It is thus necessary to ascertain the level of radioactivity existing prior to the operation of the reactor. Consequently, it was for this reason that an environmental radiation survey was instituted in 1958 preparatory to the operation of Indian Point Unit No. 1 (AEC Docket 50-3). The pre-operational survey for Unit No. 1 was continued until the reactor achieved initial criticality in August 1962. These measurements, therefore, covered a period of approximately four years. This showed the variation in the activities that may be expected from natural sources, fallout from weapons test, and other sources in the vicinity. This survey has been continued as a regular program fulfilling the requirements of the provisional operating license for Indian Point Unit No. 1 (DPR-5) and the data obtained is reported semi-annually to the Atomic Energy Commission under Docket No. 50-3. These semi-annual reports contain tabulations and charts of radioactivity in filterable air particulate, fallout, aquatic vegetation, soil, lake water, well water, drinking water, reservoir water, Hudson River water, fish and bottom sediments samples collected to date. In addition, an annual road survey measures gross gamma in the vicinity of Indian Point. Each report also tests the sampling points for the reporting period and locates these on a map.

The results of the pre-operational environmental surveys concluded that the radioactivity in the vicinity of Indian Point was low and comparable to that existing at other locations in the country.(1) The dominant source of measured radioactivity is attributed to fallout from nuclear weapons debris. Subsequent measurements obtained by Consolidated Edison's semi-annual surveys of environmental radioactivity have demonstrated that the environmental monitoring program at Indian Point Unit No. 1 plant for eight years has been an effective means of measuring fallout and naturally occuring radionuclides. Insignificant, but measureable determinations have been made in rooted aquatic vegetation which is especially collected and sampled to determine the uptake of activation products. The results of these surveys have been verified by the Bureau of Radiological Health of the New York State Department of Health.(2)(3)

For ease of presentation and simplicity in comprehension, the data obtained from the pre- and post-operational environmental monitoring surveys has been condensed and presented by graphical means. These graphs combine this data

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(1) Pre-Operational Environmental Survey of Radioactivity in the Vicinity of Indian Point Power Plant. 1958 Consolidated Edison Company of New York, Inc.

(2) Consolidated Edison Indian Point Reactor, Post Operational Survey August, 1965, Division of Environmental Health Services, New York State Department of Health, Hollis Ingraham, M.D., Commissioner.

(3) Consolidated Edison Indian Point Reactor, Environmental and Post Operational Survey, July, 1966, Division of Environmental Health Services, New York State Department of Health, Hollis Ingraham, M. D., Commissioner.

in a clear and concise manner. They show the activity of the different media before and after plant operation. Explanatory notes next to obvious higher than average readings indicate plausible explanations for the increase. The sample locations with slight mofidifications were constantly monitored until the environmental program was modified during the last week of April, 1966. The program was then modified to achieve more meaningful data in view of the experience gained since the survey began. The program was again modified in 1969. A detailed explanation of these modifications can be found in the operational section (1.2.1.1). It should be noted that substantial changes in the scope, sampling frequency and instrumentation have occurred since the environmental monitoring program was initiated.

1.2 Description of Environmental Monitoring Programs

1.2.1 Indian Point Unit No. 1

1.2.1.1 Pre-Operational and Operational Program

The pre-operational environmental survey of radioactivity in the vicinity of Indian Point nuclear power plant site commenced in June, of 1958, with the selection of twenty-four sampling locations on the east side of the Hudson River and all within a ten mile radius of the plant site. In 1959, seven new locations on the west side of the Hudson River in Orange and Rockland Counties were added as sampling sites. The preponderance of locations are north and south of the plant site along the Hudson River, and east of the site through Peekskill. Prevailing winds and population concentrations were the deciding factors in selecting these sites.

From these sites air, water, fallout, vegetation, soil, and a limited number of biological specimens were collected and measured with the results displayed as bar graphs in the pre-operational semi-annual reports. All samples were initially checked for both gross-alpha and gross-beta activity. Beginning in January, 1962, however, all monitoring results were given for gross beta-gamma determinations.

Pre-Operational Determinations and Startup Date

From June, 1958, until initial plant operation, all samples were measured approximately 48 hours after collection to eliminate the effect of short-lived natural decay products. Counting errors were based on a 95% confidence level. All samples were checked for alpha decay, but no measurable activity was noted after the 48 hour hold-up period.

The pre-operational phase of the environmental survey came to an end on August 2, 1962, when the Unit No. 1 reactor attained initial criticality. Table 1.2.1.1 contains the sampling schedule which reflects the changes that were made as the station became operational.

A detailed explanation of the environmental monitoring program for the various media follows. A pictorial representation is presented in Figures 1 through 17 and plates 14, 15 and 16. In addition, the graphical data found in the figures is supplemented by tables. These tables are given the same numbers as the figures to relate the two.

Air Particulates

Starting in June, 1958, the gross beta activity of air particulate was determined on millipore filter samples of the air at Indian Point site. Table I lists the number of samples collected each year and Figure I indicates the monthly averages.

In addition, wind direction and velocity at a point 100 feet above grade was recorded. The collection and measurement of air particulate was continued in 1959, 1960, and 1961 in approximately the same manner as in 1958. However, starting in March, 1962, air particulate sampling at the Indian Point station was increased from one to four sampling sites. Air particulate was also monitored at the off-site sampling stations by use of the mobile monitor survey unit. The unit was operated at a station for three or four days and then moved to another location. At the completion of its stay at a station, the filter paper was removed from the continuous air monitor and the activity measured in the laboratory.

Sampling was continued in the same manner as in 1962 until May, 1966, when the water meter house was used an another on-site air particulate sampling location. Its location is east of the superheater stack. (The air monitoring house located in the normal downwind direction south of the stack is the site used for the tabulation of the air particulate data found in Figure 1.) Thus the concentration of radioactive particles in the air was measured weekly from two on-site monitoring stations. In addition charcoal impregnated filters were installed downstream of the particulate membrane filters for purposes of detecting ground concentrations of radioiodine. Whiel the minimum detectable sensitivity for I^{131} is 2 pCi/M^3 by gamma spectroscopy, lower concentrations can be determined by beta counting and a half life determination to a value of 0.1 pCi/M^3. To date, no radioiodine has been detected since this monitoring method was initiated.

Fallout

In October, 1958, an open pot type of collector was placed at the Indian Point site and monthly collections of atmospheric fallout were obtained. The gross-beta activity was measured. Data starting from 1958 was plotted on Figure 2, entitled "Fallout Collected at Indian Point".

Beginning in February, 1960, an additional fallout collector was installed at Hunts Point generating station, New York City. The collection and sampling period is similar to the one at Indian Point. Fallout was collected at Hunts Point until it was discontinued in December, 1961. Figure 3 indicates this activity.

Commencing in January, 1962, fallout collection facilities were installed at Eastview, 15 miles south of Indian Point. Figure 4 indicates this activity. Fallout activity prior to 1963 was reported in the environmental monitoring reports as either suspended and dissolved or soluble and insoluble fractions. However, after 1963, fallout levels decreased to a point where it was no longer practical to report the separate fractions. Instead the activity of the total sample was given. In addition all the data used in fallout

Figures 2, 3 and 4 is given are given as the combined activity. The amount of gross beta radioactivity measured during each collection period is partially dependent on the quantity of rainfall prevailing during that period. In the event there is insufficient rainfall for a particular month, distilled water is added to the collection container to insure that any particles that may have fallen from the atmosphere, are incorporated with the sample.

Water

In 1958 the gross-beta activity of Hudson River water, drinking water, and surface water was sampled and the activity determined at various sampling sites. The drinking water is collected weekly, and consists of grab samples taken from taps and three reservoirs in the vicinity of Indian Point; (1) Peekskill Reservoir, (2) Camp Smith Reservoir, and (3) Croton Reservoir, the data is presented in Figures 6, 7 and 8 and Tables VI, VII, VIII, respectively. The individual reservoir and tap water activity was extracted and averaged over monthly periods from the drinking water data supplied in the environmental monitoring reports. This was accomplished for the years: 1958, 1959 and 1960 For the 1961 reporting period the drinking water data supplied did not indicate the activity breakdown for individual locations. However, out of a total of 209 drinking water samples, 201 were fairly equally divided among Indian Point (tap), Peekskill, Camp Smith and Croton reservoirs. Thus, for 1961, the monthly average activity is comparatively indicative to prior years' data for Figures 6 through 9. Starting with the 1962 environmental monitoring reports, individual graphs were supplied for tap water and the three reservoirs. A history of tap water activity at Indian Point is shown on Figure 9, and Table IX.

As recommended by the New York State Department of health in October, 1959, grab samples from two additional wells were taken for radioactive measurements. Two samples were collected from each well once a month. One well was located on the Indian Point property, while the other was located at Verplanck (approximately 1 1/2 miles south of Indian Point). Samples from both wells

were given to the New York State Department of Health where they performed independent measurements. Sampling of these two wells was discontinued after July, 1962, when the Department of Health indicated that it was no longer interested in these samples. Figure 10 indicates the activity history of the Indian Point well which was reinstated as a representative sample in May, 1963, and Table X gives the collection data. Water sampling for 1960, 1961, and the beginning of 1962 was basically the same as 1959, with the Hudson River, surface, drinking, and well water being sampled. For the period 1958 through 1960 Hudson River water activity data at Indian Point was extracted and averaged from the environmental monitoring reports. The 1961 average were obtained from "sampling points" collected within a 10-mile radius of Indian Point. (Fifty-six river water samples were taken with the Indian Point location accounting for 47 of these samples.) In May, 1962, two new continuous river sampling systems were placed in operation at Indian Point. One of these systems is designed to continuously sample part of the Hudson River water pumped into the station cooling systems. The other system consists of four collection lines traversing the discharge canal and pumps to continuously sample the plant liquid effluents. Both sampling systems collect water at a rate of fifty gallons per week, after which the contents of each collection tank are mixed and samples removed for gross beta-gamma measurements. In November, 1969, after winter freezing prevented river water samples from being taken, submersible pumps were installed to facilitate uninterupted sampling.

This combination inlet-outlet sampling of Hudson River water was combined with the pre-start-up sampling to form one graph depicting the average monthly activity of the river water at Indian Point. Figure 5 shows this activity. Table V lists the number of samples taken prior to continuous sampling. River water was also separated into suspended and dissolved solids prior to 1967. However, with plant liquid discharges being analyzed by gamma spectroscopy, data obtained using the separation technique was not meaningful and therefore has been eliminated. This change has permitted the use of larger samples which has improved the counting accuracy. This started with the April, 1967, water samples. The data used in the Hudson

River water samples (Figure 5) is the combined or total activity as reported in the environmental monitoring reports.

Hudson River water is collected continuously at the condenser inlet and in the discharge canal. An aliquot is removed from each weekly sample and measured for gross beta radio activity.

Tap and well water collected at Indian Point, and water samples collected from Peekskill, Camp Smith and Croton reservoirs was discontinued early in 1966.

Indian Point Lake

Gross beta activity of surface waters collected at Indian Point and other sampling locations was reported in the environmental monitoring reports. The data for Figure 11 was extracted from these surveys. Table XI indicates each year's activity. From 1958 through 1960 the surface water at Indian Point Lake was defined in these reports. The 1961 data was not defined as to which month's samples of activity were taken from the lake. However, the total number of surface water samples and the number of samples taken from the lake is given. Starting in 1961, the data was averaged from the "sampling points". These points are within a 10-mile radius of Indian Point. From 1961 through 1965 Indian Point Lake was included as part of the "surface water sampling program". Over 82% of all the surface water samples from this period ware taken from this location. Starting in March, 1966, a periodic weekly sampling program was started exclusively for beta-gamma activity of Indian Point Lake water. This program is still in existance.

Vegetation

In 1958, 1959 and 1960, although vegetation was sampled at all the sampling stations, only those samples obtained from Indian Point site were included in Figure 12 and Table XII. Vegetation consisted of grasses and

weeds growing on the Indian Point property, and picked during the growing season. From 1962 to 1965 vegetation samples were collected from within a 10-mile radius of Indian Point, and that activity is shown as an average for all the "sampling points".

In addition to the regular vegetation gross beta-gamma measurements, a gamma spectroscopy analysis was made each month on ashed samples collected at four separate on-site locations:

Period	Results
April to September, 1962	Measurable amounts of Niobium 95-Zirconium 95 Ruthinium 106,103, and Cerium 141. Iodine 131 not detected.
May to July, 1963	Same as above
August to October, 1963	Same as above
April to October, 1964	
June to October, 1965	Iodine 131 not detected

Starting with the June, 1965, vegetation samples, raw vegetation was analyzed by the gamma spectroscopy counting system. This was initiated to insure the detection of Iodine 131 which would have been removed during the ashing process. No traces of I-131 were found in the ashed or raw samples. The radionuclides detected in grass were those characteristic of weapons test fallout which was widely distributed in the Northern hemisphere at that time.

Soil

From 1958 through 1961 soil was sampled for gross beta activity only, at all existingg locations. Soil activities at Indian Point were extracted from the environmental reports for the years 1958, 1959 and 1960, averaged and presented in Figure 13 and Table XIII. Starting with the 1962 samples, gross

beta-gamma determinations were made. In addition, one soil sample was taken from each "sampling point" during 1961 and 1962. As these sites are within a 10-mile radius of Indian Point they are included in the soil activity history. Summer collection of soil samples were continued into the operational period. Gamma spectrums were also taken of soil samples. In July, 1965, a composite sample was made up of soil collected at several sampling points in Northern Westchester and Putnam counties; this sample was compared to one taken from Indian Point site. Gamma spectrums of the two samples were identical, and indicated that the isotopes present were due to fallout from nuclear weapons testing in addition to the naturally occurring radio isotopes. Starting with the 1966 soil samples, periodic isotopic analysis was performed. Soil samples were obtained from the same locations as the vegetation samples. The number of samples and their minimum and maximum activity is given on Table XIII.

Biological Samples (Hudson River Fish at Indian Point)

In 1958, fish, crabs, and eels were removed from the Hudson River at Indian Point and measured for radioactivity.

In 1959 and 1960 gross-beta measurements were made of fish and crabs removed from the Hudson River at Indian Point, and frogs removed from the lake at Indian Point. Additionally, in 1960, a periodic system of collection was established in which samples of fish were secured with the assistance of the New York State Department of Health. This program was continued in 1961 and 1962. Figure 14 and Table XIV show the results of the radioactivity measurements made on fish caught in the Hudson River at Indian Point starting with the 1958 samples. Only that data pertaining to fish at Indian Point was used for this bar graph.

Each month from mid 1961 through July, 1964, one fish was collected at random from the Hudson River at the station dock. For the period August 1, 1964, through January 31, 1965, the monthly collection and the method of

sampling was the same as prior years, except that the measurements were reported as "Portions of these fish were prepared by ashing and then the gross beta-gamma activity of the ash was measured and found to vary between 10 and 50 picocuries per gram of the fixed solids".

The February 1, through July 31, 1965, fish data "Collected from the Hudson River at the plant site averaged about 60 picocuries per gram of fixed solids". Biological samples were not reported subsequent to this date.

Algae from Indian Point Lake

Algae and aquatic vegetation is known to be a concentrator of radioactive isotopes such as: iodine, cobalt, and manganese. Therefore, start in 1966 algae was also collected from Indian Point Lake. Samples were collected once each spring, summer and fall (during the growing season). Table XVI also indicates the isotopic content (as determined by gamma spectrametry) of samples obtained from the on-site lake.

The bar graph shown on Figure 15 indicates a gradual decline in activity over the years. In 1968 only two samples were reported, one in May and the other in August. Samples of algae taken from the on site lake measured 160 PCi/gm of Ash, gross beta which is comparable to results obtained in 1967. Spectral analysis made by New York University's Laboratory exhibits peaks of relatively new fallout (less than six months old). Peaks observed include those for $Ce^{144} - Pr^{144}$, and $Zr^{95} - Nb^{95}$.

Hudson River Aquatic Vegetation

Algae samples were collected along the Hudson River shoreline both up and downstream from Indian Point. Only those samples taken 1 mile downstream are included in Figure 16 and Table XVI. Spectral analysis, and gross beta activity was recorded from samples.

In August, 1968, Hudson River Aquatic Vegetation samples were collected at four locations downstream of the plant (1/4, 1/2, 1, and 2 miles). These vegetation samples were measured at New York University Institute of Environmental Medicine using a 4 x 4 inch sodium iodide well crystal and multi-channel analysis. The predominant radionuclide observed was Mn54 in all samples. It is believed that this was attributed to Indian Point releases. Other nuclides identified were K40, found naturally and Cs 137 attributable to nuclear weapons testing.

Hudson River Sediments

Hudson River sediment samples were initially analyzed in May and July, 1966. Samples were taken from the mouth of the discharge canal, 1/2 and 1 mile downstream. Only the 1 mile downstream activity is included in Table XVII and Figure 17. Samples were measured for gross beta radioactivity and a qualitative analysis was made to determine radionuclide content.

Analysis of sediment samples consistently show that naturally occurring K40 (from sea water) is the predominant isotope. Sediment samples were taken at the same locations as the Hudson River vegetation starting with the 1968 samples.

The gross beta samples taken in June, 1969, from 5 locations downstream all show higher levels than previously reported. The manganese 54 and cobalt 60 can be attributed to plant releases while K40 is due to the natural salinity of the water and Cs^{137} partly due to nuclear weapons testing and partly attributable to plant releases.

Background Gamma Radiation

In 1961, a survey was initiated to obtain gross-gamma background measurements at all the 31 sampling stations, and at five on-site enclosed locations. These five sites were changed to the following four permanent

locations once the plant became operational: (1) Control Room; (2) Air Monitor House; (3) Gate House; (4) North Store House. Table XVIII shows the results of these measurements.

In addition measurements of background gamma radiation in the vicinity of Indian Point were made utilizing th rmoluminescent dosimeters. This was started as a trial program in February, 1969, and subsequently, changed to permanent locations at the perimeter of the plant as well as several points off site. Maps delineating sampling locations and thermoluminescent dosimeter locations were previously submitted for insertion in the Unit No. 3 FSAR Environmental Monitoring Program Figures 2.9-1 and 2.9-2, respectively. Table 2.9-1 also previously submitted indicates the routine frequency of sample collections, types of measurements made and instrumentation used for the measurements.

Initial results indicate the average level found is about 10 microroentgens per hour except in and around the air monitor house where the levels are two to three times higher although comparable to readings obtained in the past. These increased levels are due to the natural activity in the building material of which the structure is composed. The thermoluminescent dosimeters are supplied and processed by R. S. Landauer Jr. & Company.

This increased environmental sampling was in preparation for the forthcoming addition of Nuclear Units 2 and 3.

Gamma Road Survey

Additionally, in 1961 gross-gamma radiation measurements were made along the principal roads within a radius of five miles from Indian Point Station. Measurements were made at each road intersection or at 0.1 mile intervals in the absence of intersections. Plates 14, 15 and 16 illustrate the location of the mobile road survey sampling points. The tabulated data on

these plates indicate the range and average in microrems per hour. The number in the center of the circles denotes the average readings from 1963 to 1969. The numbers alongside the circle indicate the minimum and maximum readings taken during this period. The road survey was originally intended for comparison measurements, thus readings were given in counts per second. To convert this to microrems per hour it was necessary to divide counts per second by 2.7 (dividing counts per second by 2.7 gives values in ur/hr based on cobalt 60 calibration). However, starting with the August, 1969, Annual Road Survey, readings were converted from counts per minute to microrems per hour. This change in units was made to permit easier comparison with levels reported by other agencies. A change was also made in the instrumentation used for the survey. In the past, the instrument used was a Universal Atomic Corporation Model 511 Gamma Gun. This has been replaced with a Franklyn Model 15-2 survey meter having a detector consisting of a seven-inch diameter activated Polyvinyltoluene sphere. This instrument is calibrated in a Cesium-137 field of 1 mr/hr reading 2.2×10^6 counts per minute and the minium sensitivity is 1 ur/hr. The primary value in a road survey of this type is to furnish a reference to permit a rapid evaluation in the event of an accidental release of radioactive material, and to detect possible variations in background radioactivity due to releases of noble gases or the consequent build-up due to the cumulative effect of long lived radionuclides.

During the period April to July, 1965, additional measurements were incorporated in the plant systems and in the effluents to determine if concentrations of tritium were present. Subsequently, a routine sampling program was initiated to monitor the tritium concentration in each waste distillate tank prior to discharge as well as the concentration of a composite sample of the blowdown stream. Samples are sent to a consulting laboratory (Isotopes Inc.) where tritium determinations are made. The results of the measurements indicate that the tritium discharge level in the effluents was measurements indicate that the tritium discharge level in the effluents

was less than $\frac{1}{1000}$ of the limits prescribed in 10 CFR 20. Measurements were also made on a weekly basis to test specifically for I^{131} as well as for other isotopes in the air ejector effluent due to the low level leakage from the nuclear boilers. The results of these measurements indicated no measurable amounts of I^{131}. It was found that over 90% of the gaseous radioactivities consisted of Xe^{133} and Xe^{135}, of which Xe^{133} was 60% of the total. The particulate phase of the effluent consisted almost entirely of Cs^{138} (half life of 32 minutes) at a concentration of 3×10^{-7} uc/ml. Cs^{138} is the daughter product of Xe^{138}.

The environmental program was further modified in April 1966 to achieve more meaningful data in view of the experience gained since the pre-operational survey began in 1958 and to include advances and new techniques in the area of radioactive measurements and instrumentation. Samples of air, vegetation, soil and surface waters formerly assayed for gross beta-gamma from 31 sampling points within a 10 mile radius of Indian Point Station were eliminated. Instead, 11 sampling points extending approximately 2 miles in the predominant wind direction and downstream of the plant liquid effluents were substituted to provide samples for spectral analysis as well as gross beta-gamma determinations. Thus, measurements are available to compare the results of the environmental program with the facility effluents to insure compliance with the limits specified in 10 CFR 20, to detect possible increases in the background radiation directly attributable to the operation of the plant, and to provide reference data for Indian Points units 2 and 3.

Monthly isotopic analyses were made of a composite sample made up of aliquots of each hold-up tank prior to discharge and of all other radioactive liquids discharged into the Hudson River. In addition, gross beta-gamma determinations were made. These measurements provide comprehensive information as to the isotopes involved, the fraction of each isotope and the proportional part of the maximum permissible concentrations as specified in 10 CFR 20, Appendix B, Table II, Column 2.

The location of the new sampling points are shown on Figure No. 1.2.1.1-A. Table 1.2.1.1-B gives a composite description of the sampling program as revised in 1966, including the type of samples, frequency and method of collection, analysis made, instrumentation used and their minimum sensitivities. This modified program was continued until early 1969 when the environmental monitoring survey was again expended to include monthly grab samples of drinking water taken from Peekskill, Camp Smith, and Croton reservoirs. In addition indicator species of rooted aquatic vegetation, well water and fish (as well as the media previously examined) was sampled.

Hudson River water at the condenser inlet and discharge canal, fallout, well water, and lake water from three sites was initially analyzed for tritium content. (No tritium was detected on samples analyzed.) One lake is located on-site, (Indian Point Lake) approximately 600 yards east of the Unit No. 1 stack and is about 6,000 square yards in size. This lake has been previously sampled for gross beta activity of its surface water and aquatic vegetation. The other two lakes are new additions - Trap Rock Quarry and Lake Meahagh. In addition to tritium analysis at the lakes, water and aquatic vegetation samples was also measured for gross beta activity.

TABLE 1.2.1.1

Sampling Schedule for Indian Point Unit 1

Continuous Sampling:

(a) Air particulate from four points at Indian Point Station in membrane filters, changed twice each week, sampling rate: 1 cubic foot per minute. Wind direction and velocity recorded at the same time. In addition, starting in February, 1964, a similar collection and measurement of air particulate activity was made at Consolidated Edison's East 74th Stree Generating Station. This program was discontinued in February, 1966.

(b) Fall-out collection at Indian Point and Eastview in pot-type collector. Samples collected in open plastic pails which are emptied once a month.

(c) Air particulate collected at the sampling stations until October, 1965, by means of the AM 2 air monitor in the Mobile Monitor Survey Unit. Sampling rate: 5 cubic feet per minute, Hollingsworth and Vose filter paper monitored continuously and changed at each location. Wind direction and velocity are also recorded.

(d) Hudson River water at Indian Point taken at the inlet after the screenwell house and at the discharge canal. Samples removed from holdup tanks once a week and measured.

(e) Cumlative-gamma background at four points within the station boundary.

Weekly Sampling:

(a) Drinking water from Indian Point Tap, Camp Smith, Peekskill and Croton Reservoirs.

(b) Surface water from Indian Point Lake.

TABLE 1.2.1.1 (Continued)

Monthly Sampling:

(a) Hudson River water from Standard Brands and Sing Sing Prison. Collected by Westchester County Department of Health and shared with Con Edison (Discontinued January, 1966).

(b) Vegetation from four points on the perimeter of the Indian Point Plant. Gamma Spectroscopy analysis preformed.

(c) One marine life sample from Hudson River at Indian Point.

(d) One water sample from the Indian Point well.

Seasonal Sampling:

(a) Vegetation collection from each sampling station in June, August, and September. Gross beta and gamma spectroscopy preformed.

(b) Soil samples taken only once a year (usually in July) unless conditions such as unusually high activities in the other media is observed then soil collections are made on a more frequent basis.

Annual Survey:

Gross-gamma radiation background on roads in the vicinity of the station.

9
8
10
11

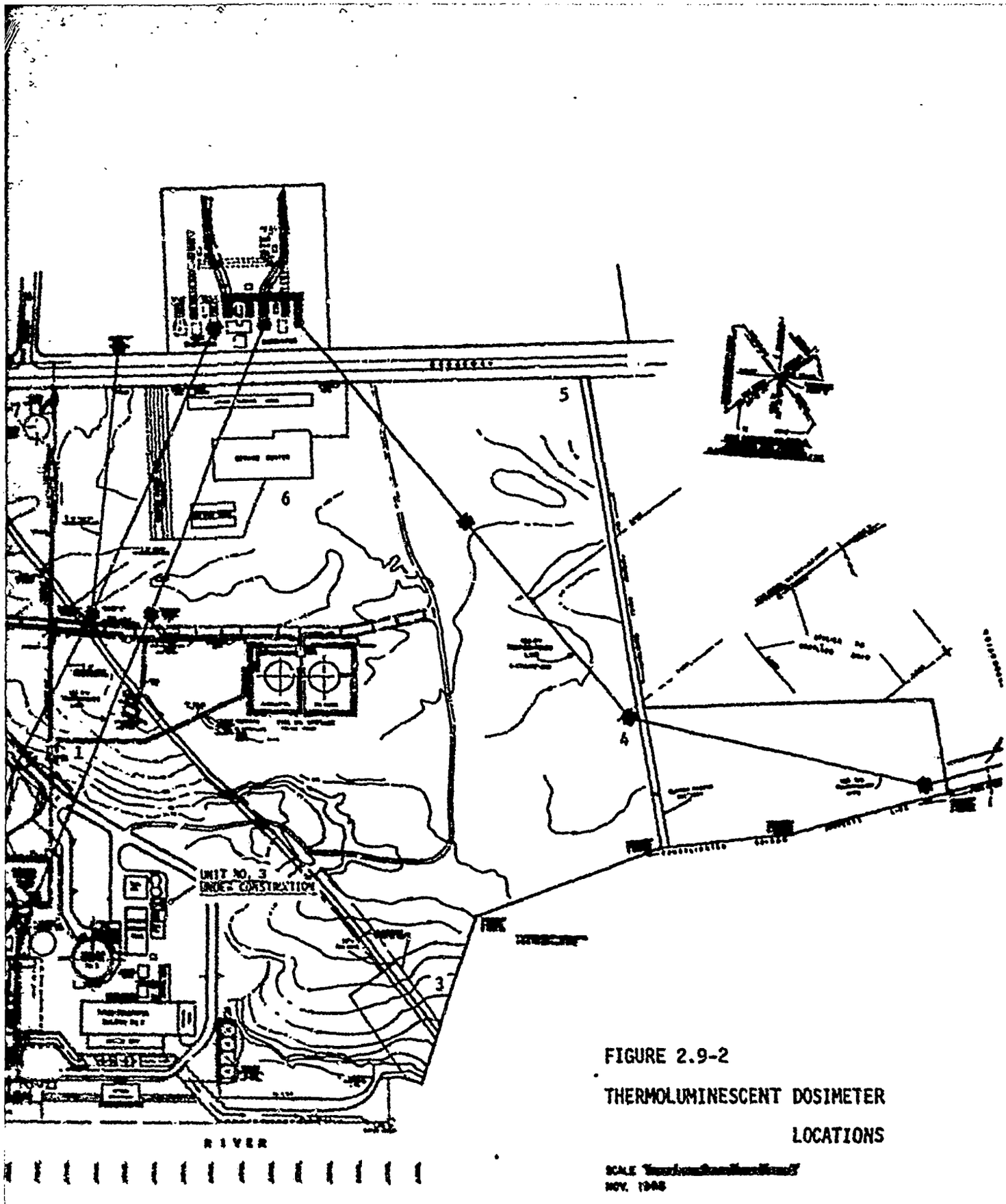

FIGURE 2.9-2

THERMOLUMINESCENT DOSIMETER

LOCATIONS

SCALE [illegible]

NOV. 1985

TABLE 1.2

INDIAN POINT ENV

CONSOLIDATED EDISON CO

No.	Media	Type	Sample Frequency	Method of Collection	Locations	
1	Air	Continuous at 1 CFM	Weekly	Two fixed membrane filters 0.8 microns preceeding charcoal filter	Points 1, 2 in predominant wind direction	Gros
2	Fallout	Continuous	Monthly	Open pot type rain collector	Point 1. Also 15 miles south of plant at Eastview	Gros of s diss
3	Vegetation	Grab	3 per year in Spring, Summer and Fall	Grab samples with 100 ft^2 area	Points 3, 4, 5 and 6	Gros and
4	Soil	Grab	1 per year	Grab samples 2" diameter and 2" deep	Points 3, 4, 5 and 6	Same
5	Lake Water	Grab	Weekly	1 liter sample off-shore	Point 7 - Lake on plant property	Gros
6	Water	Continuous in event of primary to secondary system leakage	Weekly	Continuous flow regulated to fill 50 gallon drums. Representative sample taken once a week and drums emptied.	Hudson River inlet pipe into the plant, and at the plant discharge canal	Same
7	Lake Algae	Grab	One each in Spring, Summer and Fall	Along the lake shore	Point 7	Same

[illegible] SURVEY

[illegible] OF NEW YORK, INC.

[illegible]lysis	Minimum Sensitivities	Measurement Instrumentation	Remarks
[illegible]-gamma [illegible]	0.1 picocurie per cubic meter for gross beta-gamma 2 picocurie per cubic meter for I-131	Gross beta-gamma with gas flow, windowless proportional counter. Gamma spectrum with 3" x 3" sodium iodide crystal. Nuclear Measurement Corp. Type PC 3A Type PC 11A	Measurements made soon after collection and 48 hours later to allow decay of radon-thoron daughters
[illegible]-gamma [illegible]ed and [illegible] solids	1 picocurie per liter	Gas flow, windowless proportional counter for gross beta-gammas Nuclear Measurement Corp. Type PC 3A Type PC 11A	Measurements made 48 hours after collection to allow decay of radon-thoron daughters
[illegible]-gamma [illegible] spectrum	1 picocurie per gram for gross beta-gamma 2 picocurie per gram for I-131	Same as 1 above	Raw vegetation for spectrum soon after collection. Samples ashed and counted 48 hours after collection for gross beta-gamma.
[illegible]	1 picocurie per gram for gross beta-gammas.	Same as 1	Soil dried for spectrum and measured soon after collection. Gross beta-gamma of dried soil made 48 hours after collection.
[illegible]-gamma	Same as 2	Same as 2	Same as 2
[illegible]	Same as 2	Same as 2	Same as 2
[illegible]	Same as 3	Same as 1	Dry weight for spectrum soon after collection. Sample ashed and counted 48 hours after collection for gross beta-gamma.

No.	Media	Type	Sample Frequency	Method of Collection	Locations
8	Hudson River bottom mud	Grab	Same as 7	Along river shore	Points 8, 9, 10, 11, from mouth of discharge canal at 1/2, 1 and 2 miles downstream
9	River Algae	Grab	Same as 8	Same as 8	Points 8, 9, and 10, at 1/2, 1 and 2 miles downstream
10	Direct gamma	Spot readings	Once a year	-	Plates 2, 3, 4 along principal roads within a 5 mile radius of plant
11	Direct gamma	Continuous	Weekly	-	Point 1

Analysis	Minimum Sensitivities	Measurement Instrumentation	Remarks
as 3	Same as 3	Same as 1	Mud dried for both measurements.
as 3	Same as 3	Same as 1	Same as 7
gamma background	2700 counts per second in a CO-60 field of 1 mr/hr Minimum sensitivity 1 ur/hr.	Universal Atomic Corp. Model 511 Gamma Gun using a 2" x 1½" sodium iodide crystal.	Numbers within circles are instrument readings in cps measured at approximately 1/10 mile intervals. Instrument held about 3 ft. above ground level.
gamma background	1 mr	Victoreen Ionization Chambers Model 239 0-10 mr	Average of 3 chambers used.

TABLE I

AIR PARTICULATE COLLECTED AT INDIAN POINT STATION

YEAR	NO. OF SAMPLES	pCi/m^3 AVERAGE	MIN.	MAX.
1958 (June to Dec.)	95	4.37	0.20	96.0
1959	106	2.83	0.10	12.0
1960	92	0.105	0.015	0.3
1961	105	2.17	0.010	26.8
1962	103	5.39	0.60	19.0
1963	103	5.14	0.20	15.0
1964	103	0.73	0.11	2.5
1965	104	0.26	0.1	4.1
1966	70	0.16	0.1	0.50
1967	51	0.13	0.1	0.30
1968	50	0.17	0.10	0.41
1969 (Jan. to July)	30	0.21	0.1	0.87

TABLE V

HUDSON RIVER WATER AT INDIAN POINT

YEAR	NO. OF SAMPLES	GROSS BETA pCi/l AVERAGE	MIN.	MAX.
1958 (July-Dec.)	18	35.3	4	126
1959	39	43.7	6	175
1960	41	15.3	1	60
1961	56 total (47 at Indian Pt.)	29.7	6	150
1962-1970	CONTINUOUS SAMPLING			

TABLE VI

PEEKSKILL RESERVOIR

YEAR	NO. OF SAMPLES	GROSS BETA pCi/ml		
		AVERAGE	MIN.	MAX.
1958 (July to Dec.)	7	20.9	6	62
1959	27	18.1	1.8	65
1960	43	9.9	1.0	36
1961	52	11.3	1.0	290
1962	51	88.3	2.0	130
1963	57	45.2	6.4	115
1964	50	22.4	6.0	64
1965	51	20.1	4.9	70

*

* Reservoir sampling was discontinued until June-July 1969 when samples of drinking water from two reservoirs were obtained and analyzed for gross beta and tritium. However, the gross beta readings taken during this period were later found to be erroneous. No tritium was detected.

TABLE VII

CAMP SMITH RESERVOIR

YEAR	NO. OF SAMPLES	GROSS BETA pCi/ml		
		AVERAGE	MIN.	MAX.
1958 (July to Dec.)	8	31.9	3	80
1959	33	14.3	1.05	58
1960	45	6.0	1.0	25
1961	53	11.3	1.0	290
1962	52	30.4	1.0	160
1963	53	41.0	10.0	135
1964	51	18.1	7.0	50
1965 *	52	16.8	5.0	50

* See Note on Table VI

TABLE VIII

CROTON RESERVOIR

YEAR	NO. OF SAMPLES	GROSS BETA pCi/ml		
		AVERAGE	MIN.	MAX.
1958 (July to Dec.)	10	26.6	9	76
1959	-	-	-	-
1960	40	11.4	1.5	79
1961	43	11.3	1.0	290
1962	39	29.8	2.0	130
1963	43	52.5	15.0	230
1964	40	22.6	7.2	80
1965	41	19.2	5.0	48

*

* See Note on Table VI

TABLE IX

INDIAN POINT TAP WATER

YEAR	NO. OF SAMPLES	GROSS BETA pCi/ml		
		AVERAGE	MIN.	MAX.
1958 (July to Dec.)	8	14.6	3	40
1959	31	13.5	3	43
1960	46	5.4	1	29.5
1961	53	11.3	1	290
1962	52	21.5	3.3	200
1963	52	35.2	2	130
1964	52	18.0	5	60
1965	52	15.9	4	50

*

See Fig. 9 for monthly average.

* See Note Table VI

TABLE X

WELL WATER AT INDIAN POINT

YEAR	COLLECTION DATA
1959	Monthly collections started Nov. and Dec.
1960	Collection changed to two samples per month, with Con Ed keeping one and sending the other to the New York State Dept.of Health. No sampling reported for Feb., March and Nov.
1961	None reported for Feb.
1962	Sampling discontinued after the July samples.
1963	Sampling reinstated on a once-a-month basis. No sampling reported for Jan., Feb.,March, April and Dec.
1964	None reported for Jan. and Feb.
1965	None reported for Jan. and Feb.
1966	Sampling discontinued after the January sample.
1969	Well sampling reinstated, May, June and July 1965.

TABLE XI

INDIAN POINT LAKE WATER

DATE	NO. OF SAMPLES	PCi/l Gross Beta AVERAGE	MIN.	MAX.
1958 (July to Dec.)	19 (From IP Lake)	43.4	4	230
1959	48	19.8	1.8	58
1960	31	8.5	1.4	24
1961	51 Total (41 at IP Lake)	19.3	1	86
		Gross Beta-Gamma		
1962	48 Total (36 at IP Lake)	32.1	2.5	130
1963	45 Total (37 at IP Lake)	90.4	20	200
1964	40 Total (37 at IP Lake)	22.2	5	60
1965	47 Total (41 at IP Lake)	18.4	3	50
1966	39 (From IP Lake)	15.9	0	45
1967	38 (From IP Lake)	7.3	2 ± 4*	15 ± 5
		Gross-Beta		
1968	34 (From IP Lake)	9.3	2 ± 5*	25 ± 10
1969 (April to July)	9 (From IP Lake)	23	4 ± 3	61 ± 5

*Values not statistically significant at 95% confidence level, basis of counting error.

TABLE XII

VEGETATION AT INDIAN POINT

YEAR	LOCATION	NO. OF SAMPLES	GROSS BETA pC_i/gm FIXED SOLIDS
1958	Indian Point		
	July	1	3,120
	Oct.	1	1,170
1959	Indian Point		
	June	1	350
	Aug.	1	730
	Oct.	1	380
1960	Indian Point		
	July	1	230
	Oct.	1	310
	Nov.	1	180

TABLE : XII (CONT'D)

VEGETATION AT INDIAN POINT

		pCi/gm FIXED SOLIDS GROSS BETA		
		AVERAGE	MIN.	MAX.
1961* -				
	No. of Samples			
JUNE	31	273	100	700
Aug	31	207	100	350
Oct	31	1,140	280	4,100
1962 -			Gross Beta - Gamma	
JUNE	31	1,325	350	4,265
AUG	31	1,136	460	2,150
OCT	30	2,018	980	3,700
1963 -				
JUNE	28	2,594	1,010	5,050
AUG	26	2,810	1,070	5,970
OCT	27	1,450	760	3,680
1964 -				
JUNE	20	890	310	3,995
AUG	20	425	235	825
SEPT		410	265	640
OCT	29	399	175	790

* From 1961 through 1965 data extracted from "Sampling Points"

TABLE XII (CONT'D)

VEGETATION AT INDIAN POINT

		NO. OF SAMPLES	GROSS BETA-GAMMA pCi/gm - FIXED SOLIDS		
			AVERAGE	MIN.	MAX.
1965 -					
	JUNE	29	488	300	700
	AUG.	18	419	260	880
	SEPT.	3	366	340	390
	OCT.	9	422	280	540

1966 - "½ Mile Downwind"	pCi/gm Ashed Solids	Isotopic Content	
	MAY	535	*NO ^{131}I
	JULY	310	Ce^{144}-Pr 144, Ru 103, Ru 106-Rh 106, Ba 140-La 140, Cs^{137}, Mn 54, Zr-Nb 95
	OCT.	430	Ce 144, Ru 106, K 40
	NOV.	605	Ba 140-La 140, Zr 95-Nb 95, Ru 103-106, Ce 141-144, I 131, Nd 147
1967 - "½ Mile Downwind"			
	APRIL	260	Ce 144, Ru 106, Zr-Nb 95, K 40
	JULY	325	K 40
	OCT.	330	-
1968 - "½ Mile Downwind"			
	MAY	375	Traces of Ce^{144} and Pr 144
	JUNE	90	Traces of Ce^{144} and Pr 144
	JULY	150	Traces of Ce^{144} and Pr 144
	OCT.	350	No identifiable Radionuclides revealed in the gamma spectral analysis made on this sample
1969 - "½ Mile Downwind"		200	No identifiable isotopes.

* MAY 1966 sample was analyzed for ^{131}I only and was found to be below the minimum detectable level of 2 pCi/gm for the instrumentation used.

TABLE XIII

SOIL SAMPLES COLLECTED AT INDIAN POINT & VICINITY

YEAR	LOCATION	NO. OF SAMPLES	GROSS BETA pCi/gm AVERAGE	MIN.	MAX.
1958	Indian Point				
	JULY	1		56	
	OCT.	1		155	
1959	Indian Point		AVERAGE	MIN.	MAX.
	JUNE	2	40	26	54
1960	Indian Point	1		28.5	
1961	"Sampling Points"		AVERAGE	MIN.	MAX.
	JULY	20	36.7	7	160
	AUG.	11	34.0	18	70
			GROSS BETA-GAMMA		
1962	"Sampling Points"		AVERAGE	MIN.	MAX.
	JULY	31	36.6	7	70
1963	"Sampling Points"				
	JULY	30	78.3	26	140
1964	"Sampling Points"				
	JUNE	4	86.2	44	130
	JULY	21	72.2	40	94
1965	"Sampling Points"				
	JULY	28	57.4	25	96

TABLE XIII (Cont'd)

SOIL SAMPLES COLLECTED AT INDIAN P[illegible]T & VICINITY

		pCi/gm	Isotopic Content
1966	"½ Mile Downwind"		
	JUNE	45	Ce 144-Pr 144 Pm 147, Cs 137, K 40
1967	"½ Mile Downwind"		
	JUNE	30	K 40, Cs 137, Ce 144-Pr 144
1968	"½ Mile Downwind"	95	K 40 the predominant Radionuclide, with traces of fission fragments from fall out.
1969	"½ Mile Downwind"	40	K 40, Cs 137, Ce 144-Pr 144

TABLE XIV

Biological Samples (Hudson River Fish at Indian Point)

YEAR		TYPE	GROSS BETA ACTIVITY pCi/gm Fixed Solids (1)
1958	SEPT.	Striped Bass	88±12
	OCT.	Perch	179±15
1959	APRIL	Shad	160±30
1960	JAN.	Hudson River Fish	80±15
	APRIL	"	35±15
	JULY	"	55±20
	SEPT.	"	35±20
1961	FEB.	"	50±10
	MARCH	"	60±10
	APRIL	"	25±10
	MAY	"	45±10
	JUNE	"	70±10
	*JULY	"	40±10
	AUG.	"	20±10
	SEPT.	"	45±10
	OCT.	"	35±10
	NOV.	"	30±10
	DEC.	"	40±10
1962	JAN.	"	50±10
	FEB.	"	65±10
	MARCH	"	36±10
	APRIL	"	32±10
	MAY	"	93±10
	JUNE	"	90±12
	JULY	"	22±10
	AUG.	"	22±10
	SEPT.	"	60±10
	OCT.	"	80±10
	NOV.	"	NONE
	DEC.	"	30±5

*Average of three readings

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TABLE XIV (Cont'd)

Biological Samples (Hudson River Fish at Indian Point)

YEAR		TYPE	Gross Beta-Gamma
1963	JAN.	Hudson River Fish	30±5
	FEB.	"	45±5
	MARCH	"	70±10
	APRIL	"	80±10
	MAY	"	70±10
	JUNE	"	20±10
	JULY	"	55±10
	AUG.	"	20±10
	SEPT.	"	35±10
	OCT.	"	30±10
	NOV.	"	60±10
	DEC.	"	55±10
1964	JAN.	"	15±10
	FEB.	"	30±10
	MARCH	"	25±20
	APRIL	"	13±15
	MAY	"	16±15
	JUNE	"	15±25
	JULY	"	15±25

			pCi/gm of Ash Gross Beta
1969 **	MAY	Shad	61±11
	JUNE	Sunfish	5±8
	JULY	Striped Bass	13±10

Spectral analysis of the 1969 fish samples showed no identifiable isotopes.

** Average of two readings

(1) The fish flesh is ashed at 600°C. three-hundred milligrams of the ash is placed in an aluminum plachet. It is then acidified with IN HNO_3, and a paste formed which upon drying, forms a uniform sample thickness. Each operation described is preceded by weighing to determine the percentage of dry and fixed solids. The sample is then counted in the proportional counter, and activity report4d as PCi/gm of fixed solids.

TABLE XV

GROSS BETA-GAMMA ACTIVITY OF ALGAE COLLECTED FROM LAKE AT INDIAN POINT

DATE	pCi/gm ASHED SOLIDS	ISOTOPIC CONTENT
1966 (May)	305	* No I 131
1966 (July)	135	Ce 141-144, Ba 140-La 140, Ru103-106, Cs 137, Zr95-Nb 95, Mn 54
1966 (August)	195±10	Ce 141-144, Ba 140-La 140, Ru 103-106 Cs 137, Zr 95-Nb 95, Mn 54
1967 (June)	70±10	Mn 54, Cs 137, Ce 144
1967 (July)	190±15	Ce 144, Ru 106, Cs 134, Cs 137, Mn 54
1967 (August)	75±15	Same radionuclides as found in previous samples
1968 (May)	160	Ce-Pr 144, Zr-Nb 95, Ce 141, Ru 103-106
1968 (August)	115	Ce 144, Ru 106, Cs 137, Mn 54
1969 (June)**	117	Zr 95, Nb 95, Ru 106, Ce 144

* May sample analyzed for I-131 only
** Average of two readings

TABLE XVI

HUDSON RIVER AQUATIC VEGETATION
1 MILE DOWNSTREAM

DATE	pCi/gm Ashed Solids	Isotopic Content
1966 (June)	140	Co 58, Co 60, Mn 54
1966 (July)	360	Co 58, Co 60, Mn 54
1966 (August)	90±10	Co 58, Co 60, Mn 54
1967 (June)	65±10	Mn 54, Cs 137, Ce 144
1967 (July)	50±10	Ce 144, Co 58, Cs 134, Cs 137, Mn 54, Co 60.
1967 (August)	40±15	Same radionuclides as found in previous samples
1968 (July)	30±15	No spectral analysis made
1968 (August)	not reported	Mn 54, K 40, Cs 137
1969 (June)	9±8	Mn 54, Cs 137

TABLE XVII

HUDSON RIVER SEDIMENT
1 MILE DOWNSTREAM

DATE	pCi/gm	ISOTOPIC CONTENT
1966 (May)	15	K 40, Mn 54, Ce 144-Pr 144
1966 (July)	5	K 40, Mn 54, Ce 144-Pr 144
1966 (October)	26±9	K 40, Mn 54, Ce 144-Pr 144,
1967 (January)	9±9*	K 40, Mn 54, Ce 144, -Pr 144
1967 (June)	10±10*	K 40, Mn 54, Cs 137, Ce 144
1967 (October)	10±10*	K 40 predominant radionuclide, trace quantities of Co 60
1968 (May)	±10	K 40 principal radionuclide, Cs 137 and Ce 144
1968 (July)	25±15	" " " " " "
1968 (October)	10±5	K 40, Cs 137, Ce 144, Mn 54
1969 (June)	209±12	Mn 54, Co 60, K 40, Cs 137

*Result is not statistically significant at 95% confidence level, the basis of the counting errors.

TABLE XVIII

GROSS GAMMA BACKGROUND RADIATION

MEASURED WITHIN INDIAN POINT STATION

Measuring Period	Measuring Point	Reading mr/hr	Avg. for Meas. Period mr/hr
1/61 - 12/61	Indian Point Station	.013	.013
1/62 - 1/63	Central Control Room	.02	.021
	Air Monitor House	.03	
	Gate House	.02	
	North Store House	.02	
2/63 - 7/63	Central Control Room	.02	.021
	Air Monitor House	.03	
	Gate House	.02	
	North Sto. House	.02	
8/63 - 1/64	Central Control Room	.03	.03
	Air Monitor House	.04	
	Gate House	.03	
	North Store House	.02	
2/64 - 7/64	Central Control Room	.02	.03
	Air Monitor House	.05	
	Gate House	.03	
	North Store House	.02	
8/64 - 1/65	Central Control Room	.03	.03
	Air Monitor House	.05	
	Gate House	.02	
	North Store House	.02	
1/65 - 7/65	Central Control Room	.03	.0325
	Air Monitor House	.05	
	Gate House	.03	
	North Store House	.02	
8/65 - 1/66	Central Control Room	.02	.0325
	Air Monitor House	.06	
	Gate House	.03	
	North Store House	.02	
2/66 - 7/66	Air Monitor House	.05	.05

TABLE XVIII (Cont'd)

GROSS GAMMA BACKGROUND RADIATION

MEASURED WITHIN INDIAN POINT STATION

Measuring Period	Measuring Point	Reading mr/hr	Avg. for Meas. Period mr/hr
8/66 - 1/67	Air Monitor House	.04	.04
2/67 - 7/67	Air Monitor House	.04	.04
8/67 - 1/68	Air Monitor House	.04	.04
2/68 - 7/68	Air Monitor House	.04	.04
8/68 - 1/69	Air Monitor House	.04	.04
2/69 - 7/69	Air Monitor House	.04	.04

TABLE XIX

ATMOSPHERIC NUCLEAR WEAPONS TESTING

YEAR	MONTH	SITE	
		United States	
1958	April - August	Hardtack, Eniwetok Proving Grounds	(1)
	Sept. - Oct.	Hardtack Phase II, Nevada Testing Site	(1)
	Aug. - Sept.	Argus, South Atlantic	(1)
		United Kingdom	
1958	April - Sept.	Grapple, Christmas Island Pacific Ocean	(1)
		France	
1960	Feb. - April	Sahara Desert	(1)
	December	Sahara Desert	(1)
1966	July	Moruroa, 750 mi. S.E. of Tahite	(2)
		Soviet Union	
1958	Feb. - March		(1)
	Sept. - Nov.		(1)
1961	Sept. - Oct.		(1)
1965	November	Semipalatinsk Region	(2)
1966	February	Semipalatinsk Region	(2)
	March	Semipalatinsk Region	(2)
	April	Semipalatinsk Region	(2)
	October	Semipalatinsk Region	(2)
1967	February	Semipalatinsk Region	(2)

TABLE XIX (Cont'd)

YEAR	MONTH	SITE	
		Mainland China	
1967	December	LOP nor Site	(2)
1968	December	LOP nor Site	(2)
1969	September	Tien Shan Mts., LOP nor Site	(2)

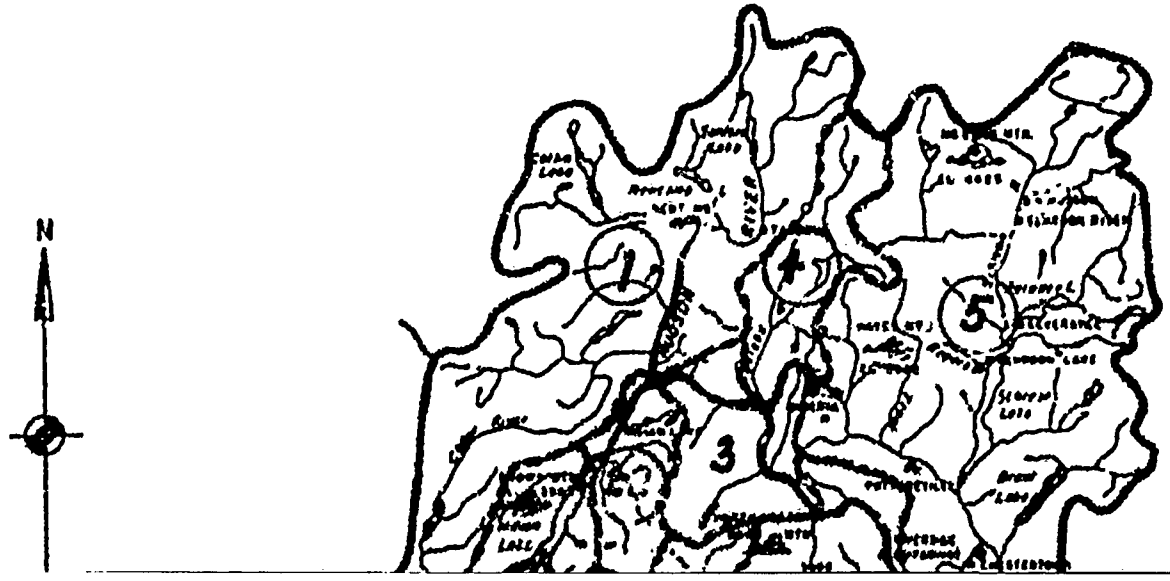
(1) "ENVIRONMENTAL RADIOACTIVITY"
by Merril Eisenbud 1963
P. 305

(2) RADIOLOGICAL HEALTH - DATA AND REPORTS
Volumes 5 through 11
(1964 - 1970)

TABLE XIX

(Cont'd)

YEAR	MONTH	SITE	
		Soviet Union (Cont'd)	
1967	April	Semipalatinsk Region	(2)
	October	Semipalatinsk, Novaya Zemlya	(2)
1968	April	Semipalatinsk	(2)
	June	Semipalatinsk	(2)
	July	North of Caspian Sea	(2)
	November	Novaya Zemlya	(2)
	December	Semipalatinsk	(2)
1969	May	Semipalatinsk	(2)
	July	Semipalatinsk	(2)
	September	Southern Ural Area, Semipalatinsk, South of Volgograd	(2)
	November	Semipalatinsk	(2)
	December	Semipalatinsk, Kazakh Desert	(2)
1970	January	Semipalatinsk	(2)
		Mainland China	
1964	October	—	(2)
1965	May	—	(2)
1966	May	—	(2)
	October	LOP nor Site	(2)
	December	LOP nor Site	(2)
1967	June	LOP nor Site	(2)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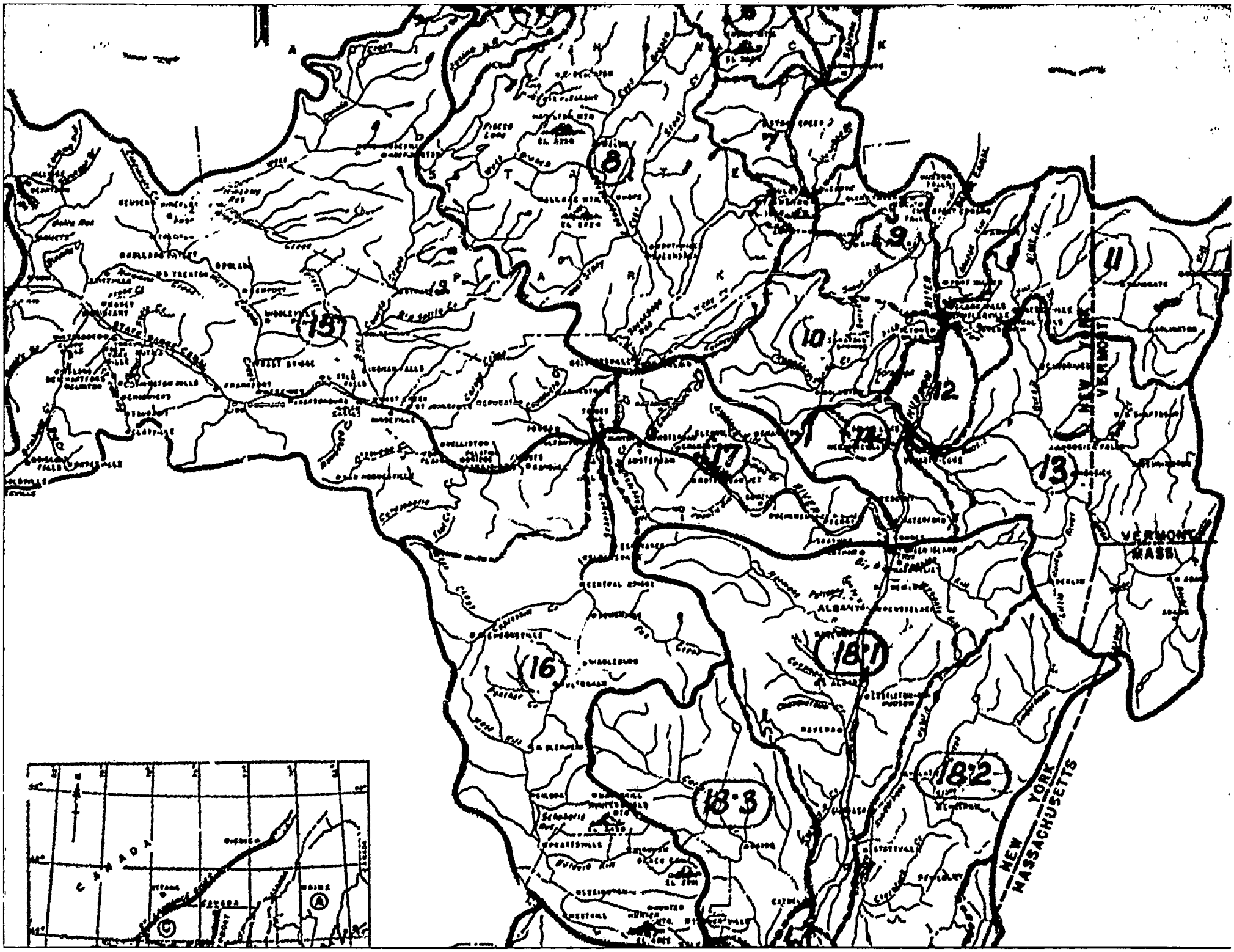
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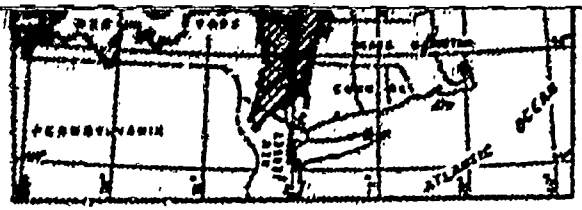
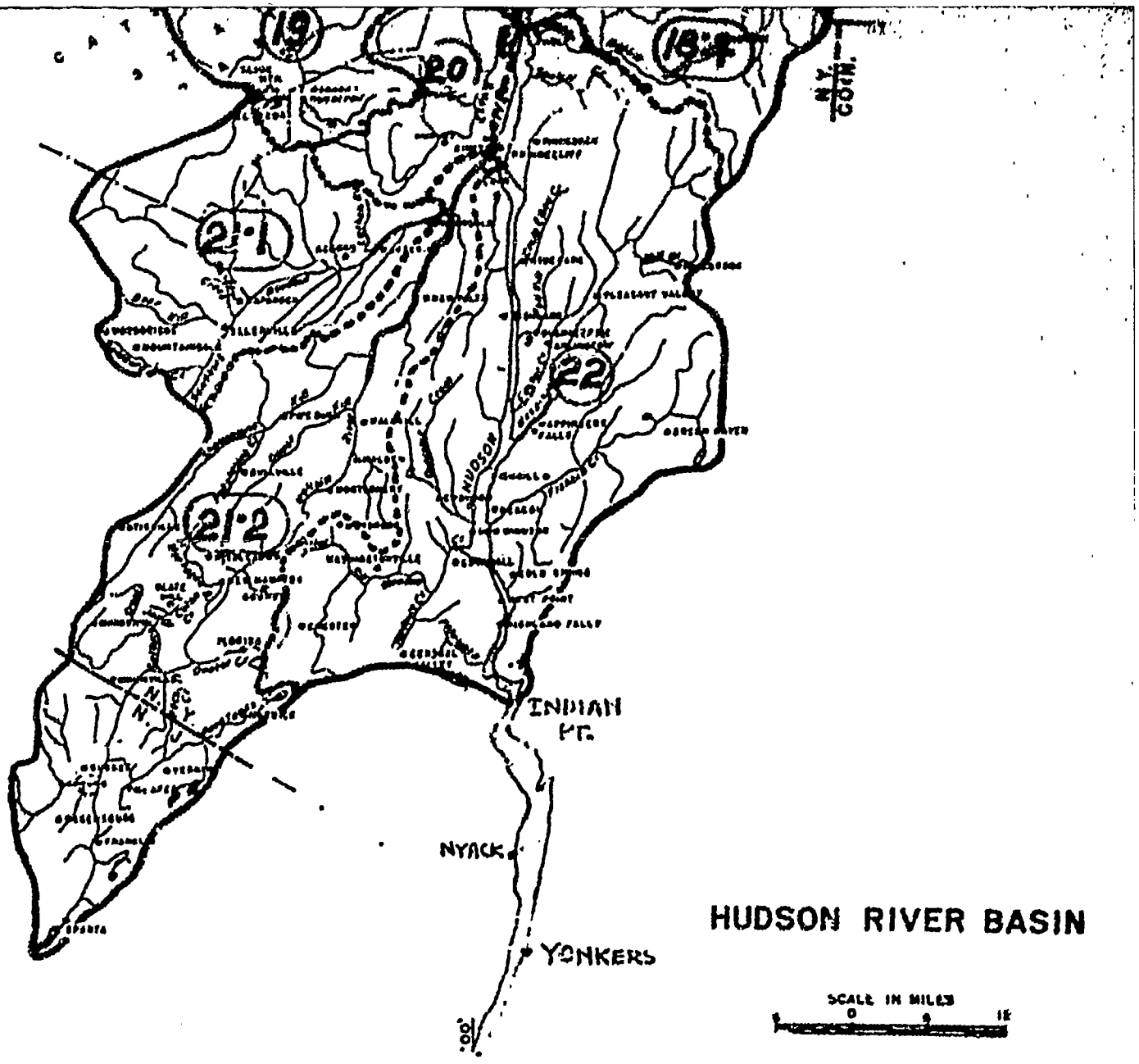


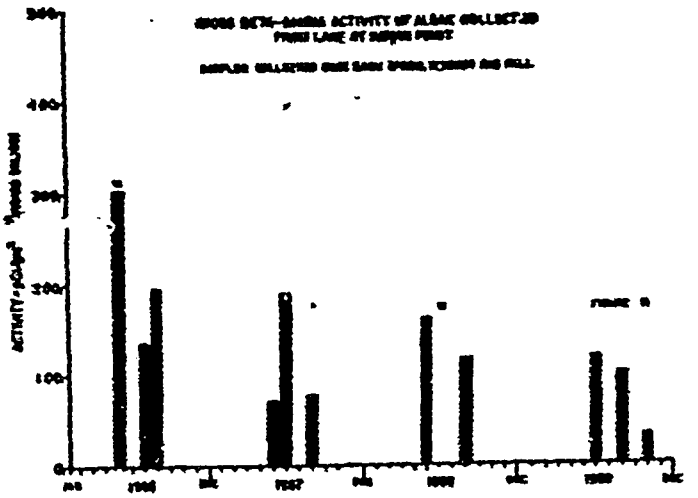
LOCATION MAP

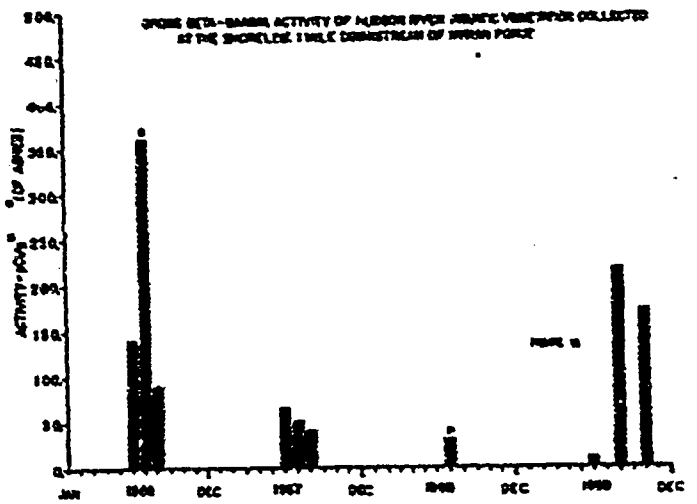
SCALE IN MILES

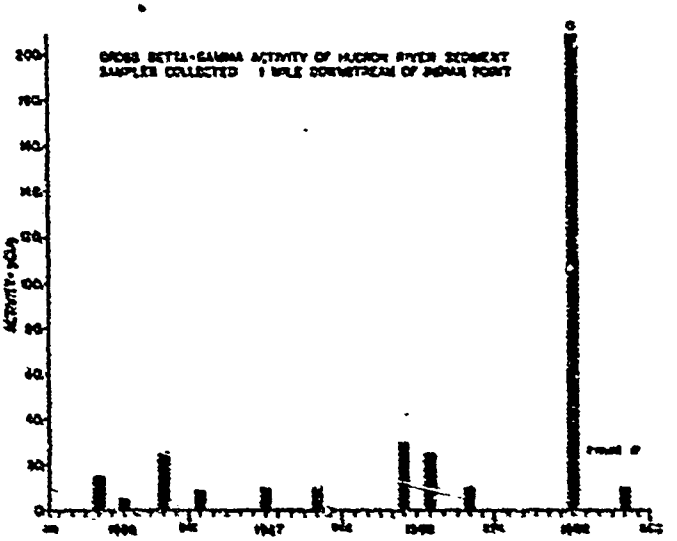


HUDSON RIVER BASIN

SCALE IN MILES







Letters on Figures Re
to Explanatory Notes
Pages 47 to 50 .

The Following Notes Pertain To Figures 1 Through 17.

(A). Intensive atmospheric nuclear weapons testing by both the United States and Russia in October 1958. Fallout from these tests was reflected in marked increases in the gross beta activity of air, water, soil and vegetation samples.

(B). Fallout resulting from the resumption of atmospheric weapons testing reversed a declining trend in the amount of background radioactivity. This is readily apparent from the results of measurements on media collected after September 1961 and is in agreement with measurements made by other agencies in this geographic location.

(C). Increase in activity (fallout) attributed to the transfer of aged radionuclides (due to high yield atmospheric weapons testing) from the stratosphere to the troposphere.

(D). Spectral analysis of the fallout samples showed predominantly fresh fission products. This increase is attributed to the Chinese atmospheric nuclear weapons testing in October and December 1966 and the increase in rainfall just prior to the January 1967 collection period.

(E). The salinity of the Hudson River increases progressively from the spring to fall of each year. This salt front pushes its way upriver, thus salinity content increases seasonally.

(F). An examination of measurements taken in the period February 1, 1963 through July 31, 1963 indicates that atmospheric fallout is still the dominating influence in most samples. In addition the samples from non-flowing surface water sources are still increasing in radioactivity from the accumulation of the longer lived fallout radionuclides.

(G). Values of Potassium 40 found in the Hudson River water samples increased in the draught years 1964-1966 when low precipitation conditions increased the sea water intrusion. Thus giving a higher activity.

(H). In 1962, and early 1963 air particulate, fallout and water samples showed higher average values than those obtained in 1961.

(I). This slightly higher activity reflects the increase due to the Chinese atmospheric nuclear weapons testing of December 1967.

(J). Effects of atmospheric nuclear weapons testing.

(K). Intermediate lived fresh fission products characteristic of weapons testing fallout such as Niobium-95, Ruthinium-106 and 103, Zirconium-95 and Cerium-141 were found in the 1963 vegetation samples. In addition, meteorological conditions for the latter half of 1963 was characterized by an unusual lack of precipitation.

(L). Samples of drinking water in 1961 were obtained from locations within a ten-mile radius of Indian Point. The following is a breakdown of that years drinking water data:

NO. OF SAMPLES	GROSS BETA ACTIVITY pCi/l		
	Minimum	Maximum	Average
209	less than 1	286	10

(M). The May 1966 samples of algae collected from the Indian Point lake was only analyzed for Iodine-131 and none was detected. A complete spectrum analysis to detect the presence of other isotopes was not performed until the July samples were taken. The July samples showed predominately fresh fission products characteristic of weapons testing. In addition, the May fallout samples taken at Indian Point and Eastview showed an increase of gross beta gamma levels of approximately eight times the average.

(N). Only two samples reported in 1958.

(O). Algae is known to be a concentrator of radioactive isotopes such as Iodine, Cobalt and Manganese. The samples of green slime scraped from Hudson River shore rocks collected at three sampling points, up to 2 miles downstream of the discharge canal, indicate the presence of Co58, Co60 and Mn54 in slightly higher concentrations than may be expected from fallout or other sources.

(P). Dredging operations in connection with the construction of Units 2 and 3 have affected algae growth to the extent that only one month (July 1968) sample was collected and analyzed.

(Q). Samples of bottom sediment were collected in the discharge canal and at four locations near the shoreline at various distances downstream of the plant. These were measured for gross beta radioactivity and a qualitative analysis made to determine radionuclide content. The gross beta radioactivity of these samples for 1968 is higher than the levels reported in the previous period and several are beyond the range of levels found in recent years which ranged from 10 to 120 picocuries per gram. Manganese-54 and Cobalt-60 can be attributed to plant releases while Potassium-40 is due to the natural salinity of the water and Cs 137 partly due to nuclear weapons testing and partly attributable to plant releases.

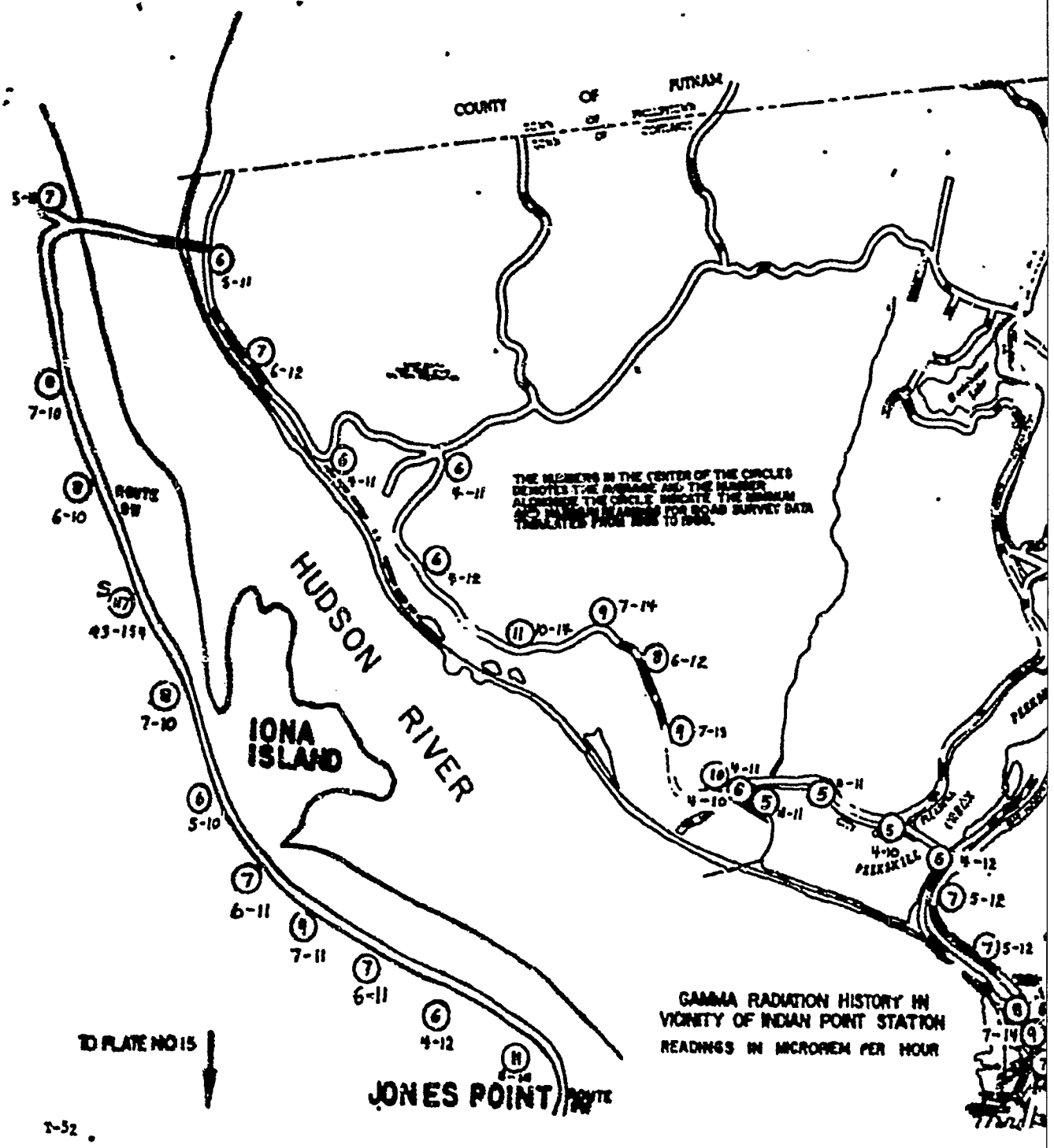
(R). Of the ten well water samples collected once each month, from Indian Point in 1964, the following data was tabulated:

	GROSS BETA - GAMMA ACTIVITY pCi/l		
Collection Month	Suspended Solids	Dissolved Solids	Total Activity
March	2 ± 3	2 ± 4	4 ± 5
April	3 ± 3	5 ± 4	8 ± 5
May	10 ± 4	5 ± 4	15 ± 6
June	50 ± 3	30 ± 4	80 ± 5
July	5 ± 3	5 ± 4	10 ± 5
August	10	6	16
September	5	10	15
October	3	3	6
November	5	5	5
December	5	5	10

Precipitation data (1) at four locations within a fifteen mile radius of Indian Point indicates that the summer months of 1964 were ones of severe draught conditions.

(S). This one isolated area, approximately 2 miles South of the Bear Mountain Bridge along Rout 9W is more than 10 times higher than other nearby sampling points. This level has remained consistantly high since readings were first taken and is believed attributable to a vein of uranium ore.

(1) Data obtained from U.S. Department of Commerce, Weather Bureau Climatological data for New York Annual Summary 1964, Volume 76, No. 13.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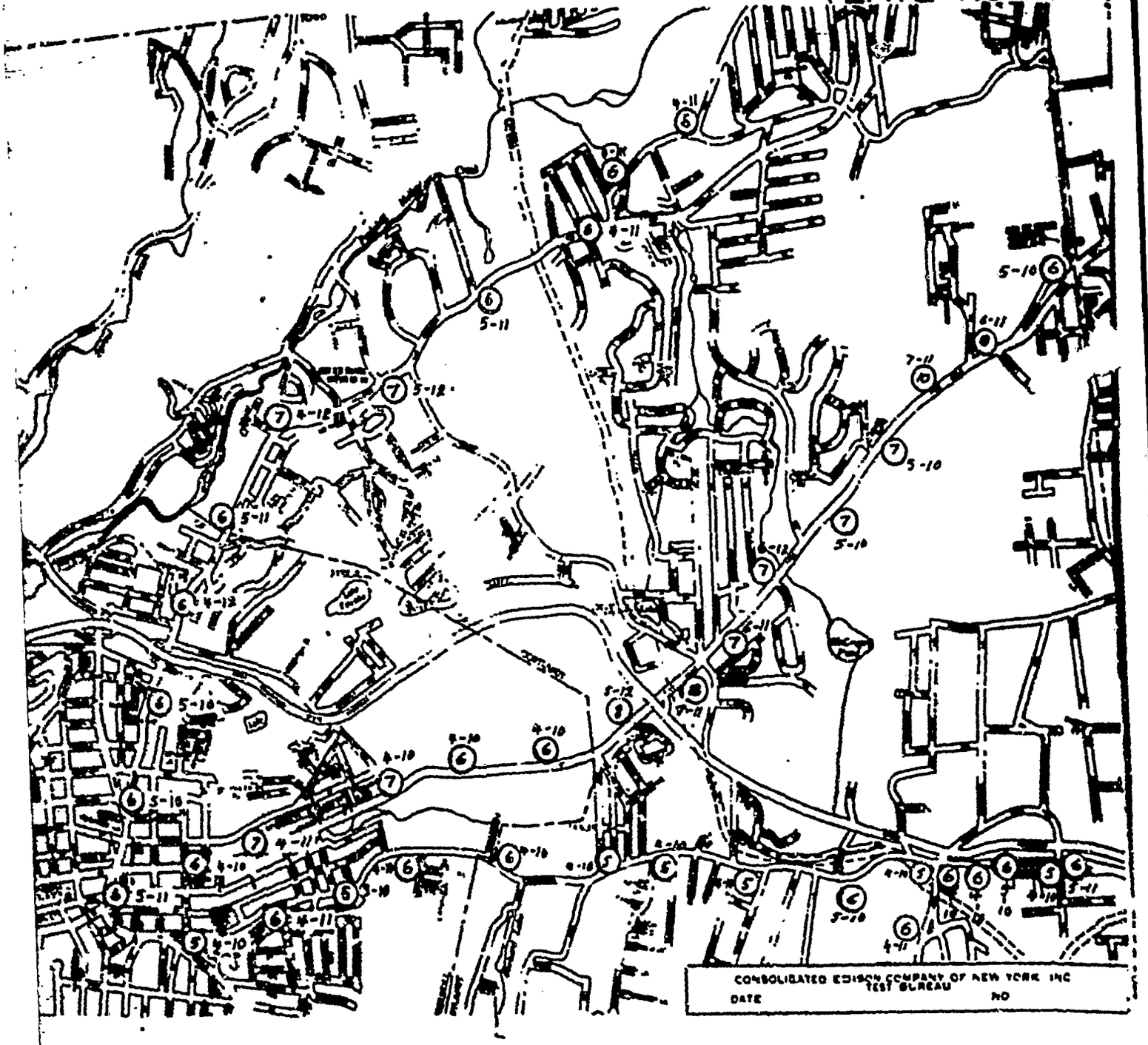
THE NUMBERS IN THE CENTER OF THE CIRCLES DENOTES THE AVERAGE AND THE NUMBER ALONGSIDE THE CIRCLE INDICATE THE MINIMUM AND MAXIMUM READINGS FOR ROAD SURVEY DATA TABULATED FROM 1959 TO 1966.
TO PLATE NO 15
GAMMA RADIATION HISTORY IN VICINITY OF INDIAN POINT STATION
READINGS IN MICROREM PER HOUR

PLATE NO 14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TEST BUREAU
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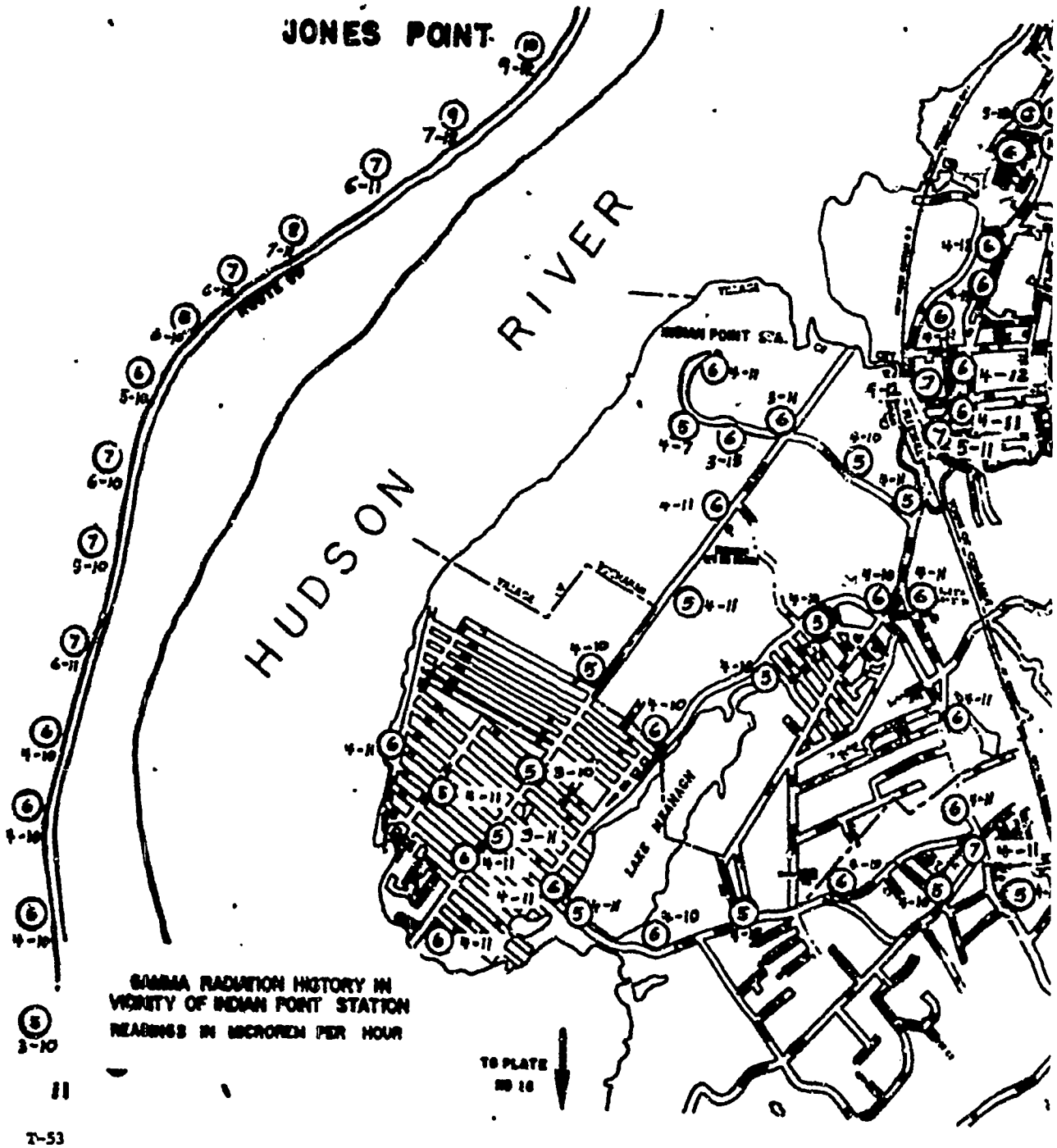
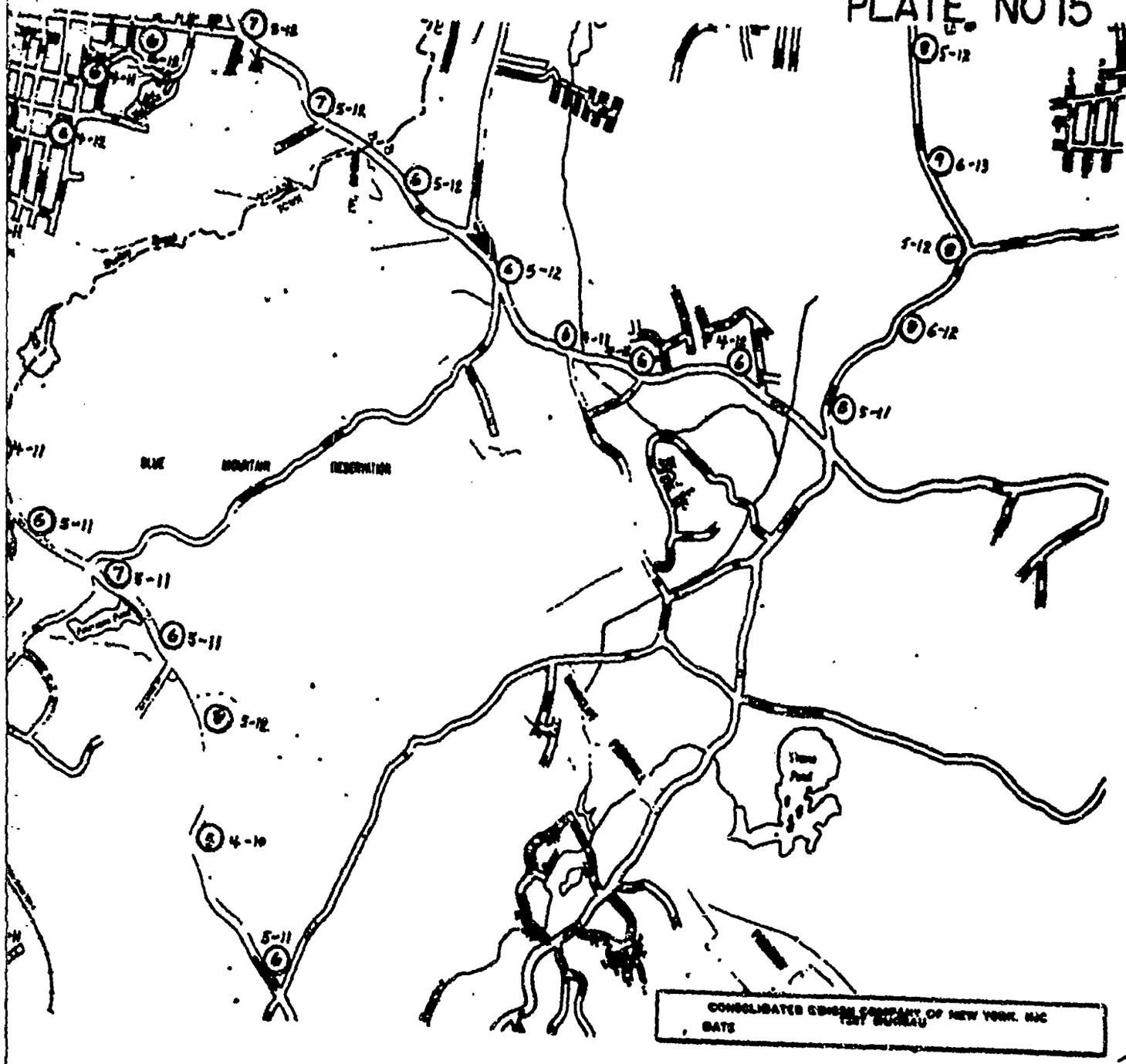
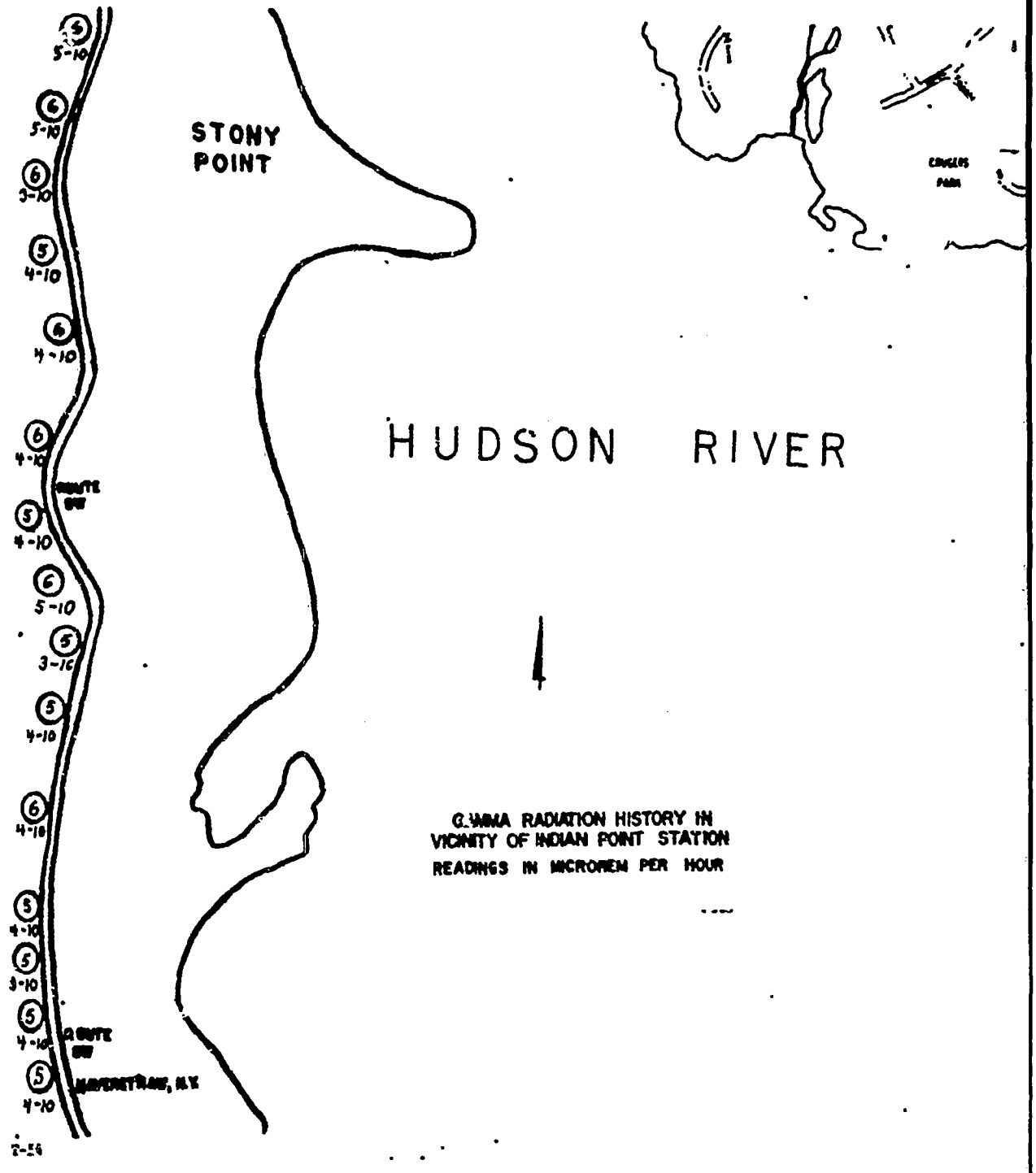


GAMMA RADIATION HISTORY IN VICINITY OF INDIAN POINT STATION
READINGS IN MICROREM PER HOUR

T-53

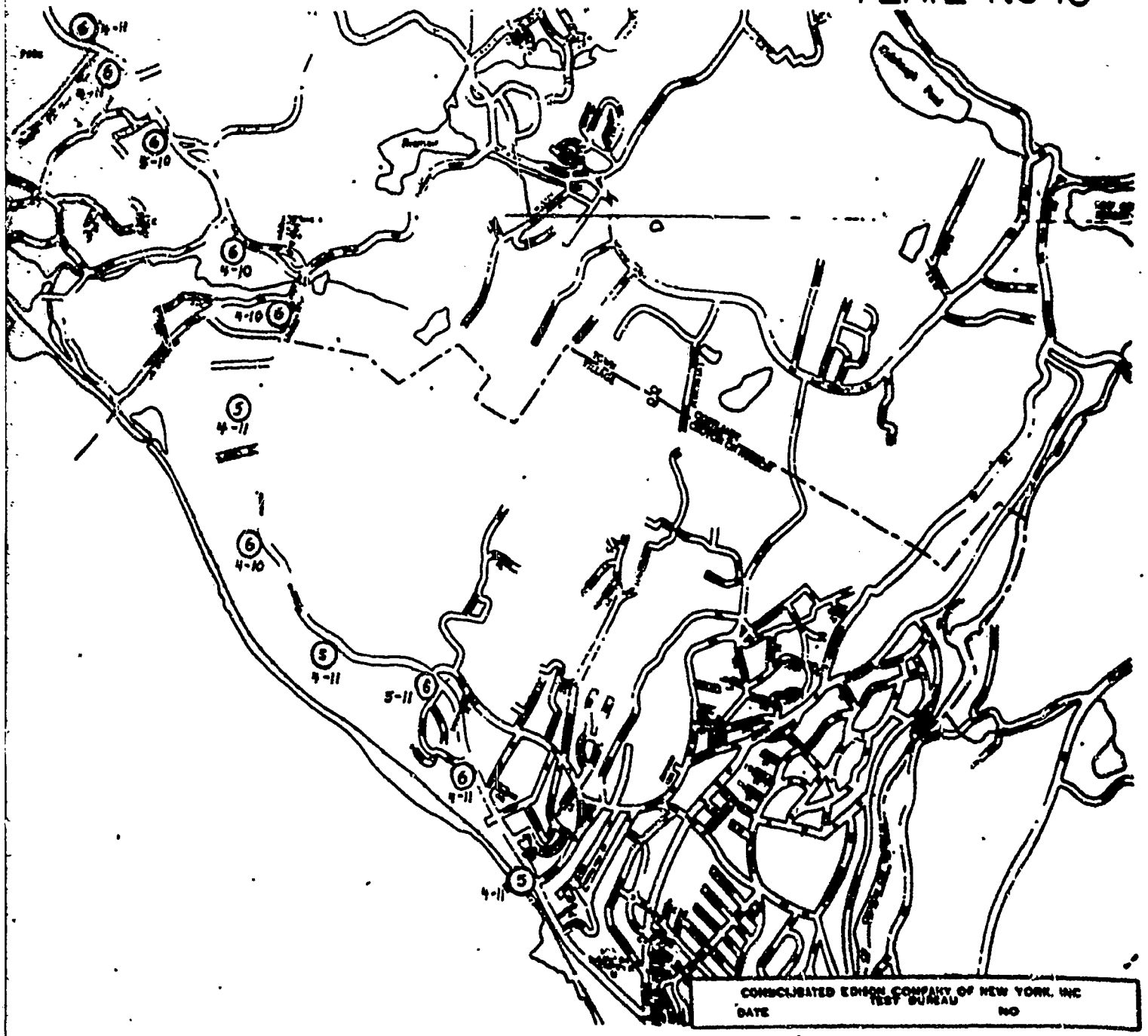
PLATE NO 15


BLUE MOUNTAIN RESERVATION
CONSOLIDATED EDISON COMPANY OF NEW YORK, INC.
GAS



GAMMA RADIATION HISTORY IN
VICINITY OF INDIAN POINT STATION
READINGS IN MICROREM PER HOUR

PLATE NO 16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1.2.2 Indian Point Unit No. 2

1.2.2.1 Pre-Operational and Operational Program

The environmental monitoring program for Unit No. 2 is essentially the same as the present environmental monitoring program for Unit No. 1. Recently, this survey has been expanded to include tritium in water measurements, marine life, station and off-site well water, and a change in location of the on-site air monitoring station. One of the original air monitoring locations was changes from the roof of the water meter house east of the stack to the roof of the service building which is downwind of the stack. The outer original location, the roof of the air monitor house remains intact. In addition, three off-site points were selected to serve as air monitors for the new nuclear units. Two sites are located approximately 0.4 and 0.9 miles south of Indian Point Plant respectively and a third site located at Standards Brands immediately North of the Indian Point.

1.2.3 Indian Point Unit No. 3

1.2.3.1 Pre-Operational and Operational Program

The pre-operational program for Unit No. 3 is the same as Unit No. 2. It is expected that the operational program for Unit No. 3 will reflect the knowledge and experience gained by the implementation of the environmental program on Indian Point Unit No. 2.

1.2.4 Indian Point Station Existing Facilities

As previously submitted in the Unit No. 3 FSAR, Section 2.9 "Environmental Monitoring Program", the following areas were covered in detail; personnel monitoring, personnel protective equipment, facilities and access provisions, radiation instrumentation and medical facilities.

1.2.5 Compliance with 10CFR20, Section 106 (e) Requirements

The environmental monitioring program conducted by Consolidated Edison supplies sufficient data to determine the necessity for evaluating whether or not additional monitoring is needed to determine the applicability of the provisions of Section 20, 106 (e). In particular, the applicant in its routine environmental program monitors air, water, sediments, terestrial and aquatic vegetation, soil and selected aquatic biota. The monitoring provides an indication of the cumulative amounts of radioactivity in the environment, and instrumentation is sufficiently sensitive to detect buildup at levels far below those specified in Section 20, 106 (e).

The results of the environmental monitoring program is reported on a semi-annual basis to the Atomic Energy Commission who are thereby advised of the short and long-term trends in the environment. In addition, discharges of radioactive liquids and gases are reported semi-annually to the AEC and New York State Department of Health and 10CFR20 makes provision for rapid reporting of any unusually high releases.

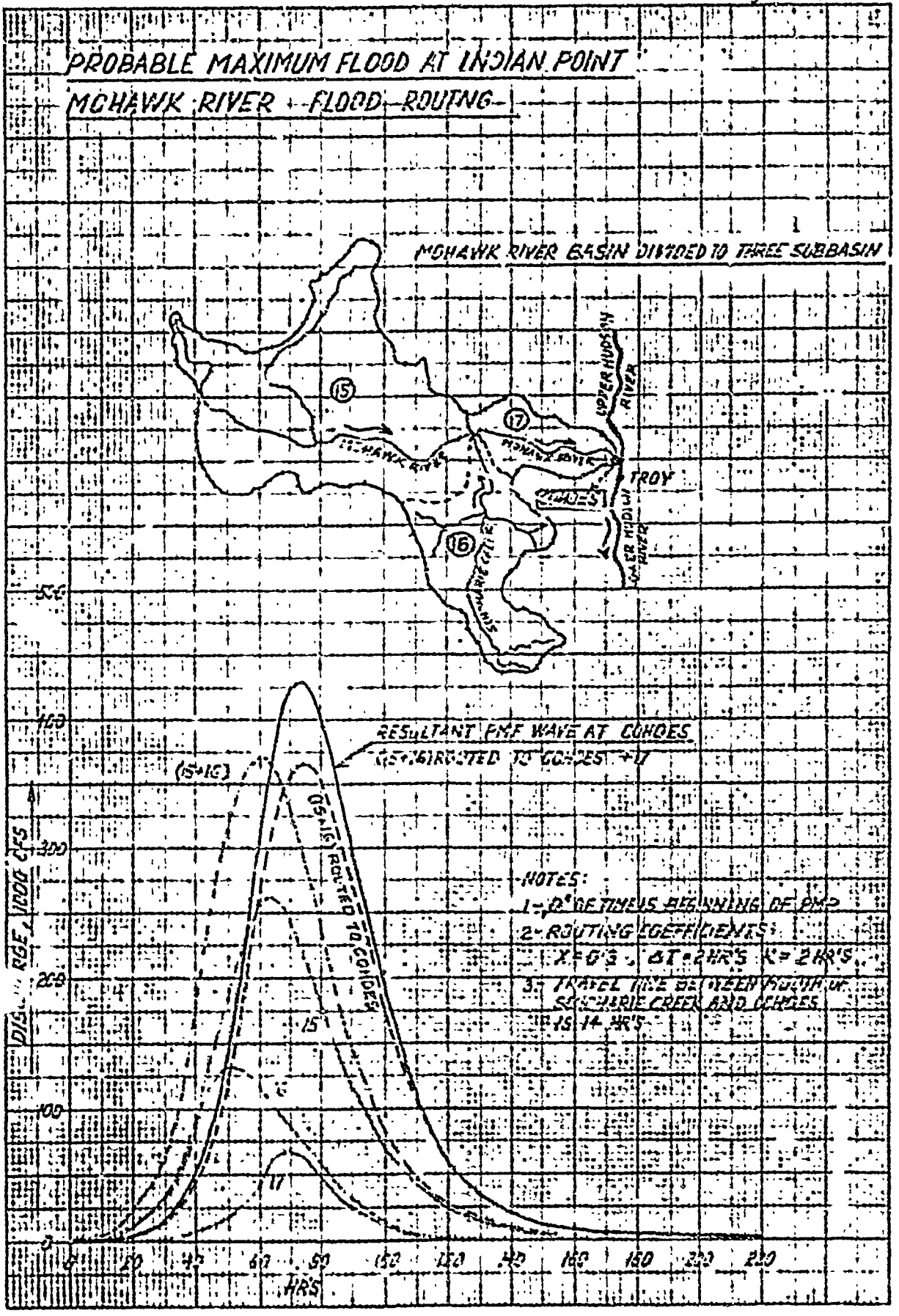
Whenever the monthly release in liquid discharges, exclusive of tritium, is greater than five curies, a report is required by agreement to be sent to the New York State Department of Health, Bureau of Radiological Health. Accordingly, the New York State Department of Health would be make aware of any significant increase in liquid discharges and their own monitoring programs can be accordingly adjusted to provide greater intensity of monitoring if necessary. It is expected that this procedure will be incorporated in the nuclear Units 2 and 3 program.

Finally, the Institute of Environmental Medicine of New York University Medical Center conducts a research program of the Hudson River concerned both with evaluation of the toral ecology and of radionuclides in relation to the ecology. The results of these studies are reported on an annual basis to the New York State Department of Health, the Department of the Interior, The U.S. Public Health Service, the Consolidated Edison Company, the Hudson River Valley Commission, the New York State Conservation Department, the New York City Environmental Protection Agency, the New York City Board

of Water Supply and the New York State Power Development association and other interested organizations.

Although not a monitoring program, the results of the studies are germane as they bear upon the manner in which radionuclides behave in the river ecosystem. Summary reports are made quarterly to Consolidated Edison who support a part of the cost of the research program on the Hudson River. In addition, results are made available rapidly through close personal cooperation.

Figure III-21



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 excesses (Table III-9), unit hydrograph ordinates (Figure III-21) above flows for the three subbasins.

The solution printouts including the flood hydro-

MOHAWK RIVER
FLOOD HYDROGRAPH PROGRAM

```
LIST

PMF        09:17   RDS3  JUN 08 78 MON

 50 DIM P(12),G(100),Q(3,140),A(140),O(140)
 70 READ F
 80 FOR Y=1 TO 3
100 READ S
110 READ B
115 READ K
120 FOR I=1 TO 12
140 READ P(I)
160 NEXT I
200 FOR J=1 TO K
210 READ G(J)
220 NEXT J
240 PRINT"UNIT HYDROGRAPH  -  SUBBASIN";S
250 PRINT
260 PRINT "HR'S ; 1000CFS"
270 PRINT
280 FOR J=1 TO K
290 PRINT 2*J;"";G(J),
300 NEXT J
301 PRINT
302 PRINT
303 PRINT
320 FOR I=1 TO12
340 FOR J=1 TO K
360 LET N=J+((I-1)*3)
380 LET Q(Y,N)=Q(Y,N)+(P(I)*G(J))
400 NEXT J
420 NEXT I
500 FOR N=1 TO 140
510 LET Q(Y,N)=Q(Y,N)+B
520 NEXT N
600 PRINT "PMF, SUBBASIN : " S
605 PRINT
607 PRINT "BASE FLOW:" B,"1000CFS"
608 PRINT
610 PRINT " HR'S;  1000CFS"
620 PRINT
632 FOR N=1 TO K+50
640 PRINT 2*N;"";Q(Y,N),
650 NEXT N
660 PRINT
663 PRINT
664 PRINT
666 PRINT
680 NEXT Y
810 FOR N=1 TO 140
820 LET A(N)=Q(1,N)+Q(2,N)
830 NEXT N
840 PRINT "PMF  -  SUBBASINS 15+16"
845 PRINT "   HR'S + 1000CFS"
850 FOR N=1 TO 100
860 PRINTA(N)
```

```
870 NEXT N
1000 LET X=0.30
1010 LET T=2.0
1020 LET D=2.0
1030 LET C1=(2*D)/((2*T*(1-X))+D)
1040 LET C2=(D-(2*T*X))/((2*T*(1-X))+D)
1045 LET R=1
1050 LET O(1)=A(1)
1060 FOR N=2 TO 140-R
1070 LET O(N)=C1*(A(N-1)-O(N-1))+C2*(A(N)-A(N-1))+O(N-1)
1075 NEXT N
1080 FOR N=1 TO 140-R
1085 LET A(N)=O(N)
1086 NEXT N
1090 LET R=R+1
1092 IF R>7 GOTO 1100
1093 GO TO 1050
1096 PRINT
1097 PRINT
1100 PRINT"PMF FFROM SUBBASINS 15+16 ROUTED TO COHOES"
1110 PRINT
1120 PRINT"HR'S          1000CFS"
1130 FOR N=1 TO 100
1140 PRINT  2*N,A(N)
1150 NEXT N
1160 PRINT
1170 PRINT
1180 FOR N=1 TO 137
1190 LET A(N)=A(N)+Q(3,N)
1195 NEXT N
1200 PRINT "          MOHAWK RIVER AT COHOES  -  PMF "
1210 PRINT
1220 PRINT               "HR'S ) 1000CFS"
1230 PRINT
1240 FOR N=1 TO 137
1250 PRINT                   2*N,A(N)
1260 NEXT N
1270 PRINT
1280 PRINT
1290 PRINT
```

PLATE III-2
(SHEET 1 OF 1)

MOHAWK RIVER
FLOOD HYDROGRAPH PROGRAM
DATA

```
2000 DATA 3,15,2.17,72                    SUBBASIN # 15
2010 DATA 0.00,0.46,0.60,0.24
2020 DATA 1.19,3.85,1.39,0.91             RUNOFF DATA
2030 DATA 0.0,0.06,0.10,0.01
2100 DATA 0.60,1.20,2.05,3.50
2110 DATA 6.00,9.60,13.00,17.00
2120 DATA 20.10,22.60,25.00,27.40
2130 DATA 30.00,32.00,33.60,33.90
2140 DATA 34.00,33.80,32.60,30.20
2150 DATA 27.80,26.00,24.29,22.80
2160 DATA 21.00,19.20,17.40,16.00
2170 DATA 14.00,12.40,10.80,9.50          UNIT HYDROGRAPH DATA
2180 DATA 8.40,7.40,6.60,6.00
2190 DATA 5.60,5.05,4.60,4.05
2200 DATA 3.60,3.20,2.90,2.65
2210 DATA 2.40,2.15,1.95,1.80
2220 DATA 1.60,1.35,1.15,1.00
2230 DATA 0.85,0.70,0.60,0.50
2240 DATA 0.40,0.35,0.27,0.22
2250 DATA 0.18,0.15,0.12,0.10
2260 DATA 0.08,0.06,0.04,0.02
2270 DATA 0.01,0.00,0.00,0.00
2280 DATA 16,0.926,40                     SUBBASIN # 16
2290 DATA 0.00,0.48,0.61,0.25
2300 DATA 1.29,3.92,1.47,0.98             RUNOFF DATA
2310 DATA 0.03,0.09,0.14,0.04
2400 DATA 1.20,2.80,4.70,7.20
2410 DATA 10.40,14.40,17.10,17.50
2420 DATA 17.35,17.00,16.30,15.60
2430 DATA 14.85,14.10,13.25,12.40
2440 DATA 11.70,10.90,10.10,9.40          UNIT HYDROGRAPH DATA
2450 DATA 8.65,7.95,7.35,6.70
2460 DATA 6.00,5.40,4.80,4.20
2470 DATA 3.60,3.00,2.50,2.00
2480 DATA 1.60,1.30,1.00,0.75
2490 DATA 0.50,0.30,0.15,0.00
2500 DATA 17,0.37,44                      SUBBASIN # 17
2510 DATA 0.00,0.46,0.60,0.24
2520 DATA 1.45,3.89,1.64,1.00             RUNOFF DATA
2530 DATA 0.06,0.13,0.18,0.08
2600 DATA 0.02,0.05,0.08,0.10
2610 DATA 0.15,0.20,0.30,0.40
2620 DATA 0.50,0.68,1.00,1.40             UNIT HYDROGRAPH DATA
2630 DATA 1.90,3.00,5.00,7.20
2640 DATA 9.30,9.70,9.70,9.30
2650 DATA 8.50,7.70,6.80,5.90
2660 DATA 5.10,4.40,3.80,3.20
2670 DATA 2.60,2.10,1.65,1.25
2680 DATA 0.95,0.75,0.55,0.40
2690 DATA 0.30,0.25,0.20,0.15
2700 DATA 0.10,0.06,0.02,0.00
9000 END
```

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

$$O_2 - O_1 = \left[\frac{2\,\Delta t}{2k(1-x)+\Delta t}\right]\left[I_1 - O_1\right] + \left[\frac{\Delta t - 2kx}{2k(1-x)+\Delta t}\right]\left[I_2 - I_1\right] \qquad \text{III-2}$$

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

PLATE III-3
(SHEET 1 OF 10)
MOHAWK RIVER
FLOOD HYDROGRAPH PROGRAM
PRINTOUT

PMF 09:10 RDS3 JUN 08 78 MON

UNIT HYDROGRAPH - SUBBASIN= 15

HR'S ; 1000CFS

2 ; .6	4 ; 1.2	6 ; 2.85	8 ; 3.5	10 ; 6
12 ; 9.6	14 ; 13	16 ; 17	18 ; 20.1	20 ; 22.6
22 ; 25	24 ; 27.4	26 ; 30	28 ; 32	30 ; 33.6
32 ; 33.9	34 ; 34	36 ; 33.8	38 ; 32.6	40 ; 30.2
42 ; 27.8	44 ; 26	46 ; 24.25	48 ; 22.8	50 ; 21
52 ; 19.2	54 ; 17.4	56 ; 16	58 ; 14	60 ; 12.4
62 ; 10.8	64 ; 9.5	66 ; 8.4	68 ; 7.4	70 ; 6.6
72 ; 6	74 ; 5.6	76 ; 5.05	78 ; 4.6	80 ; 4.25
82 ; 3.6	84 ; 3.2	86 ; 2.9	88 ; 2.65	90 ; 2.4
92 ; 2.15	94 ; 1.95	96 ; 1.8	98 ; 1.6	100 ;
1.35	102 ; 1.15	104 ; 1	106 ; .85	108 ;
.7	110 ; .6	112 ; .5	114 ; .4	116 ;
.35	118 ; .27	120 ; .22	122 ; .18	124 ;
.15	126 ; .12	128 ; .1	130 ; .08	132 ;
.06	134 ; .04	136 ; .02	138 ; .01	140 ;
0	142 ; 0	144 ; 0		

UNIT HYDROGRAPH - SUBBASIN= 16

HR'S ; 1000CFS

2 ; 1.2	4 ; 2.8	6 ; 4.7	8 ; 7.2	10 ; 10.4
12 ; 14.2	14 ; 17.1	16 ; 17.5	18 ; 17.35	20 ; 17
22 ; 16.3	24 ; 15.6	26 ; 14.85	28 ; 14.1	30 ; 13.25
32 ; 12.4	34 ; 11.7	36 ; 10.9	38 ; 10.1	40 ; 9.4
42 ; 8.65	44 ; 7.95	46 ; 7.35	48 ; 6.7	50 ; 6
52 ; 5.4	54 ; 4.8	56 ; 4.2	58 ; 3.6	60 ; 3
62 ; 2.5	64 ; 2	66 ; 1.6	68 ; 1.3	70 ; 1
72 ; .75	74 ; .5	76 ; .3	78 ; .15	80 ; 0

PMF: SUBBASIN : 15

BASE FLOW: 2.17 1000CFS

HR'S: 1000CFS

2 : 2.17	4 : 2.17	6 : 2.17	8 : 2.446	10 : 2.722
12 : 3.113	14 : 4.14	16 : 5.65	18 : 7.816	20 : 10.394
22 : 13.878	24 : 17.668	26 : 21.92	28 : 26.738	30 : 31.575
32 : 39.125	34 : 47.73	36 : 58.2865	38 : 70.967	40 : 88.888
42 : 108.182	44 : 127.861	46 : 148.774	48 : 168.502	50 : 185.79
52 : 203.843	54 : 219.599	56 : 234.375	58 : 246.776	60 : 256.68
62 : 268.215	64 : 268.586	66 : 259.417	68 : 253.785	70 : 242.75
72 : 231.556	74 : 219.838	76 : 206.888	78 : 194.582	80 : 181.66
82 : 167.148	84 : 153.595	86 : 141.731	88 : 127.927	90 : 116.14
92 : 104.66	94 : 93.654	96 : 84.232	98 : 75.756	100 :
67.7829	102 : 61.226	104 : 55.5715	106 : 50.0255	108 :
45.459	110 : 40.9875	112 : 36.8589	114 : 33.356	116 :
38.737	118 : 27.5185	120 : 24.984	122 : 22.513	124 :
20.3577	126 : 18.5497	128 : 16.7683	130 : 14.91	132 :
13.3597	134 : 12.0295	136 : 10.6671	138 : 9.4419	140 :
8.4991	142 : 7.489	144 : 6.5787	146 : 5.9832	148 :
5.3857	150 : 4.7532	152 : 4.3641	154 : 3.9803	156 :
3.6486	158 : 3.4128	160 : 3.1615	162 : 2.9549	164 :
2.7788	166 : 2.6232	168 : 2.5866	170 : 2.4876	172 :
2.3453	174 : 2.2987	176 : 2.2582	178 : 2.2292	180 :
2.2123	182 : 2.1975	184 : 2.1925	186 : 2.1878	188 :
2.1842	190 : 2.1837	192 : 2.1778	194 : 2.175	196 :
2.1728	198 : 2.1716	200 : 2.1784	202 : 2.1702	204 :
2.1701	206 : 2.17	208 : 2.17	210 : 2.17	212 :
2.17	214 : 2.17	216 : 2.17	218 : 2.17	220 :
2.17	222 : 2.17	224 : 2.17	226 : 2.17	228 :
2.17	230 : 2.17	232 : 2.17	234 : 2.17	236 :
2.17	238 : 2.17	240 : 2.17	242 : 2.17	244 :
2.17				

PMF, SUBBASIN : 16

BASE FLOW: .926 1000CFS

HR'S; 1000CFS

2 : .926	4 : .926	6 : .926	8 : 1.502	10 : 8.27
12 : 3.182	14 : 5.114	16 : 7.626	18 : 10.705	20 : 13.826
22 : 16.37	24 : 19.213	26 : 22.865	28 : 25.637	30 : 28.6605
32 : 36.691	34 : 46.404	36 : 58.1395	38 : 72.2335	40 : 86.677
42 : 103.879	44 : 117.772	46 : 123.759	48 : 128.949	50 : 132.026
52 : 131.199	54 : 130.145	56 : 127.46	58 : 122.391	60 : 116.87
62 : 110.045	64 : 104.329	66 : 98.0955	68 : 91.9215	70 : 86.4105
72 : 80.5165	74 : 74.6855	76 : 69.344	78 : 63.672	80 : 57.879
82 : 52.6105	84 : 47.265	86 : 42.034	88 : 37.153	90 : 32.3705
92 : 28.0595	94 : 23.9835	96 : 20.3525	98 : 17.2635	100 :
14.3085	102 : 11.7455	104 : 9.43	106 : 7.531	108 :
5.9905	110 : 4.546	112 : 3.781	114 : 3.1395	116 :
2.508	118 : 2.15	120 : 1.8515	122 : 1.576	124 :
1.449	126 : 1.348	128 : 1.253	130 : 1.173	132 :
1.1085	134 : 1.046	136 : 1.008	138 : .977	140 :
.946	142 : .938	144 : .932	146 : .926	148 :
.926	150 : .926	152 : .926	154 : .926	156 :
.926	158 : .926	160 : .926	162 : .926	164 :
.926	166 : .926	168 : .926	170 : .926	172 :
.926	174 : .926	176 : .926	178 : .926	180 :
.926				

PMF, SUBBASIN : 17

BASE FLOW: .37 1000CFS

HR'S; 1000CFS

2 : .37	4 : .37	6 : .37	8 : .3792	10 : .393
12 : .4068	14 : .428	16 : .469	18 : .51	20 : .5728
22 : .656	24 : .7392	26 : .9158	28 : 1.1785	30 : 1.476
32 : 1.9468	34 : 2.858	36 : 4.2312	38 : 5.842	40 : 7.9335
42 : 9.8082	44 : 11.945	46 : 14.25	48 : 15.683	50 : 17.4534
52 : 20.359	54 : 24.2328	56 : 28.9188	58 : 35.9165	60 : 44.0314
62 : 52.6536	64 : 61.4785	66 : 64.7604	68 : 66.9784	70 : 67.993
72 : 65.6984	74 : 63.1934	76 : 59.775	78 : 54.317	80 : 48.9864
82 : 43.613	84 : 38.457	86 : 33.6179	88 : 29.0275	90 : 25.0751
92 : 21.5662	94 : 18.345	96 : 15.5785	98 : 13.3055	100 :
11.112	102 : 9.1894	104 : 7.6563	106 : 6.3355	108 :
5.203	110 : 4.2735	112 : 3.468	114 : 2.8174	116 :
2.2373	118 : 1.7995	120 : 1.4979	122 : 1.2213	124 :
.9895	126 : .833	128 : .705	130 : .6075	132 :
.5436	134 : .5047	136 : .472	138 : .4456	140 :
.4236	142 : .408	144 : .3963	146 : .3856	148 :
.378	150 : .3748	152 : .3716	154 : .37	156 :
.37	158 : .37	160 : .37	162 : .37	164 :
.37	166 : .37	168 : .37	170 : .37	172 :
.37	174 : .37	176 : .37	178 : .37	180 :
.37	182 : .37	184 : .37	186 : .37	188 :
.37				

RUN

PMF 13:39 MON MAY 13, 1970

KOHAWK RIVER - PMF

SUBBASINS 15+16 - PMF

HR'S	1000CFS
2	3.096
4	3.096
6	3.096
8	3.948
10	4.992
12	6.295
14	9.254
16	13.276
18	18.521
20	24.22
22	30.248
24	36.881
26	44.765
28	52.375
30	60.236
32	75.816
34	94.139
36	116.340
38	143.2
40	174.685
42	212.061
44	244.633
46	272.533
48	297.452
50	317.319
52	334.242
54	349.744
56	361.835
58	369.167
60	372.878
62	370.26
64	364.037
66	357.512
68	345.706
70	329.146
72	312.073
74	294.543
76	276.154
78	258.254
80	239.543
82	219.739

84	200.06	[illegible]
86	183.765	[illegible]
88	165.06	
90	148.544	
92	132.719	
94	117.637	
96	[illegible]4.564	
98	93.0195	
100	82.0114	
102	72.9715	
104	65.0015	
106	57.5565	
108	51.4495	
110	45.5335	
112	40.6619	
114	36.4955	
116	32.845	
118	29.6665	
120	26.8355	
122	24.069	
124	21.6067	
126	19.6917	
128	18.0213	
130	16.083	
132	14.4662	
134	13.0775	
136	11.6751	
138	10.4189	
140	9.4451	
142	8.427	
144	7.5027	
146	6.9092	
148	6.2217	
150	5.6792	
152	5.2961	
154	4.9063	
156	4.5746	
158	4.3366	
160	4.0673	
162	3.8609	
164	3.7043	
166	3.5492	
168	[illegible]	
170	[illegible]	
172	3.2715	
174	[illegible]	
176	3.1[illegible]	
178	3.1552	
180	3.1363	
182	3.1235	
184	3.1165	
186	3.1136	
188	3.1102	
190	3.1067	
192	3.1038	
194	3.101	
196	3.0936	
198	3.0976	
200	3.0964	

PMF FROM SUBBASINS 15+16 ROUTED TO CONOES

HR'S	1000CFS
2	3.096
4	3.096
6	3.096
8	3.096
10	3.0961
12	3.09733
14	3.10[illegible]
16	3.1[illegible]
18	3.31676
20	3.72169
22	4.49435
24	5.76603
26	7.81307
28	10.7666
30	14.6661
32	19.4709
34	24.9792
36	31.1[illegible]
38	37.9749
40	45.6453
42	54.7164
44	66.2097
46	81.0554
48	99.7102
50	122.415
52	149.11
54	176.963
56	210.036
58	240.215
60	267.759
62	292.096
64	313.004
66	330.639
68	345.196
70	356.212
72	363.324
74	366.304
76	365.265
78	360.542
80	352.335
82	340.855
84	326.696
86	310.712
88	293.607
90	275.617
92	257.553
94	238.954
96	220.29
98	201.874
100	183.332
102	166.46
104	149.83

FLOW FROM [illegible] ROUTED TO [illegible] (CONTINUED)

106	134.182
108	119.7
110	106.505
112	94.6072
114	83.9793
116	74.5675
118	66.2465
120	58.8846
122	52.3776
124	46.6479
126	41.647
128	37.3006
130	33.5103
132	30.1745
134	27.208
136	24.5649
138	22.2122
140	20.0916
142	18.1445
144	16.3503
146	14.7122
148	13.2214
150	11.866
152	10.6499
154	9.55909
156	8.58734
158	7.7371
160	7.00221
162	6.36633
164	5.8287
166	5.37292
168	4.90047
170	4.65621
172	4.37735
174	4.13404
176	3.92044
178	3.74311
180	3.59163
182	3.46667
184	3.37177
186	3.29778
188	3.24186
190	3.19991
192	3.16911
194	3.14727
196	3.13241
198	3.12251
200	3.11601

MOHAWK RIVER AT COHOES - PMF

HRS X 1000CFS

2	3.466
4	3.466
6	3.466
8	3.4752
10	3.4891
12	3.50413
14	3.53494
16	3.62393
18	3.82678
20	4.29449
22	5.15035
24	6.58528
26	8.7[illegible]7
28	11.9[illegible]1
30	16.1641
32	21.4177
34	27.8372
36	35.3721
38	43.8169
40	53.5793
42	64.5186
44	78.1547
46	95.3054
48	115.393
50	139.866
52	169.469
54	203.195
56	239.004
58	276.131
60	311.68
62	344.75
64	374.476
66	395.449
68	412.174
70	424.205
72	429.083
74	429.497
76	425.04
78	414.359
80	401.343
82	384.463
84	365.153
86	344.33
88	322.635

MOHAWK RIVER AT SCHOES - PMF [illegible]

90	300.892
92	279.119
94	257.299
96	235.869
98	215.16
100	194.994
102	175.649
104	157.486
106	140.518
108	124.903
110	110.779
112	98.0752
114	86.7967
106	76.8048
118	68.046
120	60.3627
122	53.5959
124	47.6374
126	42.46
128	38.0058
130	34.1153
132	30.7221
134	27.7127
136	25.0359
138	22.656
140	20.5154
142	18.5525
144	16.7471
146	15.0973
148	13.5994
150	12.2426
152	11.0215
154	9.92909
156	8.95734
158	8.1071
160	7.37221
162	6.73833
164	6.1987
166	5.74292
168	5.35647
170	5.02821
172	4.74735
174	4.50404
176	4.29344
178	4.11311
180	3.96183
182	3.83367
184	3.74177
186	3.66776
188	3.61138

MOHAWK RIVER AT COHOES (CONTINUATION)

190	3.56991
192	3.56911
194	3.51737
196	3.50241
198	3.49261
200	3.48601
202	3.48125
204	3.47747
206	3.47465
208	3.47176
210	3.46971
212	3.46821
214	3.4672
216	3.4666
218	3.46625
220	3.46612
222	3.46605
224	3.46602
226	3.46601
228	3.466
230	3.466
232	3.466
234	3.466
236	3.466
238	3.466
240	3.466
242	3.466
244	3.466
246	3.466
248	3.466
250	3.466
252	3.466
254	3.466
256	3.466
258	3.466
260	3.466
262	3.466
264	3.466
266	3.466
268	3.466
270	3.466
272	3.466
274	3.466

the change of storage per unit change of discharge.

x = A dimensionless 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

TABLE III-16

HUDSON RIVER PROBABLE MAXIMUM FLOOD AT INDIAN POINT
ROUTING COEFFICIENTS USED IN CHANNEL FLOOD ROUTING

Subreach	Δt (hr's)	X	K (hr's)	Travel Time of Flood Wave (hr's)	Number of Sub-reaches
UPPER HUDSON RIVER					
Mouth of Indian River to Subbasin #6	3	0.4	3	3	1
Subbasin #6 to Mouth of Schroon River	3	0.3	3	3	1
Mouth of Schroon River to Mouth of Sacandaga River	3	0.1	3	6	2
Mouth of Sacandaga River to Mouth of Batten Kill	3	0.3	3	6	2
Mouth of Batten Kill to Mouth of Hoosic River	3	0.3	3	6	2
Mouth of Hoosic River to Mouth of Mohawk River	3	0.3	3	6	2
MOHAWK RIVER					
Mouth of Schoharie Creek to Cohoes	2	0.3	2	14	7
LOWER HUDSON RIVER					
Green Island to Mouth of Kinderhood Creek	2	0.3	2	6	3
Mouth of Kinderhood Creek to Mouth of Esopus Creek	2	0.3	2	2	1
Mouth of Esopus Creek to Mouth of Rondout Creek	2	0.3	2	2	1
Mouth of Rondout Creek to Indian Point	2	0.3	2	6	3

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

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

```
 * * HAPIDATA * *
TIME    09:17      SYS#3   T10
USER #:Z545,115-8
IDENT#:KANEKEKK1S
OLD OR NEW:OLD
NAME: UHUH

READY
LIST


UHUH       09:18    RDS3   APRIL 20,1970

50 DIM G(8,200)
70 FOR Y=1 TO 3
75 READ S
80 READ K
90 FOR J=0 TO K  STEP 2
100 READ G(Y,J)
110 NEXT J
120 FOR J=1 TO K-1  STEP 2
130 LETG(Y,J)=(G(Y,J-1)+G(Y,J+1))/2
140 NEXT J
150 PRINT "SUBBASIN : "S,"U.H."
155 PRINT
160 PRINT "HR'S ; 1000CFS"
165 PRINT
170 FOR J=1 TO K
180 PRINT 3*J;":";G(Y,J),
190 NEXT J
200 NEXT Y
210 PRINT
212 PRINT
220 FOR Y=4 TO 7
230 READ S
240 READ K
250 FOR J=1 TO K
260 READ G(Y,J)
270 NEXT J
271 PRINT
272 PRINT
280 PRINT "SUBBASIN *"S,"U.H."
290 PRINT "HR'S  +  1000CFS"
300 PRINT
305 FOR J=1 TO K
310 PRINT 3*J;":";G(Y,J),
320 NEXT J
330 NEXT Y
331 PRINT
332 PRINT
399 READ S
400 FOR J=0 TO 192 STEP 8
410
430 READ G(8,J)
440 NEXT J
450 FOR J=1 TO 185 STEP 8
470 LET A=1
480 LET D=A*(G(8,J+7)-G(8,J-1))/8
490 LET V=G(8,J-1)
491 LET G(8,J)=V+D
492 LET G(8,J+1)=V+(2*D)
493 LET G(8,J+2)=V+(3*D)
494 LET G(8,J+3)=V+(4*D)
```

PLATE III-4
(SHEET 1 OF 3)

UPPER HUDSON RIVER
UPSTREAM OF SACANDAGA
RIVER

FLOOD HYDROGRAPH PROGRAM

```
495 LET G(S,J+4)=V+(5*D)
496 LET G(S,J+4)=V+(6*D)
497 LET G(S,J+6)=V+(7*D)
498 GOTO 520
520 NEXT J
530 PRINT " SUBBASIN : " S,"U.H"
540 PRINT
550 PRINT "HR'S  :  1000CFS"
555 PRINT
556 PRINT
560 FOR J=1 TO 192
570 PRINT J*3;":";G(S,J),
580 NEXT J
790 DIM P(12),Q(8,200),O(200),A(200)
800 FOR Y =1 TO 8
810 READ S
820 READ B
830 FOR I=1 TO 12
840 READ P(I)
850 NEXT I
880 FOR I=1 TO 12
890 FOR J=1 TO 150
900 LET N=J+((I-1)*2)
910 LET Q(Y,N)=(G(Y,J)*P(I))+Q(Y,N)
920 NEXT J
930 NEXT I
940 FOR N=1 TO 200
950 LET Q(Y,N)=Q(Y,N)+B
960 NEXT N
970 PRINT
980 PRINT "   PMF   -    SUBBASIN : " S
990 PRINT
1000 PRINT " BASE FLOW :" B;"1000CFS"
1010 PRINT
1020 PRINT "HR'S : 1000CFS"
1030 PRINT
1040 PRINT
1045 LET Z1=50
1050 FOR N=1 TO Z1
1060 PRINT N*3;":";Q(Y,N),
1070 NEXT N
1080 PRINT
1090 PRINT
1095 PRINT
1096 GOTO 1200
1100 LET Z1=100
1110 GOTO 1050
1200 NEXT Y
1300 FOR N=1 TO 200
1310 LET A(N)=Q(1,N)+Q(2,N)+Q(4,N)
1320 NEXT N
1330 LET C1=1/1.2
1340 LET C2=1/6.0
1350 LET N5=1
1360 GOSUB 4000
1400 FOR N=1 TO 199
```

```
1410 LET A(N)=A(N)+Q(3,N)+Q(5,N)
1420 NEXT N
1430 LET C1=1/1.2
1440 LET C2=1/6
1450 LET N5=1
1460 GOSUB 4000
1500 FOR N=1 TO 197
1510 LET A(N)=A(N)+Q(6,N)+Q(8,N)
1520 NEXT N
1530 LET C1=6/8.4
1540 LET C2=2.4/8.4
1550 LET N5=2
1560 GOSUB 4000
1600 FOR N=1 TO 195
1610 LET A(N)=A(N)+Q(7,N)
1620 NEXT N
1650 PRINT
1660 PRINT
 670 PRINT"UPPER HUDSON RIVER ABOVE SACANDAGA RIVER - PMF"
1690 PRINT
1700 PRINT "HR'S ; 10 0CFS"
1710 PRINT
1740 FOR N=1 TO 100
1750 PRINT N*3;"3";A(N),
1760 NEXT N
1780 PRINT
1790 PRINT
1800 PRINT
4000 LET K=1
4010 LET O(1)=A(1)
4020 FOR N=2 TO 200-K
4030 LET O(N)=C1*(A(N-1)-O(N-1))+C2*(A(N)-A(N-1))+O(N-1)
4040 NEXT N
4050 FOR N=1 TO 200-K
4060 LET A(N)=O(N)
4070 NEXT N
4080 LET K=K+1
4090 IF K>N5 GOTO 4110
4100 GOTO 4010
4110 RETURN TO MAIN LINE
```

PLATE III-5
(SHEET 1 OF 2)

UPPER HUDSON RIVER
UPSTREAM OF SACANDAGA
RIVER

FLOOD HYDROGRAPH PROGRAM
LIST OF DATA

```
5000 DATA (1),66 . SUBBASIN
5100 DATA 0.00,0.79,3.07,4.30
5110 DATA 3.58,4.15,3.73,3.82
5120 DATA 3.21,2.60,2.18,1.77
5130 DATA 1.62,1.47,1.25,1.04   } UNIT HYDROGRAPH DATA
5140 DATA 0.93,0.83,0.72,0.62
5150 DATA 0.5 ,0.48,0.43,0.38
5160 DATA 0.33,0.28,0.23,0.18
5170 DATA 0.15,0.12,0.09,0.06
5180 DATA 0.03,0.00,0.00
5500 DATA (2.1),30
5600 DATA 0.00,0.25,2.24,3.44
5610 DATA 2.30,1.69,1.28,0.95   } U.H.
5620 DATA 0.70,0.51,0.34,0.22
5630 DATA 0.12,0.50,0.00,0.00
5800 DATA (4),22
5810 DATA 0.00,0.80,3.50,2.80   } U.H
5820 DATA 1.10,0.07,0.41,0.24
5830 DATA 0.14,0.05,0.00,0.00
5900 DATA (2.2),22
5910 DATA 0.30,0.75,1.80,3.40
5920 DATA 2.60,1.70,1.10,0.90
5930 DATA 0.68,0.53,0.45,0.38   } U.H.
5940 DATA 0.30,0.24,0.18,0.13
5950 DATA 0.09,0.07,0.05,0.30
5960 DATA 0.02,0.00
6000 DATA (3),8
6010 DATA 4.00,7.50,3.10,1.50   } U.H
6020 DATA 0.50,0.25,0.10,0.00
6100 DATA (6),10
6110 DATA 9.00,16.00,7.70,2.95
6120 DATA 1.30,0.60,0.30,0.100  } U.H
6130 DATA 0.50,0.00
6200 DATA (7),12
6210 DATA 4.40,10.50,6.70,3.250
6220 DATA 1.65,0.79,0.38,0.19   } U.H
6230 DATA 0.09,0.04,0.02,0.00
6300 DATA (5)
6310 DATA 0.00,0.65,1.40,1.53
6320 DATA 1.48,1.37,1.20,1.03
6330 DATA 0.88,0.77,0.69,0.62   } U.H
6340 DATA 0.56,0.52,0.47,0.42
6350 DATA 0.37,0.32,0.27,0.22
6360 DATA 0.17,0.12,0.07,0.02
6370 DATA 0.00
6400 DATA (1),0.40              . SUBBASIN, BASE FLOW
6410 DATA 0.00,0.35,0.26,0.23
6420 DATA 0.82,2.84,0.98,0.78   } RUNOFF DATA
6430 DATA 0.00,0.00,0.01,0.00
```

PLATE III-5
(SHEET 2 OF 2)

```
6500 DATA 2.1,0.10                          SUBBASIN, BASE FLOW
6510 DATA 0.00,0.41,0.57,0.23 }
6520 DATA 1.44,4.74,1.72,1.03 }  RUNOFF DATA
6530 DATA 0.06,0.13,0.17,0.08 }
6600 DATA 1,0.10
6610 DATA 0.00,0.38,0.56,0.22 }
6620 DATA 0.86,2.92,1.01,0.76 }
6630 DATA 0.00,0.00,0.02,0.00 }
6700 DATA 2.2,0.08
6710 DATA 0.00,0.41,0.57,0.23 }
6720 DATA 1.37,3.85,1.45,0.96 }
6730 DATA 0.02,0.08,0.13,0.04 }
6800 DATA 3,0.08
6810 DATA 0.00,0.44,0.59,0.24 }
6820 DATA 1.25,3.80,1.43,0.95 }
6830 DATA 0.02,0.08,0.12,0.03 }
6900 DATA 6,0.20
6910 DATA 0.00,0.56,0.65,0.27 }
6920 DATA 1.51,5.47,1.78,1.08 }
6930 DATA 0.08,0.15,0.19,0.09 }
7000 DATA 7,0.15
7010 DATA 0.00,0.59,0.65,0.28 }
7020 DATA 1.55,5.77,1.92,1.10 }
7030 DATA 0.08,0.15,0.20,0.11 }
7100 DATA 5,0.15
7110 DATA 0.00,0.39,0.56,0.23 }
7120 DATA 0.85,3.12,1.11,0.79 }
7130 DATA 0.00,0.01,0.05,0.00 }
9000 END


BYE
OFF AT 09:27
```

UPPER HUDSON RIVER UPSTREAM OF SACANDAGA RIVER
FLOOD HYDROGRAPH PROGRAM - PRINTOUT

UHUH 09:40 KDS3 APRIL 20, 1970

SUBBASIN : 1 U.H.

HR'S : 1000CFS

3 : .395	6 : .79	9 : 1.93	12 : 3.07	15 : 3.685
18 : 4.3	21 : 3.94	24 : 3.58	27 : 3.865	30 : 4.15
33 : 3.94	36 : 3.73	39 : 3.775	42 : 3.82	45 : 3.515
48 : 3.21	51 : 2.905	54 : 2.6	57 : 2.39	60 : 2.18
63 : 1.975	66 : 1.77	69 : 1.695	72 : 1.62	75 : 1.545
78 : 1.47	81 : 1.36	84 : 1.25	87 : 1.145	90 : 1.04
93 : .985	96 : .93	99 : .88	102 : .83	105 :
.775	108 : .72	111 : .67	114 : .62	117 :
.56	120 : .5	123 : .49	126 : .48	129 :
.455	132 : .43	135 : .405	138 : .38	141 :
.355	144 : .33	147 : .305	150 : .28	153 :
.255	156 : .23	159 : .205	162 : .18	165 :
.165	168 : .15	171 : .135	174 : .12	177 :
.105	180 : .09	183 : .075	186 : .06	189 :
.045	192 : .03	195 : .015	198 : 0	201 :
0	204 : 0	SUBBASIN : 2.1 U.H.		

SUBBASIN : 2.1 U.H

HR'S : 1000CFS

3 : .125	6 : .25	9 : 1.245	12 : 2.24	15 : 2.84
18 : 3.44	21 : 2.87	24 : 2.3	27 : 1.995	30 : 1.69
33 : 1.485	36 : 1.28	39 : 1.115	42 : .95	45 : .825
48 : .7	51 : .605	54 : .51	57 : .425	60 : .34
63 : .28	66 : .22	69 : .17	72 : .12	75 : .31
78 : .5	81 : .25	84 : 0	87 : 0	90 : 0

SUBBASIN : 4 U.H.

HR'S : 1000CFS

3 : .4	6 : .8	9 : 2.15	12 : 3.5	15 : 3.15
18 : 2.8	21 : 1.95	24 : 1.1	27 : .885	30 : .67
33 : .54	36 : .41	39 : .325	42 : .24	45 : .19
48 : .14	51 : .095	54 : .05	57 : .025	60 : 0
63 : 0	66 : 0			

SUBBASIN * 2.2 U.H.

HR'S + 1000CFS

3 : .3	6 : .75	9 : 1.8	12 : 3.4	15 : 2.6
18 : 1.7	21 : 1.1	24 : .9	27 : .68	30 : .53
33 : .45	36 : .38	39 : .3	42 : .24	45 : .18
48 : .13	51 : .09	54 : .07	57 : .05	60 : .3
63 : .02	66 : 0			

SUBBASIN * 3 U.H.
HR'S + 1000CFS

3 ; 4	6 ; 7.5	9 ; 3.1	12 ; 1.5	15 ; .5
18 ; .25	21 ; .1	24 ; 0		

SUBBASIN * 6 U.H.
HR'S + 1000CFS

3 ; 9	6 ; 16	9 ; 7.7	12 ; 2.95	15 ; 1.3
18 ; .6	21 ; .3	24 ; .1	27 ; .5	30 ; 0

SUBBASIN * 7 U.H.
HR'S + 1000CFS

3 ; 4.4	6 ; 10.5	9 ; 6.7	12 ; 3.25	15 ; 1.65
18 ; .79	21 ; .38	24 ; .19	27 ; .09	30 ; .04
33 ; .02	36 ; 0			

SUBBASIN : 5 U.H

HR'S : 1000CFS

3 : .08125	6 : .1625	9 : .24375	12 : .325	15 : .40625
18 : .4875	21 : .56875	24 : .65	27 : .74375	30 : .8375
33 : .93125	36 : 1.025	39 : 1.11875	42 : 1.2125	45 : 1.30625
48 : 1.4	51 : 1.41625	54 : 1.4325	57 : 1.44875	60 : 1.465
63 : 1.48125	66 : 1.4975	69 : 1.51375	72 : 1.53	75 : 1.52375
78 : 1.5175	81 : 1.51125	84 : 1.505	87 : 1.49875	90 : 1.4925
93 : 1.48625	96 : 1.48	99 : 1.46625	102 : 1.4525	105 :
1.43875	108 : 1.425	111 : 1.41125	114 : 1.3975	117 :
1.38375	120 : 1.37	123 : 1.34375	126 : 1.3275	129 :
1.30625	132 : 1.285	135 : 1.26375	138 : 1.2425	141 :
1.22125	144 : 1.2	147 : 1.17875	150 : 1.1575	153 :
1.13625	156 : 1.115	159 : 1.09375	162 : 1.0725	165 :
1.05125	168 : 1.03	171 : 1.01125	174 : .9925	177 :
.97375	180 : .955	183 : .93625	186 : .9175	189 :
.89875	192 : .88	195 : .86625	198 : .8525	201 :
.83875	204 : .825	207 : .81125	210 : .7975	213 :
.78375	216 : .77	219 : .76	222 : .75	225 :
.74	228 : .73	231 : .72	234 : .71	237 :
.7	240 : .69	243 : .68125	246 : .6725	249 :
.66375	252 : .655	255 : .64625	258 : .6375	261 :
.62875	264 : .62	267 : .6125	270 : .605	273 :
.5975	276 : .59	279 : .5825	282 : .575	285 :
.5675	288 : .56	291 : .555	294 : .55	297 :
.545	300 : .54	303 : .535	306 : .53	309 :
.525	312 : .52	315 : .51375	318 : .5075	321 :
.50125	324 : .495	327 : .48875	330 : .4825	333 :
.47625	336 : .47	339 : .46375	342 : .4575	345 :
.45125	348 : .445	351 : .43875	354 : .4325	357 :
.42625	360 : .42	363 : .41375	366 : .4075	369 :
.40125	372 : .395	375 : .38875	378 : .3825	381 :
.37625	384 : .37	387 : .36375	390 : .3575	393 :
.35125	396 : .345	399 : .33875	402 : .3325	405 :
.32625	408 : .32	411 : .31375	414 : .3075	417 :
.30125	420 : .295	423 : .28875	426 : .2825	429 :
.27625	432 : .27	435 : .26375	438 : .2575	441 :
.25125	444 : .245	447 : .23875	450 : .2325	453 :
.22625	456 : .22	459 : .21375	462 : .2075	465 :
.20125	468 : .195	471 : .18875	474 : .1825	477 :
.17625	480 : .17	483 : .16375	486 : .1575	489 :
.15125	492 : .145	495 : .13875	498 : .1325	501 :
.12625	504 : .12	507 : .11375	510 : .1075	513 :
.10125	516 : .095	519 : .08875	522 : .0825	525 :
.07625	528 : .07	531 : .06375	534 : .0575	537 :
.05125	540 : .045	543 : .03875	546 : .0325	549 :
.02625	552 : .02	555 : .0175	558 : .015	561 :
.0125	564 : .01	567 : .0075	570 : .005	573 :
.0025	576 : 0			

PLATE III-6
(SHEET 4 OF 7)

PMF - SUBBASIN : 1

BASE FLOW : .4 1000CFS

HR'S : 1000CFS

3 : .4	6 : .4	9 : .53825	12 : .6765	15 : 1.2967
18 : 1.9169	21 : 2.8614	24 : 3.8059	27 : 4.6104	30 : 5.4149
33 : 7.5111	36 : 9.6073	39 : 13.7396	42 : 17.8719	45 : 20.6886
48 : 23.5053	51 : 24.0102	54 : 24.5152	57 : 24.9752	60 : 25.4352
63 : 24.5853	66 : 23.7354	69 : 23.3644	72 : 22.9934	75 : 21.5868
78 : 20.1802	81 : 18.7563	84 : 17.3324	87 : 15.9545	90 : 14.5766
93 : 13.3898	96 : 12.203	99 : 11.4735	102 : 10.744	105 :
10.1442	108 : 9.5444	111 : 8.95545	114 : 8.3665	117 :
7.7995	120 : 7.2325	123 : 6.7895	126 : 6.3465	129 :
5.98055	132 : 5.6146	135 : 5.2999	138 : 4.9852	141 :
4.67795	144 : 4.3707	147 : 4.0739	150 : 3.7772	

PMF - SUBBASIN : 2.1

BASE FLOW : .1 1000CFS

HR'S : 1000CFS

3 : .1	6 : .1	9 : .15125	12 : .2025	15 : .6817
18 : 1.1609	21 : 2.0028	24 : 2.8447	27 : 3.36185	30 : 3.879
33 : 5.59235	36 : 7.3057	39 : 12.712	42 : 18.1183	45 : 21.727
48 : 25.3357	51 : 24.0666	54 : 22.7975	57 : 20.622	60 : 18.4465
63 : 16.1943	66 : 13.9422	69 : 12.4414	72 : 10.9406	75 : 9.7313
78 : 8.522	81 : 7.4991	84 : 6.4762	87 : 5.55505	90 : 4.6339
93 : 3.84645	96 : 3.059	99 : 2.78095	102 : 2.5029	105 :
2.8438	108 : 3.1847	111 : 2.2342	114 : 1.2837	117 :
1.01655	120 : .7494	123 : .4798	126 : .2102	129 :
.201	132 : .203	135 : .1988	138 : .194o	141 :
.161	144 : .14	147 : .12	150 : .1	

PMF - SUBBASIN : 4

BASE FLOW : .1 1000CFS

HR'S : 1000CFS

3 : .1	6 : .1	9 : .252	12 : .404	15 : 1.141
18 : 1.878	21 : 2.589	24 : 3.3	27 : 3.422	30 : 3.544
33 : 5.2383	36 : 6.9326	39 : 10.6208	42 : 14.309	45 : 14.0711
48 : 13.8332	51 : 11.7436	54 : 9.654	57 : 7.7261	60 : 5.7982
63 : 4.44465	66 : 3.0911	69 : 2.5083	72 : 1.9255	75 : 1.54365
78 : 1.1618	81 : .8768	84 : .5918	87 : .43105	90 : .2703
93 : .20825	96 : .1462	99 : .1255	102 : .1048	105 :
.1038	108 : .1028	111 : .1019	114 : .101	117 :
.1005	120 : .1	123 : .1	126 : .1	129 :
.1	132 : .1	135 : .1	138 : .1	141 :
.1	144 : .1	147 : .1	150 : .1	

PLATE II-6
(SHEET NO.)

PMF - SUBBASIN : 2.2

BASE FLOW : .08 1000CFS

HR'S : 1000CFS

3 : .08	6 : .08	9 : .203	12 : .3875	15 : .989
18 : 1.9015	21 : 2.241	24 : 2.8875	27 : 2.808	30 : 3.1525
33 : 5.0249	36 : 8.4068	39 : 11.5721	42 : 17.0814	45 : 14.9209
48 : 13.8549	51 : 11.0309	54 : 10.2398	57 : 7.629	60 : 5.8281
63 : 4.5837	66 : 4.0766	69 : 3.3023	72 : 3.048	75 : 2.2888
78 : 1.8908	81 : 1.354	84 : 1.3844	87 : .7756	90 : 1.6014
93 : .4292	96 : .6746	99 : .2302	102 : .4862	105 :
.1428	108 : .1181	111 : .1033	114 : .1183	117 :
.0917	120 : .1218	123 : .0846	126 : .092	129 :
.0608	132 : .08	135 : .08	138 : .08	141 :
.08	144 : .08	147 : .08	150 : .08	

PMF - SUBBASIN : 3

BASE FLOW : .08 1000CFS

HR'S : 1000CFS

3 : .08	6 : .08	9 : 1.84	12 : 3.38	15 : 3.804
18 : 5.165	21 : 3.089	24 : 2.875	27 : 6.163	30 : 9.9625
33 : 19.334	36 : 30.515	39 : 18.229	42 : 16.8175	45 : 10.338
48 : 10.3	51 : 4.2	54 : 2.0125	57 : 1.08	60 : .9475
63 : .913	66 : 1.105	69 : .613	72 : .505	75 : .241
78 : .155	81 : .107	84 : .0875	87 : .083	90 : .08
93 : .08	96 : .08	99 : .08	102 : .08	105 :
.08	108 : .08	111 : .08	114 : .08	117 :
.08	120 : .08	123 : .08	126 : .08	129 :
.08	132 : .08	135 : .08	138 : .08	141 :
.08	144 : .08	147 : .08	150 : .08	

PMF - SUBBASIN : 6

BASE FLOW : .2 1000CFS

HR'S : 1000CFS

3 : .2	6 : .2	9 : 5.24	12 : 9.16	15 : 10.362
18 : 12.252	21 : 8.363	24 : 6.7735	27 : 16.882	30 : 25.6085
33 : 61.883	36 : 92.4015	39 : 60.708	42 : 45.7603	45 : 31.325
48 : 28.164	51 : 13.948	54 : 6.281	57 : 6.839	60 : 3.662
63 : 4.383	66 : 3.8385	69 : 3.232	72 : 2.2985	75 : 1.225
78 : .5945	81 : .449	84 : .273	87 : .322	90 : .209
93 : .245	96 : .2	99 : .2	102 : .2	105 :
.2	108 : .2	111 : .2	114 : .2	117 :
.2	120 : .2	123 : .2	126 : .2	129 :
.2	132 : .2	135 : .2	138 : .2	141 :
.2	144 : .2	147 : .2	150 : .2	

PMF - SUBBASIN : 7

BASE FLOW : .15 1000CFS

HR'S : 1000CFS

3 : .15	6 : .15	9 : 2.746	12 : 6.345	15 : 6.963
18 : 8.8925	21 : 6.7105	24 : 5.6686	27 : 10.1427	30 : 17.9696
33 : 36.6851	36 : 66.1408	39 : 49.9912	42 : 40.3682	45 : 28.0017
48 : 22.804	51 : 13.3777	54 : 7.2401	57 : 4.4409	60 : 3.4496
63 : 2.8732	66 : 3.0865	69 : 2.3893	72 : 2.1327	75 : 1.3032
78 : .6972	81 : .4226	84 : .2809	87 : .2128	90 : .1769
93 : .1639	96 : .1544	99 : .1522	102 : .15	105 :
.15	108 : .15	111 : .15	114 : .15	117 :
.15	120 : .15	123 : .15	126 : .15	129 :
.15	132 : .15	135 : .15	138 : .15	141 :
.15	144 : .15	147 : .15	150 : .15	

PMF - SUBBASIN : 5

BASE FLOW : .15 1000CFS

HR-S : 1000CFS

3 : .15	6 : .15	9 : .181687	12 : .213375	15 : .290562
18 : .36775	21 : .463625	24 : .5595	27 : .724437	30 : .889375
33 : 1.31269	36 : 1.736	39 : 2.2565	42 : 2.777	45 : 3.36456
48 : 3.95812	51 : 4.55031	54 : 5.1485	57 : 5.75627	60 : 6.36405
63 : 6.94635	66 : 7.52867	69 : 8.10304	72 : 8.6774	75 : 9.18589
78 : 9.69437	81 : 9.95241	84 : 10.2104	87 : 10.3705	90 : 10.5305
93 : 10.6242	96 : 10.7178	99 : 10.7923	102 : 10.8668	105 :
10.8674	108 : 10.868	111 : 10.8356	114 : 10.8032	117 :
10.7512	120 : 10.6993	123 : 10.641	126 : 10.5827	129 :
10.4978	132 : 10.4129	135 : 10.3144	138 : 10.2159	141 :
10.1098	144 : 10.0036	147 : 9.89106	150 : 9.77852	

UPPER HUDSON RIVER ABOVE SACANDAGA RIVER - PMF

HR'S ; 1000CFS

3 ; 1.26	6 ; 1.26	9 ; 4.29674	12 ; 9.93055	15 ; 14.3076
18 ; 20.3618	21 ; 21.73	24 ; 22.8884	27 ; 29.1973	30 ; 42.2247
33 ; 73.1689	36 ; 125.97	39 ; 139.523	42 ; 148.064	45 ; 137.851
48 ; 130.131	51 ; 118.602	54 ; 109.233	57 ; 100.525	60 ; 92.8368
63 ; 85.8561	66 ; 80.0456	69 ; 73.942	72 ; 68.4701	75 ; 62.6255
78 ; 57.4099	81 ; 53.056	84 ; 49.2397	87 ; 45.7358	90 ; 42.4889
93 ; 39.4932	96 ; 36.7364	99 ; 34.2063	102 ; 31.8295	105 ;
29.6642	108 ; 27.8635	111 ; 26.4768	114 ; 25.4462	117 ;
24.5701	120 ; 23.5859	123 ; 22.4383	126 ; 21.2887	129 ;
20.2482	132 ; 19.3201	135 ; 18.4998	138 ; 17.801	141 ;
17.2151	144 ; 16.7066	147 ; 16.2406	150 ; 15.7916	153 ;
15.3464	156 ; 14.9026	159 ; 14.4718	162 ; 14.0733	165 ;
13.7104	168 ; 13.3701	171 ; 13.0424	174 ; 12.724	177 ;
12.4128	180 ; 12.1034	183 ; 11.7957	186 ; 11.4896	189 ;
11.1855	192 ; 10.8835	195 ; 10.5842	198 ; 10.2891	201 ;
10.0014	204 ; 9.72599	207 ; 9.46421	210 ; 9.21394	213 ;
8.97339	216 ; 8.74119	219 ; 8.51662	222 ; 8.29895	225 ;
8.08847	228 ; 7.88704	231 ; 7.69593	234 ; 7.51543	237 ;
7.34821	240 ; 7.19763	243 ; 7.06425	246 ; 6.94491	249 ;
6.83738	252 ; 6.74114	255 ; 6.65473	258 ; 6.57476	261 ;
6.49897	264 ; 6.42588	267 ; 6.35469	270 ; 6.28466	273 ;
6.21571	276 ; 6.14837	279 ; 6.08287	282 ; 6.01884	285 ;
5.95603	288 ; 5.8941	291 ; 5.8329	294 ; 5.7723	297 ;
5.71859	300 ; 5.65452			

PLATE III-7
(SHEET 1 OF 3)

UPPER HUDSON RIVER
BETWEEN SACANDAGA RIVER
AND MOHAWK RIVER

PROGRAM FOR
GENERATING OF FLOOD HYDROGRAPHS

```
READY
LIST

LUHR       15:04   RDS3  APRIL 30,1970

 50 DIM G(6,200)
 70 FOR Y=1 TO 2
 75 READ S
 80 READ K
 90 FOR J=0 TO K STEP 2
100 READ G(Y,J)
110 NEXT J
120 FOR J=1 TO K-1 STEP 2
130 LET G(Y,J)=(G(Y,J-1)+G(Y,J+1))/2
140 NEXT J
150 PRINT " SUBBASIN # "S,"U.H."
155 PRINT
160 PRINT "HR'S ; 1000CFS"
165 PRINT
170 FOR J=1 TO K
180 PRINT 3*J;"};G(Y,J),
190 NEXT J
191 PRINT
192 PRINT
193 PRINT
200 NEXT Y
210 PRINT
212 PRINT
220 FOR Y=3 TO 6
230 READ S
240 READ K
250 FOR J=1 TO K
260 READ G(Y,J)
270 NEXT J
280 PRINT
281 PRINT "SUBBASIN :"S,"U.H."
290 PRINT
292 PRINT "HR'S ; 1000CFS"
300 PRINT
305 FOR J=1 TO K
310 PRINT 3*J;"};G(Y,J),
320 NEXT J
321 PRINT
322 PRINT
323 PRINT
330 NEXT Y
```

```
340 PRINT
350 PRINT
790 DIM P(12),Q(6,200),O(200),A(200)
800 FOR Y=1 TO 6
810 READ S
820 READ B
830 FOR I=1 TO 12
840 READ P(I)
850 NEXT I
860 FOR I=1 TO 12
890 FOR J=1 TO 150
900 LET N=J+((I-1)*2)
910 LET Q(Y,N)=(Q(Y,J)*P(I))+Q(Y,N)
920 NEXT J
930 NEXT I
940 FOR N=1 TO 200
950 LET Q(Y,N)=Q(Y,N)+B
960 NEXT N
970 PRINT
980 PRINT "PMF - SUBBASIN :"S
990 PRINT
1000 PRINT" BASE FLOW :" B;"1000CFS"
1010 PRINT
1020 PRINT "HR'S ; 1000CFS"
1030 PRINT
1040 PRINT
1050 FOR  N=1 TO 50
1060 PRINT N*3;":";Q(Y,N),
1070 NEXT N
1080 PRINT
1090 PRINT
1100 PRINT
1110 NEXT Y
1200 FOR Y=7 TO 8
1210 READ K1
1220 FOR N=1 TO K1
1230 READ Q(Y,N)
1240 NEXT N
1250 NEXT Y
1300 FOR N=1 TO 110
1310 LET A(N)=Q(7,N)+Q(8,N)
1320 NEXT N
1340 PRINT
1350 PRINT
1360 PRINT
1370 PRINT"UPPER HUDSON RIVER BELOW SACANDAGA RIVER - PMF"
1380 PRINT
1390 PRINT"HR'S           1000CFS"
1400 FOR N=1 TO 70
1410 PRINT 3*N,A(N)
1420 NEXT N
1425 PRINT
1430 PRINT
1450 LET N5=2
```

```
1460 GOSUB 4000
1500 FOR N=1 TO 108
1510 LET A(N)=A(N)+Q(1,N)+Q(2,N)+Q(3,N)
1520 NEXT N
1550 LET N5=V
1560 GO SUB 4000
1600 FOR N=1 TO 106
1610 LET A(N)=A(N)+Q(4,N)+Q(5,N)
1620 NEXT N
1630 PRINT
1640 PRINT
1650 PRINT
1660 PRINT" HUDSON RIVER BELOW HOSIC RIVER - PF"
1670 PRINT
1680 PRINT "HR'S ; 1000CFS"
1690 FOR N=1 TO 100
1700 PRINT 3*N;"j";A(N),
1710 NEXT N
1720 PRINT
1730 PRINT
1735 PRINT
1750 LET N5=2
1760 GO SUB 4000
1800 FOR N=1 TO 98
1810 LET A(N)=A(N)+Q(6,N)
1820 NEXT N
1830 PRINT
1840 PRINT"UPPER HUDSON RIVER  ABOVE MOHAWK RIVER - PMF"
1850 PRINT
1860 PRINT" HR'S            1000CFS"
1870 PRINT
1880 FOR N=1 TO 100
1890PRINT 3*N,A(N)
1900 NEXT N
1910 PRINT
1920 PRINT" "0" OF TIME IS BIGINNING OF PMP"
1930 PRINT
1940 GOTO 9000
4000 LET R=1
4010 LET C1=1/1.2
4020 LET C2=1/6
4030 LET O(1)=A(1)
4035 FOR N=2 TO 110-R
4040 LET O(N)=C1*(A(N-1)-O(N-1))+C2*(A(N)-A(N-1))+O(N-1)
4050 NEXT N
4060 FOR N=1 TO 110-R
4070 LET A(N)=O(N)
4080 NEXT N
4090 LET R=R+1
4100 IF R>N5 GOTO 4120
4110 GOTO 4030
4120 RETURN TO MAIN LINE
```

PLATE III-8
(SHEET 1 OF 3)

UPPER HUDSON RIVER
BETWEEN SACANDAGA RIVER
AND MOHAWK RIVER

FLOOD HYDROGRAPHS PROGRAM
DATA

```
5000 DATA (10).0,76.0  SUBBASIN
5010 DATA 0.00,0.52,1.90,3.09
5020 DATA 2.69,2.40,2.14,1.89
5030 DATA 1.65,1.44,1.24,1.05
5040 DATA 0.89,0.74,0.63,0.55
5050 DATA 0.49,0.44,0.39,0.35
5060 DATA 0.32,0.29,0.26,0.24
5070 DATA 0.22,0.20,0.19,0.17
5080 DATA 0.16,0.14,0.13,0.10
5090 DATA 0.10,0.08,0.06,0.35
5100 DATA 0.00,0.00,0.00
5190 DATA (11).0,76.0
5210 DATA 0.30,0.17,1.86,3.49
5220 DATA 4.04,4.16,4.29,3.520
5230 DATA 2.92,2.32,2.02,1.71
5240 DATA 1.41,1.11,1.02,0.93
5250 DATA 0.84,0.75,0.67,0.58
5260 DATA 0.49,0.460,0.43,0.41
5270 DATA 0.38,0.35,0.32,0.29
5280 DATA 0.27,0.24,0.21,0.18
5290 DATA 0.15,0.15,0.14,0.12
5300 DATA 0.00,0.00,0.00
5390 DATA (9).0,16.0
5410 DATA 0.75,1.80,3.60,6.60
5420 DATA 11.15,10.85,8.10,5.90
5430 DATA 4.45,3.38,2.42,1.70
5440 DATA 1.18,0.80,0.52,0.28
5450 DATA 0.20,0.00
5500 DATA (18).0,8.0
5510 DATA 9.00,4.70,2.18,0.70
5580 DATA 0.25,0.10,0.04,0.00
5600 DATA (13).0,24
5610 DATA 0.94,2.35,7.00,14.20
5620 DATA 16.22,15.75,14.25,12.75
5630 DATA 11.00,9.35,7.80,6.45
5640 DATA 5.24,4.15,3.20,2.50
5650 DATA 1.95,1.50,1.13,0.83
5660 DATA 0.36,0.33,0.14,0.00
```

U.H. HYDROGRAPH DATA (SUBBASIN 10)

U.H.

U.H.

U.H.

PLATE III-0
(SHEET 2 OF 3)

```
57000 DATA 4,0,9
5710 DATA 16.10,7.25,4.35,2.10
5720 DATA 0.75,0.40,0.25,0.10
5730 DATA 0.00
6000 DATA 10.0,0.10
6010 DATA 0.00,0.44,0.58,0.23
6020 DATA 1.35,3.60,1.40,0.95
6030 DATA 0.02,0.08,0.12,0.03
6050 DATA 11.0,0.20
6060 DATA 0.00,0.30,0.59,0.23
6070 DATA 0.77,2.79,0.95,0.67
6080 DATA 0.00,0.00,0.01,0.00
6100 DATA 9.0,0.29
6110 DATA 0.00,0.46,0.59,0.24
6120 DATA 1.17,3.65,1.33,0.92
6130 DATA 0.00,0.06,0.10,0.12
6150 DATA 12.0,0.08
6160 DATA 0.00,0.44,0.58,0.23
6170 DATA 1.25,3.60,1.40,0.95
6180 DATA 0.02,0.08,0.12,0.03
6200 DATA 13.0,0.71
6210 DATA 0.00,0.38,0.58,0.22
6220 DATA 0.89,2.98,1.05,0.75
6230 DATA 0.00,0.00,0.03,0.00
6250 DATA 14.0,0.14
6260 DATA 0.00,0.44,0.58,0.23
6270 DATA 1.25,3.60,1.40,0.95
6280 DATA 0.02,0.08,0.12,0.03
7000 DATA 100
7010 DATA 1.26,1.26,4.29,9.93
7020 DATA 14.30,20.36,21.37,22.89
7030 DATA 29.20,42.22,73.16,125.97
7040 DATA 139.52,148.06,137.85,130.13
7050 DATA 118.60,109.23,100.52,92.83
7060 DATA 85.90,80.04,73.94,68.47
7070 DATA 62.63,57.41,53.06,49.24
7080 DATA 45.74,42.49,39.49,36.74
7090 DATA 34.21,31.83,29.66,27.86
7100 DATA 26.48,25.45,24.57,23.58
7110 DATA 22.44,21.29,20.24,19.32
7120 DATA 18.50,17.80,17.22,16.70
7130 DATA 16.24,15.79,15.35,14.90
7140 DATA 14.47,14.07,13.71,13.37
7150 DATA 13.04,12.72,12.41,12.10
7160 DATA 11.80,11.49,11.19,10.88
7170 DATA 10.58,10.29,10.00,9.73
7180 DATA 9.46,9.21,8.97,8.74
7190 DATA 8.52,8.30,8.08,7.89
7200 DATA 7.70,7.52,7.35,7.20
7210 DATA 7.06,6.94,6.84,6.74
7220 DATA 6.65,6.57,6.50,6.43
7230 DATA 6.35,6.28,6.21,6.15
7240 DATA 6.08,6.02,5.96,5.89
7250 DATA 5.83,5.77,5.71,5.65
```

U.H.

SUBBASIN # 10

RUNOFF DATA

DATA OF UPPER HUDSON FLOOD UPSTREAM OF SACANDAGA RIVER (OUTPUT OF PROGRAM: "SHUH")

```
7500 DATA 100
7510 DATA 0.00,0.00,0.00,0.00
7520 DATA 0.00,0.00,0.00,0.00
7530 DATA 0.00,0.00,4.54,9.08
7540 DATA 18.89,27.50,36.26,45.02
7550 DATA 50.75,56.48,59.87,63.27
7560 DATA 65.09,66.91,67.89,68.87
7570 DATA 69.19,69.508,69.84,68.91
7580 DATA 68.28,67.59,66.64,65.69
7590 DATA 64.58,63.45,62.24,61.01
7600 DATA 59.73,58.44,57.11,55.78
7610 DATA 54.41,53.05,51.09,50.24
7620 DATA 48.82,47.40,45.96,44.52
7630 DATA 43.06,41.60,40.16,38.71
7640 DATA 37.28,35.84,34.46,33.07
7650 DATA 31.77,30.46,29.24,28.03
7660 DATA 26.90,25.78,22.29,18.61
7670 DATA 17.70,16.58,15.93,15.28
7680 DATA 14.81,14.35,13.91,3.58
7690 DATA 13.23,12.87,12.56,12.25
7700 DATA 11.98,11.71,11.45,11.19
7710 DATA 10.98,10.77,10.57,10.36
7720 DATA 10.17,9.98,9.83,9.68
7730 DATA 9.53,9.38,9.25,9.12
7740 DATA 9.01,8.91,8.81,8.71
7750 DATA 8.62,8.53,8.64,8.34
9000 END
```

OUTFLOW FROM SACANDAGA RESERVOIR DATA

(OUTPUT OF PROGRAM: "RESERV", PROGRAM FOR SACANDAGA RESERVOIR FLOOD ROUTING, SEE CHAPTER IV)

PLATE III-9 (SHEET 1 OF 8)

UPPER HUDSON RIVER
BETWEEN SACANDAGA RIVER AND MOHAWK RIVER
FLOOD HYDROGRAPHS PROGRAM - PRINT OUT

.URH 14:45 RDS3 APRIL 30,1970

SUBAS. 1 * 10 U.H.

HRS ; 1000CFS

3 ; .26	6 ; .52	9 ; 1.21	12 ; 1.9	15 ; 2.495
18 ; 3.09	21 ; 2.89	24 ; 2.69	27 ; 2.545	30 ; 2.4
33 ; 2.27	36 ; 2.14	39 ; 2.015	42 ; 1.89	45 ; 1.77
48 ; 1.65	51 ; 1.545	54 ; 1.44	57 ; 1.34	60 ; 1.24
63 ; 1.145	66 ; 1.05	69 ; .97	72 ; .89	75 ; .815
78 ; .74	81 ; .685	84 ; .63	87 ; .59	90 ; .55
93 ; .52	96 ; .49	99 ; .465	102 ; .44	105 ;
.415	108 ; .39	111 ; .37	114 ; .35	117 ;
.335	120 ; .32	123 ; .305	126 ; .29	129 ;
.275	132 ; .26	135 ; .25	138 ; .24	141 ;
.23	144 ; .22	147 ; .21	150 ; .2	153 ;
.195	156 ; .19	159 ; .18	162 ; .17	165 ;
.165	168 ; .16	171 ; .15	174 ; .14	177 ;
.135	180 ; .13	183 ; .115	186 ; .1	189 ;
.1	192 ; .1	195 ; .09	198 ; .08	201 ;
.07	204 ; .06	207 ; .205	210 ; .05	213 ;
.175	216 ; 0	219 ; 0	222 ; 0	225 ;
0	228 ; 0			

SURJASIN * 11 U.H.

HRS ; 1000CFS

3 ; .085	6 ; .17	9 ; 1.015	12 ; 1.86	15 ; 2.675
18 ; 3.49	21 ; 3.765	24 ; 4.04	27 ; 4.1	30 ; 4.16
33 ; 4.225	36 ; 4.29	39 ; 3.905	42 ; 3.52	45 ; 3.22
48 ; 2.92	51 ; 2.62	54 ; 2.32	57 ; 2.17	60 ; 2.02
63 ; 1.865	66 ; 1.71	69 ; 1.56	72 ; 1.42	75 ; 1.26
78 ; 1.11	81 ; 1.065	84 ; 1.02	87 ; .975	90 ; .93
93 ; .885	96 ; .84	99 ; .795	102 ; .75	105 ;
.71	108 ; .67	111 ; .625	114 ; .58	117 ;
.535	120 ; .49	123 ; .475	126 ; .46	129 ;
.445	132 ; .43	135 ; .42	138 ; .41	141 ;
.395	144 ; .38	147 ; .365	150 ; .35	153 ;
.335	156 ; .32	159 ; .305	0162 ; .29	165 ;
.28	168 ; .27	171 ; .255	174 ; .24	177 ;
.225	180 ; .21	183 ; .195	186 ; 018	189 ;
.165	192 ; .15	195 ; .15	198 ; .15	201 ;
.145	204 ; .14	207 ; .13	210 ; .12	213 ;
.06	216 ; 0	219 ; 0	222 ; 0	225 ;
0	228 ; 0			

SUBBASIN : 9 U.H.

HR'S	1000CFS			
3	.75	6	1.8	9	3.6	12	6.6	15	11.15
18	10.85	21	8.1	24	5.9	27	4.45	30	3.32
33	2.42	36	1.7	39	1.18	42	.8	45	.52
48	.28	51	.1	54	0				

SUBBASIN : 12 U.H.

HR'S	1000CFS				
3	9	6	4.7	9	2.18	12	.7	15	.25
18	.1	21	.04	24	0				

SUBBASIN : 13 U.H.

HR'S	1000CFS				
3	.94	6	2.35	9	7	12	14.2	15	16.22
18	15.75	21	14.25	24	12.75	27	11	30	9.35
33	7.8	36	6.45	39	5.24	42	4.15	45	3.2
48	2.5	51	1.95	54	1.5	57	1.13	60	.83
63	.56	66	.33	69	.14	72	0		

SUBBASIN : 14 U.H.

HR'S	1000CFS				
3	16.1	6	7.25	9	4.35	12	2.1	15	.75
18	.4	21	.25	24	.1	27	0		

MF - SUBBASIN : 10

BASE FLOW : .1 1000CFS

R'S : 1000CFS

3 : .1	6 : .1	9 : .2144	12 : .3288	15 : .7832
18 : 1.8376	21 : 1.9594	24 : 2.6812	27 : 3.448	30 : 4.2148
33 : 5.03935	36 : 7.8639	39 : 11.3278	42 : 14.7918	45 : 17.713
48 : 20.6343	51 : 21.057	54 : 21.4798	57 : 20.9576	60 : 20.4354
63 : 19.3714	66 : 18.3075	69 : 17.386	72 : 16.4645	75 : 15.5521
78 : 14.6398	81 : 13.7184	84 : 12.7971	87 : 11.9254	90 : 11.0538
93 : 10.2649	96 : 9.4761	99 : 8.7792	102 : 8.0823	105 :
7.46985	108 : 6.857	111 : 6.3655	114 : 5.874	117 :
5.48785D	120 : 5.1017	123 : 4.80545	126 : 4.5092	129 :
4.266	132 : 4.0228	135 : 3.80695	138 : 3.5911	141 :
3.4109	144 : 3.2307	147 : 3.0817	150 : 2.9327	

MF - SUBBASIN : 11

BASE FLOW : .2 1000CFS

R'S : 1000CFS

3 : .2	6 : .2	9 : .8255	12 : .251	15 : .55465
18 : .8583	21 : 1.6209	24 : 2.3835	27 : 3.20665	30 : 4.0298
33 : 5.2853	36 : 6.5408	39 : 9.7248	42 : 12.9088	45 : 16.1907
48 : 19.4727	51 : 21.3243	54 : 23.176	57 : 23.8452	60 : 24.5144
63 : 24.3504	66 : 24.1864	69 : 22.2876	72 : 21.3889	75 : 19.8359
78 : 18.2829	81 : 16.621	84 : 14.9592	87 : 13.7994	90 : 12.6397
93 : 11.6743	96 : 10.709	99 : 9.873	102 : 9.037	105 :
8.2841	108 : 7.5312	111 : 7.0761	114 : 6.621	117 :
6.2686	120 : 5.9162	123 : 5.6323	126 : 5.3484	129 :
5.07625	132 : 4.8041	135 : 4.55975	138 : 4.3154	141 :
4.0713	144 : 3.8272	147 : 3.60685	150 : 3.3853	

MF - SUBBASIN : 9

BASE FLOW : .29 1000CFS

R'S : 1000CFS

3 : .29	6 : .29	9 : .635	12 : 1.118	15 : 2.3885
18 : 4.388	21 : 7.723	24 : 9.607	27 : 12.336	30 : 13.0955
33 : 16.7415	36 : 22.1942	39 : 33.1557	42 : 43.6253	45 : 58.9511
48 : 59.3973	51 : 54.7192	54 : 47.2207	57 : 41.0759	60 : 32.6912
63 : 24.3489	66 : 17.9178	69 : 13.661	72 : 10.38	75 : 8.1338
78 : 6.171	81 : 4.8472	84 : 3.4898	87 : 2.4636	90 : 1.6896
93 : 1.2288	96 : .9064	99 : .7296	102 : .5908	105 :
.4896	108 : .414	111 : .3624	114 : .3236	117 :
.302	120 : .29	123 : .29	126 : .89	129 :
.29	132 : .29	135 : .29	138 : .29	141 :

PMF - SUBBASIN : 12

BASE FLOW : .08 1000CFS

HR'S : 1000CFS

3 : .08	6 : .08	9 : 4.04	12 : 2.148	15 : 6.8592
18 : 3.114	21 : 3.5244	24 : 1.611	27 : 1.0994	30 : 6.1174
33 : 35.8857	36 : 17.898	39 : 20.8497	42 : 9.305	45 : 18.632
48 : 8.885	51 : 2.825	54 : .979	57 : 1.1371	60 : .565
63 : 1.3774	66 : .702	69 : .6324	72 : .313	75 : .1766
78 : .113	81 : .0923	84 : .083	87 : .0812	90 : .08
93 : .08	96 : .08	99 : .08	102 : .08	105 :
.08	108 : .08	111 : .08	114 : .08	117 :
.08	120 : .08	123 : .08	126 : .08	129 :
.08	132 : .08	135 : .08	138 : .08	141 :
.08	144 : .08	147 : .08	150 : .08	

PMF - SUBBASIN : 13

BASE FLOW : .71 1000CFS

HR'S : 1000CFS

3 : .71	6 : .71	9 : .0672	12 : 1.603	15 : 3.9152
18 : 7.469	21 : 11.1404	24 : 15.448	27 : 17.9092	30 : 19.9055
33 : 25.7546	36 : 34.764	39 : 49.4718	42 : 70.19	45 : 78.7185
48 : 83.04	51 : 81.2172	54 : 78.99	57 : 71.3093	60 : 62.4465
63 : 53.1477	66 : 44.8104	69 : 37.1204	72 : 30.4498	75 : 24.4467
78 : 19.5365	81 : 15.4486	84 : 12.1113	87 : 9.3841	90 : 7.2076
93 : 5.3864	96 : 3.8834	99 : 2.7199	102 : 1.8035	105 :
1.373	108 : 1.0325	111 : .8735	114 : .755	117 :
.7439	120 : .7349	123 : .7268	126 : .7199	129 :
.7142	132 : .71	135 : .71	138 : .71	141 :
.71	144 : .71	147 : .71	150 : .71	

PMF - SUBBASIN : 14

BASE FLOW : .14 1000CFS

HR'S : 1000CFS

3 : .14	6 : .14	9 : 7.224	12 : 3.33	15 : 11.392
18 : 5.269	21 : 6.696	24 : 3.2015	27 : 21.6105	30 : 9.9615
33 : 63.855	36 : 29.015	39 : 39.335	42 : 18.373	45 : 24.5375
48 : 11.5385	51 : 6.5445	54 : 3.8	57 : 2.5775	60 : 1.882
63 : 8.6725	66 : 1.281	69 : 1.21	72 : .6435	75 : .3805
78 : .259	81 : .1985	84 : .164	87 : .1475	90 : .143
93 : .14	96 : .14	99 : .14	102 : .14	105 :
.14	108 : .14	111 : .14	114 : .14	117 :
.14	120 : .14	123 : .14	126 : .14	129 :
.14	132 : .14	135 : .14	138 : .14	141 :
.14	144 : .14	147 : .14	150 : .14	

UPPER HUDSON RIVER BELOW SACANDAGA RIVER - PMF

HR'S	1000CFS
3	1.26
6	1.25
9	4.29
12	9.93
15	14.3
18	20.36
21	21.37
24	22.89
27	39.2
30	42.22
33	77.7
36	135.05
39	157.81
42	175.56
45	174.11
48	175.15
51	169.35
54	165.71
57	160.39
60	156.1
63	150.99
66	146.95
69	141.83
72	137.34
75	131.82
78	126.918
81	122.3
84	116.21
87	114.02
90	110.08
93	106.13
96	102.43
99	98.79
102	95.29
105	91.9
108	88.87
111	86.21
114	83.89
117	81.68
120	79.36
123	76.85
126	74.34
129	71.33

UPPER HUDSON RIVER DOWNSTREAM OF SACANDAGA RIVER - PMF
(CONTINUATION)

132	69.54
135	67.32
138	65.2
141	63.18
144	61.28
147	59.3
150	57.39
153	55.51
156	53.61
159	51.75
162	49.91
165	48.17
168	46.44
171	44.81
174	43.18
177	41.65
180	40.13
183	38.7
186	37.27
189	33.48
192	29.69
195	28.28
198	26.87
201	25.93
204	25.01
207	24.27
210	23.56

HUDSON RIVER BELOW HOSIC RIVER - PF

HR'S	1000CFS				
3	2.64	6	8.64	9	6.97301	12	5.78967	15	18.9921
18	15.6374	21	24.5871	24	34.171	27	55.456	30	59.3318
33	101.175	36	101.215	39	135.171	42	174.516	45	233.813
48	284.137	51	322.095	54	340.818	57	338.888	60	325.835
63	308.543	66	287.674	69	266.806	72	247.528	75	230.128
78	214.879	81	201.124	84	188.473	87	176.71	90	165.959
93	156.399	96	147.841	99	140.214	102	133.246	105	
127.144		108	121.407	111	116.125	114	111.183	117	
106.804		120	102.847	123	99.3409	126	96.1608	129	
93.1829		132	90.2195	135	87.2092	138	84.1562	141	
81.2184		144	78.5257	147	75.931	150	73.4178	153	
71.005		156	68.6802	159	66.4473	162	64.2862	165	
62.1308		168	60.0295	171	57.9827	174	55.9861	177	
54.0449		180	52.1587	183	50.3327	186	48.5602	189	
46.8434		192	45.1448	195	43.2941	198	40.9082	201	
37.9095		204	35.0589	207	32.9009	210	31.2685	213	
29.9864		216	28.9128	219	27.8814	222	26.3616	225	
23.905		228	22.1387	231	22.9472	234	23.4405	237	
23.4251		240	23.1747	243	22.8831	246	22.4141	249	
21.5594		252	20.7029	255	20.0791	258	19.5427	261	
19.0317		264	18.5824	267	18.2613	270	17.9967	273	
17.7594		276	17.5286	279	17.2923	282	17.0653	285	
16.8556		288	16.681	291	16.4817	294	16.313	297	
16.1459		300	15.9905						

UPPER HUDSON RIVER ABOVE MOHAWK RIVER - PMF (DOWNSTREAM OF MECHANICVILLE

HR'S	1000CFS
3	2.78
6	2.76
9	9.98436
12	7.0605
15	17.3054
18	13.0312
21	19.0514
24	20.3673
27	47.2842
30	47.1319
33	116.18
36	97.0386
39	131.786
42	127.169
45	163.587
48	192.274
51	238.935
54	282.97
57	317.683
60	334.38
63	336.548
66	324.691
69	308.208
72	288.013
75	267.622
78	248.478
81	231.126
84	215.678
87	201.751
90	189.002
93	177.255
96	166.554
99	156.951
102	148.355
105	140.649
108	133.706
111	127.472
114	121.727
117	116.417
120	111.527
123	107.122
126	103.163
129	99.6103
132	96.3892
135	93.3456
138	90.35
141	87.3381
144	84.3328
147	81.4385
150	78.7141

UPPER HUDSON RIVER UPSTREAM OF THE MOUTH OF MOHAWK RIVER -PMF (CONTINUATION)

156	73.5967
159	71.1811
162	68.8557
165	66.6112
168	64.4254
171	62.2863
174	60.1897
177	58.143
180	56.1478
183	54.207
186	52.3222
189	50.4946
192	48.7214
195	46.9891
198	45.2985
201	43.2537
204	40.8384
207	38.062
210	35.4117
213	33.2481
216	31.5606
219	30.2216
222	29.0727
225	27.8625
228	25.2206
231	24.245
234	23.0405
237	23.1623
240	23.4453
243	23.4656
246	23.2754
249	22.9539
252	22.4299
255	21.6766
258	20.9059
261	20.2592
264	19.7007
267	19.1965
270	18.7656
273	18.4279
276	18.1506
279	17.9042
282	17.6885
285	17.4356
288	17.2114
291	17.0017
294	16.807
297	16.4862
300	16.3155

0 OF TIME IS BIGINNING OF PMP

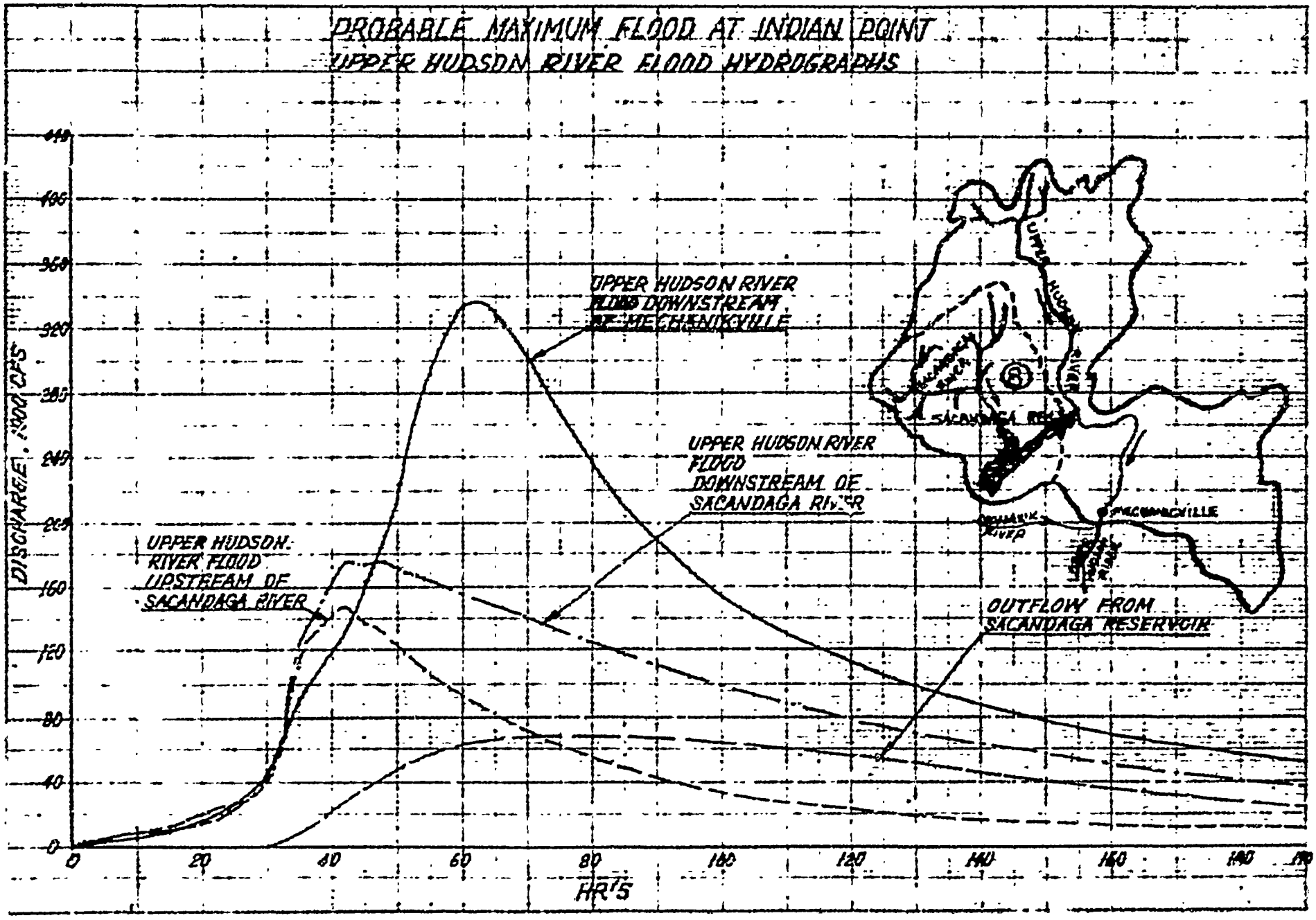


Figure III-22

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

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 emplcyed to route the river hydrographs to the Indian Point site. The Lower Hudson River computer program, input or data file and

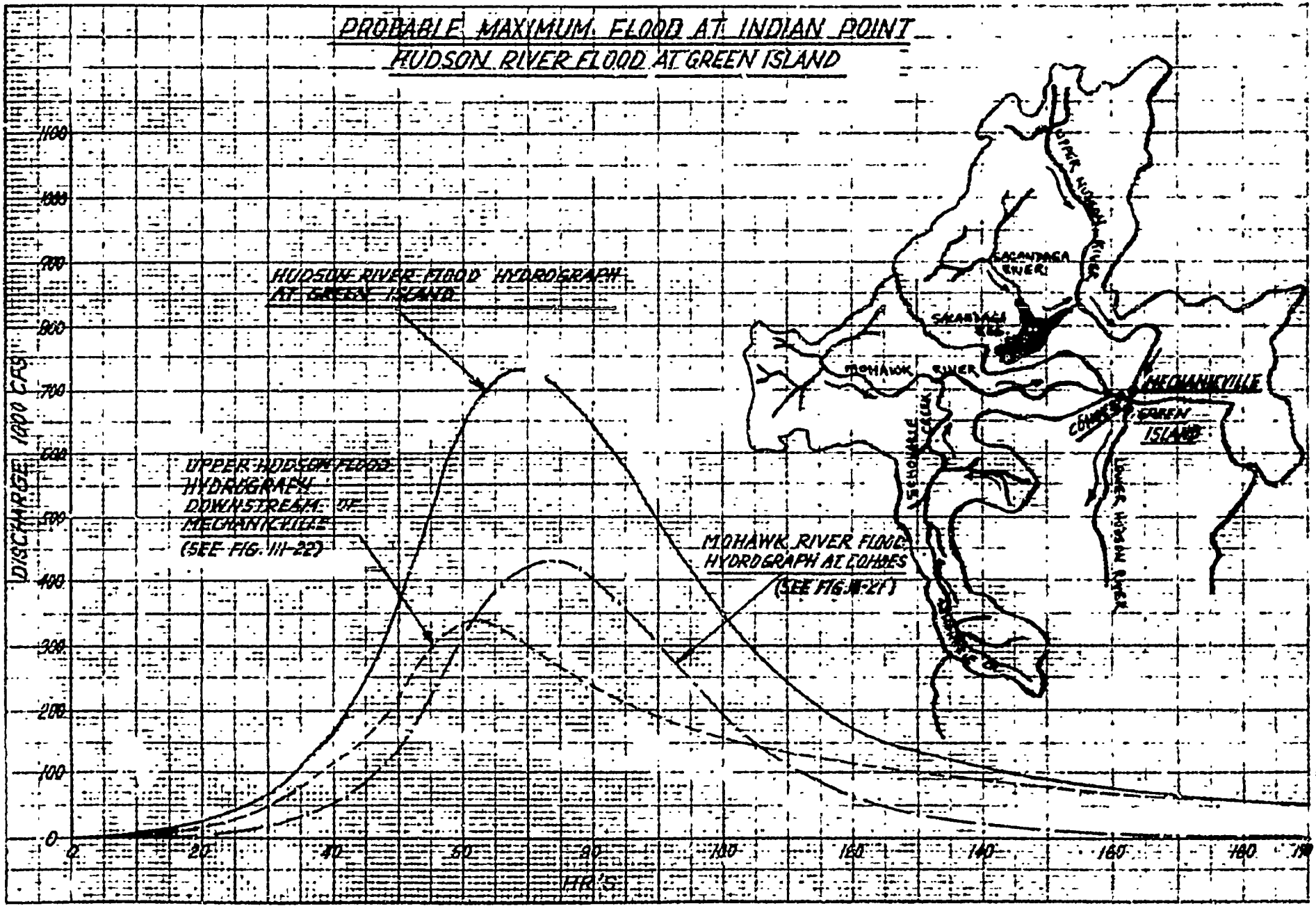


Figure III-23

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.

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

LOWER HUDSON RIVER

FLOOD HYDROGRAPHS PROGRAM

```
LISTNH
13
50 DIM G(8,200),A(150),M(150),H(150),E(150)
55 DIM W(200),L(200),U(200)
212 PRINT
220 FOR Y=1 TO 8
230 READ S
240 READ K
250 FOR J=1 TO K
260 READ G(Y,J)
270 NEXT J
280 PRINT
281 PRINT" SUBBASIN :"S,"U.H."
290 PRINT
293 PRINT" HR'S ; 1000CFS"
300 PRINT
305 FOR J=1 TO K
310 PRINT 2*J;";G(Y,J),
320 NEXT J
330 NEXT Y
790 DIM P(12),Q(8,200),O(200),A(200)
800 FOR Y=1 TO 8
810 READ S
820 READ B
830 FOR I=1 TO 12
840 READ P(I)
850 NEXT I
860 FOR I=1 TO 12
890 FOR J=1 TO 150
900 LET N=J+((I-1)*3)
910 LET Q(Y,N)=(G(Y,J)*P(I))+Q(Y,N)
920 NEXT J
930 NEXT I
940 FOR N=1 TO 200
950 LET Q(Y,N)=Q(Y,N)+B
960 NEXT N
970 PRINT
980 PRINT" PMF   -        SUBBASIN :"S
990 PRINT
1000 PRINT" BASE FLOW :"B;"1000CFS"
1010 PRINT
1020 PRINT" HR'S ;1000CFS"
1030 PRINT
1040 PRINT
```

```
1050 FOR  N=1 TO 50
1060 PRINT 2*N;"3";U(Y,N),
1070 NEXT N
1080 PRINT
1090 PRINT
1100 PRINT
1110 NEXT Y
1150 READ K1
1160 FOR N=1 TO K1
1170 READ M(N),H(N)
1180 NEXT N
1190 FOR N=1 TO K1
1200 LET A(N)=M(N)+H(N)
1210 NEXT N
1212 LET F9=13.49
1213 GOSUB 9000
1214 FOR N=1 TO K1
1215 LET U(N)=L(N)
1216 NEXT N
1220 PRINT
1225 PRINT " HUDSON RIVER AT GREEN ISLAND  -  PMF"
1230 PRINT
1235 PRINT "HR'S          1000CFS"
1240 PRINT
1245 FOR N=1 TO 70
1250 PRINT 2*N,A(N)
1255 NEXT N
1256 PRINT
1260 LET N5=3
1270 GOSUB 8000
1280 FOR N=1 TO K1
1290 LET A(N)=A(N)+U(1,N)
1300 NEXT N
1310 LET N5=1
1320 GOSUB 8000
1330 FOR N=1 TO K1
1340 LET A(N)=A(N)+U(2,N)+U(3,N)+U(4,N)
1350 NEXT N
1352 LET N5=0
1353 GOSUB 8000
1354 FOR N=1 TO K1
1355 LET A(N)=A(N)+U(5,N)
1356 NEXT N
1360 PRINT
1365 PRINT
1370 PRINT"HUDSON RIVER ABOVE ESOPUS CREEK  -  PMF"
1380 PRINT
1385 PRINT " HR'S          1000CFS"
1390 PRINT
1395 FOR N=1 TO 90
1400 PRINT 2*N,A(N)
1410 NEXT N
1420 PRINT
1430 FOR N=1 TO 100
1440 READ E(N)
1450 NEXT N
1460 FOR N=1 TO 120
1470 LET A(N)=A(N)+E(N)
```

```
1480 NEXT N
1490 PRINT
1500 PRINT "HUDSON RIVER BELOW ESOPUS CREEK  -  PMF"
1510 PRINT
1520 PRINT" HR'S          1000CFS"
1530 PRINT
1540 FOR N=1 TO 80
1550 PRINT N*2,A(N)
1560 NEXT N
1570 PRINT
1580 PRINT
1620 LET N5=1
1630 GOSUB 8000
1640  FOR N=1 TO 120
1650 LET A(N)=A(N)+Q(6,N)+Q(7,N)
1660 NEXT N
1661 PRINT "HUDSON RIVER BELOW RONDOUT CREEK  -  PMF"
1662 PRINT
1663 PRINT " HR'S       1000 CFS"
1664 PRINT
1665 FOR N=1 TO 90
1666 PRINT 2*N,A(N)
1667 NEXT N
1668 PRINT
1669 PRINT
1670 LET N5=3
1680 GOSUB 8000
1690 FOR N=1 TO 120
1700 LET A(N)=A(N)+Q(8,N)
1710 NEXT N
1712 LET F9=14.01
1713 GOSUB 9000
1720 PRINT
1730 PRINT
1740 PRINT
1750 PRINT " HUDSON RIVER AT INDIAN POINT"
1760 PRINT" PROBABLE MAXIMUM FLOOD"
1770 PRINT"-------------------------------"
1780 PRINT
1790 PRINT" HR'S          1000CFS"
1800 PRINT
1810 FOR N=1 TO 100
1820 PRINT 2*N,A(N)
1830 NEXT N
1840 PRINT
1850 PRINT
1860 PRINT " 0 OF TIME IS BEGINNING OF PMP"
1870 PRINT
1871 PRINT
1880 PRINT" S-HYDROGRAPH AT GREEN ISLAND AND AT INDIAN POINT"
1900 PRINT
1919PRINT"TIME            G.I.          I.P."
1920 PRINT"HR'S          1000CFS       1000CFS"
1930 PRINT
1935 FOR N=1 TO 85
1940 PRINT 2*N,U(N),L(N)
1950 NEXT N
1960 PRINT
1970 GOTO 90000
```

```
8000 LET R=1
8005 IF N5=0 GOTO8120
8010 LET C1=1/1.2
8020 LET C2=1/6
8030 LET O(1)=A(1)
8035 FOR N=2 TO 110-R
8040 LET O(N)=C1*(A(N-1)-O(N-1))+C2*(A(N)-A(N-1))+O(N-1)
8050 NEXT N
8060 FOR N=1 TO 110-R
8070 LET A(N)=O(N)
8080 NEXT N
8090 LET R=R+1
8100 IF R>N5 GOTO 8120
8110 GOTO 8030
8120 RETURN TO MAIN LINE
9000 FOR N=1 TO K1
9010 LET W(N)=(A(N)-A(1))/(F9-4.3)
9020 NEXT N
9030 FOR N=1 TO 36
9040 LET L(N)=W(N)
9050 NEXT N
9055 FOR N=37 TO 72
9060 LET L(N)=W(N)+L(N-36)
9070 NEXT N
9080 FOR N=73 TO K1
9090 LET L(N)=W(N)+L(N-36)
9100 NEXT N
9110 RETURN TO MAIN LINE
90000 END
```

LOWER HUDSON RIVER
FLOOD HYDROGRAPHS PROGRAM
DATA

```
3000 DATA 18.2,21   SUBBASIN # 18.2
3010 DATA 0.70,2.60,6.40,10.0
3020 DATA 13.69,13.57,15.25,14.20
3030 DATA 13.10,11.65,10.40,9.25
3040 DATA 8.10,6.95,5.65,4.90
3050 DATA 4.00,3.25,2.60,2.05
3060 DATA 1.45,1.05,0.75,0.50
3070 DATA 0.28,0.12,0.00
3100 DATA 18.3,30
3110 DATA 1.10,..75,5.60,3.65
3120 DATA 11.25,12.35,12.00,11.15
3130 DATA 10.05,8.90,7.90,6.90
3140 DATA 5.96,5.12,4.35,3.65
3150 DATA 3.10,2.64,2.25,1.86
3160 DATA 1.53,1.25,1.00,0.80
3170 DATA 0.62,0.47,0.33,0.22
3180 DATA 0.11,0.00
3200 DATA 18.4,29
3210 DATA 0.65,1.45,2.30,3.45
3220 DATA 4.70,5.43,5.40,5.12
3230 DATA 4.82,4.48,4.14,3.75
3240 DATA 3.40,3.02,2.37,2.04
3250 DATA 1.74,1.47,1.32,1.00
3260 DATA 0.82,0.65,0.50,0.37
3270 DATA 0.27,0.18,0.10,0.04
3280 DATA 0.00
3300 DATA 18.1,46
3310 DATA 0.10,0.30,0.55,0.90
3320 DATA 1.50,2.20,3.60,5.60
3330 DATA 8.60 ,9.70,16.20,17.05
3340 DATA 17.30,17.20,16.60,15.00
3350 DATA 13.80,12.50,11.90,10.20
3360 DATA 9.10,8.00,7.00,6.00
3370 DATA 5.00,4.10,3.20,2.60
3380 DATA 2.15,1.80,1.50,1.30
3390 DATA 1.05,0.90,0.75,0.60
3400 DATA 0.45,0.35,0.30,0.20
3410 DATA 0.15,0.10,0.05,0.04
3420 DATA 0.03,0.02,0.01,0.00
3450 DATA 18.5,3
3460 DATA 4.75,4.75,4.75
3500 DATA 21.1,26
3510 DATA 0.45,1.00,1.70,3.30
3520 DATA 5.40,8.00,11.30,11.45
3530 DATA 10.70,9.80,8.88,7.95
3540 DATA 7.10,6.25,5.42,4.60
3550 DATA 3.82,3.10,2.45,1.80
3560 DATA 1.30,0.94,0.65,0.40
3570 DATA 0.20,0.00
3600 DATA 21.2,45
3610 DATA 0.50,1.05,1.75,2.80
3620 DATA 4.20,6.10,8.30,10.90
3630 DATA 12.10,12.55,12.62,12.46
3640 DATA 12.10,11.48,10.92,10.35
3650 DATA 9.76,9.22,8.65,8.15
3660 DATA 7.55,7.00,6.45,5.95
3670 DATA 5.45,4.95,4.45,4.00
3680 DATA 3.55,3.10,2.65,2.30
3690 DATA 2.00,1.70,1.45,1.25
3700 DATA 1.02,0.86,0.68,0.53
3710 DATA 0.40,0.30,0.20,0.10
3720 DATA 0.00
```

UNIT HYDROGRAPH DATA

U.H.

U.H

U H

U.H

PLATE, III - 11
(SHEET 2 OF 3)

```
3810 DATA 0.05,0.10,0.15,0.22
3820 DATA 0.30,0.40,0.51,0.59
3830 DATA 0.80,1.00,1.35,1.60
3840 DATA 2.00,2.60,3.20,4.00
3850 DATA 5.10,8.40,12.00,16.20
3860 DATA 17.85,18.25,18.25,18.05
3870 DATA 17.50,16.20,14.20,12.50
3880 DATA 11.00,9.00,7.40,6.50
3890 DATA 5.75,5.15,4.75,4.40
3900 DATA 4.00,3.65,3.15,2.90
3910 DATA 2.75,2.55,2.40,2.25
3920 DATA 2.05,1.90,1.75,1.62
3930 DATA 1.52,1.38,1.25,1.15
3940 DATA 1.00,0.92,0.80,0.72
3950 DATA 0.62,0.52,0.42,0.34
3960 DATA 0.24,0.18,0.10,0.05
3970 DATA 0.00
```

... HYDROGRAPH DATA

```
4000 DATA (18.2),0.51
```

SUBBASIN # 18.2, BASE FLOW: 0.51 x 10^3 CFS

```
4010 DATA 0.00,0.45,0.59,0.24
4020 DATA 1.3,3.98,1.47,0.97
4030 DATA 0.03,0.08,0.14,0.04
```

RUNOFF DATA

```
4100 DATA (18.3),0.42
4110 DATA 0.00,0.71,0.72,0.31
4120 DATA 1.64,7.45,1.96,1.22
4130 DATA 0.09,0.17,0.23,0.12
```

RUNOFF DATA

```
4200 DATA 18.4,0.21
4210 DATA 0.00,0.45,0.58,0.24
4220 DATA 1.33,4.09,1.51,0.98
4230 DATA 0.03,0.10,0.15,0.05
430 DATA (18.1),0.75
4310 DATA 0.00,0.58,0.66,0.38
4320 DATA 1.53,5.91,1.83,1.07
4330 DATA 0.07,0.16,0.20,0.10
4400 DATA (18.5),0.04
4410 DATA 0.00,0.94,0.84,0.39
4420 DATA 1.81,9.50,2.35,1.31
4430 DATA 0.15,0.21,0.26,0.23
4500 DATA (21.1),0.40
4510 DATA 0.00,0.61,0.68,0.29
4520 DATA 1.42,3.78,1.59,1.07
4530 DATA 0.05,0.12,0.17,0.07
4600 DATA (21.2),0.80
4610 DATA 0.00,0.46,0.59,0.23
4620 DATA 1.31,3.70,1.51,1.03
4630 DATA 0.03,0.10,0.15,0.04
4700 DATA (22),0.92
4710 DATA 0.00,0.46,0.60,0.24
4720 DATA 1.35,4.10,1.43,0.93
4730 DATA 0.02,0.08,0.12,0.04
5000 DATA 100
5010 DATA 2.78,3.47,2.78,3.47
5020 DATA 2.78,3.47,2.50,3.48
5030 DATA 9.00,3.15,7.06,3.34
5040 DATA 13.50,3.53,16.50,3.63
5050 DATA 13.03,3.83,17.00,4.29
5060 DATA 19.50,15.15,20.47,6.53
5070 DATA 39.00,6.73,47.30,11.94
5080 DATA 47.13,16.16,88.00,21.42
5090 DATA 112.00,27.84,97.09,35.37
5100 DATA 119.00,43.82,130.50,53.58
5110 DATA 127.17,111.76,150.00,137.96
5120 DATA 173.00,95.31,192.27,115.39
5130 DATA 335.00,139.87,352.00,160.47
```

UPPER MOHAWK RIVER AT MECHANICVILLE AND MOHAWK RIVER AT COHOES FLOOD HYDROGRAPHS DATA (SEE PLATE III-3

```
5150 DATA 324.00,276.13,334.38,311.02
5160 DATA 336.00,344.75,334.00,374.43
5170 DATA 324.69,395.45,313.50,412.17
5180 DATA 302.00,424.21,286.01,429.02
5190 DATA 275.00,429.50,260.00,425.04
5200 DATA 248.00,414.86,238.50,401.32
5210 DATA 227.00,384.47,215.68,365.15
5220 DATA 207.00,344.33,196.00,322.64
5230 DATA 189.00,300.89,181.00,279.12
5240 DATA 174.20,257.33,166.55,235.37
5250 DATA 160.20,215.18,154.50,194.99
5260 DATA 148.36,175.65,143.80,157.49
5270 DATA 138.40,140.52,133.71,124.03
5280 DATA 129.60,110.76,125.60,98.07
5290 DATA 121.73,86.80,118.40,76.50
5300 DATA 115.00,68.05,111.53,60.38
5310 DATA 108.50,53.60,105.90,47.64
5320 DATA 103.16,42.40,100.60,36.01
5330 DATA 98.60,34.12,96.39,30.72
5340 DATA 94.30,27.70,92.50,25.04
5350 DATA 90.35,22.36,88.40,20.50
5360 DATA 88.50,18.55,84.33,16.74
5370 DATA 83.50,15.13,80.50,13.60
5380 DATA 78.71,12.24,77.00,11.02
5390 DATA 75.10,9.93,73.60,8.96
5400 DATA 72.00,8.11,70.20,7.37
5410 DATA 68.66,6.74,67.10,6.20
5420 DATA 65.90,5.74,64.43,5.36
5430 DATA 63.20,5.03,61.90,4.75
5440 DATA 60.19,4.50,59.00,4.29
5450 DATA 57.80,4.11,56.15,3.96
5460 DATA 54.90,3.84,53.60,3.74
5470 DATA 52.32,3.66,50.90,3.61
5480 DATA 49.70,3.57,48.72,3.54
5490 DATA 47.60,3.52,46.40,3.50
5500 DATA 45.23,3.49,44.20,3.49
7010 DATA 0.17,0.17,0.17,0.29
7020 DATA 0.41,0.54,0.54,1.25
7030 DATA 1.81,2.79,4.61,4.76
7040 DATA 5.15,10.17,10.02,14.66
7050 DATA 16.61,18.95,24.37,32.09
7060 DATA 35.64,51.23,66.59,60.29
7070 DATA 80.61,79.32,70.12,69.85
7080 DATA 64.37,54.37,50.89,43.50
7090 DATA 6.13,30.40,25.32,20.35
7100 DATA 17.21,14.30,10.56,9.57
7110 DATA 8.12,6.47,5.82,4.88
7120 DATA 3.96,3.43,2.95,2.53
7130 DATA 2.19,1.94,1.70,1.52
7140 DATA 1.36,1.21,1.11,1.03
7150 DATA 0.95,0.90,0.86,0.82
7160 DATA 0.82,0.81,0.80,0.80
7170 DATA 0.79,0.79,0.78,0.78
7180 DATA 0.77,0.76,0.76,0.75
7190 DATA 0.74,0.75,0.73,0.73
7200 DATA 0.72,0.71,0.70,0.70
7210 DATA 0.70,0.70,0.69,0.69
7220 DATA 0.68,0.68,0.68,0.67
7230 DATA 0.67,0.67,0.66,0.66
7240 DATA 0.66,0.66,0.66,0.65
7250 DATA 0.65,0.65,0.65,0.65
```

UPPER HUDSON RIVER AT MECHANICVILLE AND MOHAWK RIVER AT COMMES FLOOD HYDROGRAPHS DATA (SEE PLATE III-3 AND PLATE III-9)

ESOPUS CREEK FLOOD HYDROGRAPH DATA (ROUTED THROUGH ASHOKAN RESERVOIR) (SEE CHAPTER IV)

[illegible]
LOWER H DSON RIVER
[illegible]
PRINTOUT

SUBBASIN : 18.2 U.H.

HR'S : 1000CFS

2 : .7	4 : 2.6	6 : 6.4	8 : 10	10 : 13.69
12 : 15.57	14 : 15.25	16 : 14.2	18 : 13.1	20 : 11.65
22 : 10.4	24 : 9.25	26 : 8.1	28 : 6.95	30 : 5.85
32 : 4.9	34 : 4	36 : 3.25	38 : 2.6	40 : 2.05
42 : 1.45	44 : 1.05	46 : .75	48 : .5	50 : .28
52 : .12	54 : 0			

SUBBASIN: 18.3 U.H.

HR'S : 1000CFS

2 : 1.1	4 : 3.75	6 : 5.[illegible]	8 : 6.65	10 : 11.[illegible]
12 : 12.[illegible]	14 : 12	16 : 11.15	18 : 10.[illegible]	20 : [illegible]
22 : 7.9	24 : 6.9	26 : [illegible]	28 : 5.12	30 : [illegible]
32 : 3.6[illegible]	34 : 3.1	36 : [illegible]	38 : 2.2[illegible]	40 : 1.[illegible]
42 : 1.53	44 : 1.25	46 : 1	48 : [illegible]	50 : [illegible]
52 : .47	54 : .[illegible]	56 : .[illegible]	58 : .11	60 : 0

SUBBASIN: 18.4 U.H.

HR'S : 1000CFS

2 : .65	4 : 1.[illegible]	6 : [illegible]	8 : [illegible]	10 : [illegible]
12 : 5.43	14 : 5.4	16 : 5.1[illegible]	18 : 4.[illegible]	20 : [illegible]
22 : [illegible]	24 : 3.75	[illegible] : [illegible]	28 : 3.02	30 : [illegible]
32 : [illegible]	34 : 1.74	36 : 1.47	38 : 1.32	40 : 1
42 : [illegible]	44 : .65	46 : .5	48 : .3[illegible]	50 : .27
52 : .1[illegible]	54 : .1	56 : .04	58 : 0	

SUBBASIN: 18.1 U.H.

HR'S : 1000CFS

2 : .1	4 : .3	6 : .[illegible]	8 : .9	10 : 1.[illegible]
12 : 2.[illegible]	14 : 3.6	16 : 3.6	18 : 3.6	20 : [illegible]
22 : 10.[illegible]	24 : 17.05	26 : 17.[illegible]	28 : 17.[illegible]	30 : 1[illegible]
32 : 15	34 : 13.[illegible]	36 : 12.[illegible]	38 : 11.[illegible]	40 : 1[illegible]
42 : 9.1	44 : 8	46 : 7	48 : 6	50 : [illegible]
52 : 4.1	54 : 3.[illegible]	56 : 2.6	58 : 2.1[illegible]	60 : 1.[illegible]
62 : 1.5	64 : 1.3	66 : 1.0[illegible]	68 : .9	70 : .7[illegible]
72 : .6	74 : .45	76 : .3[illegible]	78 : .3	[illegible] : .[illegible]
82 : .15	84 : .1	86 : .05	88 : .04	90 : .03
92 : .0[illegible]	94 : .01	96 : 0		

SUBBASIN: 18.5 U.H.

HR'S ; 1000CFS

2 ; 4.75	4 ; 4.75	6 ; 4.75

SUBBASIN: 21.1 U.H.

HR'S ; 1000CFS

2 ; .45	4 ; 1	6 ; 1.7	8 ; 3.3	10 ; 5.4
12 ; 8	14 ; 11.3	16 ; 11.65	18 ; 10.7	20 ; 9.8
22 ; 8.68	24 ; 7.95	26 ; 7.1	28 ; 6.25	30 ; 5.42
32 ; 4.6	34 ; 3.82	36 ; 3.1	38 ; 2.45	40 ; 1.8
42 ; 1.3	44 ; .94	46 ; .65	48 ; .4	50 ; .2
52 ; 0				

SUBBASIN: 21.2 U.H.

HR'S ; 1000CFS

2 ; .5	4 ; 1.05	6 ; 1.75	8 ; 2.8	10 ; 4.2
12 ; 6.1	14 ; 8.3	16 ; 10.9	18 ; 12.1	20 ; 12.55
22 ; 12.62	24 ; 12.46	26 ; 12.1	28 ; 11.48	30 ; 10.92
32 ; 10.35	34 ; 9.76	36 ; 9.22	38 ; .65	40 ; 8.15
42 ; 7.55	44 ; 7	46 ; 6.45	48 ; .95	50 ; 5.45
52 ; 4.95	54 ; 4.45	56 ; 4	58 ; 3.55	60 ; 3.1
62 ; 2.65	64 ; 2.3	66 ; 2	68 ; 1.7	70 ; 1.45
72 ; 1.25	74 ; 1.02	76 ; .86	78 ; .68	80 ; .53
82 ; .4	84 ; .3	86 ; .2	88 ; .1	90 ; 0

SUBBASIN: 22 U.H.

HR'S ; 1000CFS

2 ; .05	4 ; .1	6 ; .15	8 ; .22	10 ; .3
12 ; .4	14 ; .51	16 ; .59	18 ; .6	20 ; 1
22 ; 1.35	24 ; 1.6	26 ; 2	28 ; 2.6	30 ; 3.2
32 ; 4	34 ; 5.1	36 ; 8.4	38 ; 12	40 ; 16.2
42 ; 17.85	44 ; 18.25	46 ; 18.25	48 ; 18.05	50 ; 17.5
52 ; 16.2	54 ; 14.2	56 ; 12.5	58 ; 11	60 ; 9
62 ; 7.4	64 ; 6.5	66 ; 5.75	68 ; 5.15	70 ; 4.75
72 ; 4.4	74 ; 4	76 ; 3.65	78 ; 3.15	80 ; 2.9
82 ; 2.75	84 ; 2.55	86 ; 2.4	88 ; 2.25	90 ; 2.05
92 ; 1.9	94 ; 1.75	96 ; 1.62	98 ; 1.52	100 ;
1.38	102 ; 1.25	104 ; 1.15	106 ; 1	108 ;
.92	110 ; .8	112 ; .72	114 ; .62	116 ;
.52	118 ; .42	120 ; .34	122 ; .24	124 ;
.18	126 ; .1	128 ; .05	130 ; 0	

PMF - SUBBASIN : 18.2

BASE FLOW : .51 1000CFS

HR'S : 1000CFS

2 : .51	4 : .51	6 : .51	8 : .825	10 : 1.68
12 : 3.39	14 : 5.423	16 : 8.2045	18 : 11.2925	20 : 15.4405
22 : 15.6011	24 : 17.1273	26 : 18.06	28 : 20.2336	30 : 24.4583
32 : 30.4745	34 : 41.3265	36 : 57.457	38 : 70.944	40 : 85.6747
42 : 96.0506	44 : 97.734	46 : 98.1428	48 : 97.7429	50 : 92.N28
52 : 87.6753	54 : 81.3424	56 : 72.7615	58 : 64.0402	60 : 55.9446
62 : 48.3722	64 : 41.5355	66 : 35.5586	68 : 30.3477	70 : 25.7629
72 : 21.1443	74 : 17.338	76 : 14.1206	78 : 11.1563	80 : 8.7259
82 : 6.7786	84 : 5.036	86 : 4.0101	88 : 3.1844	90 : 2.4875
92 : 2.0311	94 : 1.6509	96 : 1.33	98 : 1.1624	100 : 1.0206

PMF - SUBBASIN : 18.3

BASE FLOW : .42 1000CFS

HR'S : 1000CFS

2 : .42	4 : .42	6 : .42	8 : 1.201	10 : 2.3725
12 : 4.538	14 : 7.3535	16 : 10.3875	18 : 13.3645	20 : 15.509
22 : 17.289	24 : 18.2668	26 : 19.8645	28 : 22.0545	30 : 25.9171
32 : 37.1606	34 : 52.1372	36 : 75.0653	38 : 96.3402	40 : 116.245
42 : 127.472	44 : 128.765	46 : 126.988	48 : 122.45	50 : 114.31
52 : 106.508	54 : 96.7928	56 : 85.8952	58 : 75.8453	60 : 66.3606
62 : 57.6357	64 : 50.5136	66 : 44.3675	68 : 38.9708	70 : 33.9665
72 : 29.5853	74 : 25.5279	76 : 21.8208	78 : 18.5057	80 : 15.4235
82 : 12.6355	84 : 10.0668	86 : 8.061	88 : 6.2007	90 : 4.4576
92 : 3.6573	94 : 2.9426	96 : 2.2839	98 : 1.9122	100 : 1.5663

PMF - SUBBASIN : 18.4

BASE FLOW : .21 1000CFS

HR'S : 1000CFS

2 : .21	4 : .21	6 : .21	8 : .5025	10 : .8625
12 : 1.245	14 : 2.1395	16 : 3.166	18 : 3.9375	20 : 4.797
22 : 5.538	24 : 6.0804	26 : 7.0505	28 : 8.0971	30 : 9.0555
32 : 12.6814	34 : 17.3605	36 : 21.2372	38 : 26.4492	40 : 31.9603
42 : 35.2334	44 : 36.6941	46 : 37.359	48 : 37.155	50 : 36.157
52 : 34.7625	54 : 32.3631	56 : 29.3698	58 : 27.042	60 : 23.0956
62 : 20.7657	64 : 18.3395	66 : 15.746	68 : 14.1951	70 : 12.0309
72 : 10.1462	74 : 8.7621	76 : 7.2172	78 : 5.9757	80 : 4.9056
82 : 3.9124	84 : 3.0659	86 : 2.3959	88 : 1.8358	90 : 1.4306
92 : 1.1625	94 : .9134	96 : .7401	98 : .6823	100 : .5084

PMF - SUBBASIN : 18.1V

BASE FLOW : .75 1000CFS

HR'S ; 1000CFS

2 ; .75	4 ; .75	6 ; .75	8 ; .808	10 ; .924
12 ; 1.069	14 ; 1.338	16 ; 1.818	18 ; 2.389	20 ; 3.47
22 ; 5.102	24 ; 7.399	26 ; 9.247	28 ; 14.871	30 ; 17.9925
32 ; 20.322	34 ; 27.613	36 ; 31.5155	38 ; 35.564	40 ; 44.244
42 ; 52.6015	44 ; 61.997	46 ; 83.258	48 ; 102.113	50 ; 110.298
52 ; 150.718	54 ; 159.28	56 ; 160.347	58 ; 170.181	60 ; 167.9
62 ; 157.909	64 ; 154.079	66 ; 143.669	68 ; 135.211	70 ; 121.491
72 ; 110.425	74 ; 99.58	76 ; 89.001	78 ; 78.689	80 ; 69.194
82 ; 60.7045	84 ; 51.903	86 ; 44.539	88 ; 39.0235	90 ; 33.4165
92 ; 28.626	94 ; 24.7592	96 ; 20.8249	98 ; 17.9151	100 ;
15.3137				

PMF SUBBASIN : 18.5

BASE FLOW : .04 1000CFS

HR'S ; 1000CFS

2 ; .04	4 ; .04	6 ; .04	8 ; 4.505	10 ; 4.505
12 ; 4.505	14 ; 4.03	16 ; 4.03	18 ; 4.03	20 ; 1.8925
22 ; 1.8925	24 ; 1.8925	26 ; 8.6375	28 ; 8.6375	30 ; 8.6375
32 ; 45.165	34 ; 45.165	36 ; 45.165	38 ; 11.2025	40 ; 11.2025
42 ; 11.2025	44 ; 6.2625	46 ; 6.2625	48 ; 6.2625	50 ; .7525
52 ; .7525	54 ; .7525	56 ; 1.0375	58 ; 1.0375	60 ; 1.0375
62 ; 1.275	64 ; 1.275	66 ; 1.275	68 ; 1.1325	70 ; 1.1325
72 ; 1.1325	74 ; .04	76 ; .04	78 ; .04	80 ; .04
82 ; .04	84 ; .04	86 ; .04	88 ; .04	90 ; .04
92 ; .04	94 ; .04	96 ; .04	98 ; .04	100 ;
.04				

PMF - SUBBASIN : 21.1

BASE FLOW : .4 1000CFS

HR'S ; 1000CFS

2 ; .4	4 ; .4	6 ; .4	8 ; .6745	10 ; 1.01
12 ; 1.437	14 ; 2.719	16 ; 4.374	18 ; 6.436	20 ; 9.6875
22 ; 11.3465	24 ; 12.86	26 ; 15.658	28 ; 16.5888	30 ; 17.2595
32 ; 21.059	34 ; 25.0194	36 ; 30.0012	38 ; 40.1115	40 ; 47.8164
42 ; 56.4191	44 ; 69.44	46 ; 71.4547	48 ; 71.1468	50 ; 72.6199
52 ; 69.6032	54 ; 65.8324	56 ; 63.1337	58 ; 57.1741	60 ; 50.6321
62 ; 44.4881	64 ; 38.4137	66 ; 32.9883	68 ; 28.4033	70 ; 23.6943
72 ; 19.7709	74 ; 16.7377	76 ; 13.509	78 ; 10.9	80 ; 8.8111
82 ; 6.6116	84 ; 5.3329	86 ; 4.2913	88 ; 3.328	90 ; 2.7429
92 ; 2.234	94 ; 1.7354	96 ; 1.4824	98 ; 1.2613	100 ;
1.0514				

PMF - SUBBASIN : 21.2

BASE FLOW : .8 1000CFS

HR'S ; 1000CFS

2 ; .8	4 ; .8	6 ; .8	8 ; 1.03	10 ; 1.283
12 ; 1.605	14 ; 2.383	16 ; 3.3515	18 ; 4.6385	20 ; 6.385
22 ; 8.5335	24 ; 10.3675	26 ; 12.769	28 ; 15.3777	30 ; 17.3661
32 ; 21.1975	34 ; 25.4206	36 ; 30.4236	38 ; 37.5745	40 ; 46.3699
42 ; 55.4133	44 ; 65.562	46 ; 77.2335	48 ; 84.3305	50 ; 89.222
52 ; 93.3696	54 ; 95.1258	56 ; 95.3885	58 ; 94.5328	60 ; 92.211d
62 ; 88.8935	64 ; 84.8634	66 ; 80.783	68 ; 76.5455	70 ; 72.3981
72 ; 68.0406	74 ; 63.802	76 ; 59.7587	78 ; 55.6422	80 ; 51.5012
82 ; 47.4494	84 ; 43.3189	86 ; 39.3591	88 ; 35.5382	90 ; 31.8496
92 ; 28.2363	94 ; 25.08	96 ; 22.1857	98 ; 19.3626	100 ;
16.9785				

PMF - SUBBASIN : 22

BASE FLOW : .92 1000CFS

HR'S ; 1000CFS

2 ; .92	4 ; .92	6 ; .92	8 ; .943	10 ; .966
12 ; .989	14 ; 1.0512	16 ; 1.118	18 ; 1.194	20 ; 1.2986
22 ; 1.3954	24 ; 1.564	26 ; 1.8063	28 ; 2.102	30 ; 2.4345
32 ; 3.0644	34 ; 3.8896	36 ; 4.699	38 ; 5.862	40 ; 7.3195
42 ; 10.0225	44 ; 13.1221	46 ; 16.8195	48 ; 21.0905	50 ; 25.2099
52 ; 30.4287	54 ; 34.448	56 ; 38.3127	58 ; 43.2482	60 ; 50.078
62 ; 57.9736	64 ; 67.8813	66 ; 82.5795	68 ; 97.5307	70 ; 115.281
72 ; 125.063	74 ; 130.849	76 ; 134.492	78 ; 135.37	80 ; 133.889
82 ; 129.936	84 ; 119.819	86 ; 109.967	88 ; 100.626	90 ; 88.271
92 ; 78.0515	94 ; 70.7165	96 ; 62.665	98 ; 56.062	100 ;
51.5945				

HUDSON RIVER AT GREEN ISLAND - PMF

HR'S	1000CFS
2	6.25
4	6.25
6	6.25
8	10.98
10	12.15
12	10.4
14	17.03
16	20.13
18	24.65
20	27
22	31.8
24	36.59
26	47.73
28	59.24
30	63.29
32	109.42
34	139.84
36	132.46
38	162.62
40	184.08
42	238.93
44	287.98
46	268.31
48	307.66
50	364.87
52	421.47
54	486.17
56	546
58	600.13
60	646.2
62	680.75
64	708.48
66	720.14
68	725.67
70	726.21
72	717.03
74	704.5
76	685.04
78	662.86
80	639.82
82	611.47
84	580.83
86	551.33
88	518.64
90	489.89
92	460.12
94	431.5
96	402.42
98	375.38
100	349.49

102	324.01
104	301.29
106	276.92
108	257.74
110	240.38
112	223.67
114	208.53
116	195.2
118	183.05
120	171.91
122	162.1
124	153.54
126	145.64
128	138.61
130	132.72
132	127.11
134	122
136	117.54
138	113.01
140	108.9

HUDSON RIVER ABOVE ESOPUS CREEK - PEAK

HR'S	1000CFS
2	8.18
4	8.18
6	8.18
8	13.8326
10	15.9031
12	19.9153
14	26.3143
16	34.6419
18	43.4022
20	50.0151
22	59.2524
24	67.1072
26	81.8421
28	96.2385
30	111.904
32	176.662
34	220.476
36	275.209
38	305.734
40	379.48
42	437.516
44	471.34
46	517.397
48	563.595
50	593.124
52	649.299
54	663.406
56	679.931

58	716.61
60	748.313
62	776.405
64	814.702
66	840.636
68	863.63
70	872.631
72	875.741
74	868.364
76	855.965
78	837.405
80	813.725
82	786.071
84	754.176
86	721.231
88	687.955
90	652.153
92	616.449
94	580.999
96	545.41
98	511.359
100	479.203
102	447.38
104	416.673
106	387.503
108	359.624
110	333.514
112	308.766
114	285.701
116	264.413
118	245.571
120	228.451
122	212.947
124	199.1
126	186.387
128	175.342
130	165.389
132	156.469
134	148.516
136	141.468
138	135.21
140	129.544
142	124.406
144	119.674
146	115.22
148	111.051
150	107.136
152	103.492
154	99.9926
156	96.4200
158	93.1176
160	90.0568
162	87.2114
164	84.5805
166	82.0653
168	79.7268
170	77.5333
172	75.4802
174	73.6054
176	71.843
178	70.1604
180	68.4779

PLATE III-12
(SHEET 9 OF 13)

HUDSON RIVER BELOW BOOMS CREEK - PAR

HR'S	1000CFS
2	8.35
4	8.35
6	8.35
8	14.1226
10	16.3131
12	20.4553
14	27.1543
16	36.0919
18	45.2122
20	52.8051
22	63.8624
24	71.8672
26	89.9931
28	106.408
30	121.924
32	190.942
34	237.286
36	294.159
38	350.104
40	411.57
42	474.156
44	532.62
46	583.967
48	623.885
50	673.734
52	726.619
54	733.526
56	749.781
58	781.16
60	802.633
62	829.295
64	858.202
66	846.966
68	894.03
70	897.951
72	896.091
74	886.074
76	870.265
78	847.983
80	823.295
82	794.191
84	760.648
86	727.051
88	692.835
90	656.133
92	619.879
94	583.949
96	547.94
98	514.049
100	481.143
102	449.08

104	415.193
106	388.863
108	361.034
110	334.424
112	309.796
114	286.651
116	265.313
118	246.431
120	229.271
122	213.767
124	199.91
126	187.367
128	176.142
130	166.159
132	157.259
134	149.296
136	142.246
138	135.98
140	130.304
142	125.166
144	120.424
146	115.96
148	111.601
150	107.666
152	104.222
154	100.713
156	97.1389
158	93.8176
160	90.7566

HUDSON RIVER BELOW RONDOUT CREEK - PMF

Hr's	1000 CFS
2	9.55
4	9.55
6	9.55
8	11.0166
10	15.979
12	19.6076
14	26.0256
16	35.331
18	47.272
20	61.0277
22	73.223
24	86.6708
26	101.911
28	121.943
30	140.881
32	173.072
34	239.085
36	299.083
38	368.586
40	431.334
42	521.43
44	606.477
46	673.012
48	736.17
50	786.836
52	837.731
54	881.418
56	892.58
58	904.1

104	418.193
106	388.863
108	361.034
110	334.424
112	309.796
114	286.651
116	265.313
118	246.431
120	229.271
122	213.767
124	199.91
126	187.367
128	176.142
130	166.159
132	157.259
134	149.296
136	142.248
138	135.98
140	130.304
142	125.166
144	120.424
146	115.96
148	111.601
150	107.866
152	104.222
154	100.713
156	97.1389
158	93.8176
160	90.7566

HUDSON RIVER BELOW RONDOUT CREEK - PMF

HR'S	1000 CFS
2	9.55
4	9.55
6	9.55
8	11.0166
10	15.979
12	19.6076
14	26.0256
16	35.331
18	47.272
20	61.0277
22	73.223
24	86.6708
26	101.911
28	121.943
30	140.881
32	173.072
34	239.085
36	299.083
38	368.586
40	431.334
42	521.43
44	606.477
46	673.012
48	736.17
50	786.836
52	837.731
54	881.418
56	892.58
58	904.1

HUDSON RIVER DOWNSTREAM OF RINDOUT CREEK – PMF (CONTINUATION)

PLATE III-12
(SHEET 12 OF 13)

60	922.81
62	936.714
64	953.063
66	965.364
68	960.53
70	984.368
72	983.84
74	974.951
76	958.1
78	935.534
80	907.682
82	876.518
84	841.963
86	804.143
88	765.788
90	726.991
92	686.605
94	646.749
96	607.616
98	568.916
100	532.302
102	497.034
104	462.733
106	430.109
108	399.111
110	369.678
112	341.869
114	316.033
116	291.943
118	269.845
120	250.03
122	232.414
124	216.518
126	202.262
128	189.517
130	178.103
132	167.926
134	158.932
136	150.91
138	143.791
140	137.446
142	131.719
144	126.522
146	121.736
148	117.252
150	113.061
152	109.131
154	105.458
156	101.907
158	98.3801
160	95.0679
162	92.0011
164	89.1545
166	86.5137
168	84.0044
170	81.6475
172	79.4435
174	77.3949
176	75.5082
178	73.7317
180	72.0335

PLATE III-12
(SHEET 12 OF 13)

HUDSON RIVER AT INDIAN POINT
PROBABLE MAXIMUM FLOOD

HR'S	1000CFS
2	10.47
4	10.47
6	10.47
8	10.4996
10	10.6306
12	11.3254
14	13.4831
16	17.2165
18	22.0309
20	28.9043
22	38.2391
24	49.8195
26	62.6324
28	75.8465
30	90.2153
32	107.013
34	127.044
36	152.894
38	192.91
40	248.841
42	312.784
44	382.081
46	457.672
48	542.24
50	624.551
52	699.307
54	764.655
56	822.999
58	875.673
60	918.294
62	947.329
64	973.464
66	1004.28
68	1034.45
70	1065.96
72	1085.32
74	1098.01
76	1111.65
78	1113.47
80	1103.95
82	1084.25
84	1052.16
86	1015.33
88	975.06
90	928.436
92	881.463
94	836.049
96	788.998
98	742.721
100	698.665
102	654.608
104	612.07

108	532.686
110	496.082
112	461.327
114	427.623
116	396.179
118	366.664
120	338.541
122	312.894
124	289.427
126	268.061
128	249.07
130	231.729
132	216.152
134	202.255
136	189.557
138	178.393
140	168.33
142	159.347
144	151.319
146	144.027
148	137.44
150	131.477
152	125.066
154	120.791
156	116.01
158	111.595
160	107.443
162	103.633
164	99.9756
166	96.5313
168	93.3625
170	90.4269
172	87.6672
174	85.1552
176	82.7736
178	80.5501
180	78.4807
182	76.5527
184	74.7392
186	73.0003
188	71.3075
190	69.6609
192	68.0702
194	66.5166
196	64.997
198	63.523
200	62.093

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

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.

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.

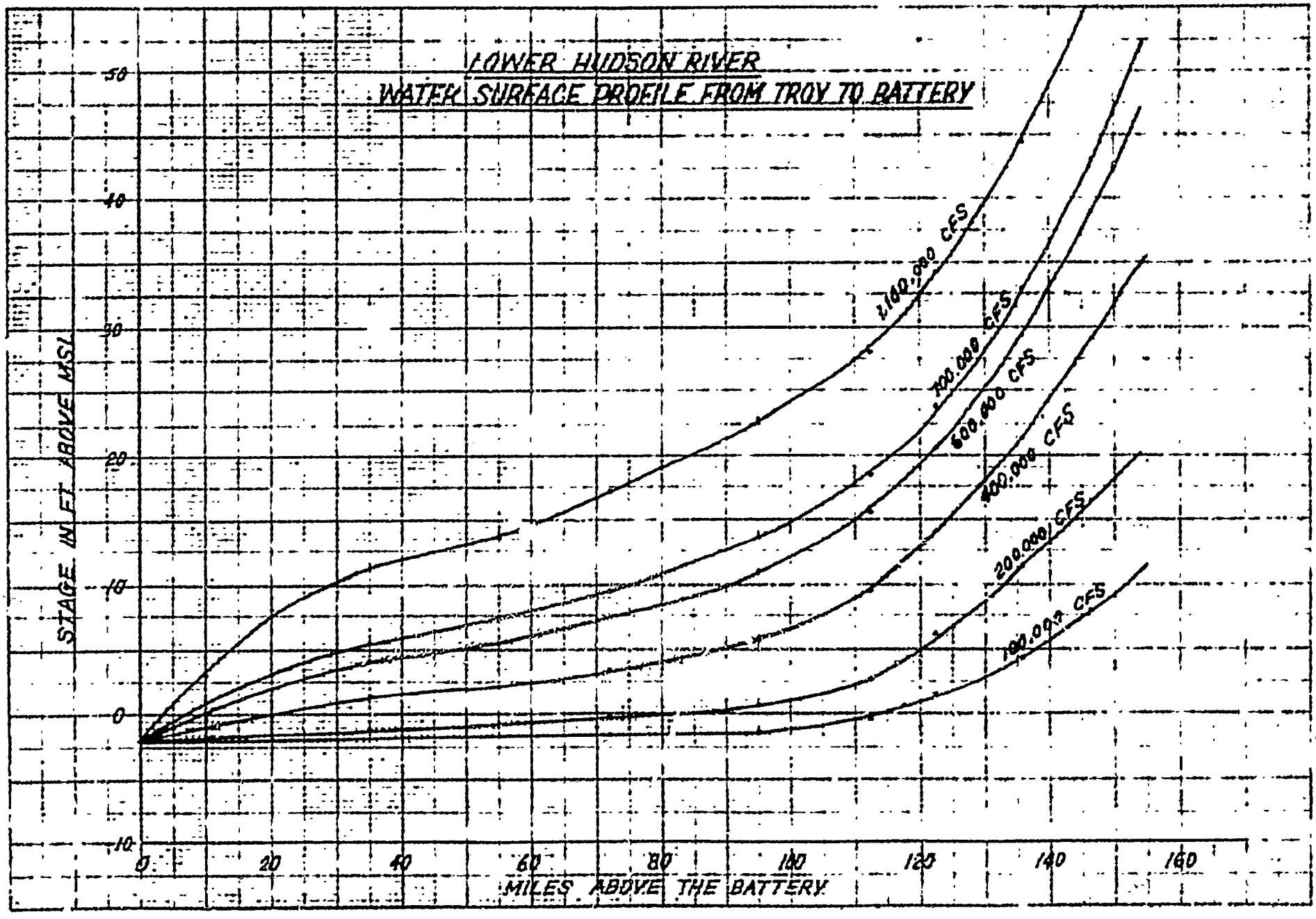
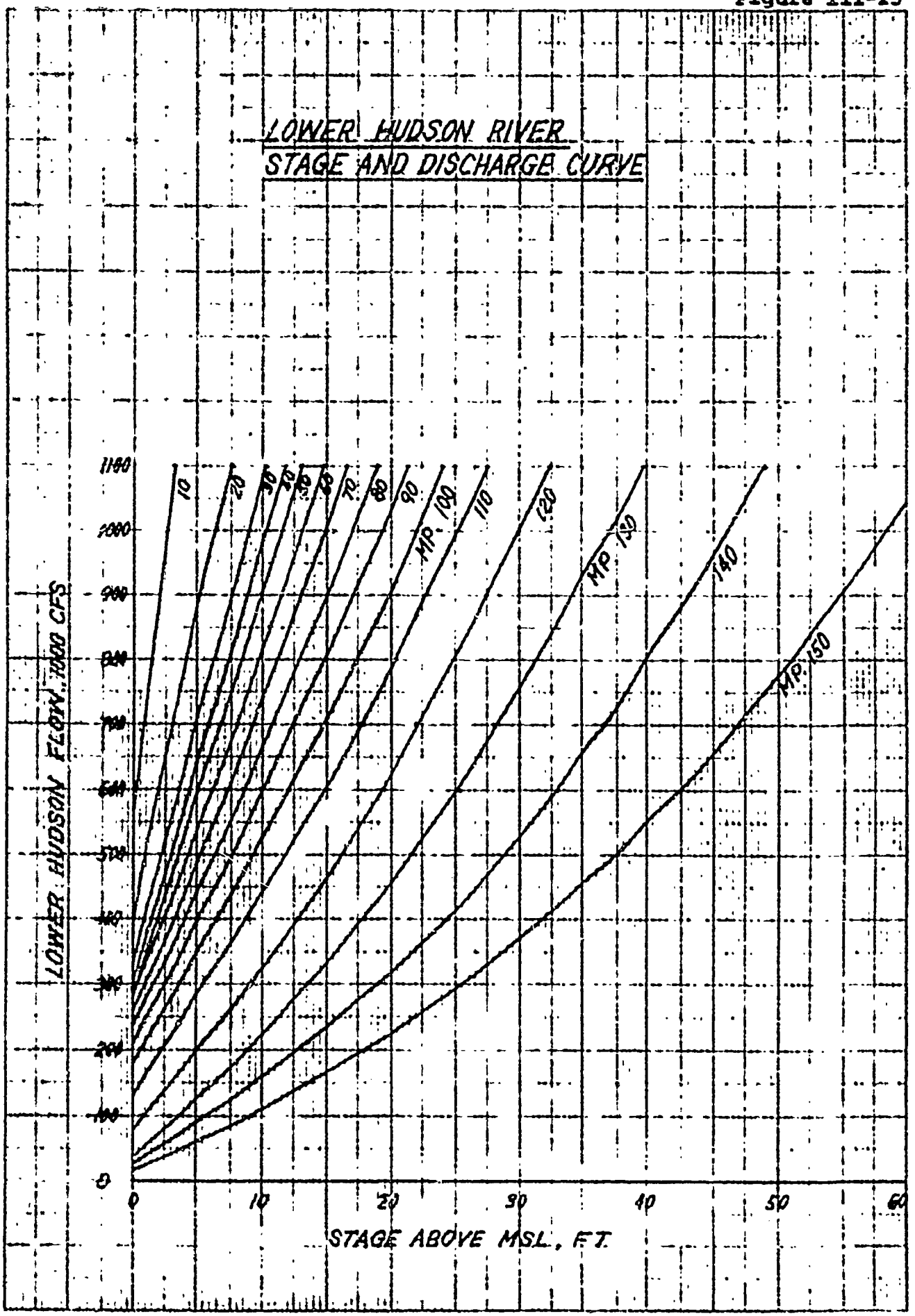
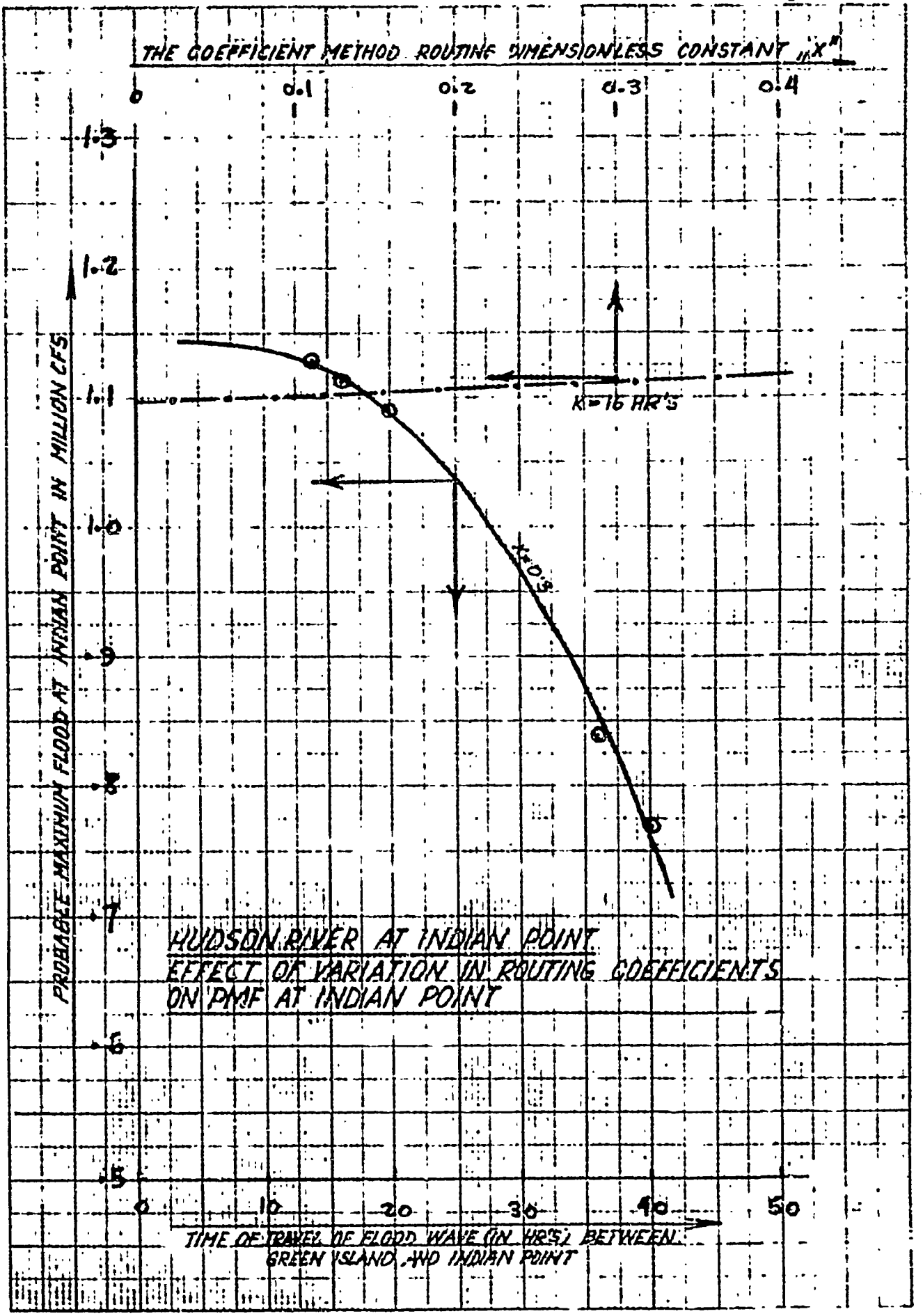


Figure III-24

Figure III-25





HUDSON RIVER AT INDIAN POINT
EFFECT OF VARIATION IN ROUTING COEFFICIENTS ON PMF AT INDIAN POINT

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.

On 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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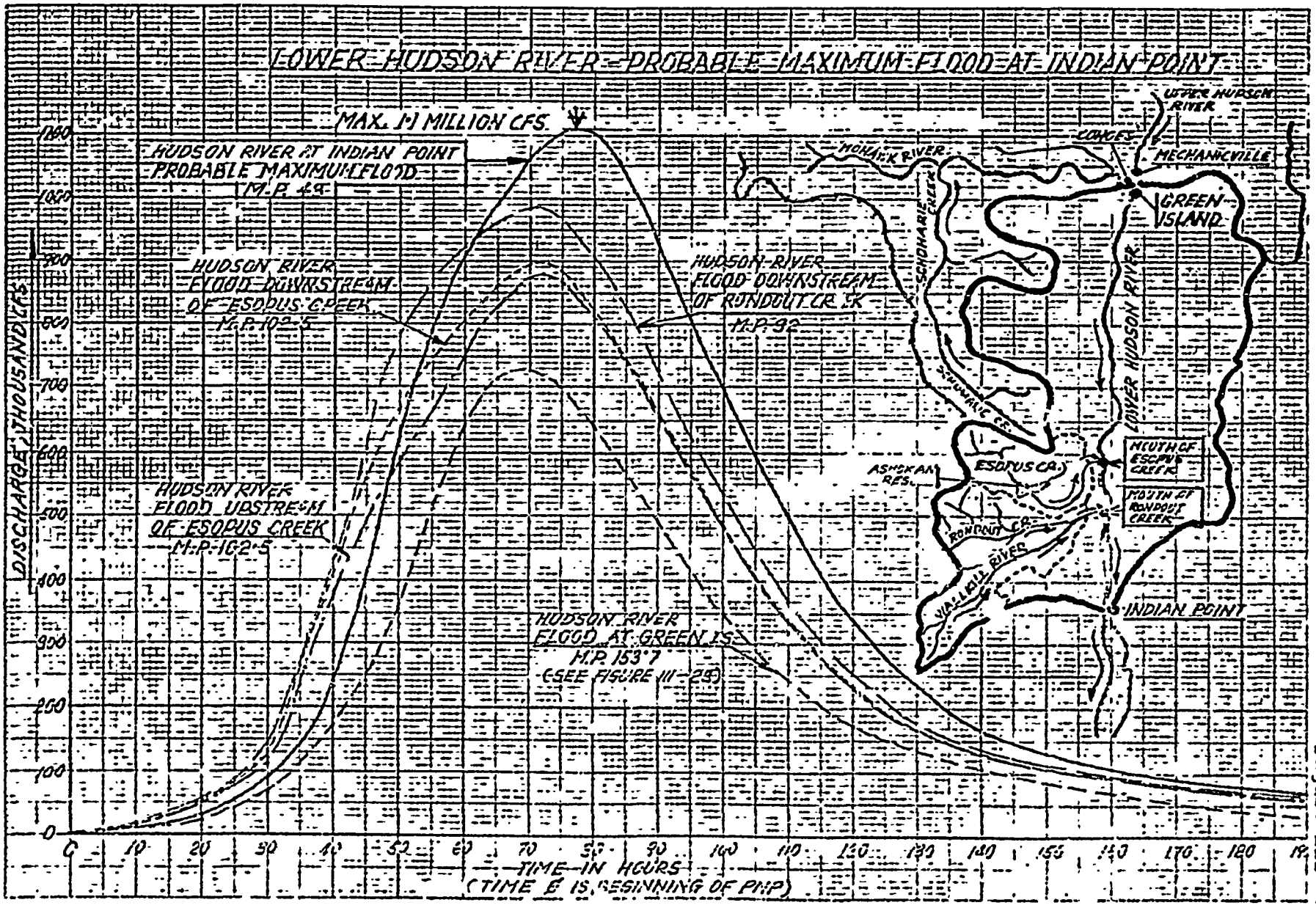


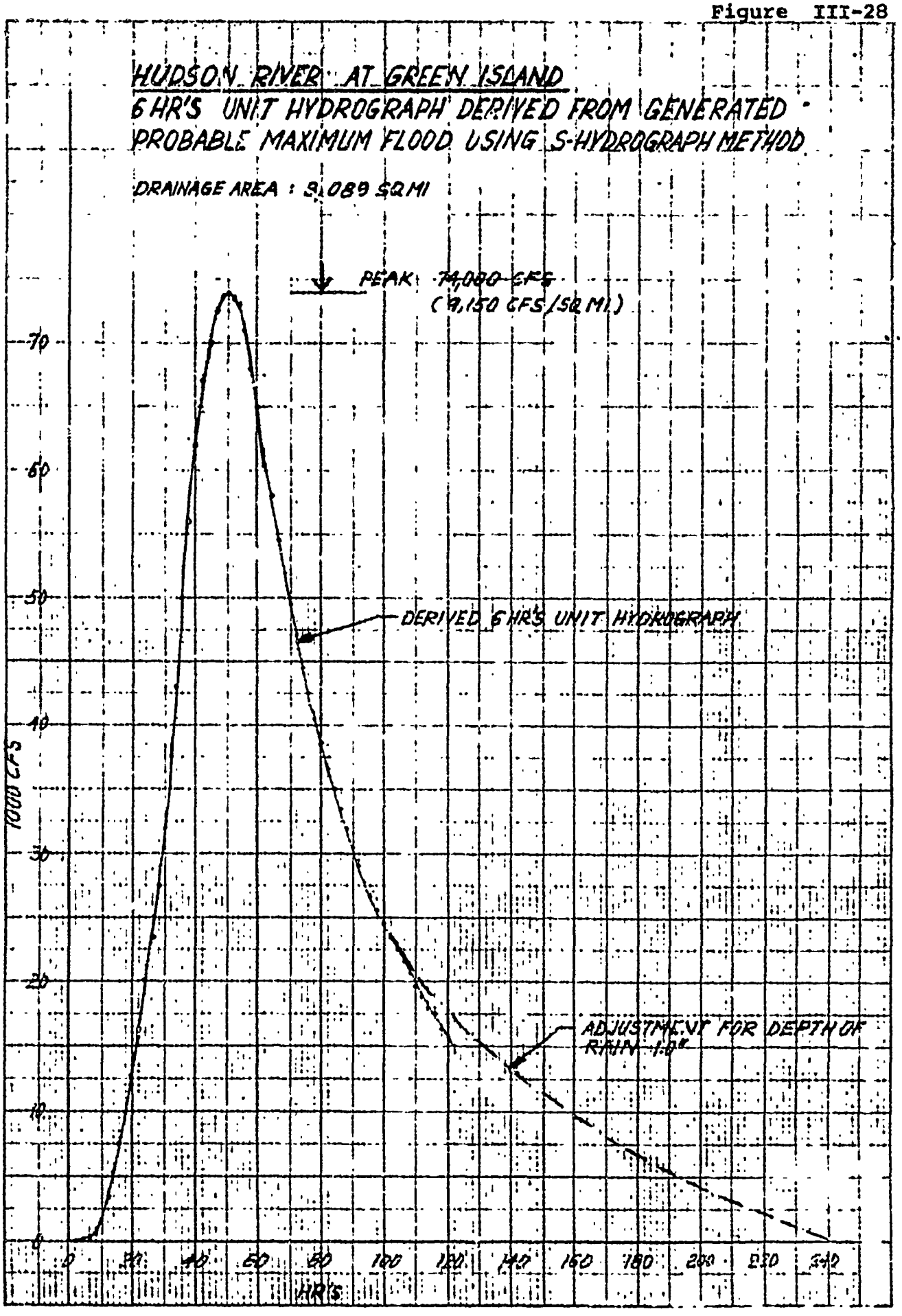
Figure III-27

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 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]

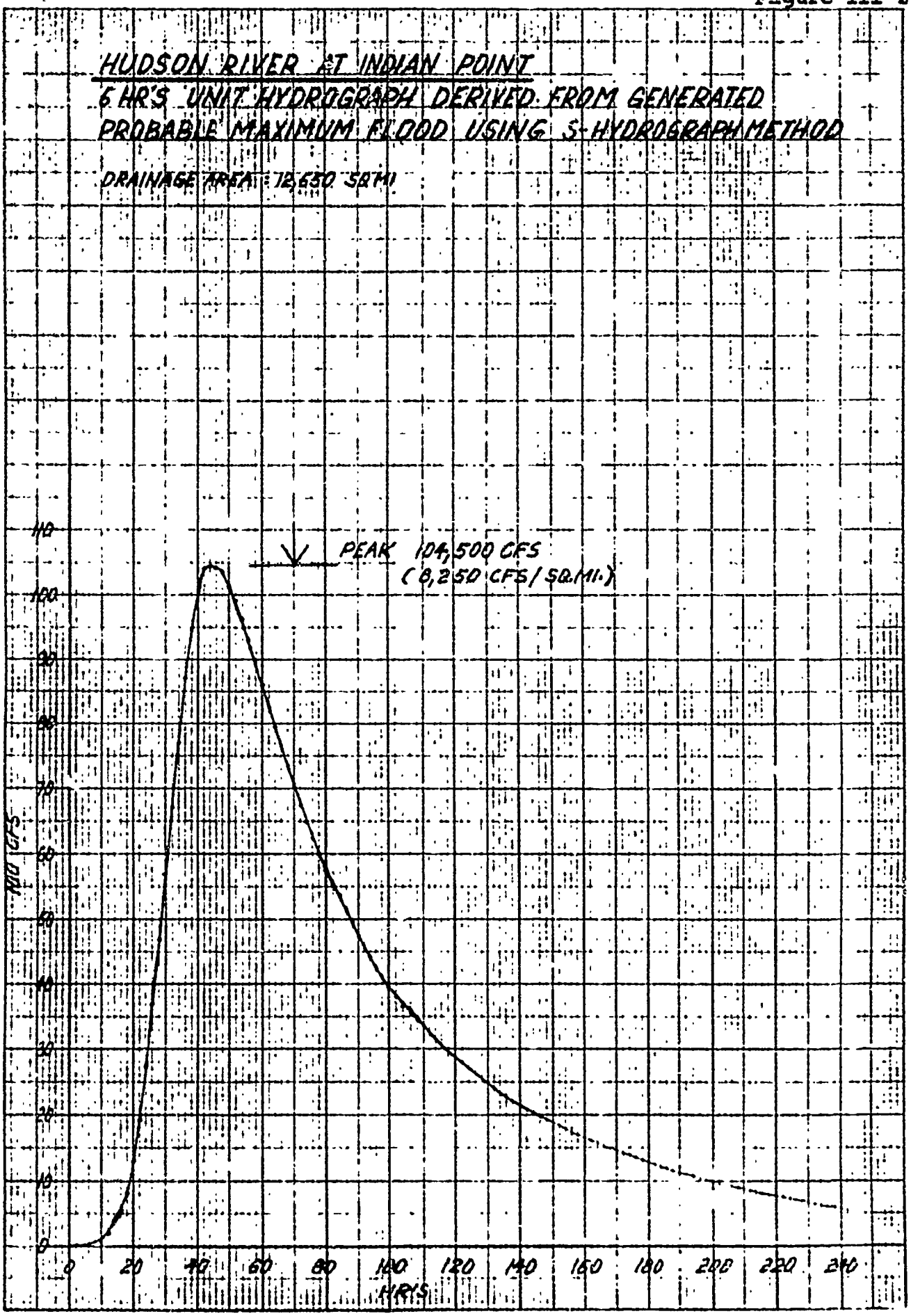
The Green Island and Indian Point unit hydrographs thus obtained are shown in Figures III-28 and III-29 respectively. As shown in Figure III-15 these unit hydrographs produced peak discharges in excess of the Corps of Engineers mean line values. It was then concluded that the Indian Point PMF of 1,100,000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

Figure III-28



TTT-143

Figure III-29



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.

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.

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

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

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

where

B = Channel width

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

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

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

$$Q' = \frac{W_o Q_o}{W_o + Q_o La} \qquad \text{(IV-5)}$$

where

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

of the dam site.

Q_o = 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 $.1 \times 10^{-4}$ to $.5 \times 10^{-4}$

v_1 = The wave front velocity.

v_2 = The wave tail velocity.

Equations IV-4 and IV-5 were used to determine the flow at Indian 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 then the two major reservoirs, concentrated the detailed analysis 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

SACANDAGA RESERVOIR

PLATE IV-1

Approximate computation of max. flow after Conklingville dam failure:

Conditions: Water stored in Hinkley Reservoir, Silboa reservoir, Delta Reservoir and in Lake are located at Sacandaga Reservoir.

At the dam site: $Q_0 = \frac{8}{27} B g^{1/2} H^{3/2}$ $H_{AVG} = 48 FT$

$Q_0 = \frac{8}{27} \times 2000 \times 5.68 \times 48^{3/2}$

$Q_0 = 1.121 \times 10^6 CFS$

At Indian Point: $Q_1 = \frac{W_0' Q_0}{W_0' - Q_0 L \cdot a}$, *

where

Q_0 is initial discharge after dam failure

W_0' is amount of water stored in reservoir (59.15 $\times 10^9$ CF)

L is distance from the dam (180 miles)

$a = 0.5 \left(\frac{1}{V_2} - \frac{1}{V_1} \right)$

V_1 = velocity of wave front

V_2 = velocity of wave tail

For river basins $< 50000 km^2$ (19,300 SQMI)

River slope 10÷50 cm/km ($S = 0.1 \times 10^{-2} \div 0.5 \times 10^{-2}$)

$V_2 = 0.50 m/s \doteq 1.65 FT/S$

$V_1 = 2.0 m/s \doteq 6.05 FT/S$

$a = 0.5 \left(\frac{1}{1.65} - \frac{1}{6.05} \right) = 0.224$

$Q_1 = \frac{59.15 \times 10^9 \times 1.121 \times 10^6}{59.15 \times 10^9 + 1.121 \times 10^6 \times 0.950 \times 10^6 \times 0.224}$

$Q_1 = 223{,}000 CFS \approx \underline{225 \times 10^3 CFS}$

* Lishtvan, L.L. "Computation of maximum discharge caused by dam failure in design of bridges," Moscow 1948

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SUBJECT _____

SHEET NO. 2 OF 2
JOB NO. _____

PLATE IV-1 (continued)

Approximate computation of max. flow after Ashokan Dam failure:

Conditions: Water stored in Rondout Reservoir is "located" @ Ashokan Reservoir

At the dam site: $Q_o = \frac{5}{27} \times 5200 \times 568 \times 46^{3/2}$

$Q_o = 2600 \times 10^6$ CFS

Using same equation as that used for Cacandaga reservoir the maximum flow at Indian Point is:

$$Q = \frac{24.0 \times 10^9 \times 2.6 \times 10^6}{24.0 \times 10^9 + 2.6 \times 10^6 \times 0.296 \times 10^6 \times 0.224} = 266 \times 10^3 \text{ CFS}$$

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 regulating 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 construction of the Conklingville Dam in 1930. The Conklingville Dam is located on the Sacandaga River which discharges into the Upper Hudson River some 70 miles upstream 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 three components:

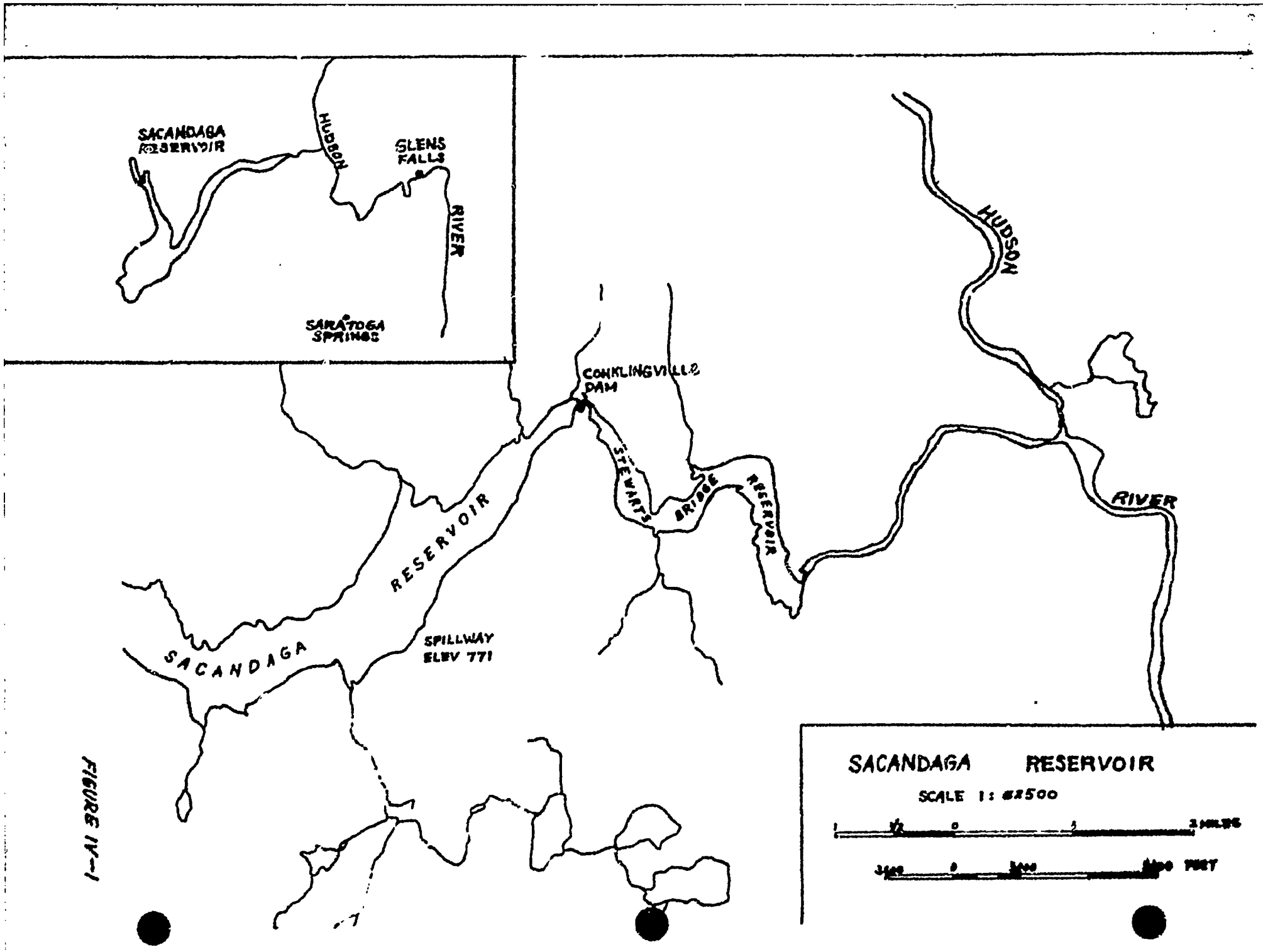

SACANDAGA RESERVOIR
HUDSON
GLENS FALLS
RIVER
SARATOGA SPRINGS
CONKLINGVILLE DAM
STEWARTS BRIDGE RESERVOIR
HUDSON
RIVER
SACANDAGA RESERVOIR
SPILLWAY ELEV 771
SACANDAGA RESERVOIR
SCALE 1: 62500
1 ½ 0 1 2 MILES
3000 0 3000 6000 FEET

FIGURE IV-1

Use	Elevation	Storage Capacity Billion Cubic Feet
Dead Storage	700 - 735	4.60
Stream Regulation Storage	735 - 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 = 3,000 to 28,000 feet

Length of Shore Line = 125 miles

Flow Line Elevation = 771.0' above sea level

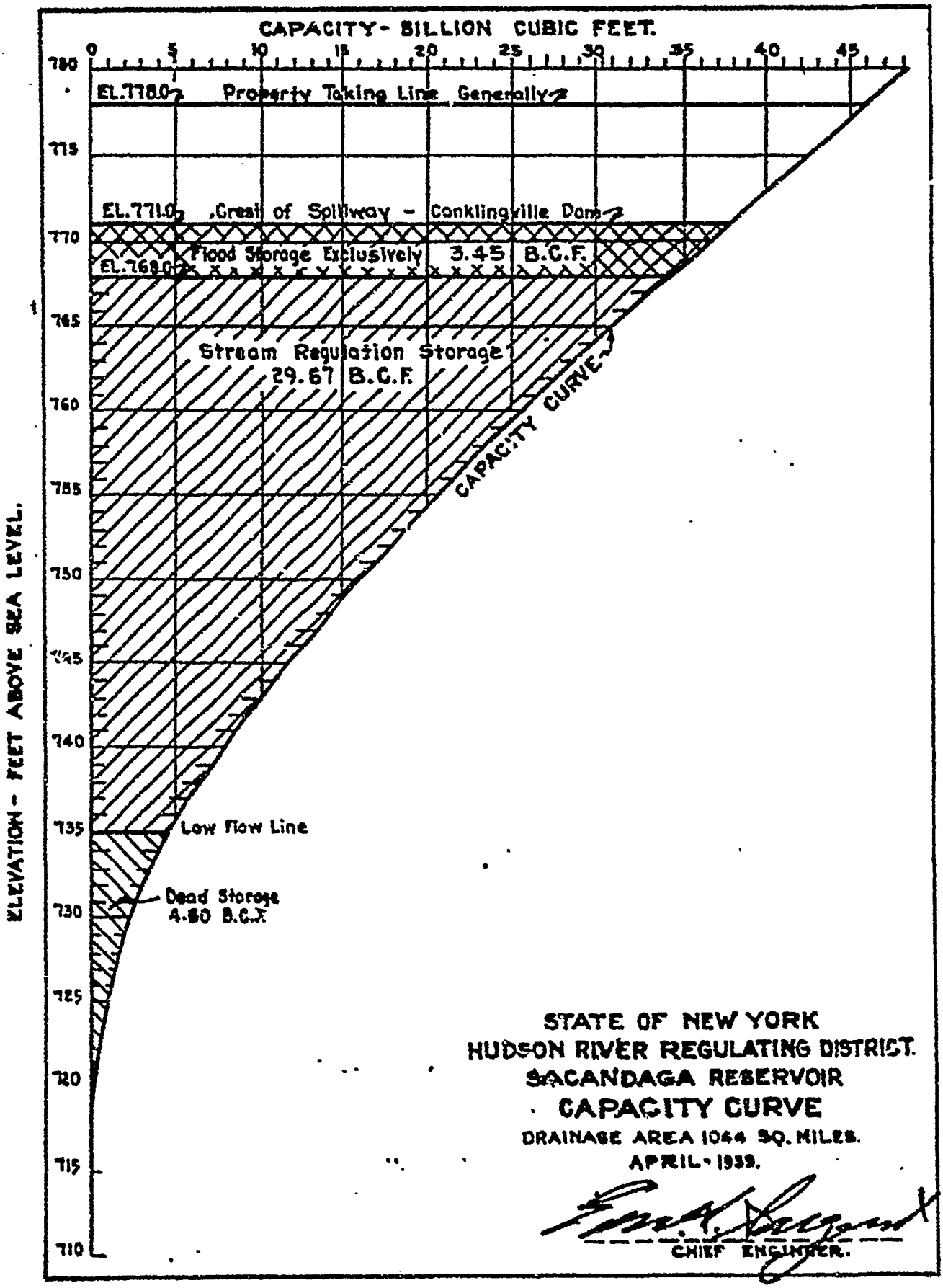
Storage Above Elevation 768.0 = 12 billion cubic feet (total flood storage)

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

FIGURE IV-2



STATE OF NEW YORK
HUDSON RIVER REGULATING DISTRICT.
SACANDAGA RESERVOIR
CAPACITY CURVE
DRAINAGE AREA 1044 SQ. MILES.
APRIL - 1939.

CHIEF ENGINEER.

K-E 10 X 10 TO THE INCH 46 1320

SACANDAGA RESERVOIR
CONKLINGVILLE DAM
TYPICAL CROSS-SECTION

CONKLINGVILLE DAM

670000 Cubic Yards of Earth Fill
120000 Cubic Yards of Impervious Fill
200000 Cubic Yards of Rock Excavation
15800 Cubic Yards of Concrete
Length of Dam 1180 Feet
Spillway - 400' Weir and 2-8'x18' Bypass

SACANDAGA RESERVOIR

Area - 27000 Acres 42.3 Square Miles
Capacity - 37.8 Billion Cubic Feet - 283 Bil Gallons
Length 27 Miles
Width - 3000 to 28000 Feet
Length of Shore Line - 125 Miles
Flow Line Elevation 1110' above Sea Level

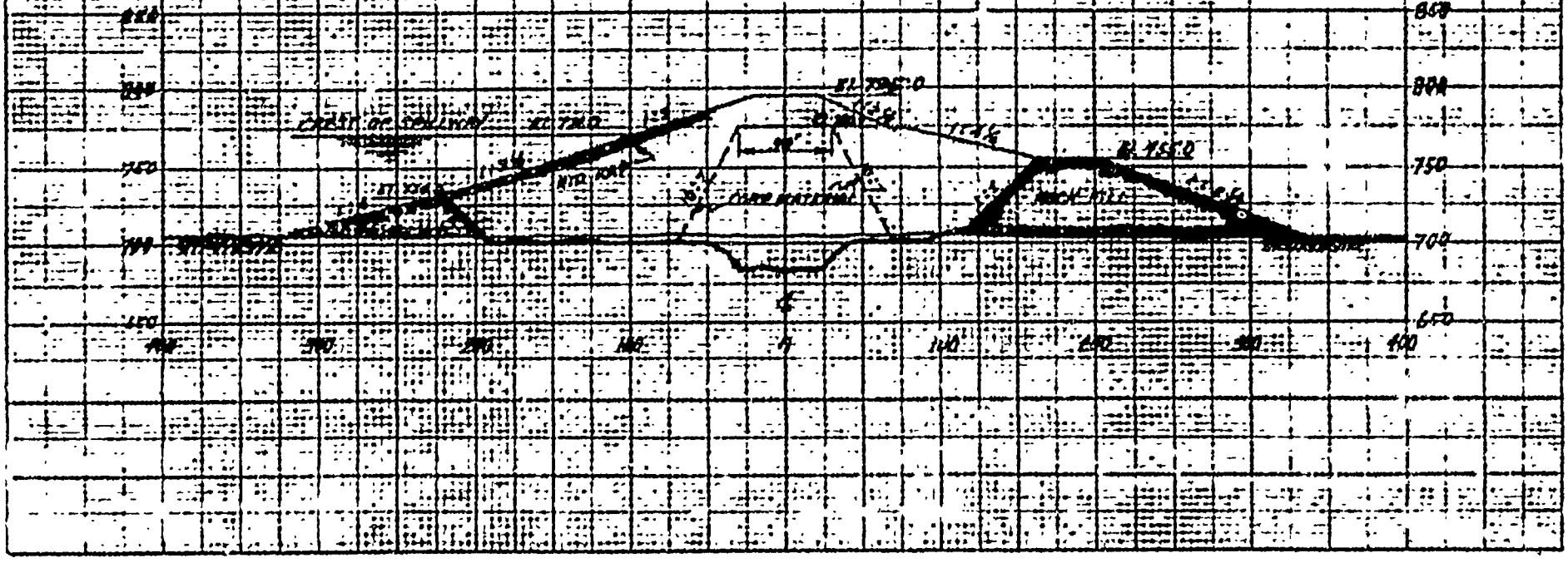


FIGURE IV-3

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 as 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 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]

TABLE IV-12

STATE OF NEW YORK—HUDSON RIVER REGULATING DISTRICT

SUBJECT: CORE SAMPLES — BY STATIONS — FILE — Acc. No. — SHEET 15

COMPUTED: R.E.D. March 1930 — CHECKED BY — 19

MADE IN CONNECTION WITH: CONKLINGVILLE DAM

REFERENCE — Corr's Work Acc.

STATION	% PASSING 100 Mesh Sum	No. of Tests	Av.	% Passing 200 Mesh Sum	No. of Tests	Av.
101+0	115.9	2	57.9	65.5	2	32.7
102+0	166.0	2	83.0	113.0	2	56.5
103+0	1286.6	18	71.6	912.8	18	50.7
104+0	806.1	12	67.2	609.2	18	50.8
105+0	2292.4	32	71.6	1779.8	32	55.6
106+0	3181.0	42	75.7	2547.3	42	60.6
107+0	2291.1	32	71.6	1737.2	32	54.3
108+0	2186.8	34	64.4	1648.8	34	48.5
109+0	12830.1	185	69.3	9335.4	185	50.6
110+0	1617.2	22	73.2	1161.2	22	53.2
111+0	167.9	2	84.0	121.3	2	60.6
TOTAL	26938.1	383	70.3	20064.0	383	52.4

COMPLETE FILING SYMBOL

Corr's Work Acc.

FIGURE IV-3a

STATE OF NEW YORK—HUDSON RIVER REGULATING DISTRICT

SUBJECT: CORE SAMPLES BY DISTANCE FROM CENTERLINE

FILE

ASS. NO.

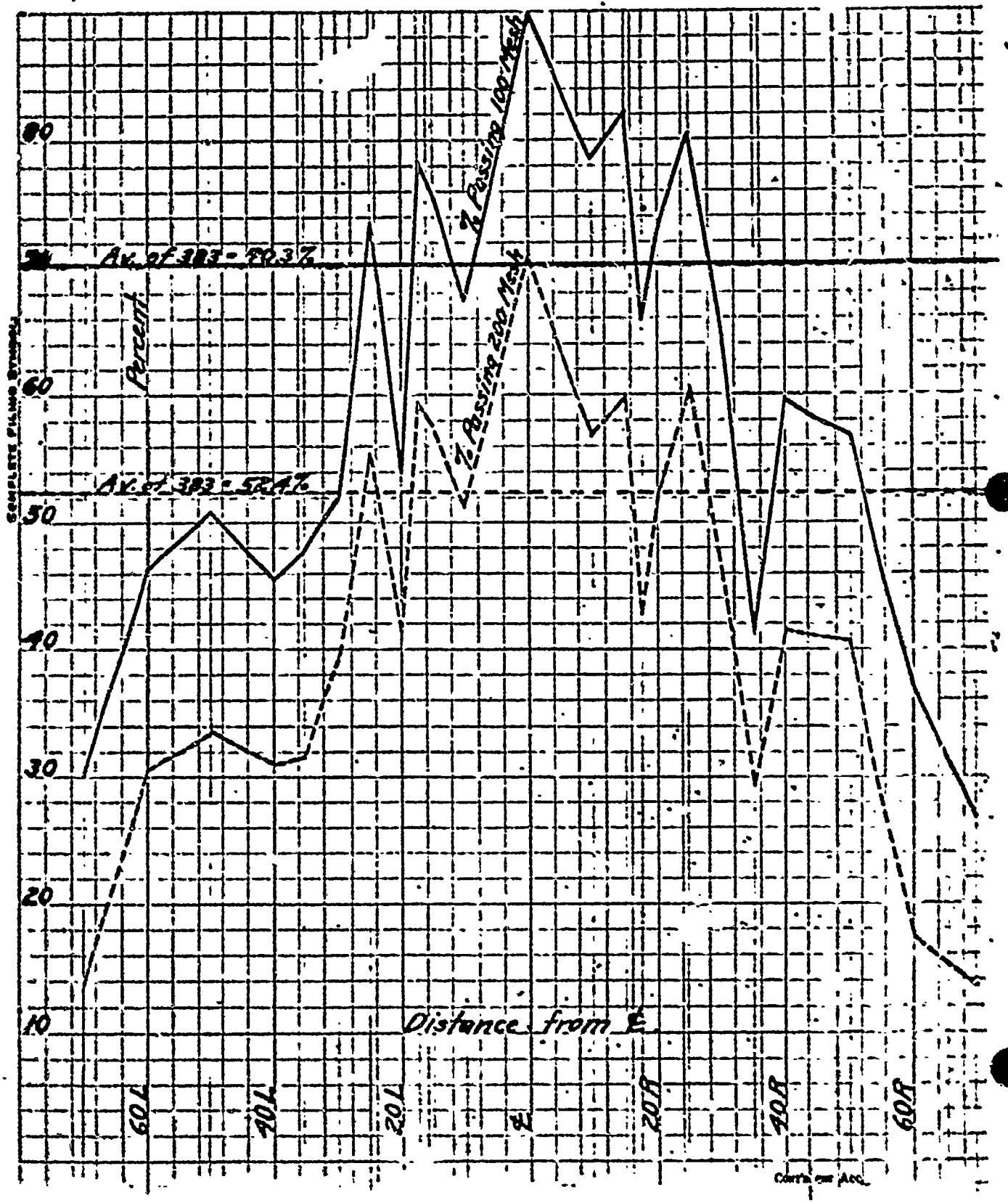
SHEET 18

COMPUTER R.B. March 19 30 CHECKED BY 19

MADE IN CONNECTION WITH CONKLINGVILLE DAM

REFERENCE

CONT'D FROM ACC.



COMPLETE FILING SYMBOL

CONT'D FROM ACC.

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 dimensions 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 Elevation = 769.72 feet (on May 21,1969)

The outlet works consist of three 8 foot diameter Dow Valves and 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 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-

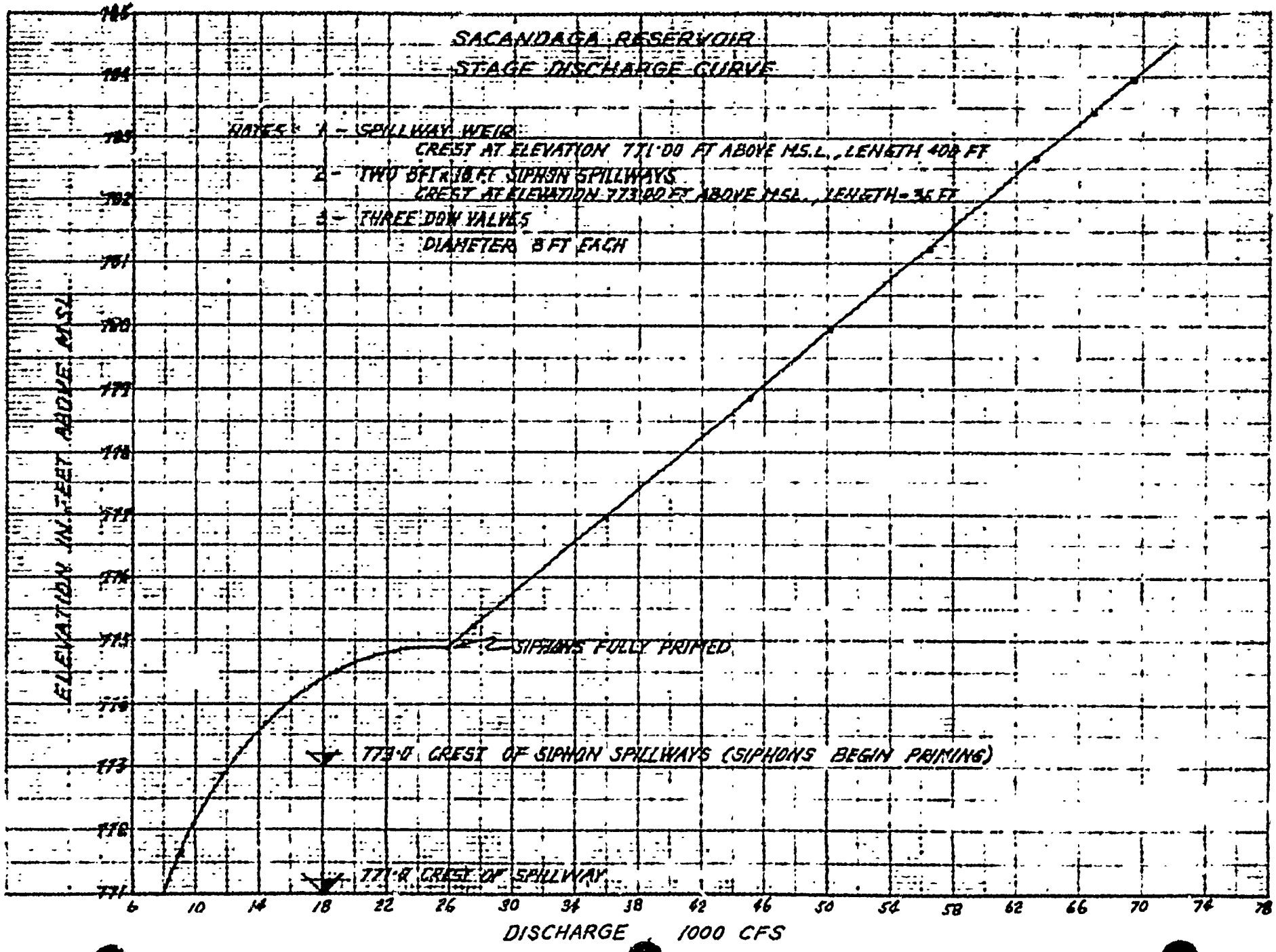

SACANDAGA RESERVOIR
STAGE DISCHARGE CURVE
RATES: 1 - SPILLWAY WEIR
CREST AT ELEVATION 771.00 FT ABOVE M.S.L., LENGTH 400 FT
2 - TWO 8FT x 38 FT SIPHON SPILLWAYS
CREST AT ELEVATION 773.00 FT ABOVE MSL, LENGTH = 76 FT
3 - THREE DOW VALVES
DIAMETER 8 FT EACH
ELEVATION IN FEET ABOVE MSL
DISCHARGE 1000 CFS
2 - SIPHONS FULLY PRIMED
773.0 CREST OF SIPHON SPILLWAYS (SIPHONS BEGIN PRIMING)
771.0 CREST OF SPILLWAY
6
10
14
18
22
26
30
34
38
42
46
50
54
58
62
66
70
74
78

FIGURE II-4

sults of the January - February, 1931 tests 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 as follows:

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

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.

SACANDAGA RESERVOIR FLOOD ROUTING PROGRAM

```
RESERV     13:31    RDS3  APRIL 30,1970

50 DIM C(350),O(350),G(50),P(12),Q(200)
55 DIM A(200),B(200),D(200),H(200)
100 READ H(0)
105 READ M
110 READ T
160 FOR Y=0 TO 29
180 LET O(Y)=0
200 NEXT Y
220 FOR Y=30 TO 67
240 READ O(Y)
260 NEXT Y
270 FOR Y=68 TO M
280 LET O(Y)=0.4825*(Y-68)+25.4
290 NEXT Y
300 FOR Y=0 TO M
310 LET O(Y)=O(Y)*(10^3)
320 NEXT Y
390 READ K
400 FOR J=1 TO K
410 READ G(J)
420 NEXT J
430 FOR I=1 TO 12
440 READ P(I)
450 NEXT I
460 FOR I=1 TO 12
470 FOR J=1 TO K
480 LET N=J+I-1
490 LET Q(N)=(G(J)*P(I))+Q(N)
500 NEXT J
510 NEXT I
515 FOR N=1 TO 200
520 LET Q(N)=(Q(N)*(10^3))+800
530 NEXT N
600 LET D(0)=0
605 LET Q(0)=0
610 FOR N=1 TO 200
615 LET S=((Q(N)+Q(N-1))/2)*T
620 LET V2=(S/1.17E8)+((H(N-1)-H(0))*10)
625 LET W2=INT(V2)
630 IF W2=V2 GOTO 660
650 LET D2=((V2-W2)*(O(W2+1)-O(W2)))+O(W2)
655 GOTO 665
```

```
660 LET D2=O(V2)
665 LET D3=((D(N-1)+D2)/2)*T
670 LET S1=S-D3
675 LET Y2=(S1/1.17E8)+((H(N-1)-H(0))*10)
680 LET Y3=V2-Y2
685 IF ABS(Y3)<0.2 GOTO700
690 LET V2=(V2+Y2)/2
695 GOTO 625
700 LET H(N)=V2/10 +H(0)
710 LET D(N)=D2
720 NEXT N
730 INPUT A9
740 IF A9=99.9 GOTO 2000
760 PRINT
770 PRINT
780 PRINT
790 PRINT " SACANDAGA RESERVOIR - PMF - OUTFLOW"
800 PRINT
810 PRINT"HR'S;1000CFS"
820 FOR N=1 TO 75
900 PRINT 6*N;";";D(N)/(10↑3)
910 PRINT (2*N+1)*3;";";((D(N)+D(N+1))/(2*(10↑3))
920 NEXT N
930 PRINT
940 PRINT
2000 PRINT
2010 PRINT
2020 PRIN. "SACANDAGA RESERVOIR   -   PMF"
2030 PRINT
2040 PRINT" TIME      INTAKE      OTFLOW       ELEVATION"
2050 PRINT"  HR'S     [redacted]CFS     [redacted]CFS      FT M.S.L."
2060 FOR N=1 TO 75
2070 PRINT 6*N,Q(N),D(N),H(N)
2080 NEXT N
2090 PRINT
2100 PRINT"    0 OF TIME IS BEGINNING OF PMP"
2110 PRINT
2120 GOTO 9000
9000 END
```

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 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

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 from 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

SACANDAGA RESERVOIR FLOOD ROUTING PROGRAM

DATA

```
1000 DATA 768.0
1010 DATA 340
1020 DATA 21.6E3
1100 DATA 8.09,8.21,8.35,8.50
1110 DATA 8.65,8.80,8.95,9.10
1120 DATA 9.27,9.46,9.66,9.85
1130 DATA 10.05,10.30,10.52,10.78
1140 DATA 11.01,11.26,11.55,11.83
1150 DATA 12.10,12.40,12.70,13.02
1160 DATA 13.35,13.68,14.01,14.36
1170 DATA 14.72,15.10,15.53,16.00
1180 DATA 16.55,17.18,13.95,18.90
1190 DATA 20.05,23.80
1230 DATA 24
1310 DATA 13.00,28.00,14.50,10.00
1320 DATA 7.60,6.20,5.10,4.30
1330 DATA 3.60,3.10,2.70,2.40
1340 DATA 2.10,1.90,1.70,1.40
1350 DATA 1.20,1.00,0.70,0.50
1360 DATA 0.20,0.10,0.00,0.00
1380 DATA 0.00,0.72,0.73,0.31
1390 DATA 1.64,7.10,1.99,1.17
1400 DATA 0.11,0.18,0.23,0.12
```

STAGE DISCHARGE DATA

U.H. DATA SUBBASIN 8

RUNOFF DATA

SACANDAGA RESERVOIR FLOOD ROUTING PROGRAM

PRINTOUT

Time hr's	Intake cfs	Outflow cfs	Elevation Ft. M.S.L.
6	800	0	768.007
12	10160.	0	768.109
18	30450	0	768.423
24	35710.	0	769.094
30	48585.	0	769.572
36	156287.	9084.48	771.69
42	262362.	27497.7	775.235
48	201634.	45017.5	778.866
54	156050.	56483.6	781.242
60	114525.	63267.3	782.648
66	95826.	66910.	783.403
72	82325.	68872.1	783.81
78	68628.	69508.6	783.942
84	56159.	68975.1	783.831
90	47703.	67593.9	783.545
96	41278.	65694.2	783.151
102	36209.	63464.2	782.689
108	31764.	61016.2	782.182
114	28294.	58436.9	781.647
120	24905.	55782.3	781.097
126	21151.	53046.6	780.53
132	17924.	50243.7	779.949
138	14747.	47403.7	779.36
144	11268.	44519.1	778.763
150	8275.	41601.4	778.158
156	5050.	38710.8	777.559
162	3194.	35843.6	776.964
168	1788.	33075.7	776.391
174	1310	30457.4	775.848
180	1046	28025.3	775.344
186	924	25778.9	774.879
192	847	18809.5	774.498
198	812	16582.1	774.205
204	800	15280.3	773.942
210	800	14354.8	773.699
216	800	13581.2	773.47
222	800	12874.5	773.255
228	800	12251.9	773.051

Time hr's	Intake cfs	Outflow cfs	Elevation Ft. M.S.L.
234	800	11709.6	772.857
240	800	11191.5	772.673
246	800	10770.8	772.496
252	800	10360.7	772.328
258	800	9981.22	772.166
264	800	9678.33	772.01
270	800	9381.87	771.859
276	800	9122.1	771.713
282	800	8906.97	771.571
288	800	8714.73	771.443
294	800	8527.12	771.318
300	800	8344.42	771.196
306	800	8182.17	771.077
312	800	5499.2	770.968
318	800	2141.26	770.926
324	800	1290.93	770.916
330	800	1075.6	770.913
336	800	1021.08	770.913
342	800	1007.27	770.912
348	800	1003.77	770.912
354	800	1002.88	770.912
360	800	1002.66	770.912
366	800	1002.6	770.912
372	800	1002.59	770.912
378	800	1002.59	770.912
384	800	1002.58	770.912
390	800	1002.58	770.912
396	800	1002.58	770.912
402	800	1002.58	770.912
408	800	1002.58	770.912
414	800	1002.58	770.912
420	800	1002.58	770.912
426	800	1002.58	770.912
432	800	1002.58	770.912
438	800	1002.58	770.912
444	800	1002.58	770.912
450	800	1002.58	770.912

"0" OF TIME IS BEGINNING OF PMP

KEUFFEL & ESSER CO

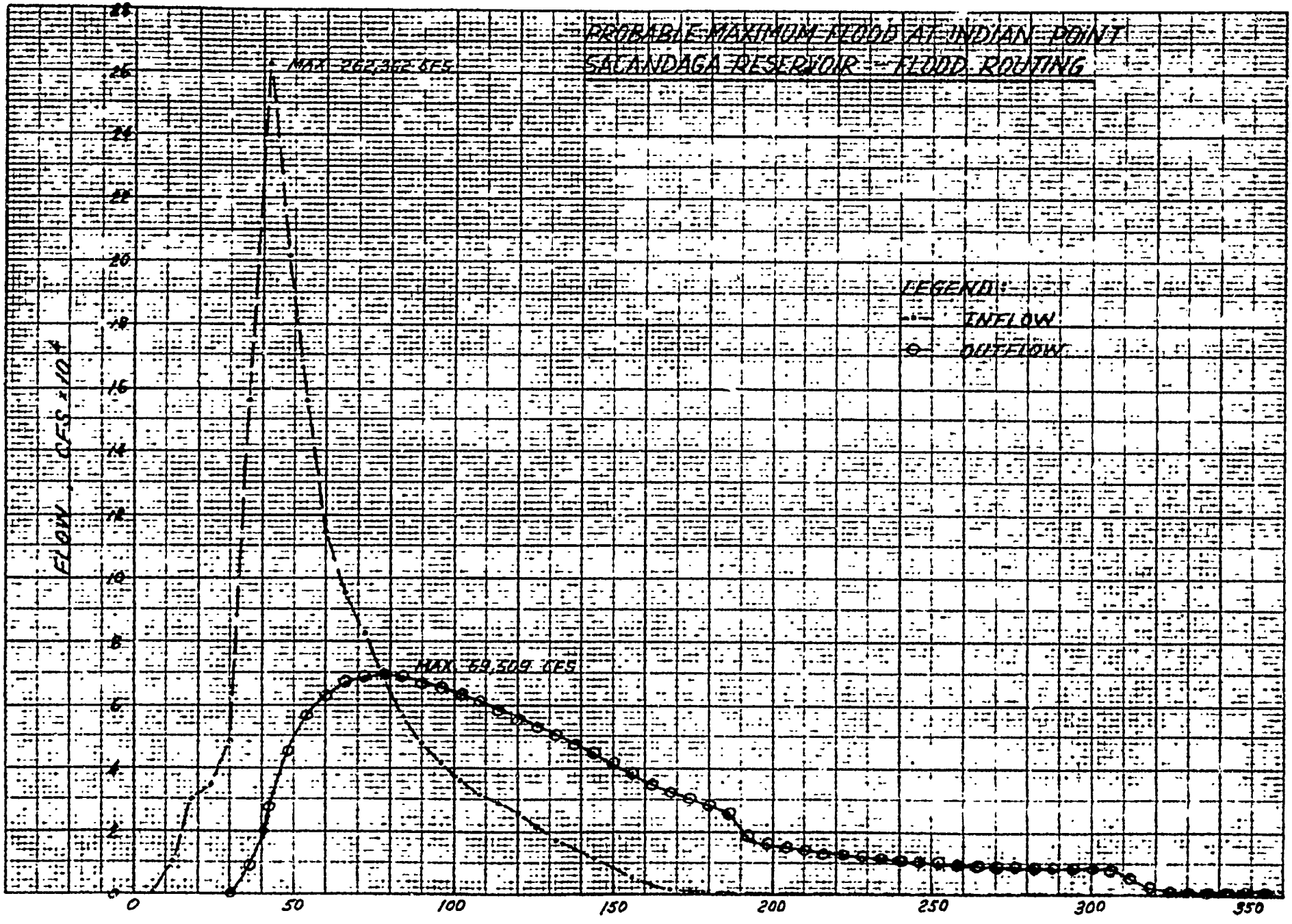


FIGURE IV-5

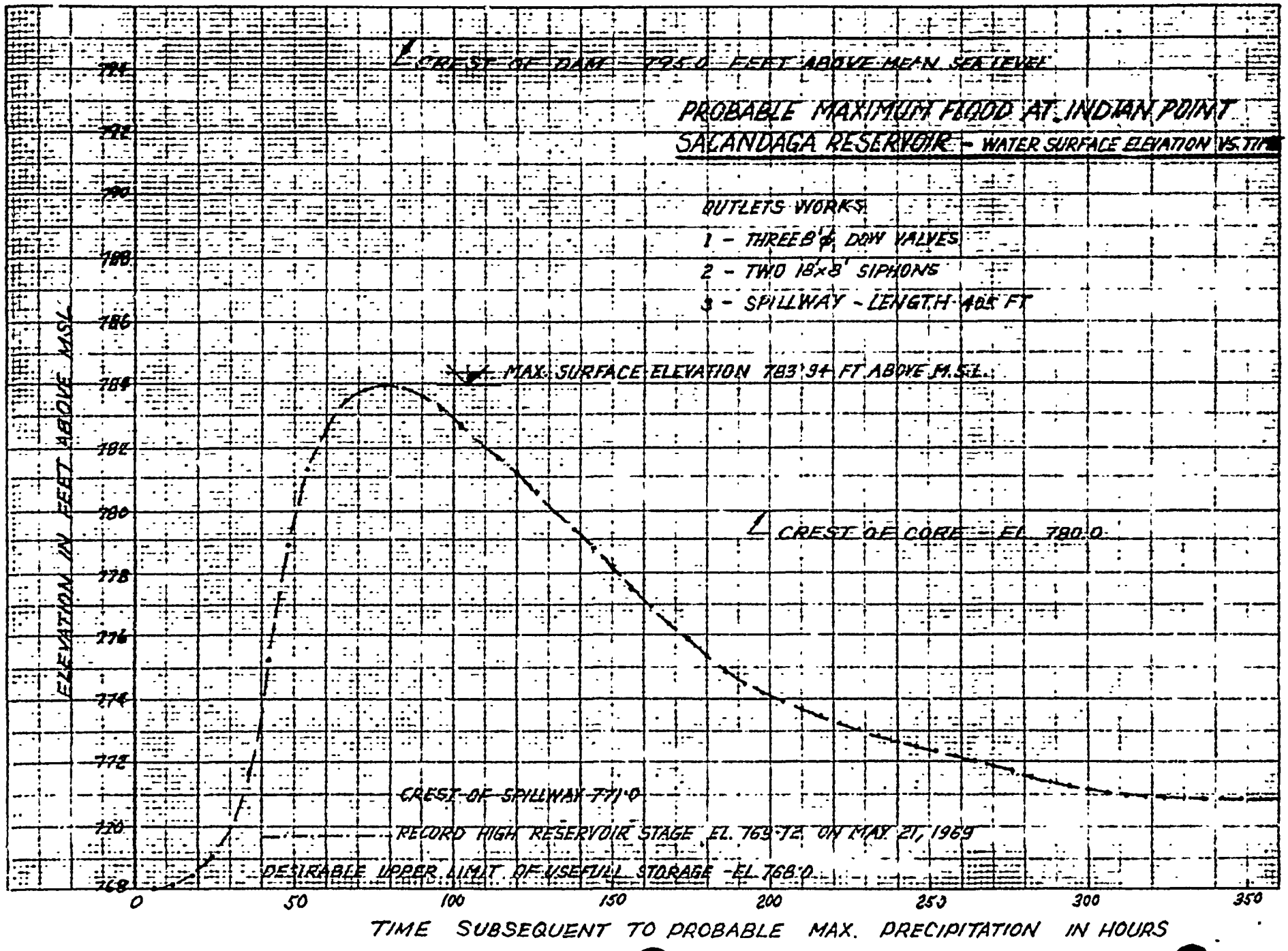

CREST OF DAM - 795.0 FEET ABOVE MEAN SEA LEVEL
PROBABLE MAXIMUM FLOOD AT INDIAN POINT
SACANDAGA RESERVOIR - WATER SURFACE ELEVATION VS. TIME
OUTLETS WORKS
1 - THREE 8'ϕ DOW VALVES
2 - TWO 18'x8' SIPHONS
3 - SPILLWAY - LENGTH 405 FT
MAX. SURFACE ELEVATION 783.94 FT ABOVE M.S.L.
CREST OF CORE - EL. 780.0
CREST OF SPILLWAY 771.0
RECORD HIGH RESERVOIR STAGE EL. 769.72 ON MAY 21, 1969
DESIRABLE UPPER LIMIT OF USEFULL STORAGE - EL. 768.0
ELEVATION IN FEET ABOVE MSL
784
782
780
778
776
774
772
770
768
0
50
100
150
200
250
300
350
TIME SUBSEQUENT TO PROBABLE MAX. PRECIPITATION IN HOURS

FIGURE IV-6

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 severe 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.

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 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

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:

$$q = kiy \qquad \text{IV-7}$$

or $$q/k = iy \qquad \text{IV-8}$$

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

QUINK, LAWLER & MATUSKY ENGINEERS
ENVIRONMENTAL SCIENCE & ENGINEERING CONSULTANTS
505 FIFTH AVENUE
NEW YORK, NEW YORK 10017

BY _____ DATE _____ SHEET NO. _____ OF _____

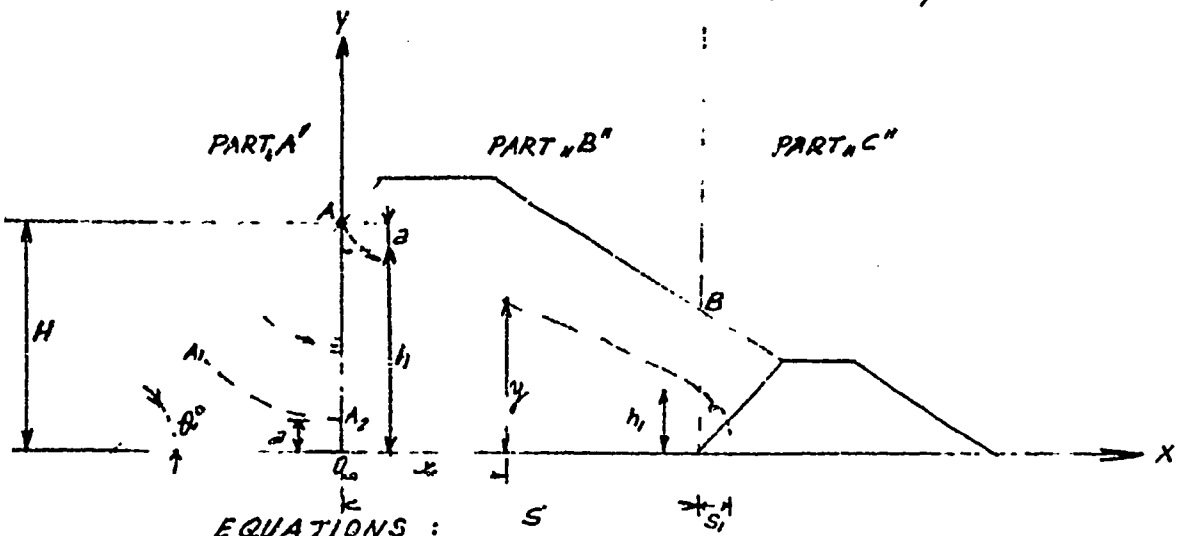
CHKD. BY _____ DATE _____ JOB NO. _____

SUBJECT CONKLINGVILLE DAM

COMPUTATION OF CURVE OF SATURATION

Assumptions: 1) The dam is homogeneous

2) Foundations are practically impervious



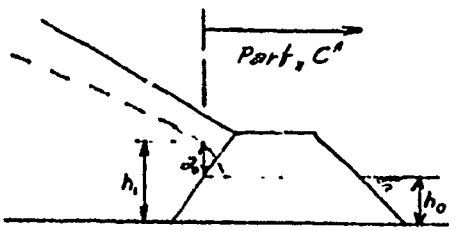
EQUATIONS:

$$Q = k\,\frac{115\,(H-h)}{90^\circ-\theta^\circ} \qquad (1)$$ - FOR PART „A"

$$Q = k\,\frac{h^2-h_1^2}{2S} \qquad (2)$$ - FOR PART „B"

(illegible) (3) - FOR PART „C"

$Q =$ (illegible) $= k a_o$ (3b) - FOR PART „C". IF THERE IS WATER BELOW THE DAM



Equation for computation of curve of saturation in part „B"

$$x = \frac{k}{Q}\cdot\frac{h^2-y^2}{2}$$

Q is seepage (cfs/FT)

k is permeability coefficient (FT/sec)

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 upstream face of the dam. Substitution in equation IV-9 yields:

$$q/k = \frac{ah}{\frac{\pi(90 - \theta)h}{360}} \cong \frac{115(H-h)}{90-\theta} \qquad \text{IV-?}$$

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

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

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 = 0 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

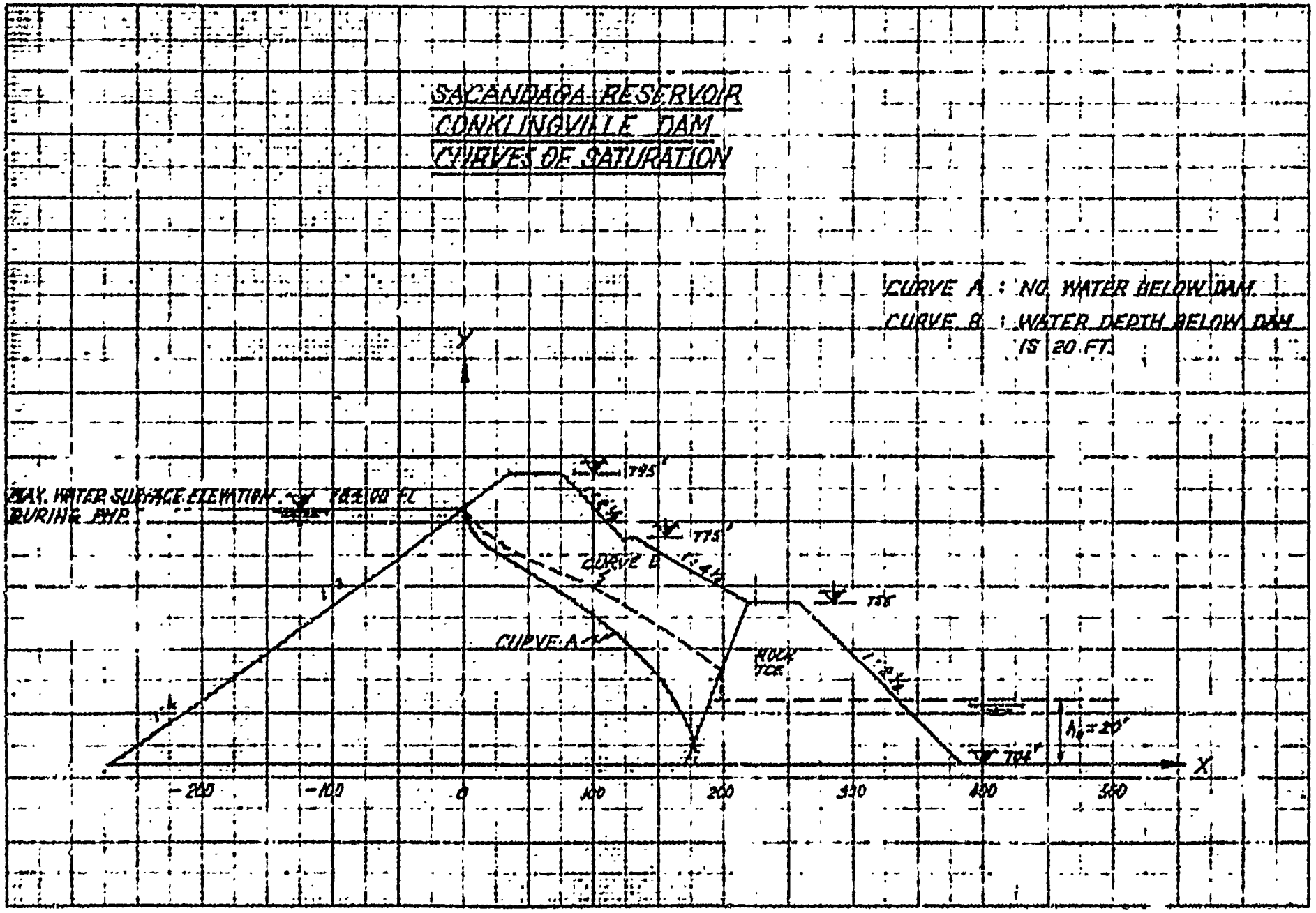

SACANDAGA RESERVOIR
CONKLINGVILLE DAM
CURVES OF SATURATION
CURVE A : NO WATER BELOW DAM.
CURVE B : WATER DEPTH BELOW DAM IS 20 FT.
MAX. WATER SURFACE ELEVATION
DURING PMP
795'
775'
755
704'
CURVE B
CURVE A
ROCK TOE
$h_o = 20'$
Y
X
-200
-100
0
100
200
300
400
500

FIGURE IV-7

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.

D. 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

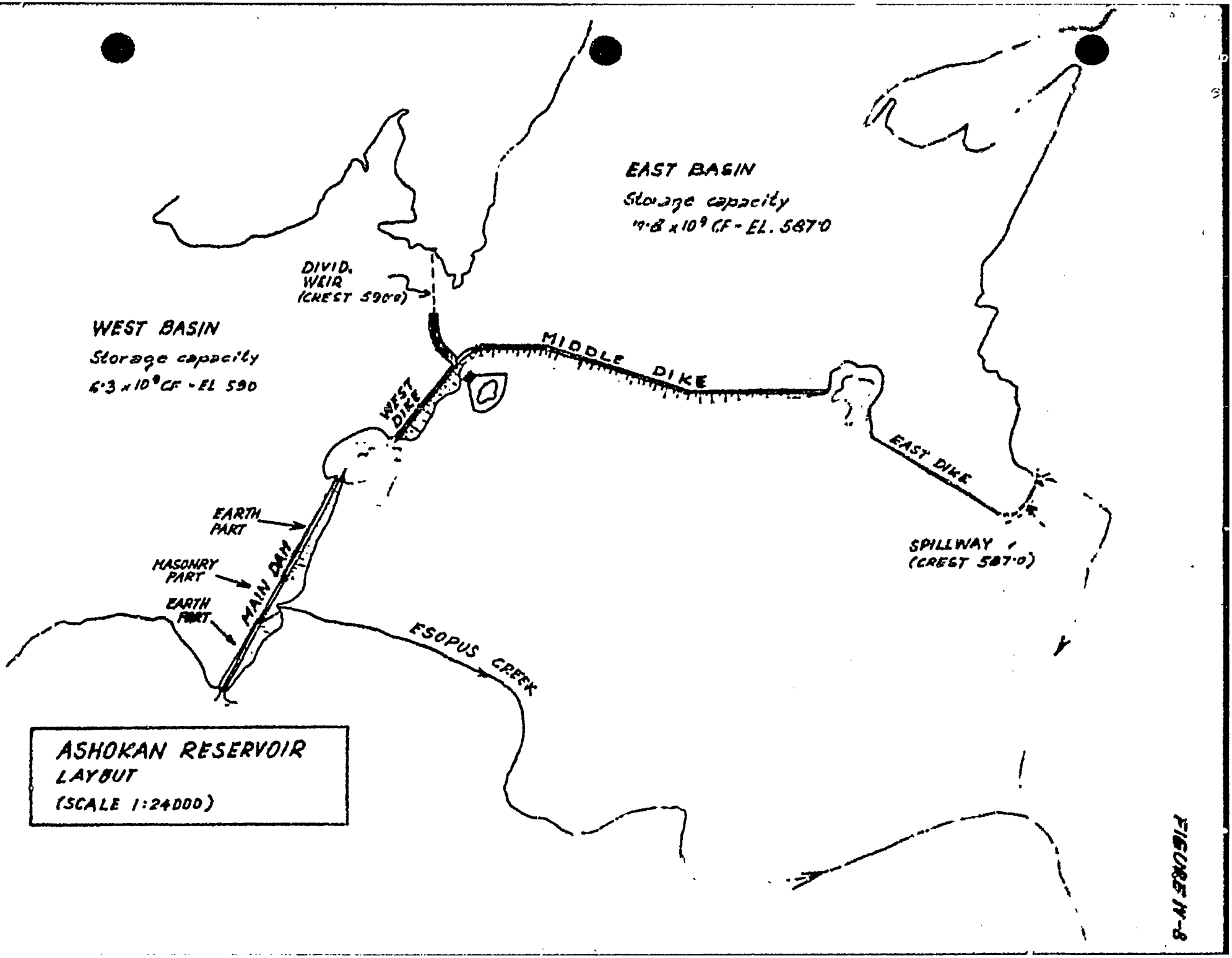

EAST BASIN
Storage capacity
19·8 x 10⁹ C.F - EL. 587·0
DIVID. WEIR (CREST 590·0)
WEST BASIN
Storage capacity
6·3 x 10⁹ CF - EL 590
MIDDLE DIKE
WEST DIKE
EAST DIKE
EARTH PART
MASONRY PART
EARTH PART
MAIN DAM
SPILLWAY (CREST 587·0)
ESOPUS CREEK
ASHOKAN RESERVOIR
LAYOUT
(SCALE 1:24000)

FIGURE IV-8

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.

The reservoir has a surface area of 6180 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.

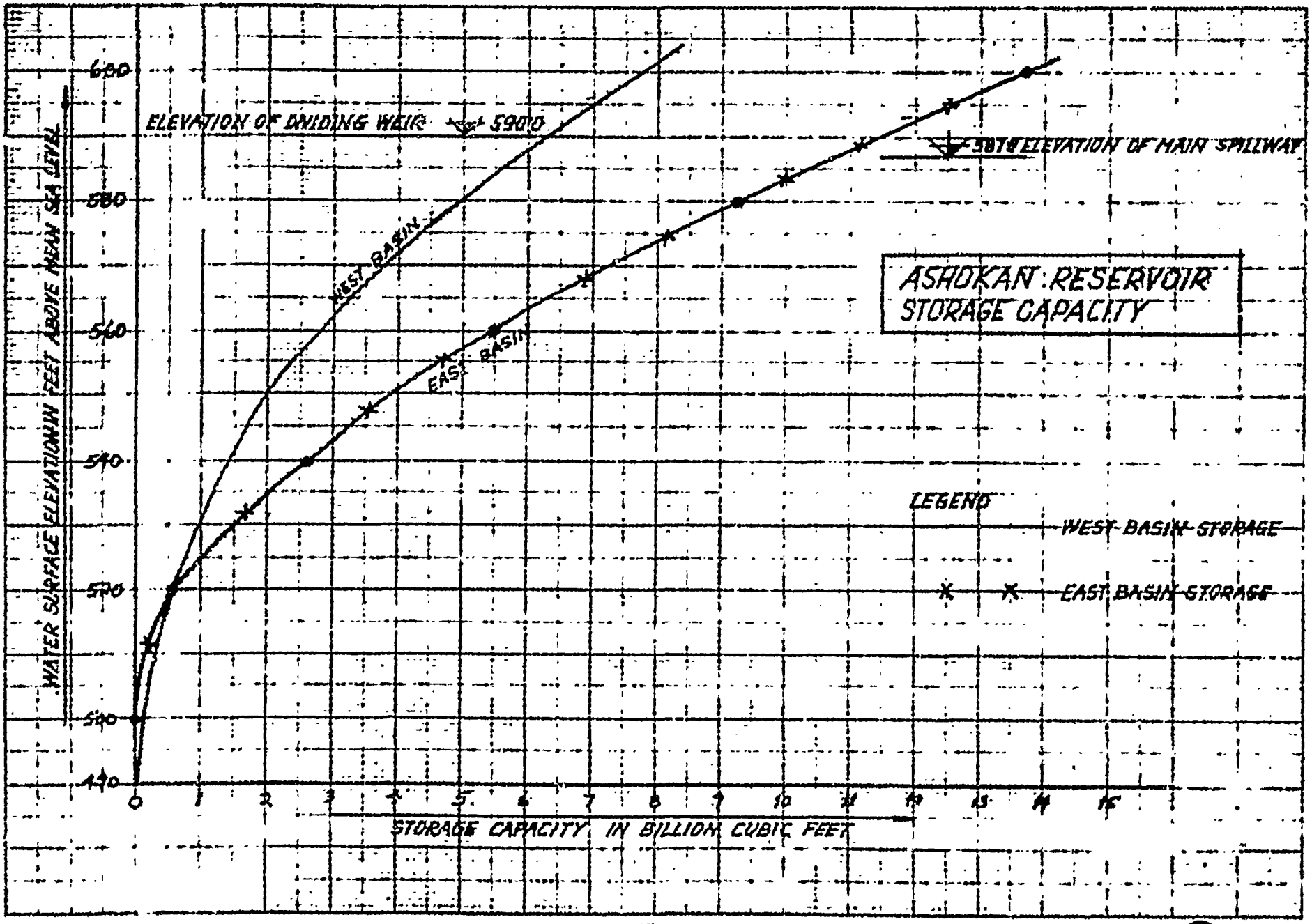

ASHOKAN RESERVOIR
STORAGE CAPACITY
ELEVATION OF DIVIDING WEIR 590'0
590'0 ELEVATION OF MAIN SPILLWAY
WEST BASIN
EAST BASIN
LEGEND
WEST BASIN STORAGE
EAST BASIN STORAGE
WATER SURFACE ELEVATION IN FEET ABOVE MEAN SEA LEVEL
600
580
560
540
520
500
480
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
STORAGE CAPACITY IN BILLION CUBIC FEET

FIGURE IV-5

TABLE IV-1

LENGTH AND ELEVATION

OF ASHOKAN DAM PARTS

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	

Typical cross-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:

$$\left[\left(\frac{I_{wn} + I_{wn+1}}{2} - \frac{O_{wn} + O_{wn+1}}{2}\right) + \left(\frac{I_{en} + I_{en+1}}{2} + \frac{O_{wn} + O_{wn+1}}{2} - \frac{O_{en} + O_{en+1}}{2}\right)\right]$$

$$\left(t_{n+1} - t_n\right) = S_{wn+1} - S_{wn} + S_{en+1} - S_{en} \qquad \text{IV-14}$$

in which

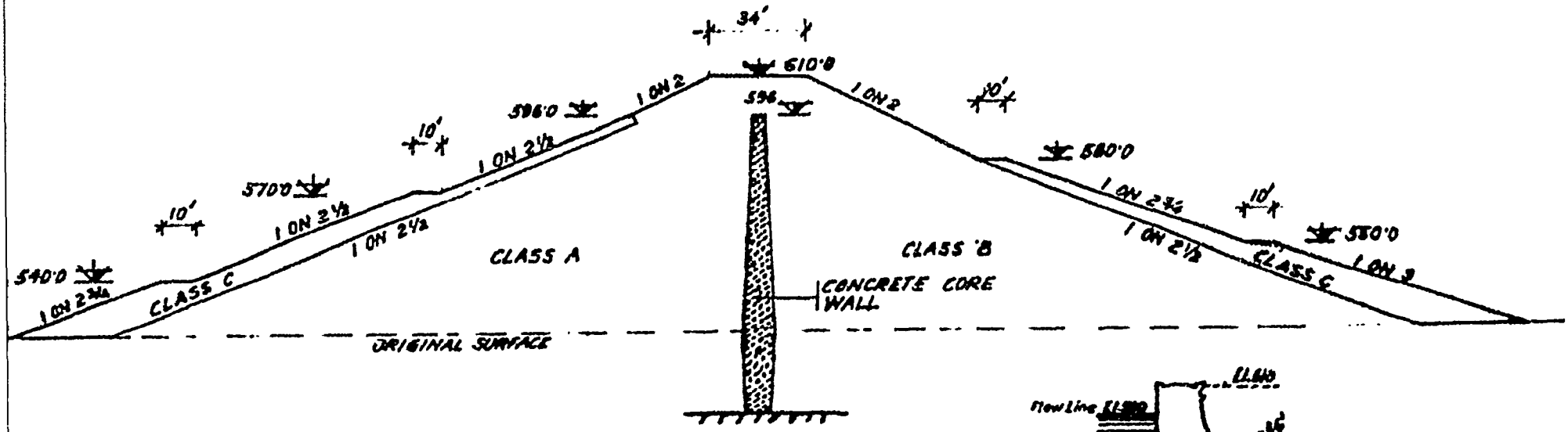
e and w = subscripts denoting east basin and west basin respectively.

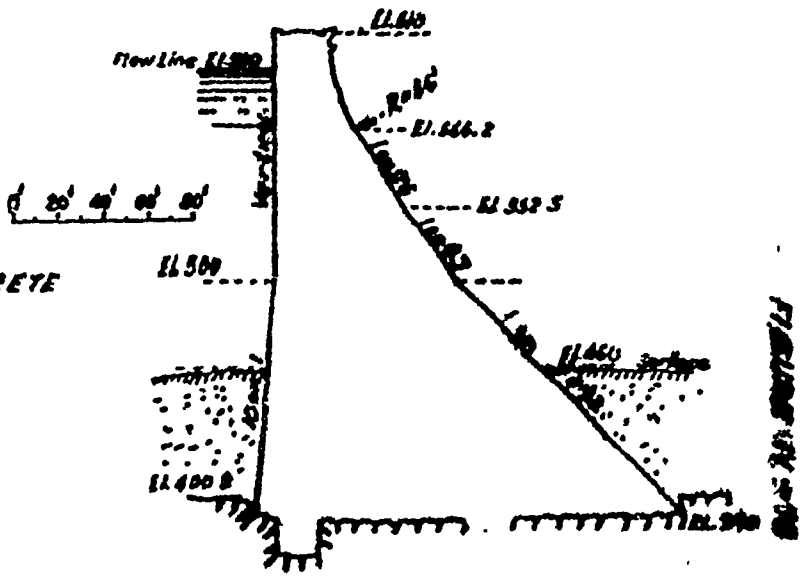
Equation IV-14 was programmed as shown on Plate IV-6 and the input or data file included:

a. Subbasin No. 19 unit hydrograph - see Table III-11

b. Subbasin No. 19 inflow storm hydrograph computed using

ASHOKAN RESERVOIR
CROSS-SECTON OF EARTH PARTS OF MAIN DAM AND WEST DIKE

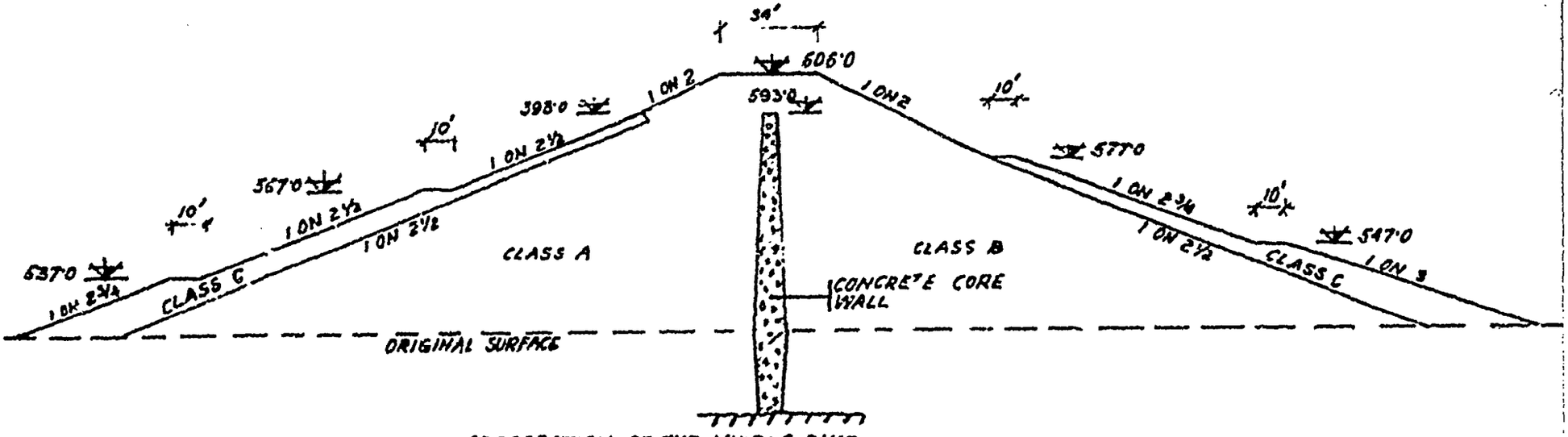




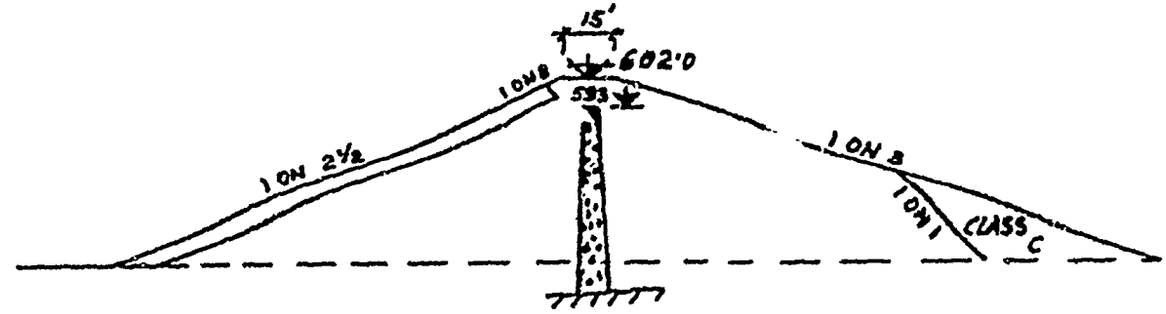
CROSS-SECTION OF CONCRETE PART OF MAIN DAM

ASHOKAN RESERVOIR

CROSS-SECTIONS OF EAST BASIN DIKES



CROSSECTION OF THE MIDDLE DIKE



CROSSECTION OF THE EAST DIKE

FIGURE IV-10

ESOPUS CREEK

PROGRAM FOR THE GENERATION OF FLOOD HYDROGRAPHS
AND FLOOD ROUTING THROUGH ASHOKAN RESERVOIR

```
ESASH     10:49   RDS3  APRIL 28, 1970

50 DIM G(2,200),Q(2,200),P(12)
55 DIM A(150),C(150),H(150),Z(150)
56 DIM E(130)
100 READ H(0)
105 READ Z(0)
110 READ T
390 FOR Y=1 TO2
400 READ K
410 FOR J=1 TO K
420 READ G(Y,J)
430 NEXT J
440 READ B
450 FOR I=1 TO 12
460 READ P(I)
470 NEXT I
480 FOR I=1 TO 12
490 FOR J=1 TO K
500 LET N=J+(I-1)*3
510 LET Q(Y,N)=(G(Y,J)*P(I))+Q(Y,N)
520 NEXT J
530 NEXT I
540 FOR N=0 TO 200
550 LET Q(Y,N)=(Q(Y,N)*(10:3))+B
560 NEXT N
570 NEXT Y
600 LET A(0)=0
610 LET C(0)=0
620 FOR N=1 TO 120
630 LET X9=0
635 LET R=0
640 LET D1=C(N-1)
650 LET S=(((D1+C(N-1))+((Q(1,N-1)+Q(1,N))/10))/2)*T
660 LET V2=(S/((3.03/1.3)*(10:8)))+H(N-1)-H(0)
670 LET D2=4.0*900.0*(V2:1.5)
680 LET O2=(D2+A(N-1))*T
690 LET S2=S-O2
700 LET Y2=(S2/((3.03/1.3)*(10:8)))+H(N-1)-H(0)
710 IF ABS(V2-Y2)<0.02 GOTO 735
720 LET V2=(V2+Y2)/2
730 GOTO 670
735 IF R=1 GOTO 760
740 LET S3=((Q(1,N-1)+Q(1,N))/2)*T*0.9
750 LET V1=(S3/(1.4*(10:8)))+Z(N-1)-Z(0)
760 LET X=V2-3
770 IF X<0 GO TO 790
780 GOTO 795
```

```
790 LET X=0
795 IF V1>X GOTO 800
796 LET V1=V1+0.1
797 GOTO 795
800 LET D1=4.0*100.0*(V1^1.5)*(1-((X/V1)^1.5))
810 LET O1=((D1+C(N-1))/2)*T
820 LET S1=S3-O1
830 LET Y1=(S1/(1.4*(10^8)))+Z(N-1)-Z(0)
840 IF ABS(V1-Y1)<0.02 GOTO 870
850 LET V1=(V1+Y1)/2
860 GOTO 795
870 IF ABS (V1-X9)<0.05 GOTO 900
875 LET R=1
880 LET X9=V1
890 GOTO 650
900 LET A(N)=D2
910 LET H(N)=V2+H(0)
915 PRINT N;")";H(N),
920 LET C(N)=D1
930 LET Z(N)=V1+Z(0)
940 NEXT N
950 FOR N=1 TO 120
960 LET A(N)=A(N)/(10^3)
970 LET Q(1,N)=Q(1,N)/(10^3)
971 NEXT N
972 PRINT
973 PRINT
975 PRINT " ASHOKAN RESERVOIR  -  PMF"
980 PRINT
990 PRINT
992 PRINT"TIME   INFLOW    ELEVATION      ELEVATION      OUTFLOW"
993 PRINT"                   WEST           EAST           "
1010 PRINT"HR'S  1000CFS    FT M.S.L.      FT M.S.L.      1000CFS"
1020 PRINT
1030 FOR N=1 T0100
1040 PRINT 2*N,Q(1,N),Z(N),H(N),A(N)
1050 NEXT N
1060 PRINT
1070 PRINT
1080 PRINT
1090 FOR N=1 TO 200
1100 LET Q(2,N)=Q(2,N)/(10^3)
1110 NEXT N
1150 FOR N=1 TO 120
1160 LET E(N)=A(N)+Q(2,N)
1170 NEXT N
1180 PRINT
1190 PRINT
1200 PRINT "ESOPUS CREEK    -     PMF"
1210 PRINT
1220 PRINT" HR'S          1000CFS"
1230 PRINT
1235 FOR N=1 TO 120
1240 PRINT 2*N,E(N)
1245 NEXT N
1260 PRINT
1270 PRINT " 0 OF TIME IS BEGINNING OF PMP"
1280 PRINT
1290 PRINT
```

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 stage - 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:

$$O_e = CLH^{3/2} \qquad \text{(IV-15)}$$

in which

C = Constant equal to 4 feet$^{\frac{1}{2}}$/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:

$$Q_w = CL'H_w^{3/2}\left[1 - \left(\frac{H_e}{H_w}\right)^{3/2}\right] \qquad \text{(IV-16)}$$

in which

C = Constant equal to 4 feet$^{\frac{1}{2}}$ /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 IV-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

ESOPUS CREEK

DATA FOR THE GENERATION OF FLOOD HYDROGRAPHS
AND FLOOD ROUTING THROUGH ASHOKAN RESERVOIR

```
2000 DATA 587.0,590.0
2010 DATA 7200.0
2020 DATA 22
2030 DATA 0.35,1.15,2.50,4.50
2040 DATA 7.30,10.72,10.72,9.82
2050 DATA 8.52,6.90,5.60,4.50
2060 DATA 3.55,2.80,2.20,1.60
2070 DATA 1.10,0.75,0.50,0.40
2080 DATA 0.13,0.00
2180 DATA 260
2190 DATA 0.00,0.45,0.58,0.24
2200 DATA 1.29,3.73,1.45,0.96
2210 DATA 0.03,0.09,0.13,0.04
2300 DATA 24.0
2310 DATA 0.20,0.40,0.60,0.86
2320 DATA 1.25,1.80,2.70,4.62
2330 DATA 3.35,6.25,5.60,4.75
2340 DATA 4.00,3.35,2.74,2.25
2350 DATA 1.82,1.44,1.11,0.83
2360 DATA 0.59,0.39,0.20,0.00
2380 DATA 170.0
2390 DATA 0.00,0.60,0.67,0.29
2400 DATA 1.60,6.43,1.98,1.30
2410 DATA 0.10,0.17,0.22,0.11
3040
9000 END
```

U.H. SUBBASIN 19

RUNOFF DATA SUBBASIN 19

U.H. SUBBASIN 20

RUNOFF, SUBBASIN 20

ESOPUS CREEK

PRINTOUT FOR THE GENERATION OF FLOOD HYDROGRAPHS AND FLOOD ROUTING THROUGH ASHOKAN RESERVOIR

Time Hr's	Inflow 1000 cfs	Elevation West Ft M.S.L.	Elevation East Ft M.S.L.	Outflow 1000 cfs
2	.26	590.012	587.001	8.19431E-5
4	.26	590.024	587.002	2.71671E-4
6	.26	590.036	587.003	6.20301E-4
8	.4175	590.052	587.005	1.43247E-3
10	.7775	590.079	587.01	3.37456E-3
12	1.385	590.129	587.018	8.41018E-3
14	2.488	590.219	587.034	2.22966E-2
16	4.212	590.355	587.065	.060278
18	6.534	590.549	587.124	.157331
20	7.778	590.768	587.28	.37043
22	9.189	590.989	587.341	.716088
24	10.9116	591.202	587.482	1.20292
26	11.1141	YYYY91.388	587.641	1.8493
28	11.7111	591.529	587.792	2.53938
30	13.0844	591.658	587.939	3.27821
32	15.5428	591.815	588.086	4.07455
34	2 .8313	592.08	588.239	4.96494
36	49.0586	592.461	588.443	6.84327
38	35.8163	592.967	588.721	8.13081
40	45.2873	593.524	589.075	10.7578
42	57.5549	594.193	589.509	13.3104
44	58.0126	594.737	589.978	18.5038
46	57.2916	595.003	590.357	22.1457
48	56.8101	595.142	590.604	24.6328
50	51.125	595.137	590.727	25.9045
52	46.5375	594.978	590.74	26.0405
54	42.8586	594.763	590.676	25.3751
56	36.1482	594.47	590.557	24.1477
58	30.0887	594.088	590.385	22.4228
60	24.7155	593.673	590.173	20.352
62	19.4166	593.252	589.934	18.0955
64	15.4161	592.857	589.677	15.7658
66	12.2806	592.509	589.42	13.5505
68	9.6238	592.213	589.188	11.649
70	8.0818	591.958	588.974	9.98263
72	6.3398	591.738	588.781	8.55391
74	4.8221	591.535	588.606	7.32751
76	4.0526	591.358	588.449	6.27794
78	3.1759	591.207	588.308	5.38799
80	2.4333	591.07	588.183	4.63167
82	2.0498	590.961	588.072	3.99277
84	1.5311	590.863	587.983	3.50923
86	1.1565	590.773	587.902	3.08144
88	.959	590.692	587.827	2.70566
90	.7974	590.623	587.759	2.37847
92	.655	590.563	587.697	2.09491
94	.551	590.519	587.642	1.85282
96	.4572	590.48	587.594	1.64603
98	.389	590.443	587.55	1.46907

Time hr's	Inflow 1000 cfs	Elevation West Ft M.S.L.	Elevation East Ft M.S.L.	Outflow 1000 cfs
100	.356	590.411	587.521	1.35147
102	.3069	590.382	587.492	1.24361
104	.28	590.356	587.466	1.144
106	.276	590.333	587.441	1.05316
108	.2652	590.313	587.417	.970845
110	.26	590.305	587.396	.898148
112	.26	590.297	587.378	.835398
114	.26	590.29	587.361	.780893
116	.26	590.284	587.346	.733278
118	.26	590.279	587.333	.691469
120	.26	590.273	587.321	.654585
122	.26	590.269	587.32	.650927
124	.26	590.264	587.318	.646256
126	.26	590.261	587.316	.640306
128	.26	590.257	587.314	.634776
130	.26	590.254	587.312	.628329
132	.26	590.251	587.31	.621603
134	.26	590.248	587.308	.614709
136	.26	590.246	587.305	.60774
138	.26	590.243	587.303	.600771
140	.26	590.241	587.301	.593863
142	.26	590.239	587.298	.587063
144	.26	590.238	587.296	.580409
146	.26	590.236	587.294	.573931
148	.26	590.235	587.292	.56765
150	.26	590.233	587.29	.561583
152	.26	590.232	587.288	.555741
154	.26	590.231	587.286	.550131
156	.26	590.23	587.284	.544757
158	.26	590.229	587.282	.539619
160	.26	590.228	587.28	.534718
162	.26	590.227	587.279	.530049
164	.26	590.227	587.277	.525609
166	.26	590.226	587.276	.521392
168	.26	590.225	587.274	.517392
170	.26	590.225	587.273	.513603
172	.26	590.224	587.272	.510016
174	.26	590.224	587.271	.506624
176	.26	590.223	587.269	.50342
178	.26	590.223	587.268	.500395
180	.26	590.223	587.267	.497542
182	.26	590.222	587.266	.494853
184	.26	590.222	587.265	.49232
186	.26	590.222	587.265	.489936
188	.26	590.221	587.264	.487692
190	.26	590.221	587.263	.485583
192	.26	590.221	587.262	.483601
194	.26	590.221	587.262	.481738
196	.26	590.221	587.261	.47999
198	.26	590.22	587.26	.47835
200	.26	590.22	587.26	.476811

OUTFLOW HYDROGRAPH
FROM
ESOPUS CREEK - PMF

HR'S	1000CFS
2	.170082
4	.170272
6	.17062
8	.291432
10	.413375
12	.53841
14	.842297
16	1.24827
18	1.80933
20	2.79463
22	4.61159
24	4.78292
26	8.1477
28	10.1673
30	10.0247
32	14.2771
34	16.8087
36	18.9493
38	24.3771
40	32.0858
42	36.6437
44	51.2795
46	66.5912
48	60.2927
50	80.6142
52	79.3178
54	70.1165
56	69.8509
58	64.3704
60	54.3723
62	50.8873
64	43.4989
66	36.1325
68	30.3955
70	25.3233
72	20.3509
74	17.2125
76	14.1344
78	10.5793

hr's	1000 cfs
80	9.57047
82	8.11847
84	6.46953
86	5.81944
88	4.88106
90	3.97757
92	3.42761
94	2.95222
96	2.53453
98	2.19707
100	1.93877
102	1.70181
104	1.5219
106	1.35846
108	1.20575
110	1.11105
112	1.0274
114	.950893
116	.903278
118	.861469
120	.824585
122	.820927
124	.818186
126	.804776
128	.798329
130	.791603
132	.784709
134	.77774
136	.770771
138	.763863
140	.757063
142	.750409
144	.743931
146	.73765
148	.731583
150	.725741
152	.720131
154	.714757
156	.709619
158	.704718
160	.700049
162	.695609
164	.691392
166	.687392
168	

hr's	1000 cfs
170	.683603
172	.680016
174	.676624
176	.67342
178	.670395
180	.667542
182	.664853
184	.66232
186	.659936
188	.657692
190	.655583
192	.653601
194	.651738
196	.64999
198	.64835
200	.646811
202	.645368
204	.644015
206	.642748
208	.641561
210	.64045
212	.63941
214	.638437
216	.637527
218	.636676
220	.63588
222	.635137
224	.634443
226	.633794
228	.633189
230	.632623
232	.632096
234	.631604
236	.631145
238	.630717
240	.630317

0 OF TIME IS BEGINNING OF PMP

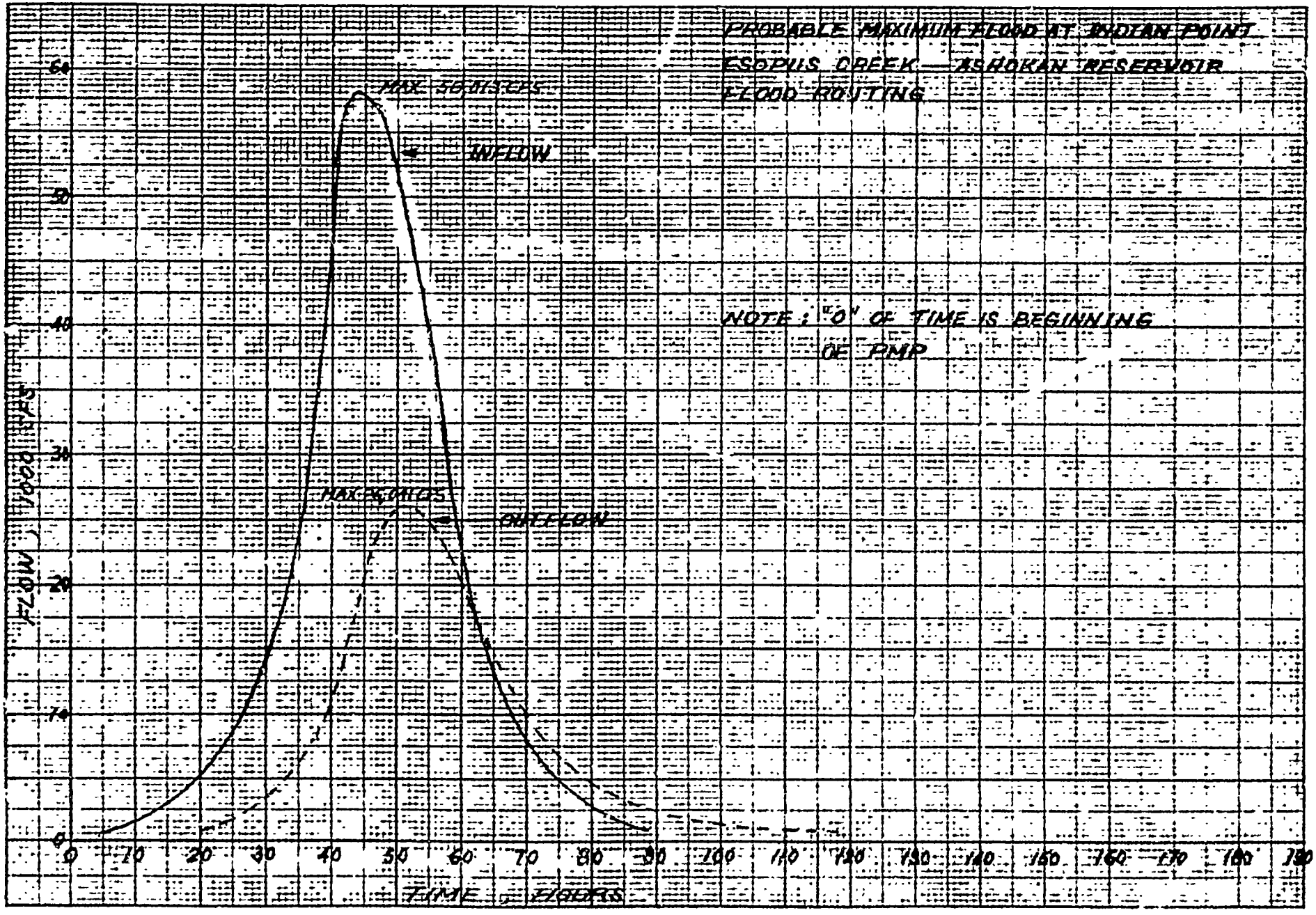

PROBABLE MAXIMUM FLOOD AT INDIAN POINT
ESOPUS CREEK — ASHOKAN RESERVOIR
FLOOD ROUTING
NOTE: "0" OF TIME IS BEGINNING OF PMP
INFLOW
OUTFLOW
FLOW, 1000 CFS
TIME, HOURS
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190

FIGURE IV-11

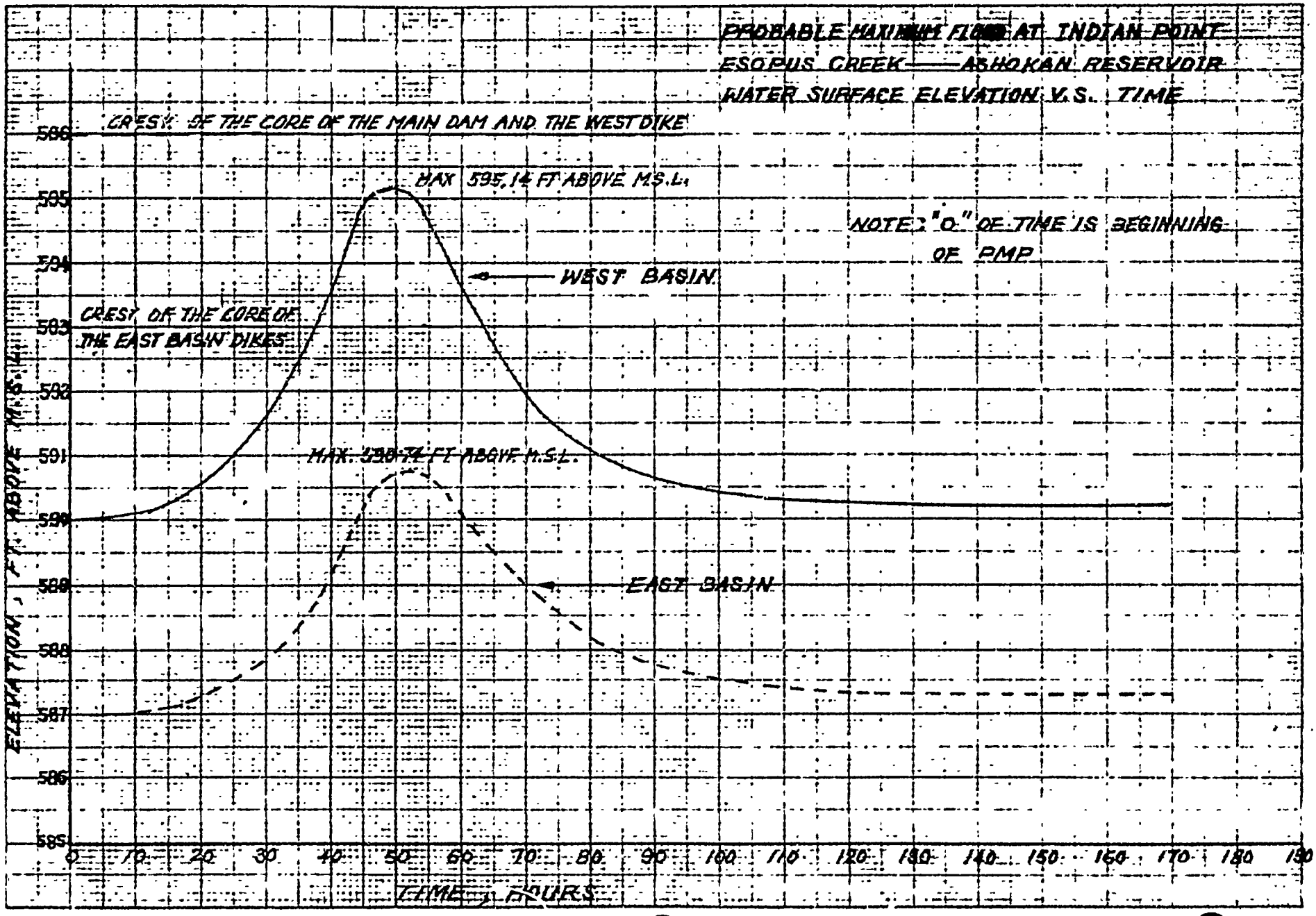


FIGURE IV-11a

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 an 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 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.

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-hour 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.

FIGURE IV-12

ESOPUS CREEK -ASHOKAN RESERVOIR

PROBABLE MAXIMUM PRECIPITATION OVER ESOPUS CREEK DRAINAGE BASIN, AT ASHOKAN DAM (257 SQ. MI)
DEPTH - DURATION CURVE

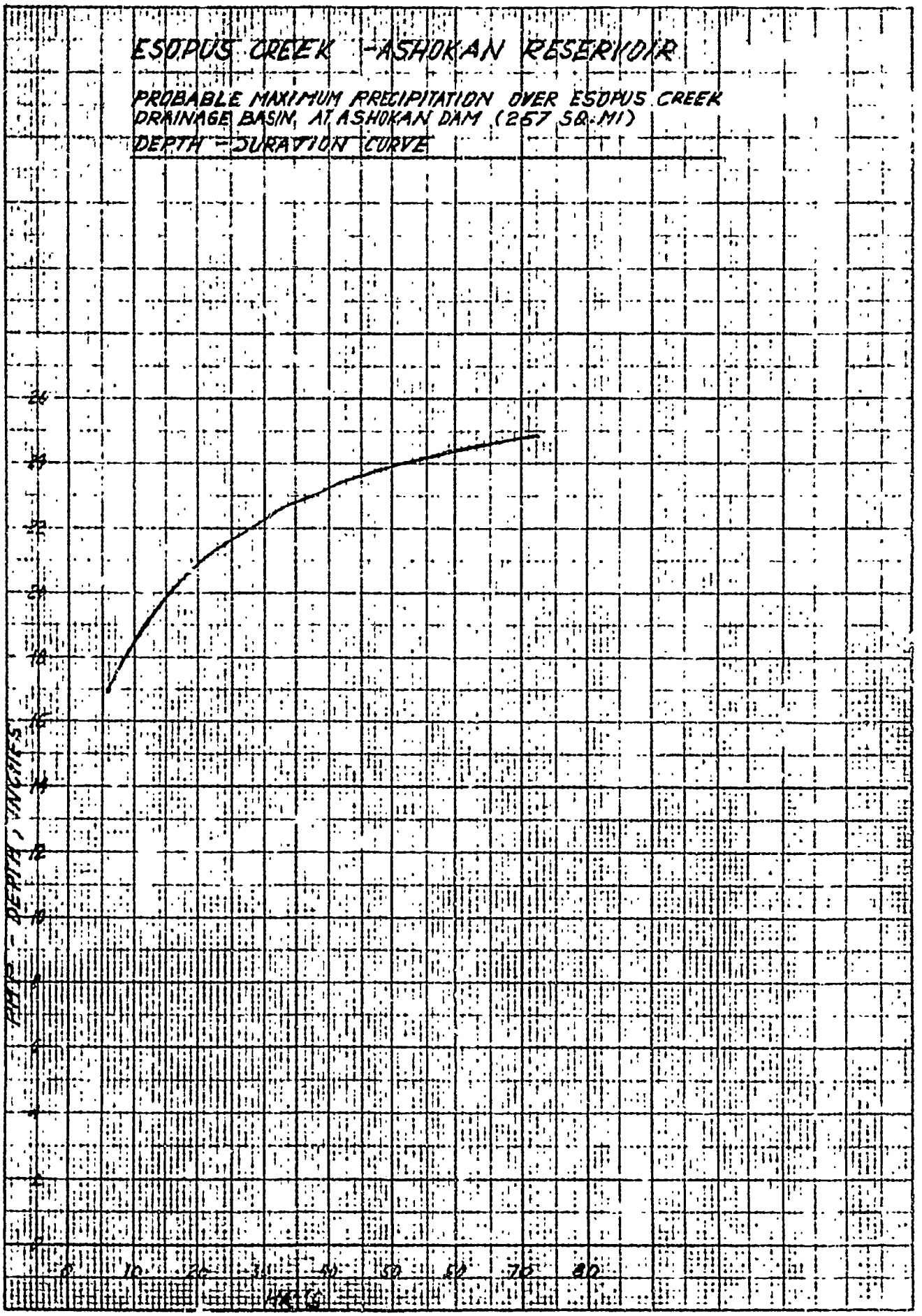
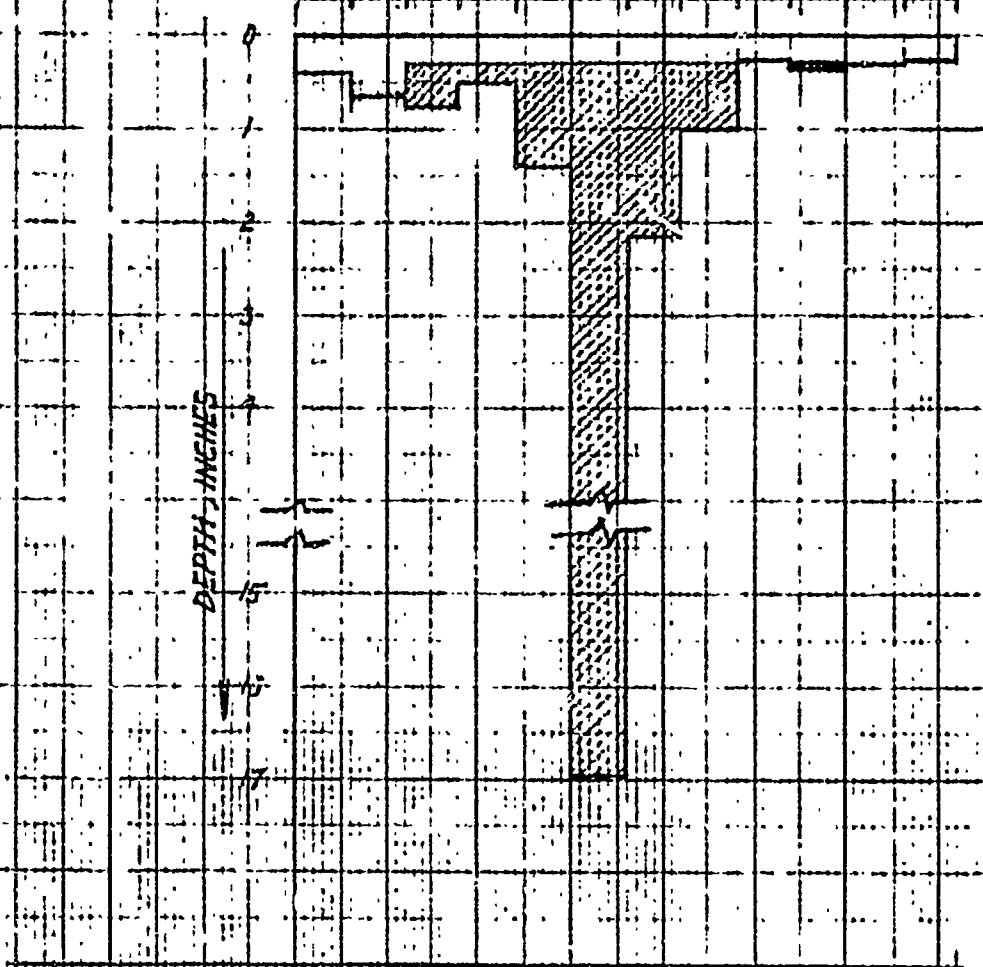


FIGURE IV-13

PROBABLE MAXIMUM PRECIPITATION OVER ESOPUS CREEK DRAINAGE BASIN

DRAINAGE AREA = 257 SQ MI.



TIME FROM BEGINNING OF PMP (HOURS)	6	12	18	24	30	36	42	48	54	60	66	72
PMP 6-HR INCREMENT	8	5	7	5	3	1	2	4	1	9	10	12
PMP INCREMENT OF RAIN (INCHES)	0.42	0.65	0.75	0.50	1.40	16.25	2.15	1.00	0.27	0.35	0.30	0.25
RAINFALL EXCESS (INCHES)	0.12	0.40	0.55	0.20	1.10	16.05	1.85	0.70	0.00	0.05	0.00	0.00

NOTE - INITIAL LOSSES = 1.0"
INFILTRATION LOSSES = 0.05"/HR.

PROBABLE MAXIMUM PRECIPITATION OVER ESOPUS CREEK DRAINAGE BASIN

VALUE OF PROBABLE MAXIMUM FLOOD TAKEN FROM FIG. III-27

	6 HR's	12 HR's	24 HR's	48 HR's	72 HR's
INCREMENT OF PMP (INCHES)	16.95	19.10	21.50	23.80	24.95

TABULATION OF DEPTH AND 6-HR INCREMENTS OF PMP

DURATION (HRS)	6	12	18	24
DEPTH (INCHES)	16.95	19.10	20.50	21.50
# OF 6HR INCREMENT	1	2	3	4
6HR-INCREMENT (INCHES)	16.95	2.15	1.40	1.00

DURATION (HRS)	30	36	42	48
DEPTH (INCHES)	22.25	22.90	23.40	23.80
# OF 6HR INCREMENT	5	6	7	8
6HR INCREMENT (INCHES)	0.75	0.65	0.50	0.40

DURATION (HRS)	54	60	66	72
DEPTH (INCHES)	24.15	24.45	24.72	24.95
# OF 6HR INCREMENT	9	10	11	12
6HR-INCREMENT (INCHES)	0.35	0.30	0.27	0.23

SEQUENCE OF 6HR INCREMENT OF THE PMP STORM

# OF 6HR INCREMENT	8	6	5	7
DEPTH (INCHES)	0.40	0.65	0.75	0.50

# OF 6 HR INCREMENT	3	1	2	4
DEPTH (INCHES)	1.40	16.95	2.15	1.00

# OF 6 HR INCREMENT	11	9	10	12
DEPTH (INCHES)	0.27	0.35	0.30	0.23

RAINFALL EXCESS (RUNOFF)

# OF 6 HR INCREMENT	8	6	7	5
RUNOFF (INCHES)	0	0	0.45	0.20

# OF 6 HR INCREMENT	3	1	2	4
RUNOFF (INCHES)	1.10	16.05	1.85	0.70

# OF 6 HR INCREMENT	11	9	10	12
RUNOFF (INCHES)	0.00	0.05	0.00	0.00

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 200,000 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

PROBABLE MAXIMUM FLOOD OVER ESOPUS CREEK DRAINAGE BASIN

ASHOKAN RESERVOIR FLOOD ROUTING

Time Hr's	Inflow 1000 cfs	Elevation West Ft M.S.L.	Elevation East Ft M.S.L.	Outflow 1000 cfs
2	.26	590.012	587.001	8.19431E-5
4	.26	590.024	587.002	2.7167E-4
6	.26	590.036	587.003	6.20301E-4
8	.26	590.048	587.005	1.20919E-3
10	.26	590.06	587.007	2.27127E-3
12	.26	590.072	587.01	3.86278E-3
14	.4175	590.088	587.015	6.36166E-3
16	.7775	590.116	587.021	1.08789E-2
18	1.385	590.166	587.031	2.01173E-2
20	2.355	590.241	587.05	4.01227E-2
22	3.775	590.363	587.082	8.49512E-2
24	5.584	590.526	587.138	.183557
26	6.439	590.704	587.222	.376881
28	7.634	590.883	587.323	.662324
30	9.488	591.076	587.441	1.05259
32	17.1865	591.391	587.602	1.68231
34	33.3815	592.055	587.863	2.88587
36	59.55	593.233	588.347	5.62718
38	92.746	594.885	589.19	11.6649
40 DAM FAILURE	139.078	CORE 596.0 $\boxed{596.974}$	590.468	23.2536
42	196.339	599.64	592.065	41.0399
44	197.718	601.873	593.643	61.6425
46	186.408	602.859	594.718	77.1951
48	170.327	603.05	595.151	83.7698
50	143.287	602.574	595.118	85.2696
52	120.817	601.649	594.766	77.9158
54	101.519	600.587	594.253	70.3234
56	81.834	599.451	593.671	62.0329
58	65.6815	598.293	593.063	53.7467
60	52.305	597.196	592.463	45.9706
62	39.1725	596.147	591.886	38.8854
64	28.56	595.147	591.333	32.474
66	20.6725	594.243	590.813	26.8045
68	14.566	593.464	590.335	21.9216
70	11.406	592.828	589.904	17.8166
72	5.778	592.301	589.507	14.292
74	2.65	591.837	589.153	11.3725
76	2.05	591.475	588.839	8.97871
78	1.2505	591.21	588.574	7.1068
80	.7875	591.002	588.353	5.66493

THE VALUES COMPUTED FOR TIME >40HRS DO NOT TAKE IN ACCOUNT THE DAM FAILURE

ASHOKAN RESERVOIR - WAVE AFTER DAM FAILURE

CONDITIONS : PROBABLE MAXIMUM PRECIPITATION ON ESOPUS CREEK DRAINAGE BASIN

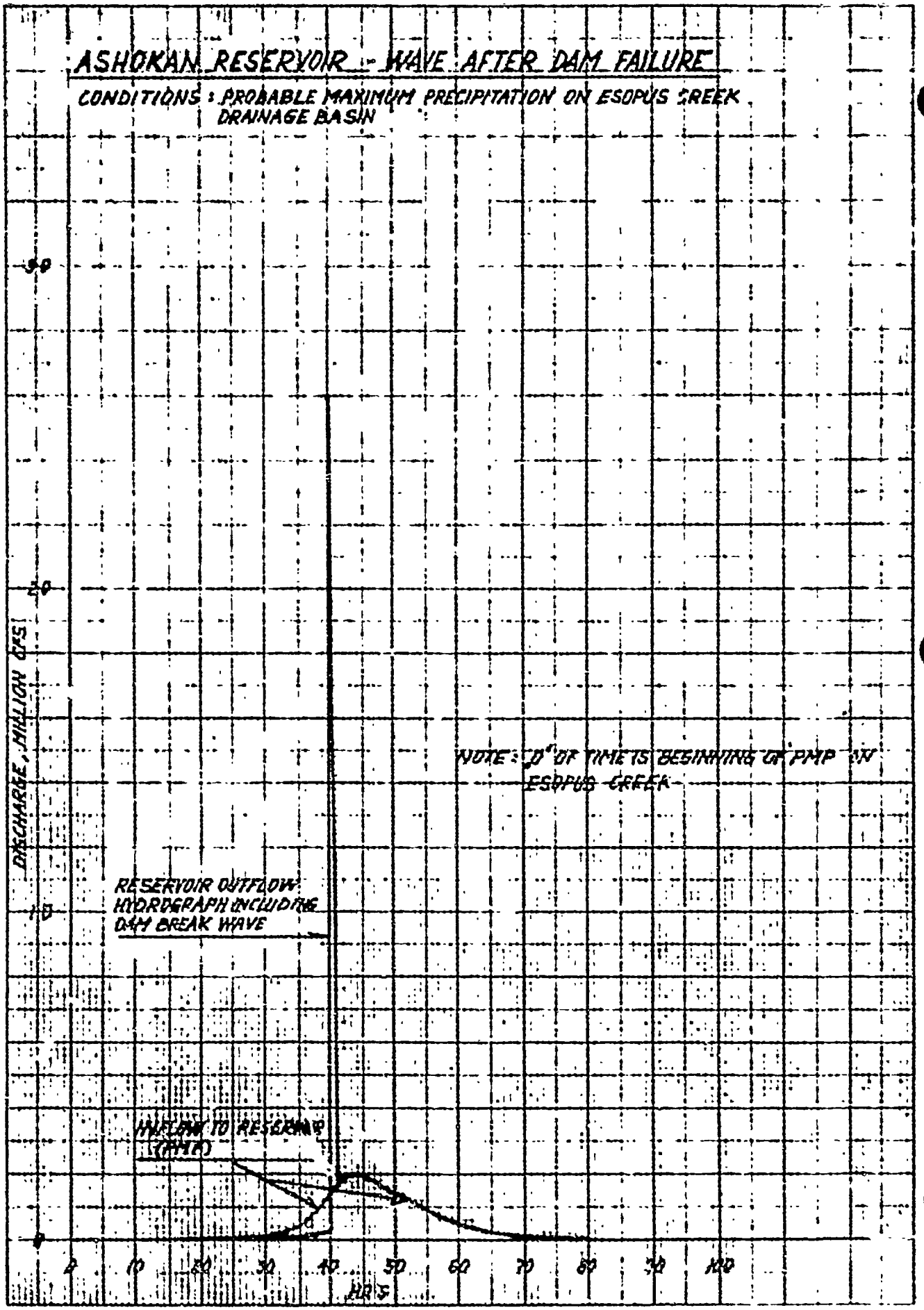
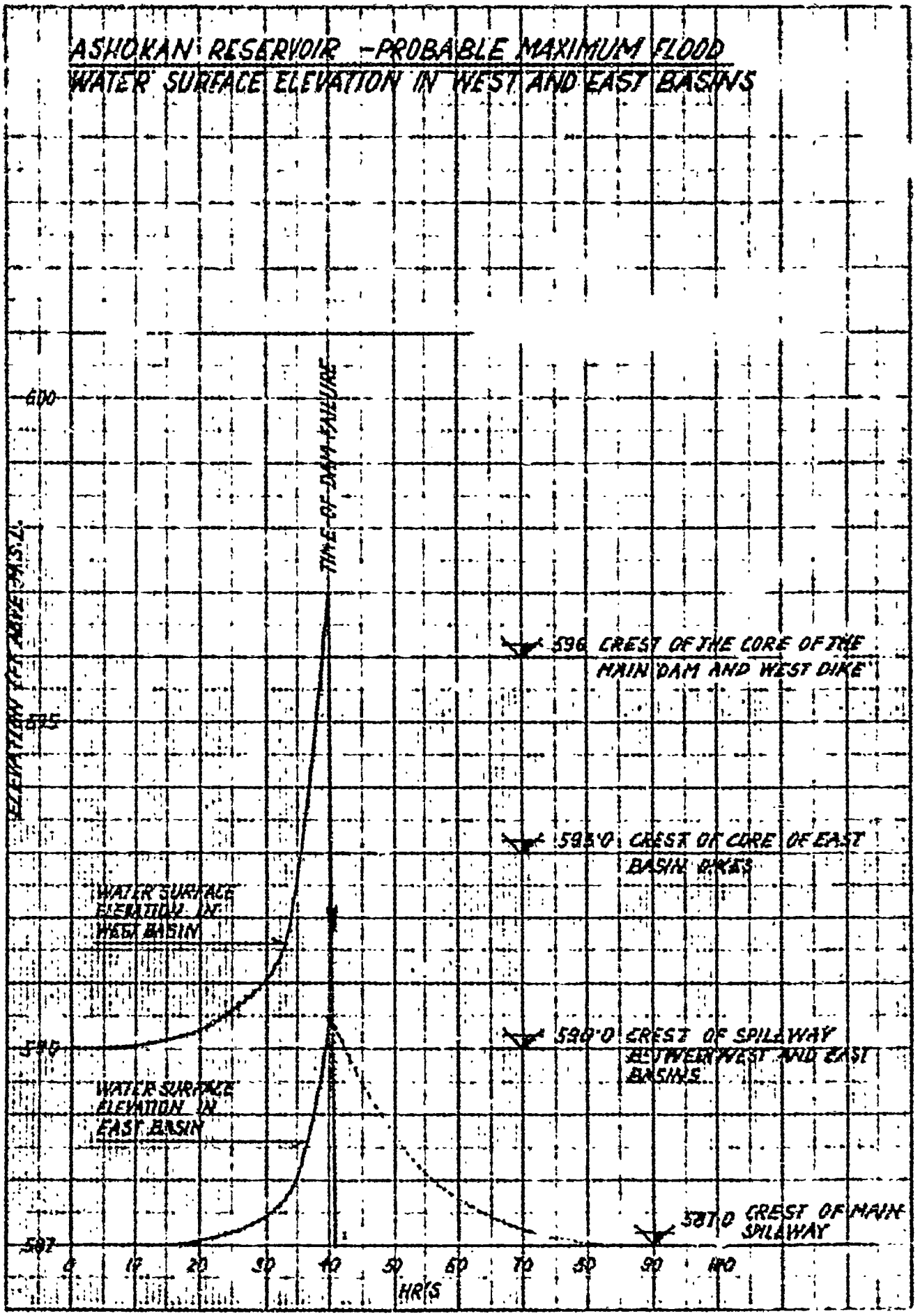


FIGURE IV-15


ASHOKAN RESERVOIR - PROBABLE MAXIMUM FLOOD
WATER SURFACE ELEVATION IN WEST AND EAST BASINS
ELEVATION FEET ABOVE M.S.L.
600
595
590
587
TIME OF DAM FAILURE
596 CREST OF THE CORE OF THE MAIN DAM AND WEST DIKE
593.0 CREST OF CORE OF EAST BASIN DIKES
590.0 CREST OF SPILLWAY BETWEEN WEST AND EAST BASINS
587.0 CREST OF MAIN SPILLWAY
WATER SURFACE ELEVATION IN WEST BASIN
WATER SURFACE ELEVATION IN EAST BASIN
0
10
20
30
40
50
60
70
80
90
100
HRS

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

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 would not fail under the Esopus Creek PMP conditions. The following two conditions were evaluated:

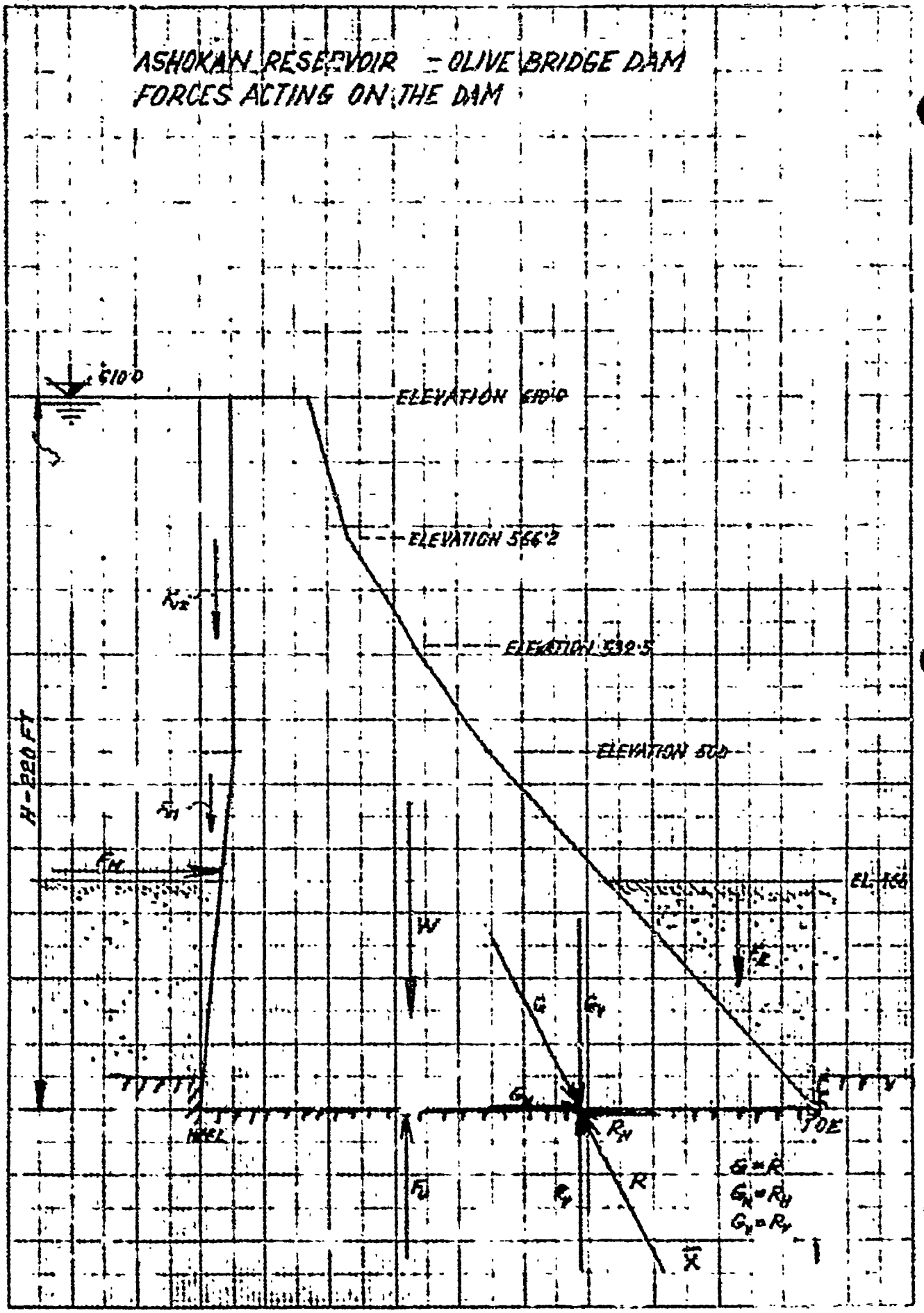
Condition i - An upstream water level at elevation 610.0 and no water downstream of the dam.

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:

FIGURE IV-16


ASHOKAN RESERVOIR - OLIVE BRIDGE DAM
FORCES ACTING ON THE DAM
610.0
ELEVATION 610.0
ELEVATION 566.2
ELEVATION 532.5
ELEVATION 500
H-220 FT
W
G
R
HEEL
TOE
G = R
G_H = R_H
G_V = R_V

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

= $\frac{1}{2}\gamma H^2$, where γ= specific weight of water.

P_V = vertical component of hydrostatic pressure

= 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.

P_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 approximately linearly from full hydrostatic head at the heel to full tailwater head, and P_u is approxi-

mately $\frac{1}{2}\gamma HB$, 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_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 equilibrium of the dam, by equating to zero the sum of the moments of all the foregoing forces about the toe of the dam.

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 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 $1,510 \times 10^3$ and 2542×10^3 pounds respectively. The location of the resultant is in the middle third of the base about 72.5 feet from the toe.

The factors 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.

ASHOKAN RESERVOIR - OLIVE BRIDGE DAM STRUCTURAL ANALYSIS

TABLE IV-3

CONDITIONS : WATER SURFACE ELEVATION ABOVE THE DAM IS 610 C ET ABOVE M.S.L. (= ELEVATION OF THE DAM CREST)
NOTE : FOR EXPLANATION OF THE VARIABLES . SEE TABLE IV-4

FORCE	EQUATION FOR COMPUTATION OF THE FORCE	EQUATION FOR MOMENT ARM ABOUT THE TOE	FORCE IN LB	MOMENT ARM* ABOUT TOE FT	MOMENT IN LB-FT
F_H - HORIZONTAL COMPONENT OF HYDROSTATIC PRESSURE ON UPSTREAM FACE	$F_H = \frac{1}{2}\gamma H^2$	$r_H = -\frac{1}{3}H$	1,510·8	- 73·33	-110,742
W - WEIGHT OF DAM	$W = \Sigma W_i \qquad W_i = \gamma_c \cdot A_i$	$r_W = \frac{\Sigma W_i r_{Wi}}{W}$	2,979·1	+ 124·83	-371,409
F_V - VERTICAL COMPONENT OF HYDROSTATIC PRESSURE ON UPSTREAM FACE	$F_V = \gamma$ (Area of water mass vertically above the upstream face)	r_V = Horizontal distance between the centroid of the water mass and the toe	103·0	+ 185·50	+ 19,114
F_U - UPLIFT FORCE ON THE BASE OF THE DAM	$F_U = C\frac{1}{2}\gamma HB \quad , C = 0·6$	$r_U = -\frac{2}{3}B$	806·3	- 126·60	-102,078
F_E - VERTICAL COMPONENT OF EARTH PRESSURE ON THE DOWNSTREAM FACE	$F_E = \gamma_E$ (Area of earth mass vertically above the downstream face)	r_E = Horizontal distance between the centroid of the earth mass and the toe	266·7	+ 25·40	+ 6,774
G_V - VERTICAL COMPONENT OF RESULTANT FORCE	$G_V = W + F_V + F_E - F_U$	$r_{GV} = \frac{\text{Moment of all forces}}{G_V}$	2,542·5	+ 72·50	184,477
G_H - HORIZONTAL COMPONENT OF RESULTANT FORCE	$G_H = F_H$	$r_{GH} = 0$	1,510·8	0·00	0

* The moment arm is considered positive if the force causes a resisting moment (anti-clockwise) and negative if the force causes an overturning moment at the toe of the dam.

FACTORS OF SAFETY OF THE DAM:

1) Factor of safety against sliding:

$$(FS)_S = \frac{G_V \cdot C_f}{G_H} = \frac{2,542·5 \times 10^6 \times 0·65}{1,510·8 \times 10^6}$$

$$(FS)_S = 1·09$$

2) Factor of safety against overturning

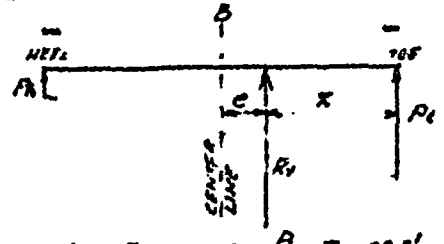
$$(FS)_O = \frac{\Sigma \text{ resisting moments}}{\Sigma \text{ overturning moments}}$$

$$(FS)_O = \frac{W \cdot r_W + F_V \cdot r_V + F_E \cdot r_E}{-(F_H \cdot r_H + F_U \cdot r_U)}$$

$$(FS)_O = \frac{397,297 \times 10^3}{212,820 \times 10^3}$$

$$(FS)_O = 1·87$$

BEARING PRESURE AND STRESS ANALYSIS:



$R_V = G_V$, $x = r_{GV}$, $e = \frac{B}{2} - x = 8·25'$

$$p_t = \left(\frac{R_V}{B}\right)\left(1 + \frac{6e}{B}\right)$$

$$p_h = \left(\frac{R_V}{B}\right)\left(1 - \frac{6e}{B}\right)$$

$$\left(\frac{R_V}{B}\right) = \frac{2,542·5 \times 10^3}{190 \times 144} = 93·0 \text{ psi}$$

$p_t = 93·0\left(1 + \frac{6 \times 8·25}{190}\right) = 169·5$ psi

$p_h = 93·0\left(1 - \frac{6 \times 8·25}{190}\right) = 16·5$ psi

Bearing pressures just above the base ie no uplift

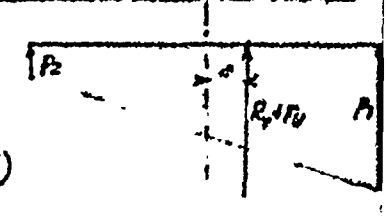
$$p_1 = \left(\frac{R_V - F_U}{B}\right)\left(1 + \frac{6e}{B}\right)$$

$$p_1 = 122·0\left(1 + \frac{6 \times 8·25}{190}\right)$$

$p_1 = 209$ psi

$p_2 = 122·0\left(1 - \frac{6 \times 8·25}{190}\right)$

$p_2 = 35$ psi



FIRST PRINCIPAL STRESS σ_1 PARALLEL TO DOWNSTREAM FACE :

$$\sigma_1 = p_1(1 + \tan^2\theta)$$

$$\sigma_1 = 209(1 + 0·846)$$

$\sigma_1 = 386$ psi < 600 psi (max allowed)

MAXIMUM HORIZONTAL SHEARING STRESS :

$$\tau_H = p_1 \cdot \tan\theta = 209 \times 0·92$$

$\tau_H = 192$ psi < 300 psi (max allowed)

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 the resultant are 1088×10^3 and 2044×10^3 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

ASHOKAN RESERVOIR – OLIVE BRIDGE DAM STRUCTURAL ANALYSIS

TABLE IV-4

CONDITION: WATER SURFACE ELEVATION UPSTREAM OF THE DAM IS 597.0 FT ABOVE M.S.L.
DEPTH OF WATER ABOVE THE BASE DOWNSTREAM OF THE DAM IS 90 FT
MASS OF EARTH VERTICALLY ABOVE THE DOWNSTREAM FACE IS ERODED

FORCE	EQUATION FOR COMPUTATION OF THE FORCE	EQUATION FOR MOMENT ARM ABOUT THE TOE *	FORCE 10^3 LB	MOMENT ARM ABOUT TOE FT	MOMENT 10^3 LB-FT
F_H – HORIZONTAL COMPONENT OF HYDROSTATIC PRESSURE ON UPSTREAM FACE	$F_H = \frac{1}{2}\gamma H^2$	$r_H = -\frac{1}{3}H$	1,338.0	- 68.0	- 90,984
F_{HI} – HORIZONTAL COMPONENT OF HYDROSTATIC PRESURE ON DOWNSTREAM FACE	$F_{HI} = \frac{1}{2}\gamma H_1^2$	$r_{HI} = +\frac{1}{3}H_1$	250.0	+ 30.0	+ 7,500
W – WEIGHT OF DAM	$W = \Sigma W_i \qquad W_i = \gamma_c \cdot A_i$	$r_W = \frac{\Sigma W_i \cdot r_{wi}}{W}$	2,979.1	+ 124.8	+371,809
F_V – VERTICAL COMPONENT OF HYDROSTATIC PRESSURE ON UPSTREAM FACE	$F_V = \gamma \cdot$ (area of water mass vertically above upstream face)	r_V = horizontal distance between the centroid of the water mass and the toe	95.2	+195.6	+17,634
F_{VI} – VERTICAL COMPONENT OF HYDROSTATIC PRESSURE ON DOWNSTREAM FACE	$F_{VI} = \gamma \cdot$ (area of water mass vertically above downstream face)	r_{VI} = as above	230.0	+ 27.6	+ 6,348
F_U – UPLIFT FORCE ON THE BASE OF THE DAM	$F_U = \frac{1}{2}\gamma B(cH + H_1)$ $\quad c = 0.5$	$r_U = \frac{\frac{1}{2}B(c \cdot H - H_1) + BH_1}{cH + H_1}$	1,260.0	- 100.0	-126,000
G_V – VERTICAL COMPONENT OF RESULTANT FORCE	$G_V = W + F_V + F_{VI} - F_U$	$r_{GV} = \frac{\text{Moment of all forces}}{G_V}$	2044.1	+ 91.0	+185,907
G_H – HORIZONTAL COMPONENT OF RESULTANT FORCE	$G_H = F_H - F_{HI}$	$r_{GH} = 0$	1,088.0	0.0	0

* SEE TABLE IV-3

FACTORS OF SAFETY OF THE DAM:

1) Factor of safety against sliding

$$(FS)_S = \frac{G_V \cdot C_f}{G_H} = \frac{2,044.1 \times 10^6 \times 0.65}{1,088 \times 10^6}$$

$$(FS)_S = 1.22$$

2) Factor of safety against overturning

$$(FS)_O = \frac{\Sigma \text{ resisting moments}}{\Sigma \text{ overturning moments}}$$

$$(FS)_O = \frac{W \cdot r_W + F_V \cdot r_V + F_{VI} \cdot r_{VI} + F_H \cdot r_{HI}}{-(F_H \cdot r_H + F_U \cdot r_U)}$$

$$(FS)_O = \frac{402,691 \times 10^3}{216,964 \times 10^3}$$

$$(FS)_O = 1.86$$

EXPLANATION OF VARIABLES:

H – Water head upstream of the dam
H_1 – Depth of water above the base downstream of the dam
W_i – Component of the weight of the dam
A_i – Part of crossectional area of the dam related to W_i
γ – Specific weight of water (62.4 lb/cf)
γ_c – Specific weight of concrete (150 lb/cf)
γ_e – Specific weight of earth (= ko lb/cf)
B – Length of the base
$(FS)_S$ – Factor of safety against sliding
$(FS)_O$ – Factor of safety against overturning
p_t – Bearing pressure at the toe
p_h – Bearing pressure at the heel
p_1 – Bearing pressure just above the toe
p_2 – Bearing pressure just above the heel
σ_1 – First principal stress parallel to the downstream face
τ_x – Maximum horizontal stress
ϕ – Angle between the downstream face and the vertical
C_f – Coefficient of friction between the dam and the foundation (0.65)

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

a. Computation of Reservoir Outflow Hydrograph after Dam Failure

The work required to achieve this objective includes:

i. The use of Equation IV-4 to determine 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 the east basin outflow over the main spillway to obtain the reservoir outflow hydrograph. The cast 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

previous step with the flood hydrograph of the remaining part of the Esopus Creek Basin, i.e. Subbasin No. 20. Since the contribution of Subbasin No. 20 represents only a fraction of the reservoir outflow hydrograph, the resultant is essentially the same as the reservoir outflow hydrograph shown in Figure IV-14.

ii. Computation of the Valley storage in Subbasin 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

Y' = Depth at the crest of the wave

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.

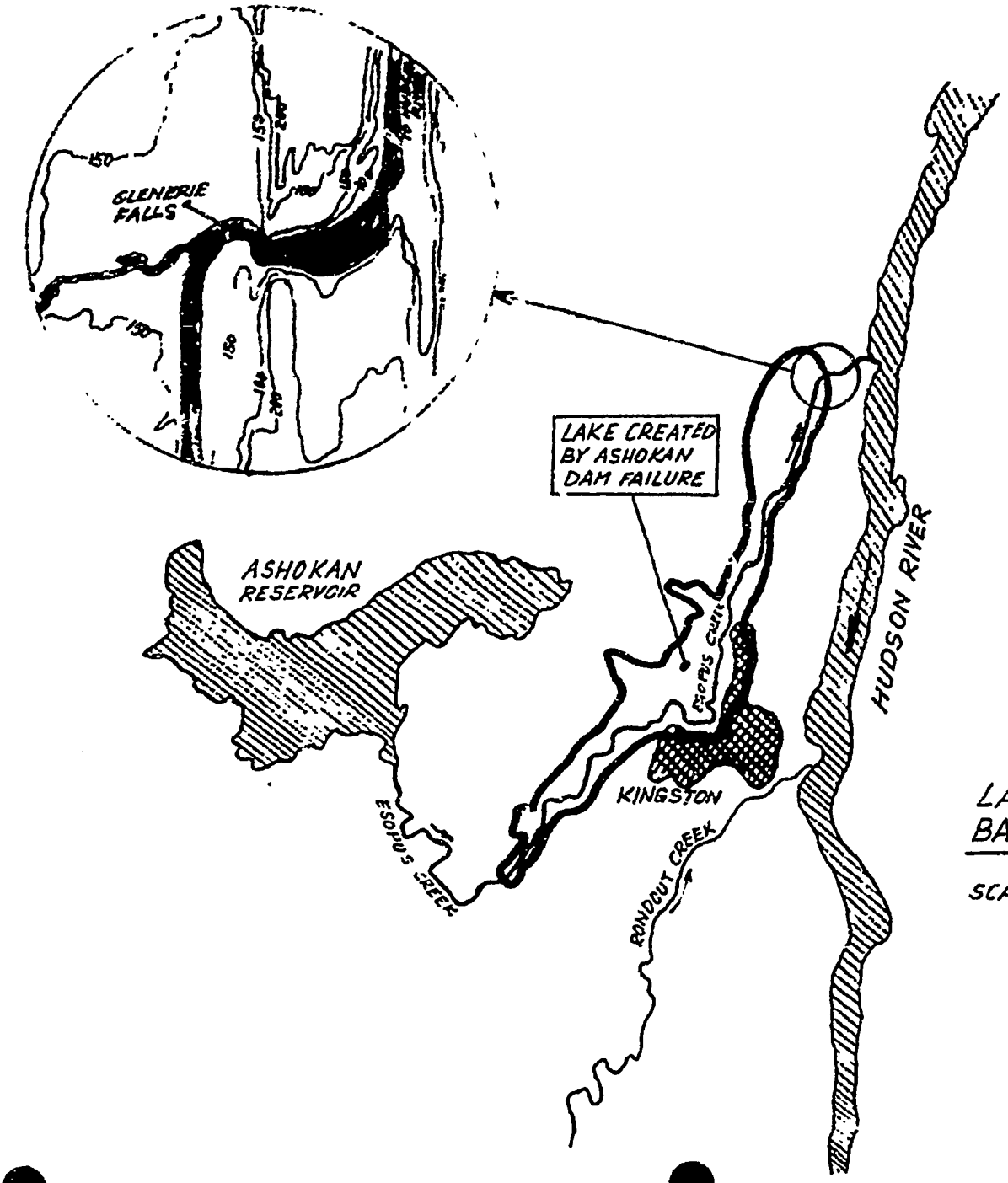
The lake would have a control section at Glenerie Falls where the width decreases

suddenly from 6000 feet to 500 feet. A sketch showing the size, direction of flow 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, delineates the shore lines of the lake. The storage capacity 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:



LAKE CREATED IN ESOPUS CREEK BASIN BY ASHOKAN DAM FAILURE

SCALE : 1 : 250,000

FIGURE IV-17

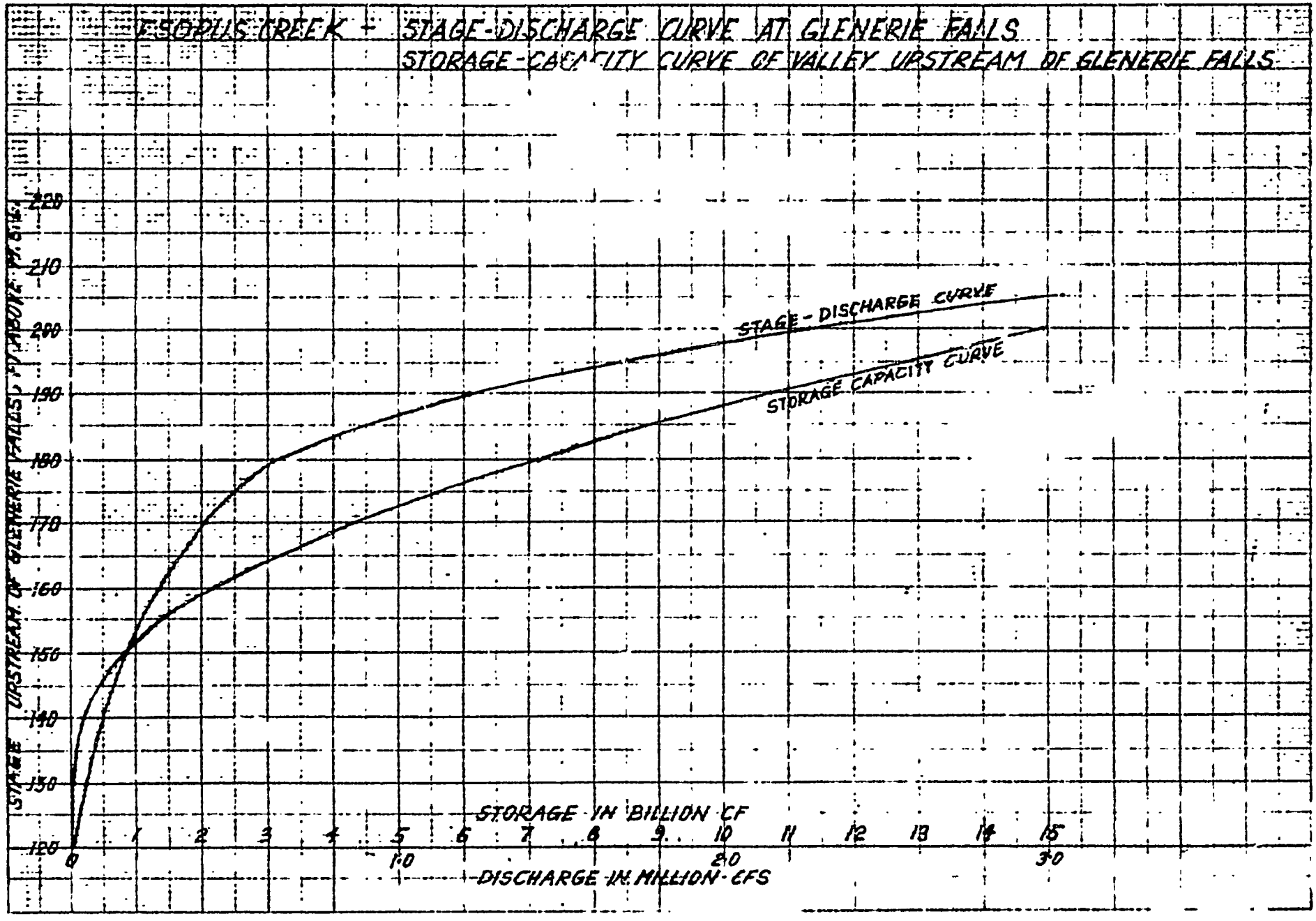

ESOPUS CREEK - STAGE-DISCHARGE CURVE AT GLENERIE FALLS
STORAGE-CAPACITY CURVE OF VALLEY UPSTREAM OF GLENERIE FALLS
STAGE - DISCHARGE CURVE
STORAGE CAPACITY CURVE
STAGE UPSTREAM OF GLENERIE FALLS IN ABOVE M.S.L.
220
210
200
190
180
170
160
150
140
130
120
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
STORAGE IN BILLION CF
1.0
2.0
3.0
DISCHARGE IN MILLION CFS

FIGURE IV-18

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

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

iii. 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 Flook.

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

PROGRAM FOR COMPUTATION OF OUTFLOW FROM ESOPUS CREEK AFTER ASHOKAN DAM FAILURE INCLUDING ASHOKAN RESERVOIR FLOOD ROUTING AND EFFECT OF LAKE UPSTREAM OF GLENERIE FALLS

```
ESASH1     15:45   RDS3  MAY 09,1970

50 DIM G(2,200),Q(2,200),P(12)
55 DIM O(200),W(100),D(200),S(200),H(200)
56 DIM M(200)
57 DIM V(290)
100 READ H(0)
105 READ Z(0)
110 READ T
390 FOR Y=1 TO2
400 READ K
410 FOR J=1 TO K
420 READ G(Y,J)
430 NEXT J
440 READ B
450 FOR I=1 TO 12
460 READ P(I)
470 NEXT I
480 FOR I=1 TO 12
490 FOR J=1 TO K
500 LET N=J+(I-1)*3
510 LET Q(Y,N)=(G(Y,J)*P(I))+Q(Y,N)
520 NEXT J
530 NEXT I
540 FOR N=0 TO 200
550 LET Q(Y,N)=(Q(Y,N)*(10↑3))+B
560 NEXT N
570 NEXT Y
575 GOTO 1045
600 LET A(0)=0
610 LET C(0)=0
620 FOR N=1 TO 120
630 LET X9=0
635 LET R=0
640 LET D1=C(N-1)
650 LET S=(((D1+C(N-1))+((Q(1,N-1)+Q(1,N))/10))/2)*T
660 LET V2=(S/((3.03/1.3)*(10↑8)))+H(N-1)-H(0)
670 LET D2=4.0*900.0*(V2↑1.5)
680 LET O2=(D2+A(N-1))*T
690 LET S2=S-O2
700 LET Y2=(S2/((3.03/1.3)*(10↑8)))+H(N-1)-H(0)
710 IF ABS(V2-Y2)<0.02 GOTO 735
720 LET V2=(V2+Y2)/2
730 GOTO 670
735 IF R=1 GOTO 760
740 LET S3=((Q(1,N-1)+Q(1,N))/2)*T*0.9
750 LET V1=(S3/(1.4*(10↑8)))+Z(N-1)-Z(0)
```

```
760 LET X=V2-3
770 IF X<0 GO TO 790
780 GOTO 795
790 LET X=0
795 IF V1>X GOTO 800
796 LET V1=V1+0.1
797 GOTO 795
800 LET D1=4.0*1100.0*(V1↑1.5)*(1-((X/V1)↑1.5))
810 LET O1=((D1+C(N-1))/2)*T
820 LET S1=S3-O1
830 LET Y1=(S1/(1.4*(10↑8)))+Z(N-1)-Z(0)
840 IF ABS(V1-Y1)<0.02 GOTO 870
850 LET V1=(V1+Y1)/2
860 GOTO 795
870 IF ABS (V1-X9)<0.05 GOTO 900
875 LET R=1
880 LET X9=V1
890 GOTO 650
900 LET A(N)=D2
910 LET H(N)=V2+H(0)
915 PRINT N;";";H(N),
920 LET C(N)=D1
930 LET Z(N)=V1+Z(0)
940 NEXT N
950 FOR N=1 TO 120
960 LET A(N)=A(N)/(10↑6)
970 LET Q(1,N)=Q(1,N)/(10↑3)
971 NEXT N
972 PRINT
973 PRINT
975 PRINT " ASHOKAN RESERVOIR  -  PMF"
980 PRINT
990 PRINT
992 PRINT"TIME   INFLOW    ELEVATION      ELEVATION      OUTFLOW"
993 PRINT"                   WEST           EAST           "
1010 PRINT"HR'S  1000CFS    FT M.S.L.      FT M.S.L.      1000CFS"
1020 PRINT
1030 FOR N=1 TO100
1040 PRINT 2*N,Q(1,N),Z(N),H(N),A(N)
1041 NEXT N
1045 FOR Z=0 TO 70
1050 READ O(Z),W(Z)
1052 LET O(Z)=O(Z)*(10↑3)
1053 LET W(Z)=W(Z)*(10↑9)
1055 NEXT Z
1060 FOR N=20 TO 100
1065 LET M(N)=Q(1,N)+((Q(2,N))/2)
```

```
1070 NEXT N
1072 LET T=7200.0
1075 LET D(20)=620E3
1080 LET S(20)=7.15E9
1082 LET O1=620E3
1085 LET H(20)=160
1090 FOR N=21 TO 100
1095 LET V(N)=(M(N)+M(N-1))/2
1095 LET D(N)=D(N-1)
1100 LET S(N)=(V(N)-D(N))*T+S(N-1)
1103 IF S(N)<0 GOTO 1160
1105 FOR Z=1 TO 70
1110 LET K=(S(N)-W(Z-1))/(W(Z)-W(Z-1))
1115 LET D(N)=(K*(O(Z)-O(Z-1)))+O(Z-1)
1120 LET H(N)=((K+Z-1)*0.1)+120
1125 IF W(Z)>S(N) GOTO 1135
1130 NEXT Z
1135 IF ABS(D(N)-O1)<3000 GOTO 1150
1140 LET O1=D(N)
1145 GOTO1100
1150 NEXT N
1160 FOR N=20 TO 100
1170 LET D(N)=D(N)/10↑3
1180 NEXT N
1185 FOR N=1 TO 19
1190 READ Q(1,N)
1200 NEXT N
1210 FOR N=1 TO 19
1220 LET Q(2,N)=Q(2,N)/(10↑3)
1230 NEXT N
1250 PRINT "ESOPUS CREEK  -  OUTFLOW"
1260 PRINT
1270 PRINT"CONDITIONS : DAM FAILURE , PMP ABOVE ASHOKAN"
1280 PRINT
1290 PRINT "HR'S        1000CFS"
1300 PRINT
1310 FORN=1 TO 19
1320 PRINT 2*N,Q(1,N)+Q(2,N)
1330 NEXT N
1340 FOR N=20 TO 70
1350 PRINT 2*N,D(N)
1360 NEXT N
1370 PRINT
1380 PRINT " 0 OF PIME IS BMGINNING OF THE STORM"
```

COMPUTATION OF OUTFLOW FROM ESOPUS CREEK AFTER ASHOKAN DAM FAILURE INCLUDING ASHOKAN RESERVOIR FLOOD ROUTING AND EFFECT OF LAKE UPSTREAM OF GLENERIE FALLS

DATA

```
2090 DATA 587.0,590.0
2010 DATA 7200.0   * T
2020 DATA 22
2030 DATA 0.35,1.15,2.50,4.50
2040 DATA 7.30,10.72,10.72,9.82
2050 DATA 8.52,6.90,5.60,4.50
2060 DATA 3.55,2.80,2.20,1.60
2070 DATA 1.10,0.75,0.50,0.40
2080 DATA 0.13,0.00
2180 DATA 260 BASE FLOW
2190 DATA 0.00,0.00,0.45,0.20
2200 DATA 1.30,16.65,1.85,0.70
2210 DATA 0.00,0.05,0.00,0.00
2300 DATA 24.0
2310 DATA 0.20,0.40,0.60,0.86
2320 DATA 1.25,1.80,2.70,4.62
2330 DATA 3.35,6.25,5.60,4.75
2340 DATA 4.00,3.35,2.74,2.25
2350 DATA 1.82,1.44,1.11,0.83
2360 DATA 0.59,0.39,0.20,0.00
2380 DATA 170.0 BASE FLOW
2390 DATA 0.00,0.60,0.67,0.29
2400 DATA 1.60,6.43,1.92,1.30
2410 DATA 0.10,0.17,0.22,0.11
8490 DATA 0.00,0.00
```

UNIT HYDROGRAPH DATA (SUBBASIN 20)

RUNOFF DATA (SUBBASIN 20)

U'H DATA (SUBBASIN 20)

RUNOFF DATA (SUBBASIN 20)

```
2500DATA 5,0.005,10,0.010
2510 DATA 15,0.015,20,0.020
2520 DATA 25,0.025,30,0.030
2530 DATA 35,0.035,40,0.040
2540 DATA 05,0.045,50,0.550
2550 DATA 55,0.065,60,0.080
2560 DATA 65,0.095,70,0.110
2570 DATA 75,0.125,80,0.140
2580 DATA 85,0.155,90,0.170
2590 DATA 95,0.185,100,0.200
2600 DATA 107,0.265,114,0.330
2610 DATA 121.0,0.395,128,0.460
2620 DATA 135,0.525,142,0.590
2630 DATA 149,0.655,156,0.720
2640 DATA 163,0.785,170,0.850
2650 DATA 180,0.980,189,1.11
2660 DATA 200,1.24,210,1.37
2670 DATA 220,1.50,230,1.63
2680 DATA 240,1.76,250,1.89
2690 DATA 260,2.02,270,2.15
2700 DATA 284,2.37,298,2.59
2710 DATA 312,2.79,326,3.01
2720 DATA 340,3.23,354,3.45
2730 DATA 368,3.67,382,3.89
2740 DATA 396,4.11,410,4.35
2750 DATA 431,4.63,452,4.91
2760 DATA 473,5.19,494,5.47
2770 DATA 515,5.75,536,6.03
2780 DATA 557,6.31,578,6.59
2790 DATA 599,6.87,620,7.15
2800 DATA 680,7.51,740,7.87
2810 DATA 800,8.23,860,8.59
2820 DATA 920,8.95,980,9.31
2830 DATA 1040,9.67,1100,10.03
2840 DATA 1160,10.39,1220,10.75
3000 DATA 0.00,0.00,0.00,0.00
3010 DATA 0.00,0.00,0.00,0.00
3020 DATA 0.00,0.00,0.08,0.18
3030 DATA 0.38,0.66,1.05,1.68
3040 DATA 2.89,5.63,11.59,23.25
9000 END
```

STAGE DISCHARGE CURVE AT GLENERIE FALLS
+
STORAGE CAPACITY OF THE LAKE UPSTREAM
OF GLENERIE FALLS

COMPUTATION OF OUTFLOW FROM ESOPUS CREEK AFTER ASHOKAN DAM FAILURE INCLUDING ASHOKAN RESERVOIR FLOOD ROUTING AND EFFECT OF LAKE UPSTREAM OF GLENERIE FALLS

PRINTOUT

ESOPUS CREEK - OUTFLOW

CONDITIONS : DAM FAILURE , PMP ABOVE ASHOKAN

HP'S	1000CFS
2	.17
4	.17
6	.17
8	.29
10	.41
12	.53
14	.82
16	1.1[illegible]
18	1.652
20	2.4242
22	3.9755
24	3.74
26	6.6784
28	8.2879
30	7.7965
32	11.[illegible]
34	14.7338
36	18.336
38	27.[illegible]363
40	6[illegible]0
42	466.276
44	381.807
46	328.765
48	286.062
50	[illegible]51.[illegible]57
52	219.326
54	189.728
56	163.059
58	107.[illegible]
60	86.3253
62	71.9643
64	56.2931
66	43.2545
68	33.3125
70	26.7467
72	17.575
74	12.535
76	9.9065
78	6.4418
80	5.7263
82	4.8047
84	3.4213
86	3.078
88	2.4904
90	1.6966

Hr's	1000 cfs
92	1.6177
94	1.38
96	1.155
98	.988
100	.8468
102	.7182
104	.637
106	.5653
108	.4949
110	.4729
112	.452
114	.43
116	.43
118	.43
120	.43
122	.43
124	.43
126	.43
128	.43
130	.43
132	.43
134	.43
136	.43
	.43
	.43
142	.43
144	.43
146	.43
148	.43
150	.43
152	.43
154	.43
156	.43
158	.43
160	.43
162	.43
164	.43
166	.43
168	.43
170	.43
172	.43
174	.43
176	.43
178	.43
180	.43
182	.43
184	.43
186	.43
188	.43
190	.43
192	.43
194	.43
196	.43
198	

essentially the same as the Ashokan west basin capacity.

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

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.

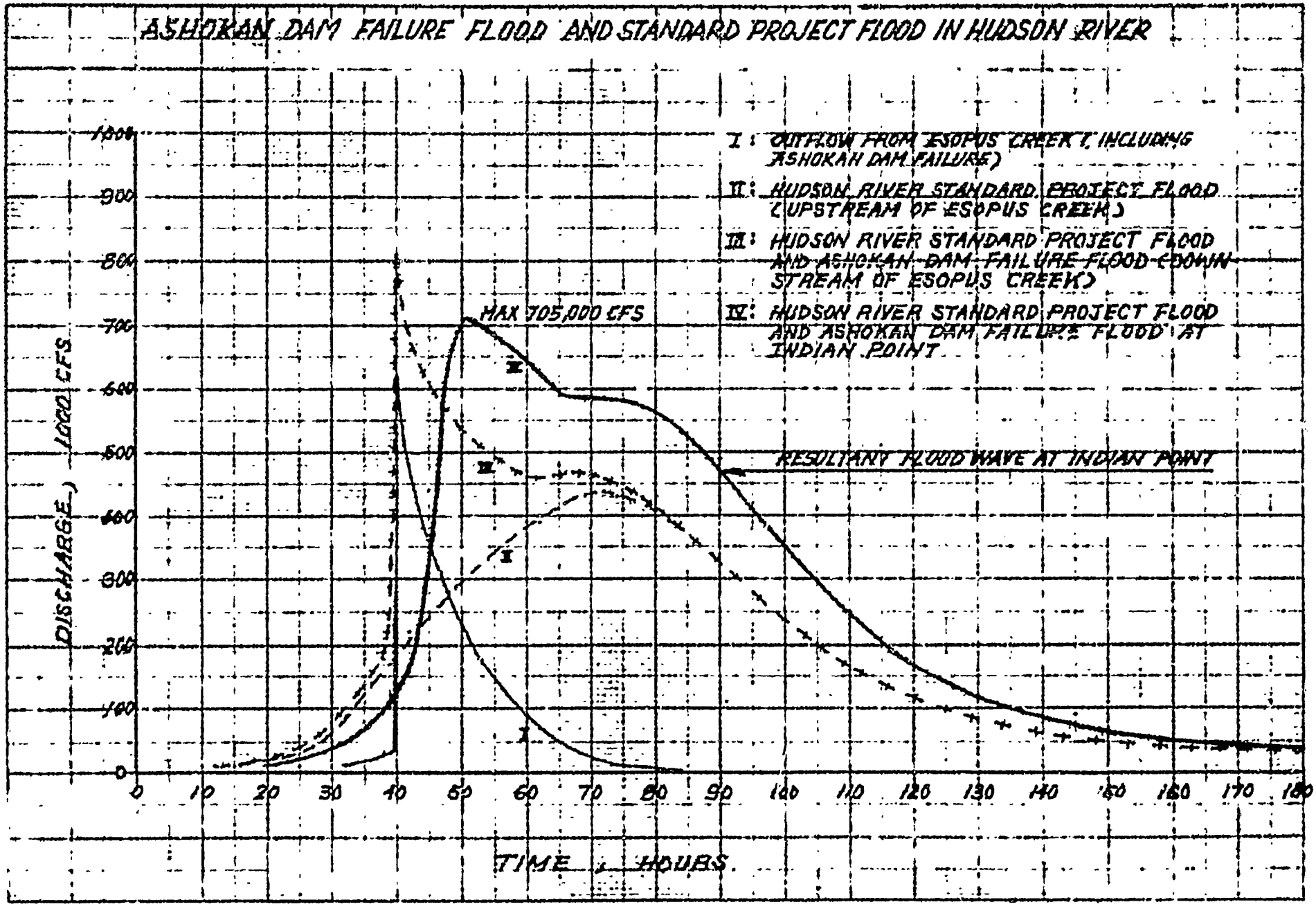

ASHOKAN DAM FAILURE FLOOD AND STANDARD PROJECT FLOOD IN HUDSON RIVER
I : OUTFLOW FROM ESOPUS CREEK (INCLUDING ASHOKAN DAM FAILURE)
II: HUDSON RIVER STANDARD PROJECT FLOOD (UPSTREAM OF ESOPUS CREEK)
III: HUDSON RIVER STANDARD PROJECT FLOOD AND ASHOKAN DAM FAILURE FLOOD (DOWN STREAM OF ESOPUS CREEK)
IV: HUDSON RIVER STANDARD PROJECT FLOOD AND ASHOKAN DAM FAILURE FLOOD AT INDIAN POINT
MAX 705,000 CFS
RESULTANT FLOOD WAVE AT INDIAN POINT
DISCHARGE, 1000 CFS
1000
900
800
700
600
500
400
300
200
100
0
0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180
TIME, HOURS.

FIGURE IV-19

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.

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.

PLATE IV-13
(SHEET 1 OF 4)

LOWER HUDSON RIVER
PROGRAM FOR FLOOD HYDROGRAPH
GENERATION
CONDITIONS:
- SPF IN HUDSON RIVER
- ASHOKAN DAM FAILURE

```
LIST

HDSN1      17:28   HDS3  MAY 09,1970

 13
 50 DIM G(8,200),A(150),M(150),H(150),E(150)
 55 DIM W(200),L(200),U(200)
 212 PRINT
 220 FOR Y=1 TO 8
 230 READ S
 240 READ K
 250 FOR J=1 TO K
 260 READ G(Y,J)
 270 NEXT J
 305 FOR J=1 TO K
 320 NEXT J
 330 NEXT Y
 790 DIM P(12),Q(8,200),O(200),A(200)
 800 FOR Y=1 TO 8
 810 READ S
 820 READ B
 830 FOR I=1 TO 12
 840 READ P(I)
 850 NEXT I
 880 FOR I=1 TO 12
 890 FOR J=1 TO 150
 900 LET N=J+((I-1)*3)
 910 LET Q(Y,N)=(G(Y,J)*P(I))+Q(Y,N)
 920 NEXT J
 930 NEXT I
 940 FOR N=1 TO 200
 950 LET Q(Y,N)=Q(Y,N)+B
 960 NEXT N
 1050 FOR  N=1 TO 50
 1070 NEXT N
 1100 PRINT
 1110 NEXT Y
 1150 READ K1
 1160 FOR N=1 TO K1
 1170 READ M(N),H(N
 1180 NEXT N
 1190 FOR N=1 TO K1
 1200 LET A(N)=M(N)+H(N)
 1210 NEXT N
 1212 LET F9=13.49
 1213 GOSUB 9000
 1214 FOR N=1 TO K1
 1215 LET U(N)=L(N)
 1216 NEXT N
```

```
1220 ONINP
1225 PRINT " HUDSON RIVER AT GREEN ISLAND  - MMF"
1230 PRINT
1235PRINT "HR'S          1000CFS"
1240 PRINT
1245 FOR N=1 TO 70
1250 PRINT 2*N,A(N)
1255 NEXT N
1256 PRINT
1260 LET N5=3
1270 GOSUB 8000
1280 FOR N=1 TO K1
1290 LET A(N)=A(N)+U(1,N)
1300 NEXT N
1310 LET N5=1
1320 GOSUB 8000
1330 FOR N=1 TO K1
1340 LET A(N)=A(N)+U(2,N)+U(3,N)+U(4,N)
1350 NEXT N
1352 LET N5=0
1353 GOSUB 8000
1354 FOR N=1 TO K1
1355 LET A(N)=(A(N)+U(5,N))/2
1356 NEXT N
1360 PRINT
1365 PRINT
1370 PRINT"HUDSON RIVER ABOVE ESOPUS CREEK  -  SPF"
1380 PRINT
1385 PRINT "HR'S          1000CFS"
1390 PRINT
1395 FOR N=1 TO 90
1400 PRINT 2*N,A(N)
1410 NEXT N
1420 PRINT
1430 FOR N=1 TO 56
1440 READ E(N)
1450 NEXT N
1452 FOR N=57 TO 120
1453 LET E(N)=0.43
1454 NEXT N
1460 FOR N=1 TO 120
1470 LET A(N)=A(N)+E(N)
1480 NEXT N
1490 PRINT
1500 PRIT "HUDSON RIVER BELOW ESOPUS CREEK  -  SPF"
1510 PRINT
```

```
1520 PRINT" R'S          1000CFS"
1530 PRINT
1540 FOR N=1 TO 80
1550 PRINT N*2,A(N)
1560 NEXT N
1570 PRINT
1580 PRINT
1620 LET N5=1
1630 GOSUB 6000
1640  FOR N=1 TO 120
1650 LET A(N)=A(N)+((U(6,N)+U(7,N))/2)
1660 NEXT N
1661 PRINT "HUDSON RIVER BELOW RONDOUT CREEK  -  SRF"
1662 PRINT
1663 PRINT "HR'S       1000 CFS"
1664 PRINT
1665 FOR N=1 TO 90
1666 PRINT 2*N,A(N)
1667 NEXT N
1668 PRINT
1669 PRINT
1670 LET N5=3
1680 GOSUB 6000
1690 FOR N=1 TO 120
1700 LET A(N)=A(N)+ U(6,N)/2)
1710 NEXT N
1712 LET F9=14.01
1713 GOSUB 9000
1720 PRINT
1730 PRINT
1740 PRINT
1750 PRINT " HUDSON RIVER AT INDIAN POINT"
1760 PRINT " SRF + ASHOKAN RELEASE"
1770 PRINT"------------------------------"
1780 PRINT
1790 PRINT" HR'S          1000CFC"
1800 PRINT
1810 FOR N=20 TO 100
1820 PRINT 2*N,A(N)
1830 NEXT N
1840 PRINT
1850 PRINT
1860 PRINT " 0 OF TIME I BEGINNINGOF PRE"0
1870 PRONT
1875 PRINT
1880 PRINT" S-HYDROGRAPH AT GREEN ISLAND AND AT INDIAN POINT"
1900 PRINT
1919PRINT"TIME            G.I.          I.P."
1920 PRINT"HR'S          1000CFS       1000CFC"
1930 PRINT
1935 FOR N=1TO 35
1940 PRINT 2*N,U(N),L(N)
1950 NEXT N
1960 PRINT
1970 GOTO 90000
```

```
8000 LET K=1
8005 IF N5=0 GOTO8120
8010 LET C1=1/1.2
8020 LET C2=1/6
8030 LET O(1)=A(1)
8035 FOR N=2 TO110-K
8040 LET O(N)=C1*(A(N-1)-O(N-1))+C2*(A(N)-A(N-1))O(N-1)
8050 NEXT N
8060 FOR N=1 TO 110-K
8070 LET A(N)=O(N)
8080 NEXT N
8090 LET K=K+1
8100 IF K>N5 GOTO 8120
8110 GOTO 8030
8120 RETURN TO MAIN LINE
9000 FOR N=1 TO K1
9010 LET W(N)=(A(N)-A(1))/F9
9020 NEXTN
9030 FOR N=1 TO 36
9040 LET L(N)=W(N)
9050 NEXT N
9055 FOR N=37 TO 72
9060 LET L(N)=W(N)+L(N-36)
9070 NEXT N
9080 FOR N=73 TO K1
9090 LET L(N)=W()+L(N-36)
9100 NEXT N
9110 RETURN TOMAIN LINE
90000 END
```

PLATE IV-14
(SHEET 1 OF 1)

LOWER HUDSON RIVER

PROGRAM FOR FLOOD HYDROGRAPHS GENERATION

CONDITIONS:

- SPF IN HUDSON RIVER
- ASHOKAN DAM FAILURE

DATA

DATA IN LINES 3000 THROUGH 3500 SEE PLATE III-11.

```
7010 DATA 0.17,017,0.17,0.29
7020 DATA 0.41,0.53,0.82,1.18
7030 DATA 1.65,2.42,3.98,3.74
7040 DATA 6.68,8.29,7.80,11.88
7050 DATA 14.73,18.34,27.84,620.00
7060 DATA 466.28,381.81,328.77,286.06
7070 DATA 251.24,219.33,189.73,163.06
7080 DATA 107.63,86.33,71.96,56.29
7090 DATA 43.25,33.31,26.75,17.58
7100 DATA 12.54,9.91,6.44,5.73
7110 DATA 4.81,3.42,3.08,2.49
7120 DATA 1.90,1.62,1.38,1.16
7130 DATA 0.99,0.85,0.72,0.64
7140 DATA 0.56,0.49,0.47,0.45
7150 DATA 0.95,0.90,0.86,0.82
7160 DATA 0.82,0.81,0.80,0.80
7170 DATA 0.79,0.79,0.78,0.78
7180 DATA 0.77,0.76,0.76,0.75
7190 DATA 0.74,0.75,0.73,0.73
7200 DATA 0.72,0.71,0.70,0.70
7210 DATA 0.70,0.70,0.69,0.69
7220 DATA0.68,0.68,0.68,0.67
7230 DATA 0.67,0.67,0.66,0.66
7240 DATA 0.66,0.66,0.66,0.65
7250 DATA 0.65,0.65,0.65,0.65
```

PLATE IV-15
(SHEET 1 OF 7)

LOWER HUDSON RIVER

PROGRAM FOR FLOOD HYDROGRAPHS GENERATION

CONDITIONS:
- SPF IN HUDSON RIVER
- ASHOKAN DAM FAILURE

PRINTOUT

HUDSON RIVER ABOVE ESOPUS CREEK - SPF

HR'S	1000CFS
2	4.09
4	4.09
6	4.09
8	6.91632
10	7.95157
12	9.95767
14	13.1572
16	17.421
18	21.7011
20	25.0076
22	29.6262
24	33.5536
26	40.921
28	48.1192
30	55.9519
32	88.3311
34	110.236
36	137.604
38	152.867
40	189.74
42	218.759
44	235.67
46	258.699
48	281.797
50	296.562
52	324.649
54	331.703
56	339.965
58	358.405
60	374.156
62	389.202
64	407.351
66	420.418
68	431.815
70	436.316
72	437.871
74	434.432
76	427.992
78	418.701
80	406.882

HUDSON RIVER UPSTREAM OF ESOPUS CREEK - SPF (CONTINUATION)

82	393.035
84	377.089
86	360.615
88	343.977
90	326.076
92	308.225
94	290.499
96	272.705
98	255.929
100	239.602
102	223.69
104	208.336
106	193.752
108	179.912
110	166.657
112	154.383
114	142.651
116	132.207
118	122.785
120	114.225
122	106.473
124	99.55
126	93.2833
128	87.6708
130	82.6847
132	78.2345
134	74.2578
136	70.734
138	67.6052
140	64.772
142	62.2028
144	59.8371
146	57.6102
148	55.5256
150	53.5682
152	51.7459
154	49.9963
156	48.2145
158	46.5568
160	45.0264
162	43.6057
164	42.2903
166	41.0426
168	39.8634
170	38.7667
172	37.7401
174	36.8027
176	35.9215
178	35.0803
180	34.2389

HUDSON RIVER BELOW ESOPUS CREEK - SPF+ ASHOKAN DAM FAILURE FLOOD

HR'S	1000CFS
2	4.26
4	4.26
6	4.26
8	7.20632
10	8.36157
12	10.4877
14	13.9772
16	18.601
18	23.3511
20	27.4276
22	33.6062
24	37.2936
26	47.601
28	56.4092
30	63.7519
32	100.211
34	124.968
36	155.964
38	180.707
40	809.74
42	685.039
44	617.48
46	587.469
48	567.857
50	547.802
52	543.979
54	521.433
56	503.025
58	466.035
60	460.486
62	461.162
64	463.641
66	463.668
68	465.125
70	463.066
72	455.451
74	446.972
76	437.902
78	425.141
80	412.592
82	397.845
84	380.509
86	363.695
88	346.467
90	327.976
92	309.845
94	291.879
96	273.865
98	256.919
100	240.452
102	224.41
104	208.976
106	194.312
108	180.402
110	167.127
112	154.833
114	143.281
116	132.637

118	123.215
120	114.655
122	106.903
124	99.96
126	93.7133
128	88.1006
130	83.1147
132	78.8645
134	74.6676
136	71.164
138	68.0352
140	65.202
142	62.6326
144	60.2671
146	58.0402
148	55.9556
150	53.9982
152	52.1759
154	50.4263
156	48.6445
158	46.9388
160	45.4564

HUDSON RIVER BELOW HORDOS CREEK - SPF AND ASHOKAN DAM FAILURE FLOOD

HR'S	1000 CFS
2	4.86
4	4.56
6	4.66
8	5.6033
10	8.13615
12	10.0033
14	13.3369
16	16.0705
18	24.1977
20	31.275
22	37.7009
24	44.3603
26	52.5505
28	63.5063
30	73.4651
32	89.6901
34	124.202
36	156.029
38	193.907
40	388.365
42	756.795
44	743.92
46	696.645
48	667.744
50	649.127
52	632.052
54	621.799
56	600.941
58	575.823
60	542.186
62	529.003
64	523.406
66	520.219

68	516.329
70	512.616
72	505.953
74	495.407
76	483.455
78	470.533
80	455.226
82	439.245
84	421.677
86	402.359
88	383.06
90	363.542
92	343.234
94	323.284
96	303.712
98	284.355
100	266.044
102	248.98
104	231.24
106	214.933
108	199.436
110	184.728
112	170.849
114	157.951
116	145.927
118	134.902
120	125.015
122	116.223
124	108.279
126	101.156
128	94.7876
130	89.0824
132	83.9973
134	79.5018
136	75.4945
138	71.9361
140	68.7681
142	65.9086
144	63.3119
146	60.923
148	58.6845
150	56.5876
152	54.6291
154	52.7945
156	51.0237
158	49.2651
160	47.6131
162	46.0604
164	44.6572
166	43.3352
168	42.0665
170	40.9095
172	39.811
174	38.7874
176	37.8449
178	36.9602
180	36.1117

HUDSON RIVER AT INDIAN POINT
SPF + ASHOKAN DAM FAILURE FLOOD

HR'S	1000CFS
20	14.7532
22	19.545
24	25.5014
26	32.1377
28	39.0486
30	46.5658
32	55.3647
34	65.9554
36	79.406
38	100.133
40	129.926
42	171.468
44	255.629
46	429.631
48	632.541
50	704.983
52	705.31
54	687.497
56	670.132
58	655.95
60	643.337
62	626.077
64	607.21
66	590.654
68	582.788
70	583.271
72	583.088
74	581.376
76	578.507
78	571.744
80	561.229
82	547.606
84	529.304
86	509.531
88	488.683
90	464.989
92	441.116
94	418.219
96	394.56
98	371.351
100	349.259
102	327.212
104	305.943
106	285.965
108	266.326
110	247.919

112	230.543
114	213.696
116	197.985
118	183.247
120	169.206
122	156.403
124	144.692
126	134.038
128	124.346
130	115.884
132	108.1
134	101.156
136	94.8097
138	89.2325
140	84.2012
142	79.7125
144	75.7011
146	72.0585
148	68.7686
150	65.79
152	62.9876
154	60.4525
156	58.0645
158	55.8605
160	53.788
162	51.8879
164	50.0624
166	48.3434
168	46.7606
170	45.2947
172	43.9253
174	42.6615
176	41.473
178	40.3634
180	39.3303
182	38.3679
184	37.4631
186	36.5951
188	35.7503
190	34.9289
192	34.1345
194	33.3582
196	32.5989
198	31.8632
200	31.15

0 OF TIME IS BEGINNING OF PMP

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 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-

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 an integral part of the system and its influence was simultaneously coupled with the other relevant 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.

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.

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.

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

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:

$$h_f = \bar{S}_f \, \Delta x = \tfrac{1}{2}(S_i + S_{i+1}) \, \Delta x \qquad \text{(V-2)}$$

where

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

Δx = Longitudinal distance between the two end sections.

Manning's formula was used to compute the friction slope

$$S_f = \left(\frac{n\bar{v}}{1.49}\right)^2 \frac{1}{R_l^{4/3}} \qquad \text{(V-3)}$$

where

n = Manning's roughness coefficient

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

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

in which

Z_i = Water depth below mean sea level

No advantage was taken of the increase in surface width

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

PLATE V-1
(SHEET 1 OF 1)

LEVEL 19:00 'RDS3 JUN 11 70 THU

BACK WATER COMPUTATION
STANDART STEP METHOD PROGRAM

```
100 DIM M(200),Z(200),B(200),Y(200),S(200),H(200)
105 DIM A(40)
120 READ Q
125 READ Y(1)
200 READ G,N5
210 LET K=2.815/(10↑4)
220 FOR I=1 TO N5+1
240 READ M(I),Z(I),B(I)
260 NEXT I
300 LET D=Z(1)+Y(1)
320 LET V=Q/(D*B(1))
330 LET H(1)=Y(1)+((V↑2)/(2*G))
350 LET S(1)=K*(V↑2)/(D↑(4.0/3.0))
1000 FOR I=1 TON5
1020 LET L=(M(I+1)-M(I))*5280.0
1040 LET W=S(I)*L
1060 LET Y(I+1)=Y(I)+W
1080 LET D=Z(I+1)+Y(I+1)
1120 LET V=Q/(D*B(I+1))
1140 LET H(I+1)=Y(I+1)+((V↑2)/(2*G))
1200 LET S(I+1)=K*(V↑2)/(D↑(4.0/3.0))
1220 LET S1=(S(I)+S(I+1))/2.0
1240 LET F=S1*L
1260 LET P=H(I+1)-H(I)
1280 LET R=F-P
1300 LET T=ABS(R)
1320 LET T= INT((T*(10↑2))+0.5)/10↑2
1340 IF T<=0.05 GO TO 2000
1360 LET Y(I+1)=Y(I+1)+R
1380 GOTO 1080
2000 NEXT I
2400 PRINT
2420 PRINT
2500 PRINT"  HUDSON RIVER"
2505 PRINT
2510 PRINT "BACK WATER PROFILE FROM M.P. 0.0  TO  M.P.  153.7"
2520 PRINT
2560 PRINT
2580 PRINT
2600 PRINT "Q";"=";Q;"CFS"
2620 PRINT
2640 PRINT
2650 PRINT"WATER SURFACE ELEVATION AT M.P. 0.0 IS";Y(1);"FT M.S.L."
2655 PRINT
2656 PRINT
2660 PRINT" M.P.     WATER LEVEL"
2670 PRINT"           ELEVATION"
2680 PRINT
2900 FOR I=1 TO 35
2950 LET Y(I)=INT((Y(I)*10↑2)+0.5)/(10↑2)
2960 PRINT M(I),Y(I)
2970 NEXT I
3000 FOR I=36 TO N5+1 STEP 4
3020 LET Y(I)=INT((Y(I)*10↑2)+0.5)/(10↑2)
3040 PRINT M(I),Y(I)
3059 NEXT I
3070 GOTO 9000
3500 DATA 1100E3
3510 DATA 0.0
3520 DATA 32.2,135
```

I-7

RUN

PLATE V-2
(SHEET 1 OF 3)

BACK WATER COMPUTATION

BASIC DATA OF HUDSON RIVER CHANNEL

LEVEL 17:20 RD53 MAY 18 70 MON

M(I) (MILES)	Z(I) MEAN DEPTH BELOW M.S.L. (FT)	B(I) SURFACE WIDTH (FT)
0	36	6700
2	37.7	6000
4	37.8	4920
6	34.6	4340
8	30.6	4870
10	29.6	5240
12	26.4	5000
14	31.7	5830
16	30.1	5160
18	29.9	5160
20	30	5000
21	35.5	5680
22.7	30.5	6520
24.9	29.5	7330
27.3	15.5	13280
29.7	17.4	13130
32.2	19.4	10610
33.8	28.4	6980
35	15.4	14880
36.9	18.4	15760
38.8	27.4	7600
40.1	35.4	5620
41.4	45.4	4040
43.3	40.4	5020
44.4	57.4	2900
46.5	72.4	2480
49.3	84.4	1500
51.6	56.4	2200
52.9	103.4	1660
53.9	49.4	3780
55	61.4	2080
56.2	48.4	2660
57.5	19.4	8550
58.1	20.4	7080
61.3	34.4	4360
63.6	30	5060
66.2	45	3360
67.7	60	2250
69	36	4500
70.3	54	2490
70.9	57	2030
71.1	52	2380
71.6	44	2760
72	46	2750
72.5	47	2710
72.9	46	2660
73.3	50	2490
73.6	54	2270
74	54	2410
74.3	62	1910
74.6	50	2460
74.9	51	2450
75.4	45	2920

M.C.	Z(I)	B(I)
75.9	51	2480
76.4	51	2490
76.7	52	2400
77.2	52	2410
77.6	49	2590
78	50	2490
78.4	49	2490
78.9	47	2620
79.4	51	2220
79.8	62	1850
80.2	73	1680
80.6	64	1770
81	60	1920
81.3	50	2280
81.6	44	2500
82	42	2880
82.3	44	2810
82.7	42	2850
83.2	46	2500
83.5	47	2470
84.1	44	2550
84.4	37	3360
84.8	32	4010
85.4	36	3330
85.8	43	2680
86.3	42	2720
86.9	35	2680
87.4	25	4430
87.8	21	6160
88.5	19	5500
89.2	25	4360
89.7	25	4270
90.1	34	3650
90.4	31	3850
90.7	31	3480
91.2	22	4850
91.8	24	4265
93.2	21	6050
95	28	3960
96	24	4300
97.1	27	3700
98.5	28	5320
100.2	21	4400
101	22	3700
102.5	33	1900
104	19	3900
105.4	21	3390
107.6	19	3750
109.1	17	6210
110.5	16	3820
112.4	28	1750
113.8	12	3230
114.9	20	1650
116.3	11.7	4100
117.4	17.3	2990
118.9	15.2	3100
120	11.2	3620
121	12.9	3670
122.3	21.1	2000
123.7	20	1320
125.9	14	2000

M.A	Z(I)	B(I)
126	17	1870
126.6	19.7	1920
127.8	15.6	1920
128.9	18.5	1570
129.8	18.5	2270
131.2	19.5	1580
132.4	13.5	2250
133.5	17.5	1480
134.8	18.5	1250
135.8	23.5	1040
137.3	17.5	1220
138.5	20.5	1200
139.9	22.4	1000
141.2	21.4	870
142.5	25.3	860
143.7	28.3	980
145	11.2	1010
146.4	17.2	1050
147.9	23.1	575
150.2	13.1	600
152.2	14	640
153.7	4.2	1060

USED: CPU 10.5 I/O 5.2

READY

Hudson River flow resistance coefficient. These estimates were derived from velocity measurements made by the U. S. Geological Survey at the Poughkeepsie gaging station. 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$^{\frac{1}{2}}$/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

value Q equivalent to the peak of the flood hydrograph. This is a conservative assaumption 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. Y_{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.

$\Delta H = H_{i+1} - H_i$

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 [illegible] 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."

The method is based on the following two equations:

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

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

a previous calculation.

b. Assume $D_{i+1} = D'_i$ 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} from equation V-6 using the approximate 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.

PLATE V-3
(SHEET 1 OF 2)

BACK WATER COMPUTATION

PRASAD'S METHOD PROGRAM

```
MAIN      10:43   MON 14, 1970

1000        REAL N,M
1010        INTEGER OF1,K,Z
1020        DIMENSION Y(50)
1030        DIMENSION H(50)
1040       CALL OPENF(1,"TEM")
1050        IF1=1
1060        OF1=66
1070        J=1
1080        WRITE(OF1,5)
1090      5 FORMAT(//,1H ,27HINPUT Y(1),S1,A1,B,P,K,N,A1,M1)
1100        READ(50, ) Y(1),S1,X1,B,P,K,N,A1,B1
1110    220 READ(IF1, ) A2,A3,S2,B2
1120        H(1)=Y(1)-S1
1130        S0=(S1-S2)/((A2-A1)*5280.)
1135        B=(B1+B2)/2.
1140        WRITE(OF1,10)J
1150    10 FORMAT(////,1H ,30HNUMBER OF EACH = ,I5)
1160        WRITE(OF1,11)
1170     11 FORMAT(//,1H ,10HMILE POINT,3X,10H+ ON - MSL)
1180        WRITE(OF1,12)
1190     12 FORMAT(1H ,2X,5HMILES,7X,3HF(1))
1200        WRITE(OF1,13) A1,H(1)
1210     13 FORMAT(/,1H ,F10.1,3X,F10.2)
1220        M=(A2-A1)/A3
1230          M1=0
1240        AM1=M1
1250        IF((M-AM1)-0.0001) 620,620,50
1260     50 A4=A2-(A1+3*M1)
1270        GO TO 660
1280    620 A4=0.
1290    660 I=1
1300        GO TO 1000
1310    710 CONTINUE
1320        IF (I-M1) 740,760,760
1330    740 I=I+1
1340        GO TO 1000
1350    760 IF ((A2-A1)-0.001) 860,860,800
1360    800 A3=A4
1370        I=I+1
1380        GO TO 1000
1390    860 IF (J-N) 880,9999,9999
1400    880 J=J+1
1410        Y(1)=Y(I+1)
1420        X1=X2
```

```
1430          S1=S2
1435          B1=B2
1440          GO TO 220
1450   1000   A1=A1+A3
1460          S1=S1-A3*5280.*50
1470          A=I
1480          IS=1
1490          GO TO 5000
1500   1080   D4=D3
1510          D5=D4
1520          Z=1
1530   1140   I5=D5
1540          Y(I+1)=Y(I)+(D4+D5)*A3*5280.*0.5
1550          A=I+1
1560          IS=2
1570          GO TO 5000
1580   1220   D5=D3
1590          IF (ABS(I5-D5)-0.00001) 1320,1320,1260
1600   1260   IF (Z-15) 1280,1280,1320
1610   1280   Z=Z+1
1620          GO TO 1140
1630   1320   H(I+1)=Y(I+1)-S1
1640          WRITE(OF1,13) A1,H(I+1)
1650          GO TO 710
1660   5000   A1=S+Z.*Y(I)
1600          A=B*Y(I)
1680          D1=50-((N**2)*(B**2)*A1**(4./3.))/(2.22*A**(10./3.))
1690          D2=1.-(A1*(B**2)*B)/(32.2*A**3)
1700          D3=(D1/D2)**
1710          GO TO (1080,1220),IS
9000 9999 CALL EXIT
10000     END
```

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

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 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 cf 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.

TABLE V-1

COMPARISON BETWEEN HUDSON RIVER
TIDAL CHARACTERISTICS AT MOUTH AND HEAD
-1922 MONTHLY AVERAGE VALUES IN FEET-

Parameter	Mouth (Battery) referred to 5' below MSL	Head (Albany) referred to 2' below MSL
High Water		
Maximum	7.62	9.66
Minimum	6.86	4.57
Annual Average	7.36	6.09
April	7.27	9.66
Low Water		
Maximum	2.99	8.63
Minimum	2.38	1.81
Annual Average	2.77	3.53
April	2.72	8.68
Half-tide Level		
Maximum	5.3	9.14
Minimum	5.62	3.48
Annual Average	5.06	4.81
April	5.00	9.14

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

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 stage 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).

This 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 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 & GS tidal measurements at the Battery. A trial and error procedure was then employed to determine the length of influenced reach, amount of

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

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

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 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 conditions (Figure III-27) indicates that the duration of the peak discharge (PMF) is only several hours.

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 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.

RUN

LEVEL 19:17 RD53 JUN 11 78 THU

PLATE V-4
(SHEET 1 OF 1)

BACK WATER COMPUTATION
STANDARD STEP METHOD PROGRAM
PRINTOUT

CASES:
1 - PEAK AT MOUTH
2 - SLACK WATER PHASE
STAGE AT THE BATTERY 0.0 FT ABOVE M.S.L.
AVERAGE ENCROACHMENT INCREASE

HUDSON RIVER

BACK WATER PROFILE FROM M.P. 0.0 TO M.P. 153.7

Q= 1.1E 6 CFS

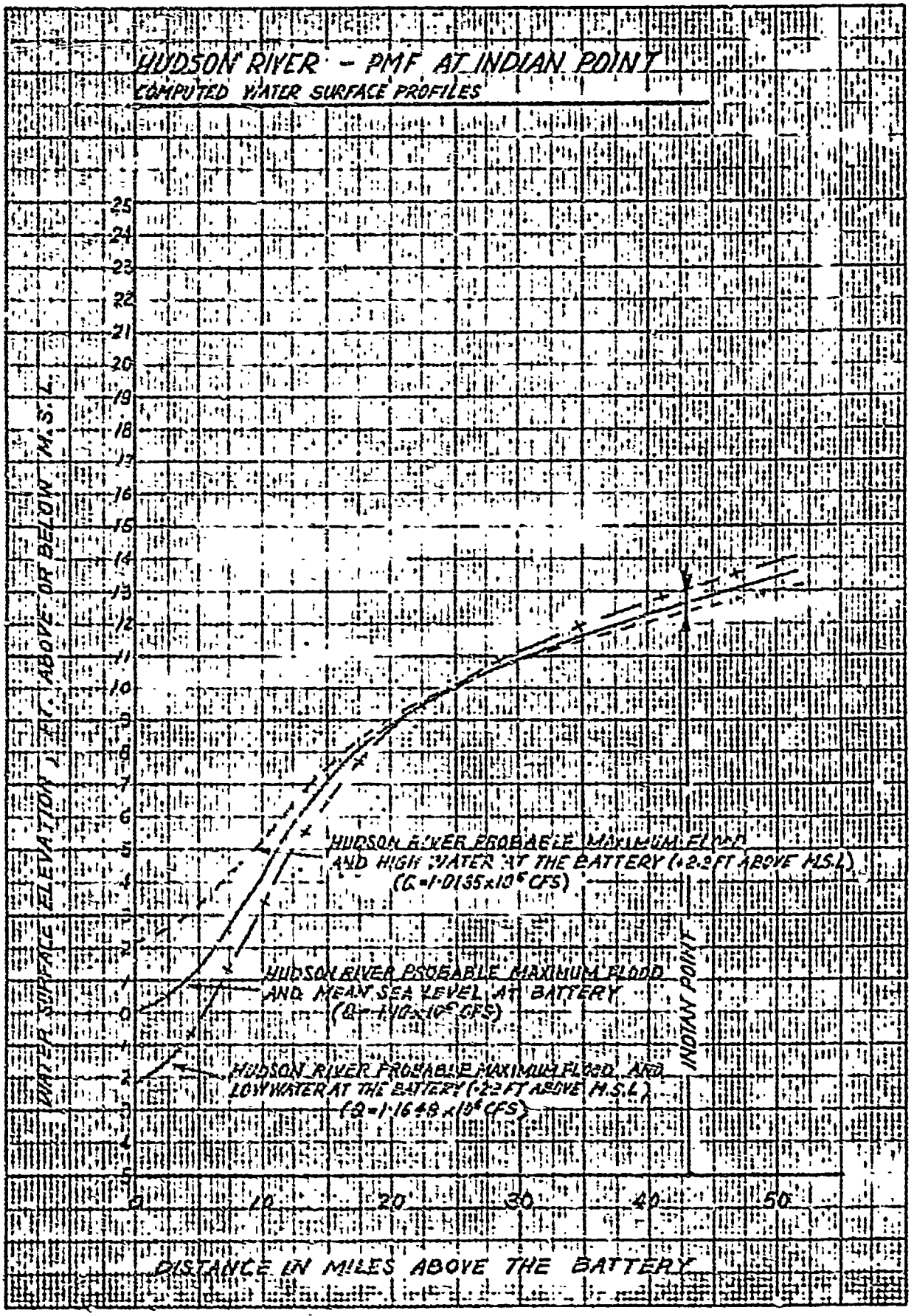
WATER SURFACE ELEVAION AT M.P. 0.0 IS 0 FT M.S.L.

M.P.	WATER LEVEL ELEVATION
0	0
2	.52
4	1.05
6	1.8
8	3.07
10	4.3
12	5.34
14	6.7
16	7.25
18	8.03
20	8.76
21	9.24
22.7	9.54
24.9	9.96
27.3	10.46
29.7	10.93
32.2	11.31
33.6	11.52
35	11.83
36.9	12.08
38.8	12.18
40.1	12.31
41.4	12.42
43.3	12.76
44.4	12.72
46.5	13.04
49.3	12.99
51.6	13.6
52.9	14.18
53.9	14.39
55	14.15
56.2	14.53
57.5	15.27
59.1	15.58
61.3	15.89

USED: CP 7.9 I/O 4.4

READY

FIGURE V-1


HUDSON RIVER - PMF AT INDIAN POINT
COMPUTED WATER SURFACE PROFILES
WATER SURFACE ELEVATION IN FEET ABOVE OR BELOW M.S.L.
25
24
23
22
21
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-1
-2
-3
-4
HUDSON RIVER PROBABLE MAXIMUM FLOOD AND HIGH WATER AT THE BATTERY (+2.2 FT ABOVE M.S.L.) (Q=1.0135×10⁶ CFS)
HUDSON RIVER PROBABLE MAXIMUM FLOOD AND MEAN SEA LEVEL AT BATTERY (Q=1.40×10⁶ CFS)
HUDSON RIVER PROBABLE MAXIMUM FLOOD AND LOW WATER AT THE BATTERY (-2.2 FT ABOVE M.S.L.) (Q=1.1648×10⁶ CFS)
INDIAN POINT
10
20
30
40
50
DISTANCE IN MILES ABOVE THE BATTERY

BACK WATER COMPUTATION
PRASAD'S METHOD PROGRAM
PRINTOUT

CONDITIONS :

1) PMF AT INDIAN POINT
2) WATER SLACK PHASE

STAGE AT THE BATTERY 0.0 FT ABOVE N.S.L.
AVERAGE DISCHARGE 1.18X10⁶ CFS

RSIN 10:27 MDS3 MAY 14, 1970

INPUT Y(1),S1,X1,Q,P,R,N,A1

INPUT:01100
736.0,36.0,0.0,1100000.0,-1.0,.27,0.025,1.0,6700.0

REACH = 1

MILE POINT MILES	+ OR - MSL FT.
0.0	0.00
0.5	0.15
1.0	0.29
1.5	0.42
2.0	0.54

REACH = 2

MILE POINT MILES	+ OR - MSL FT.
2.0	0.54
2.5	0.71
3.0	0.88
3.5	1.04
4.0	1.20

REACH = 3

MILE POINT MILES	+ OR - MSL Ft.
4.0	1.[illegible]
4.5	1.39
5.0	1.60
5.5	1.82
6.0	2.05

REACH = 4

MILE POINT MILES	+ OR - MSL Ft.
6.0	2.05
6.5	2.28
7.0	2.54
7.5	2.81
8.0	3.11

REACH = 5

MILE POINT MILES	+ OR - MSL Ft.
8.0	3.11
8.5	3.40
9.0	3.69
9.5	3.98
10.0	4.27

REACH = 6

MILE POINT MILES	+ OR - MSL FT.
10.0	4.27
10.5	4.52
11.0	4.80
11.5	5.08
12.0	5.39

REACH = 7

MILE POINT MILES	+ OR - MSL FT.
12.0	5.39
12.5	5.73
13.0	6.02
13.5	6.27
14.0	6.49

REACH = 8

MILE POINT MILES	+ OR - MSL FT.
14.0	6.49
14.5	6.65
15.0	6.81
15.5	6.97
16.0	7.14

REACH = 9

MILE POINT MILES	+ OR - MSL FT.
16.0	7.14
16.5	7.34
17.0	7.54
17.5	7.74
18.0	7.94

REACH = 10

MILE POINT MILES	+ OR - MSL FT.
18.0	7.94
18.5	8.14
19.0	8.33
19.5	8.53
20.0	8.72

REACH = 11

MILE POINT MILES	+ OR - MSL FT.
20.0	8.72
20.5	8.92
21.0	9.09

REACH = 12

MILE POINT MILES	+ OR - MSL FT.
21.0	9.09
21.5	9.15
22.0	9.22
22.5	9.30
22.7	9.33

REACH = 13

MILE POINT MILES	+ OR - MSL FT.
22.7	9.33
23.2	9.42
23.7	9.51
24.2	9.60
24.7	9.68
24.9	9.72

REACH = 14

MILE POINT MILES	+ OR - MSL FT.
24.9	9.72
25.4	8.75
25.9	9.78
26.4	9.82
26.9	9.88
27.3	9.96

REACH = 15

MILE POINT MILES	+ OR - MSL FT.
27.3	9.96
27.8	10.08
28.3	10.18
28.8	10.28
29.3	10.38
29.7	10.45

REACH = 16

MILE POINT MILES	+ OR - MSL FT.
29.7	10.45
30.2	10.55
30.7	10.65
31.2	10.74
31.7	10.83
32.2	10.91

REACH = 17

MILE POINT MILES	+ OR - MSL FT.
32.2	10.91
32.7	11.07
33.2	11.20
33.7	11.30
34.2	11.32

REACH = 18

MILE POINT MILES	+ OR - MSL FT.
33.8	11.32
34.3	11.32
34.8	11.34
35.0	11.35

REACH = 19

MILE POINT MILES	+ OR - MSL FT.
35.0	11.35
35.5	11.43
36.0	11.49
36.5	11.56
36.9	11.60

REACH = 20

MILE POINT MILES	+ OR - MSL FT.
36.9	11.60
37.4	11.70
37.9	11.78
38.4	11.85
38.8	11.89

REACH = 21

MILE POINT MILES	+ OR - MSL FT.
38.8	11.89
39.3	12.02
39.8	12.13
40.1	12.19

REACH = 22

MILE POINT MILES	+ OR - MSL FT.
40.1	12.19
40.6	12.33
41.1	12.44
41.4	12.50

REACH = 23

MILE POINT MILES	+ OR - MSL FT.
41.4	12.50
41.9	12.55
42.4	12.60
42.9	12.66
43.3	12.70

REACH = 24

MILE POINT MILES	+ OR - MSL FT.
43.3	12.70
43.8	12.90
44.3	13.02
44.4	13.04

REACH = 25

MILE POINT MILES	+ OR - MSL FT.
44.4	13.04
44.9	13.19
45.4	13.31
45.9	13.42
46.4	13.51
46.5	13.53

TABLE V-2

BACK WATER COMPUTATION - COMPERISON OF WATER SURFACE PROFILES COMPUTED USING STANDARD STEP METHOD AND PRASAD'S METHOD

CONDITIONS: 1-STAGE AT THE BATTERY 0.0 FT. ABOVE M.S.L.
2-Q=1.10 x 10^6 CFS

M.P.	Water Surface Elevation Standard Step Method (Ft. above M.S.L.)	Prasad's method (Ft. above M.S.L.)	Difference 3 - 2 Ft.
1	2	3	4
0	0	0	0
2	0.52	0.54	+ 0.02
4	1.05	1.20	+ 0.15
6	1.80	2.05	+ 0.25
8	3.07	3.11	+ 0.04
10	4.30	4.27	- 0.03
12	5.34	5.39	+ 0.05
14	6.70	6.49	- 0.21
16	7.25	7.14	- 0.11
18	8.03	7.94	- 0.09
20	8.76	8.72	- 0.04
21	9.24	9.09	- 0.15
22.7	9.54	9.33	- 0.21
24.9	9.96	9.72	- 0.24
27.3	10.46	9.96	- 0.50
29.7	10.93	10.95	- 0.48
32.2	11.31	10.91	- 0.40

TABLE V-2 (Continued)

BACK WATER COMPUTATION - COMPERISON OF WATER SURFACE PROFILES COMPUTED USING STANDARD STEP METHOD AND PRASAD'S METHOD

CONDITIONS: 1-STAGE AT THE BATTERY 0.0 FT. ABOVE M.S.L.
2-Q=1.10 x 10^6 CFS

M.P.	Water Surface Elevation Standard Step Method (Ft.above M.S.L.)	Water Surface Elevation Prasad's Method (Ft.above M.S.L.)	Difference 3 - 2 Ft.
1	2	3	4
33.8	11.52	11.32	- 0.20
35.0	11.83	11.35	- 0.48
36.9	12.08	11.60	- 0.48
38.8	12.18	11.89	- 0.29
40.1	12.31	12.19	- 0.12
41.4	12.42	12.50	+ 0.08
43.3	12.76	12.70	- 0.06
44.9	12.72	13.04	+ 0.36

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

To 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.

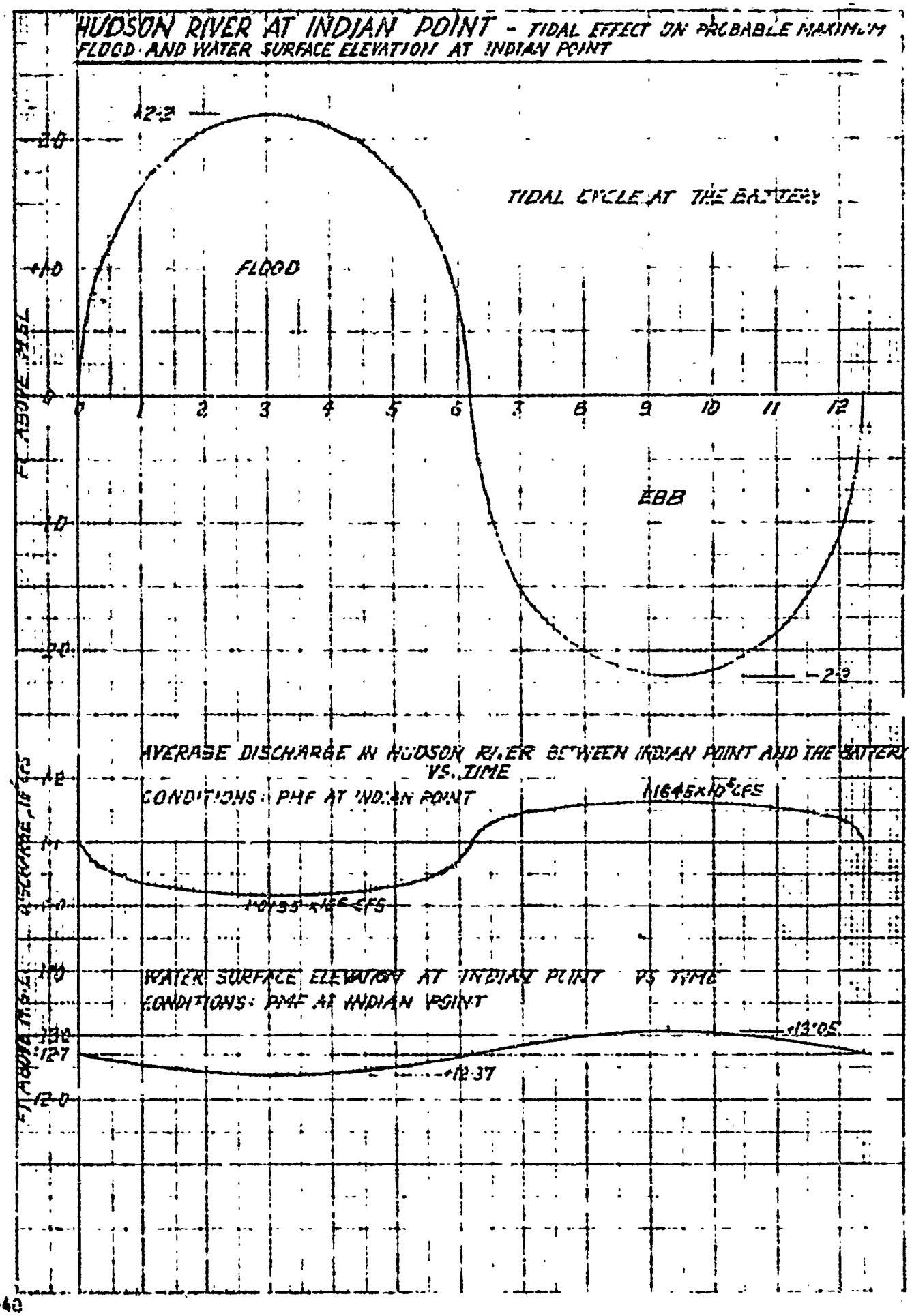
Therefore, the Indian Point river stage corresponding to Case No. 1 conditions and including the influence of the local oscillatory wave would be 13.7 feet above mean sea level.

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

HUDSON RIVER AT INDIAN POINT - TIDAL EFFECT ON PROBABLE MAXIMUM FLOOD AND WATER SURFACE ELEVATION AT INDIAN POINT



HUDSON RIVER AT INDIAN POINT - PMF

TABLE V-3

TIDAL EFFECT ON PMF AND WATER SURFACE ELEVATION AT INDIAN POINT

TIDAL PHASE	WATER SURFACE ELEVATION AT THE BATTERY (M.P. 0.0) FT ABOVE M.S.L.	COMPUTED AVERAGE DISCHARGE BETWEEN INDIAN P. AND THE BATTERY 10^6 CFS	COMPUTED WATER SURFACE ELEVATION AT INDIAN POINT FT ABOVE M.S.L.
LOW WATER SLACK	0.0	1.1000	12.7
FLOOD	1.0	1.0600	/
	1.6	1.0378	/
	1.9	1.0255	/
	2.2	1.0130	12.37
HIGH WATER SLACK	3.0	1.1400	12.7
EBB	-1.0	1.1323	/
	-1.6	1.1494	/
	-1.9	1.1577	/
	-2.2	1.1648	+13.05

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.

This 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 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).

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

42

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. 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 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

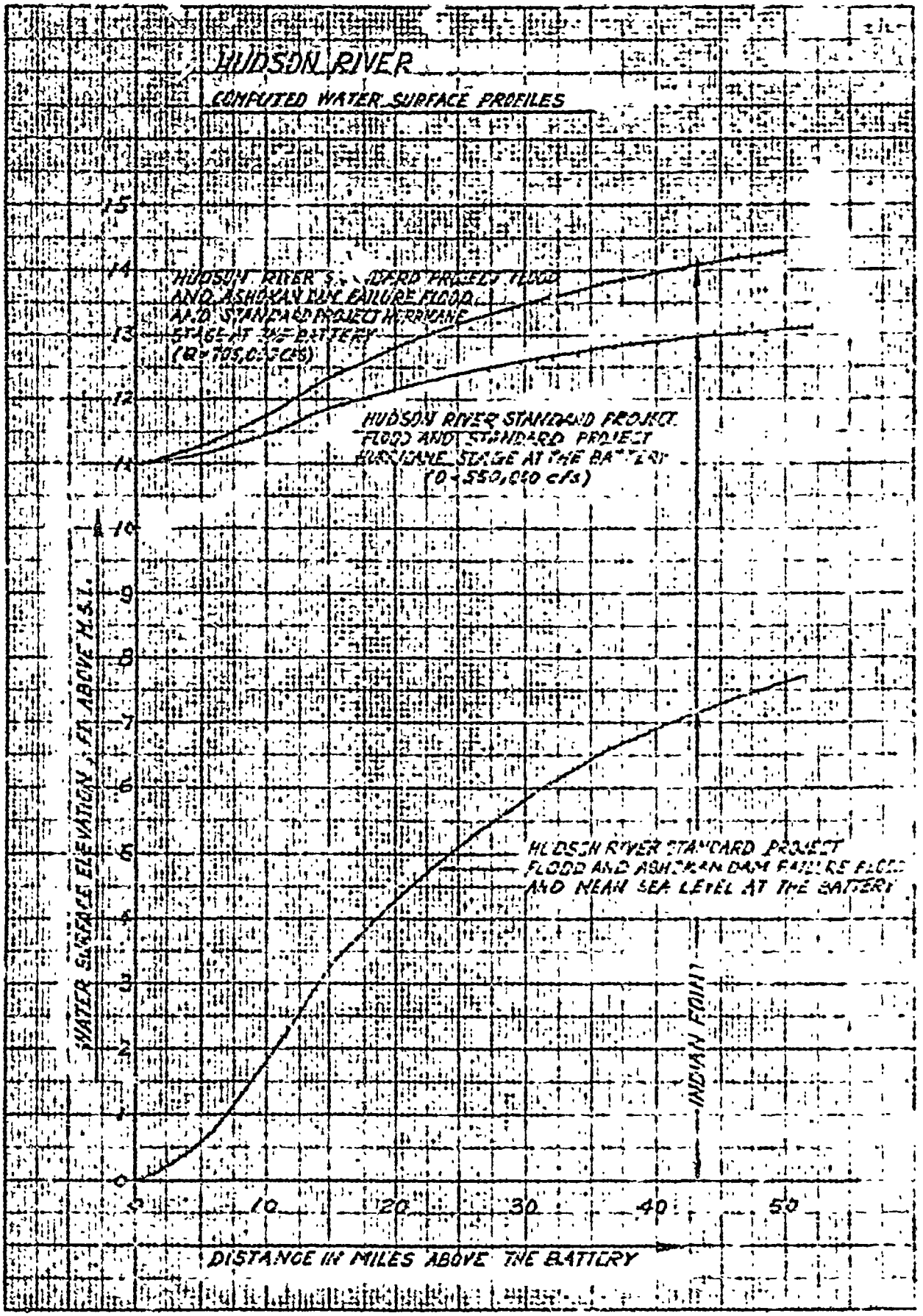
result in a river stage of Elevation 7.2 above mean sea level at the Indian Point site. This value corresponds to a mean sea level elevation at the ocean entrance. Because this condition is less critical than the probable maximum flood conditions (Cases No. 1 through 3), additional evaluation concerning 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.

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


HUDSON RIVER
COMPUTED WATER SURFACE PROFILES
HUDSON RIVER STANDARD PROJECT FLOOD AND ASHOKAN DAM FAILURE FLOOD AND STANDARD PROJECT HURRICANE STAGE AT THE BATTERY (Q=705,000 CFS)
HUDSON RIVER STANDARD PROJECT FLOOD AND STANDARD PROJECT HURRICANE STAGE AT THE BATTERY (Q=550,000 cfs)
HUDSON RIVER STANDARD PROJECT FLOOD AND ASHOKAN DAM FAILURE FLOOD AND MEAN SEA LEVEL AT THE BATTERY
WATER SURFACE ELEVATION, FT ABOVE M.S.L.
DISTANCE IN MILES ABOVE THE BATTERY
INDIAN POINT
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
10
20
30
40
50

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

hurricane "Donna" that struck on September 12, 1960. Hurricane "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

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.

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-

TABLE V-4

WATER SURFACE ELEVATION AT INDIAN POINT
RESULTING FROM STATED FLOW & ELEVATION CONDITIONS

Component Flow at Indian Point	Elevation at the Battery	Flow at Indian Point in Million cfs	Elevation at Indian Point (MSL Datum)	Elevation at Indian Point including local oscillatory wave height (MSL Datum)
1. Probable Max. Flood	Mean Sea Level (0.00)	1.100	12.7	13.7
2. Probable Max. Flood & Tidal Flow	High Water (+2.20)	1.014	12.4	13.4
3. Probable Max. Flood & Tidal Flow	Low Water (-2.20)	1.165	13.9	14.0
4. Standard Project Flood & Ashokan Dam Failure	Mean Sea Level (0.00)	0.705	7.2	8.2
5. Standard Project Flood	Standard Project Hurricane (+11.00)	0.550	13.0	14.0
6. Standard Project Flood & Ashokan Dam Failure	Standard Project Hurricane (+11.00)	0.705	14.0	15.0
7. Probable Max. Hurricane & Spring High tide	Probable Max. Hurricane (+17.5)		13.5	14.5

PLATE V-8
(SHEET 1 OF 1)

BACK WATER COMPUTATION
STANDARD STEP METHOD
PRINTOUT

CONDITIONS:
1) PMF AT [illegible] AN POINT
2) FLOOD PHASE

STAGE AT THE BATTERY = 2.2 FT ABOVE M.S.L.

AVERAGE DISCHARGE 1.0135x10⁶ CFS

RUN

LEVEL 19:20 RDS3 JUN 11 70 THU

HUDSON RIVER

BACK WATER PROFILE FROM M.P. 0.0 TO M.P. 153.7

Q= 1.0135E 6 CFS

WATER SURFACE ELEVATION AT M.P. 0.0 IS 2.2 FT M.S.L.

M.P.	WATER LEVEL ELEVATION
0	2.2
2	2.56
4	2.94
6	3.48
8	4.41
10	5.33
12	6.13
14	7.24
16	7.69
18	8.32
20	8.93
21	9.33
22.7	9.58
24.9	9.94
27.3	10.37
29.7	10.77
32.2	11.1
33.8	11.28
35	11.55
36.9	11.77
38.8	11.91
40.1	12.08
41.4	12.17
43.3	12.41
44.4	12.38
46.5	12.65
49.3	12.62
51.6	13.13
52.9	13.59
53.9	13.82
55	13.61
56.2	13.94
57.5	14.58
59.1	14.85
61.3	15.15

SED: CPU 7.4 I/O 4.8

READY

PLATE V-7
(SHEET 1 OF 1)

BACK WATER COMPUTATION
STANDARD STEP METHOD
PRINTOUT

CONDITIONS:
1) PMF AT INDIAN POINT
2) EBB PHASE

STAGE AT THE BATTERY -2.2 F.T ABOVE M.S.L.
AVERAGE DISCHARGE 1.1648×10^6 cfs

```
RUN

LEVEL      19:22    RD53   JUN 11 78 THU


  HUDSON RIVER

BACK WATER PROFILE FROM M.P. 0.0  TO  M.P.  153.7

Q= 1.1648E 6 CFS

WATER SURFACE ELEVATION AT M.P. 0.0 IS -2.2 FT M.S.L.

  M.P.      WATER LEVEL
            ELEVATION

  0              -2.2
  2              -1.48
  4              -.77
  6               .22
  8              1.85
  10             3.41
  12             4.69
  14             6.3
  16             6.95
  18             7.84
  20             8.68
  21             9.22
  22.7           9.56
  24.9           10.03
  27.3           10.58
  29.7           11.1
  32.2           11.52
  33.8           11.75
  35             12.09
  36.9           12.37
  38.8           12.47
  40.1           12.61
  41.4           12.73
  43.3           13.11
  44.4           13.06
  46.5           13.41
  49.3           13.36
  51.6           14.03
  52.9           14.6
  53.9           14.91
  55             14.64
  56.2           15.06
  57.5           15.87
  59.1           16.18
  61.3           16.53

USED: CPU     8.1   I/O     4.8

READY
```

BACK WATER COMPUTATION

STANDARD STEP METHOD
PRINT OUT

LEVEL 17:46 RDS3 JUN 11 78 THU

CONDITIONS

1 - HUDSON RIVER STANDARD PROJECT FLOOD AND ASHOKAN DAM FAILURE (Q=105,000 CFS)
2 - MEAN SEA LEVEL AT THE BATTERY

HUDSON RIVER

BACK WATER PROFILE FROM M.P. 0.0 TO M.P. 153.7

Q= 705000 CFS

WATER SURFACE ELEVATION AT M.P. 0.0 IS 0 FT M.S.L.

M.P.	WATER LEVEL ELEVATION
0	0
2	.21
4	.44
6	.76
8	.3
10	1.91
12	2.45
14	3.19
16	3.51
18	3.96
20	4.4
21	4.62
22.7	4.8
24.9	5.07
27.3	5.41
29.7	5.81
32.2	6.11
33.8	6.32
35	6.52
36.9	6.73
38.8	6.85
40.1	6.98
41.4	7.1
43.3	7.26
44.4	7.26
46.5	7.43
49.3	7.43
51.6	7.72
52.9	8
53.9	8.12
55	8.02
56.2	8.17
57.5	8.57
59.1	8.82
61.3	9.11
63.6	9.42
70.3	10.86
72 0	

USED: CPU 8.6 I/O 3.6

RUN

PLATE V-9
(SHEET 1 OF 1)

LEVEL 1714Ø RD33 JUN 1 78 THU

BACK WATER COMPUTATION
STANDARD STEP METHOD
PRINTOUT

CONDITIONS:
1-HUDSON RIVER STANDARD PROJECT FLOOD (Q=550,000 CFS)
2-STANDARD PROJECT HURRICANE STAGE AT THE BATTERY (+11.0 FT ABOVE M.S.L.)

HUDSON RIVER

BACK WATER PROFILE FROM M.P. Ø.Ø TO M.P. 153.7

Q= 55ØØØØ CFS

WATER SURFACE ELEVATION AT M.P. Ø.Ø IS 11 FT M.S.L.

M.P.	WATER LEVEL ELEVATION
Ø	11
2	11.Ø5
4	11.11
6	11.2
8	11.34
1Ø	11.49
12	11.62
14	1.82
16	11.91
18	12.Ø4
2Ø	12.17
21	12.24
22.7	12.3
24.9	12.39
27.3	12.46
29.7	12.56
32.2	12.63
33.8	12.7
35	12.74
36.9	12.8
38.8	12.83
4Ø.1	12.88
41.4	12.93
43.3	12.99
44.4	13.Ø3
46.5	13.11
49.3	13.Ø9
51.6	13.2
52.9	13.29
53.9	13.31
55	13.25
56.2	13.32
57.5	13.51
59.1	13.6
61.3	13.74
63.6	13.88
7Ø.3	14.24
72	14.3
73.6	14.41
74.9	Ø14.4

USED: CPU 8.5 I/O 4.4

(SHEET 1 OF 1)

LEVEL 17.43 RDS3 JUN 11 70 THU

BACK WATER COMPUTATION
STANDARD STEP METHOD
PRINTOUT

CONDITIONS:
1- HUDSON RIVER STANDARD PROJECT FLOOD AND ASHOKAN DAM FAILURE (Q=705,000 CFS)
2- STANDARD PROJECT HURRICANE STAGE AT THE BATTERY (+11.0 FT ABOVE M.S.L.)

HUDSON RIVER

BACK WATER PROFILE FROM M.P. 0.0 TO M.P. 153.7

Q= 705000 CFS

WATER SURFACE ELEVATION AT M.P. 0.0 IS 11 FT M.S.L.

M.P.	WATER LEVEL ELEVATION
0	11
2	11.09
4	11.18
6	11.33
8	11.55
10	11.79
12	12.01
14	12.32
16	12.46
18	12.67
20	12.88
21	12.98
22.7	13.08
24.9	13.21
27.3	13.33
29.7	13.47
32.2	13.58
33.8	13.67
35	13.74
36.9	13.83
38.8	13.88
40.1	13.95
41.4	14.02
43.3	14.12
44.4	14.1
46.5	14.23
49.3	14.2
51.6	14.44
52.9	14.65
53.9	14.76
55	14.66
56.2	14.77
57.5	15.07
59.1	15.19
61.3	15.34
63.6	15.53
70.3	16
72	16.13
73.	

USED: CPU 9.8 I/O 5.2

Y.55 READY

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 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 river. 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.

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) 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.

It 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.

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

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 track 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 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.

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 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 downstrea 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 conditions. The spring high tide at Indian Point is estimated to increase the stage by 3.0 feet above M.T.L. (very conservative).

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

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.

BY M.U.W. DATE _____ ENVIRONMENTAL SCIENCE & ENGINEERING CONSULTANTS SHEET NO _____ OF _____
CHKD. BY _____ DATE _____ 505 FIFTH AVENUE JOB NO _____
NEW YORK, NEW YORK 10017
SUBJECT _____

COMPUTATION OF MAXIMUM WATER LEVEL AT INDIAN PT.

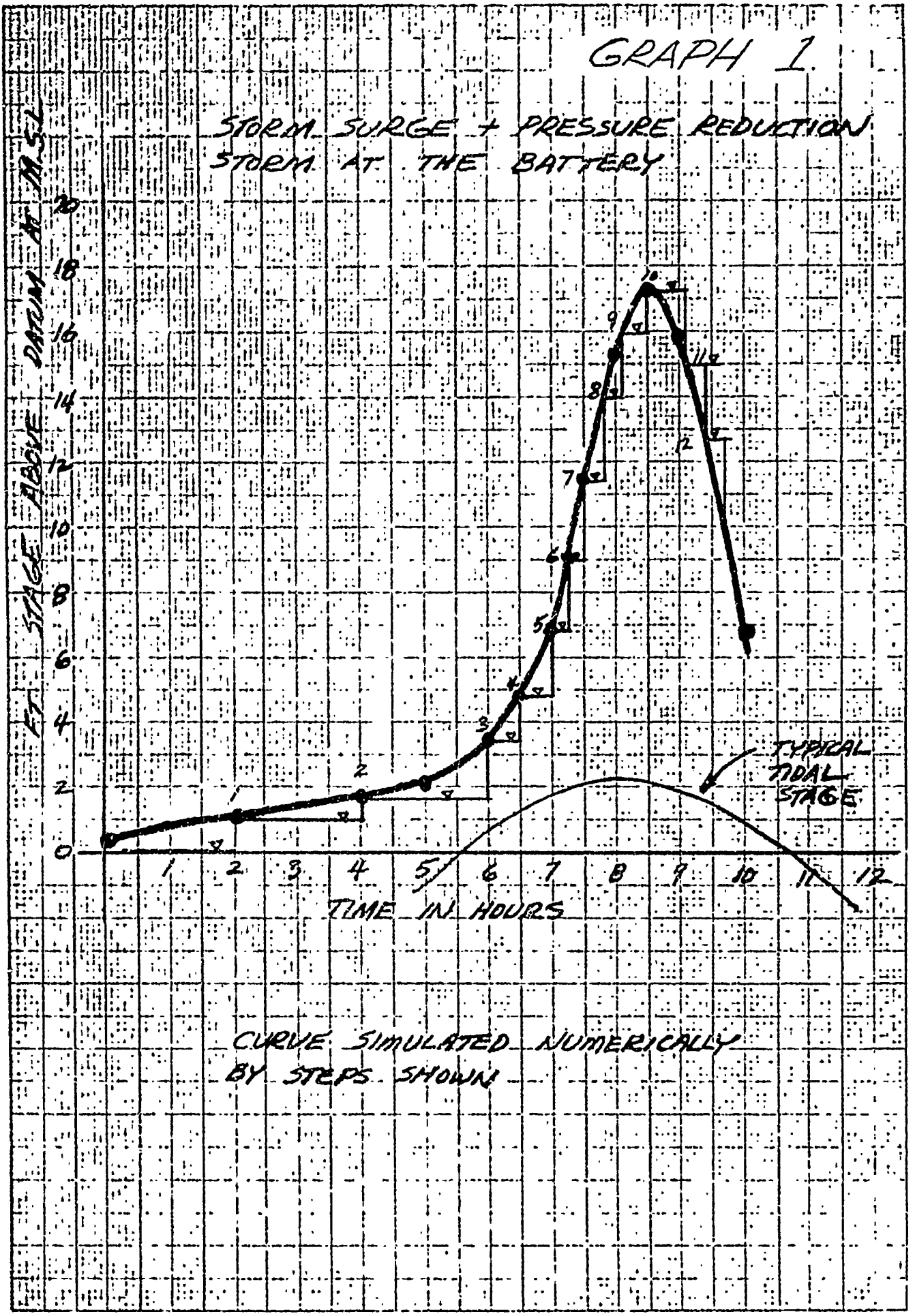
COMPUTATIONS OF HURRICANE SURGE ENTERING NEW YORK HARBOR, MADE BY JOHN LEKSTUTIS, SHOW THAT A MAXIMUM STAGE OF 15.6 FT ABOVE M.S.L. CAN BE EXPECTED AT THE BATTERY.

SUCH A HURRICANE SURGE WOULD, OF COURSE FLOOD MANY SQUARE MILES OF LOW ELEVATION AREAS IN NEW YORK CITY AND NEW JERSEY.

BASIL W. WILSON OF TEXAS A & M PREDICTED A MAXIMUM SURGE OF 15.3 FT IN THE HARBOR. (SEE TECH. MEMO. NO 120 BEACH EROSION BOARD, CORPS OF ENGINEERS, "PREDICTION OF HURRICANE STORM-TIDES IN NEW YORK BAY," 1960)

	15.6	FT	
+	1.9	FT	PRESSURE REDUCTION HEIGHT RISE
	17.5	FT	MAX SURGE @ BATTERY

STAGE-TIME HYDROGRAPH CONSTRUCTED ACCORDING TO SHAPE PREDICTED BY WILSON (SEE HIS FIG. 47) LABELED GRAPH 1



K&E 10 X 10 TO 10 X 15 INCH 7 X 10 INCHES
NO. 1320
KEUFFEL & ESSER CO.

ROUTING OF SURGE THROUGH SUCCESSIVE REACHES OF THE HUDSON ESTUARY

$$\eta = F(x)\left\{1 - 1.5\frac{X F'}{C A} - \frac{|F|}{H}\right\}$$

SEE PRODMAN'S PAPER "THE PROPAGATION OF TIDE & SURGE IN AN ESTUARY"

EQ #37

WHERE

η = EL. OF WATER SURFACE

$F(x)$ = STAGE FUNCTION OF SURGE

C = CELERITY

A = HYDRAULIC DEPTH

X = DISTANCE

H = FRICTION PARAMETER $= \frac{2A^2}{K X} = 620\frac{A^2}{X}$

NOTE THIS FORM OF EQUATION ASSUMES

A = CONSTANT FOR A GIVEN REACH

& $P = \left(\frac{b_o C_o}{b_x C_x}\right)^{1/2} \approx$ CONSTANT WITHIN REACH

BY M.D.D DATE

ENVIRONMENTAL SCIENCE & ENGINEERING CONSULTANTS

NEW YORK, NEW YORK 10017

CHKD. BY DATE

SUBJECT

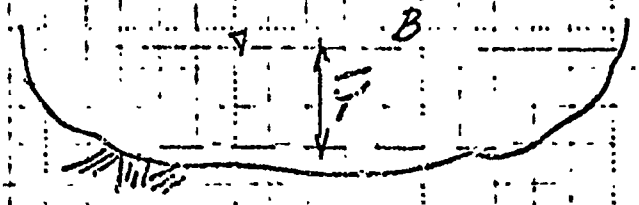
SHEET NO.

JOB NO.

CELERITY OF PROGRESSIVE WAVE

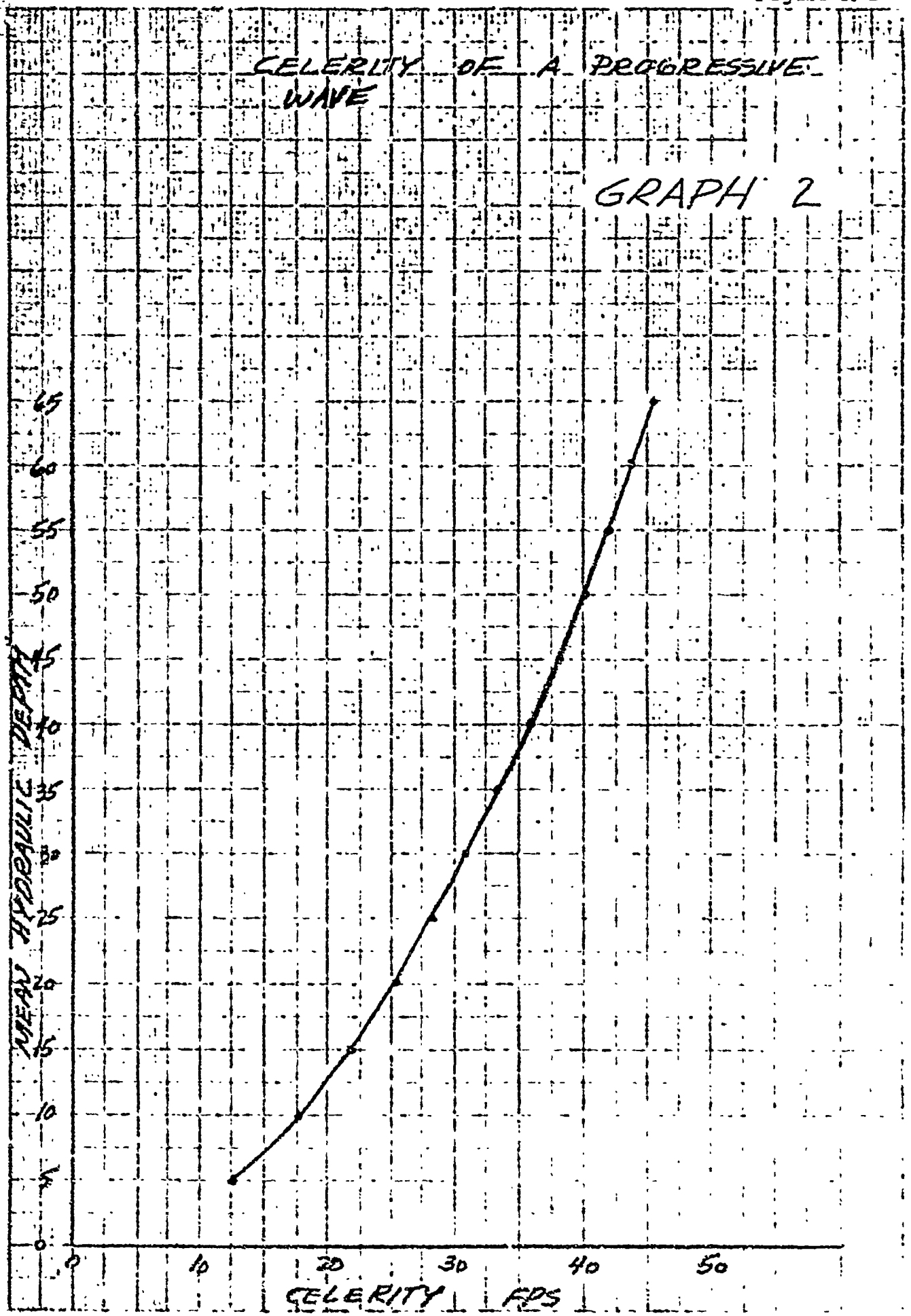
MEAN DEPTH FT	$g\bar{y}$ / 32.2	$\sqrt{g\bar{y}} = C$ FT/SEC
5	161	12.7
10	322	17.9
15	483	21.9
20	644	25.3
25	805	28.3
30	965	31.0
35	1128	33.5
40	1288	35.8
45	1450	38.0
50	1610	40.1
55	1770	42.0
60	1930	43.8
65	2090	45.6

PLOTTED ON GRAPH 2



$\frac{\text{AREA}}{B} = \bar{y}$

Figure A-2


CELERITY OF A PROGRESSIVE WAVE
GRAPH 2
MEAN HYDRAULIC DEPTH
0
5
10
15
20
25
30
35
40
45
50
55
60
65
0
10
20
30
40
50
CELERITY FPS

FRICTION TERM IN PROUDMAN'S EQUATION

PROUDMAN'S EQ. OF MOTION

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} = -g\frac{\partial \eta}{\partial x} - \frac{k}{h}u|u|$$

COMMON EXPRESSION OF EQ. OF MOTION

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} = -g\frac{\partial \eta}{\partial x} - g\frac{u|u|}{C_{CHEZY}^2 h}$$

$$\therefore \quad k = \frac{g}{C_{CHEZY}^2}$$

USING $C_{CHEZY} \approx 100$

SEE ALSO REFERENCE

IPPEN, Estuary & Coastline Hydrodynamics PG 527

$C \approx 80 \rightarrow 140$ IN ESTUARIES

COMPARES TO STREAM IN FLAT CONDITION

MANNING'S $n \approx 0.025 \rightarrow 0.030$

MEAN AND DEPTH $\approx 30-40$ FT

$$\therefore \quad k = \frac{32.2}{100(1.3)} = .00322$$

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PAGE OF

JOB NO.

PARAMETERS FOR SURGE ROUTING BATTERY TO DOBBS FERRY

STATION	DISTANCE FROM BATTERY		ΔX
BATTERY	X = 0 MILES	0 FT	28,800
WEST N.Y	X = 5.45	28,800	28,000
G.W. BRIDGE	X = 10.75	56,800	11,800
SPUYTEN DUYVIL	X = 13.0	68,600	22,900
YONKERS	X = 17.33	91,500	27,000
DOBBS FERRY	X = 22.45	118,500	

Σ = 118,500

STATION	SURFACE WIDTH	HYDRAULIC DEPTH M.L.W.	DEPTH TO [illegible]
BATTERY	4700 FT	40 FT	2.5 FT
WEST N.Y	3800 FT	28 FT	2.0 FT
G.W. BRIDGE	3450 FT	26 FT	1.9
SPUYTEN DUYVIL	5300 FT	28 FT	1.6
YONKERS	4750 FT	28 FT	1.4
DOBBS FERRY	6300 FT	21 FT	1.3

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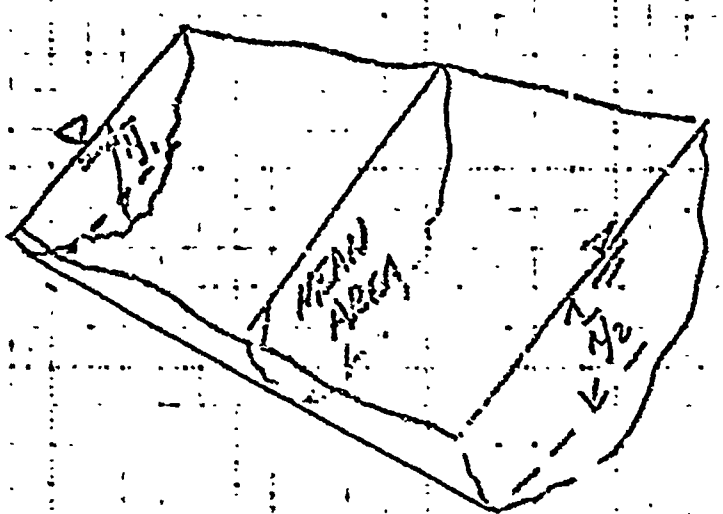
COMPUTATION FOR EFFECTIVE HYDRAULIC DEPTH OF EACH ESTUARINE REACH

REACH	SURFACE WIDTH $\circ \bar{Y}$	MEAN WIDTH	MEAN HYD DEPTH	
BATTERY	4700(40) = 188,000 ft^2	4250 ft	188,000 106,500 294,500	$\frac{294,500}{8500} = 34.6$
WEST N.Y.	3800(28) = 106,500	7250/2	106,500 89,800 196,300	$\frac{196,300}{7250} = 27.1$
G.W. BRIDGE	3400(25) = 89,800	8750/2	89,800 148,300 238,100	27.2
S. DOYVIL	5300(28) = 148,300	10050/2	148,300 133,000 281,300	28.0
YONKERS	4750(28) = 133,000	11050/2		24.0
DOBBS FERRY	6300(21) = 133,000			

M.L.W.	MEAN TIDE
34.6 + 2.5 =	37.1
27.1 + 2.2 =	29.3
27.2 + 1.9 =	29.1
29.0 1.8 =	29.8
24.0 + 1.7 =	25.7



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EVALUATION OF SECTION "P"

REACH	b SURFACE WIDTH, FT	C MEAN CELERITY M.S.L.	b C $\times 10^4$	$\left(\frac{b_{c}C_{c}}{bC}\right)^{1/2} = P$
BATTERY	4700	37.2 FPS	17.5	0.91
WEST N.Y.	3800	31.0 FPS	11.8	1.13
G.W. BRIDGE	3450	29.8	10.3	1.19
SPUYTEN DUYVIL	5300	30.9	16.4	0.95
YONKERS	4750	30.8	14.7	0.99
DOBBS FERRY	6300	27.0	17.0	0.93

ABOVE IS VARIATION OF P AT M.T.L.

Σ 87.7 $\quad \Sigma = \frac{6.10}{6} = 1.0$

$\frac{87.7}{6} = 14.6$

BELOW IS COMPUTATION WHEN SURGE IS 12 FT. ABOVE M.T.L

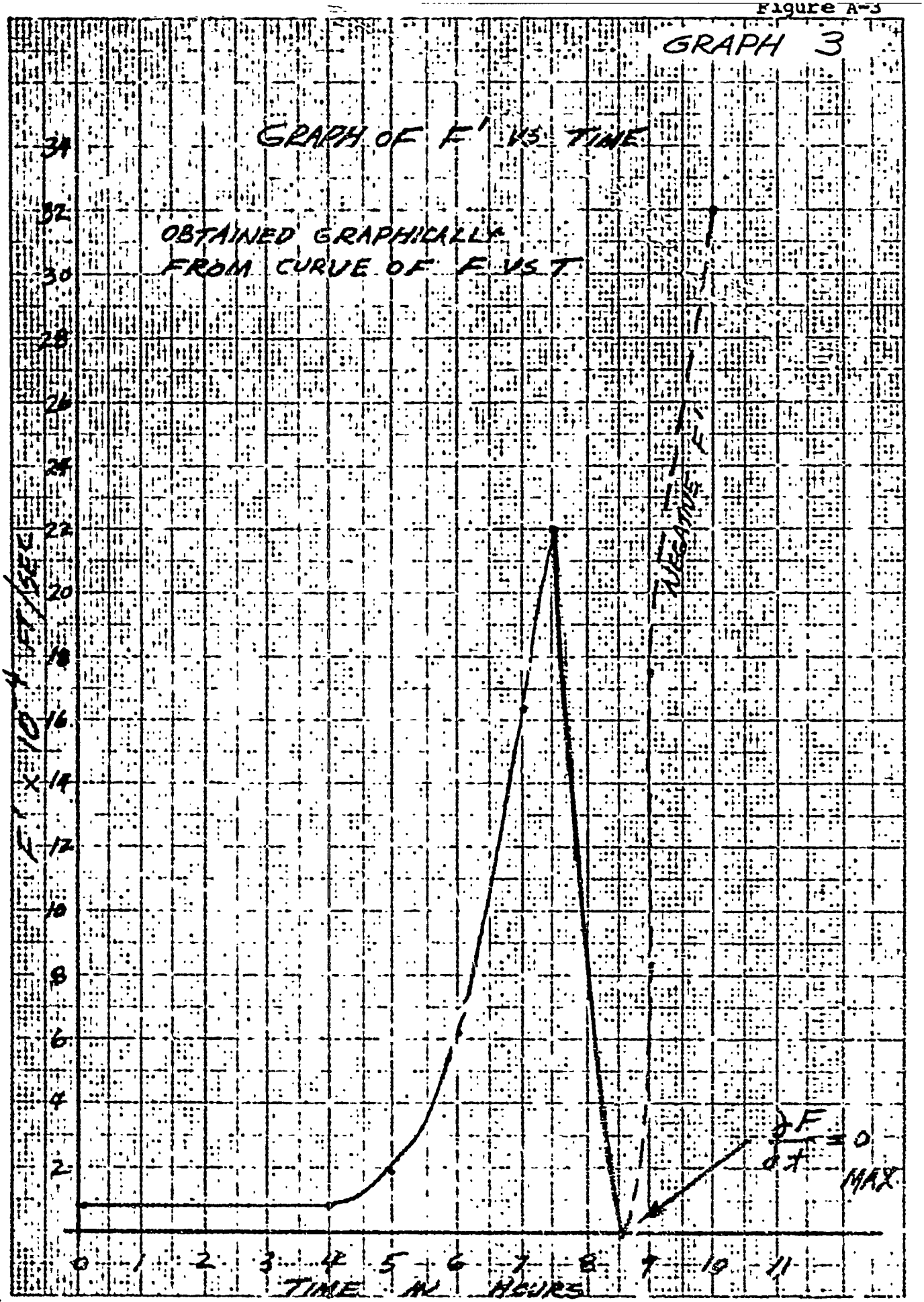
REACH	b	C	b C $\times 10^4$	
BATTERY	4700	42.0	19.8	0.935
WEST, N.Y.	3800	37.0	14.1	1.11
G.W. BRIDGE	3450	36.0	12.4	1.18
SPUYTEN DUYVIL	5300	36.0	19.1	0.95
YONKERS	4750	36.0	17.1	1.01
DOBBS FERRY	6300	33.5	21.1	0.91

$\Sigma = 103.6 \quad \Sigma = \frac{6.10}{6} = 1.$

$\frac{103.6}{6} = 17.3$

CONCLUSION IS THAT ASSUMPTION OF P ≈ 1.0 IS REASONABLE SINCE P AVERAGES 1.02

Figure A-3


GRAPH 3
GRAPH OF F' VS TIME
OBTAINED GRAPHICALLY
FROM CURVE OF F VS T
NEGATIVE F'
$\frac{\partial F}{\partial t} = 0$
MAX
F' x 10^4 FT/SEC
34
32
30
28
26
24
22
20
18
16
14
12
10
8
6
4
2
0
1
2
3
4
5
6
7
8
9
10
11
TIME IN HOURS

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	REACH	(ΔX) X FT	MTL. h FT	C FPS	t HRS	F FT	F' FT/SEC	H $620\frac{h^2}{X}$	① $\frac{3XF'}{2Ch}$	② $\frac{\lvert F\rvert}{H}$	NEW OLD F = F(1-①-②)
①	BATTERY				2.0						
		28,800	37.1	34.5		1.0	$.93\times10^{-4}$	29.5	0.0032	0.034	0.963 ✓
	WEST N.Y.				2.23						
		28,800	29.3	30.0		0.963	$.93\times10^{-4}$	18.5	0.0045	0.052	0.908 ✓
	G.W. BRIDGE				2.50						
		11,800	29.1	30.5		0.908	$.93\times10^{-4}$	44.4	0.0019	0.020	0.886 ✓
	S.D.				2.61						
		22,900	29.8	31.0		0.886	$.93\times10^{-4}$	23.8	0.0035	0.037	0.850
	YONKERS				2.83						
		27,000	25.7	28.5		0.850	$.93\times10^{-4}$	15.2	0.0051	0.055	0.798
	DOBBS F.				3.09						
②	BATTERY				4.0						
		28,800	38.1	35.0		0.67	$.93\times10^{-4}$	31.24	.0030	.021	.654
	WEST N.Y.				4.23						
		28,800	30.2	31.0		0.654	↓	19.63	.0043	.033	.630
	G.W.B				4.49						
		11,900	30.0	31.0		0.630		47.29	.0018	.013	.620
	S.D				4.59						
		22,900	30.6	31.5		0.62[illegible]		25.35	.0033	.024	.603
	YONKERS				4.79						
		27,000	26.5	29.0		0.603		16.13	.0049	.037	.576 ✓
	DOBBS F				5.05						

.80
.58
1.38

	REACH	X FT FT	A FT	C FPS	t HR	F FT	F' FT/SEC	H $G\,20\frac{C^2}{T}$	$\frac{3XF'}{2CA}$	$\frac{\lvert F\rvert}{H}$	NOW F FT	
3)	BAT				6.00							
		28800	38.8	35.5		1.80	6.1×10^{-4}	32.6	.019	.055	1.67 ✓	
	W. N.Y				6.23							
		28900	30.8	31.3		1.67		20.4	.027	.082	1.49 ✓	
	G.W.B				6.48							
		11900	30.6	31.0		1.49		49.2	.011	.030	1.43	
	S. D				6.59							
		22900	31.2	31.5		1.43		26.4	.021	.054	1.32	
	YON.				6.79							
		27000	27.1	29.3		1.32		16.7	.031	.079	1.15	1.33 1.15 2.55
	D.F				7.05							
4)	BAT				6.50							
		28800	40.6	36.5		1.40	10.5×10^{-4}	35.7	.030	.039	1.33	
	W.N.Y				6.72							
		28800	32.5	32.2		1.33		22.7	.043	.059	1.19	
	G.W.B				6.97							
		11300	32.1	32.0		1.19		54.5	.018	.022	1.14	
	S. D				7.07							
		22900	32.6	32.5		1.14		29.1	.034	.039	1.06	
	YON				7.27							
		27000	28.3	30.0		1.06		18.6	.050	.051	0.95	2.53 .75 3.45
	D.F				7.52							

REACH	X FT	R FT	C FPS	T hr	F FT	FT/sec	H 620 $\frac{R^2}{X}$	$\frac{3 \times FT}{2CR}$	$\frac{IFL}{H}$	New F ft
(5) BATTERY				7.00						
W. N.Y	29800	42.0	31.0		2.0	16.4×10^{-4}	38.0	.046	.053	1.80
G.W.B	22500	33.3	33.0	7.22	1.80		24.6	.064	.073	1.56
S.D.	11800	33.3	32.8	7.46	1.56		58.3	.026	.027	1.48
YONK.	22700	33.7	33.0	7.56	1.48		30.7	.051	.048	1.33
D.F.	27000	29.2	30.5	7.75	1.33		19.6	.074	.068	1.14
				8.00						
(6) BATTERY				7.30		19.6×10^{-4}				
W. N.Y	28200	44.0	32.0		2.2		41.7	.051	.053	1.97
G.WB	22500	35.6	34.0	7.51	1.97		27.3	.070	.072	1.69
S.D	11800	34.9	33.5	7.75	1.69		64.0	.030	.026	1.60
YONK	22700	35.2	33.8	7.94	1.60		33.5	.056	.048	1.44
D.F	27000	30.3	31.3	8.13	1.44		21.1	.064	.068	1.27

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	REACH	X	h	c	t	F	F'	H $620\frac{h^2}{r}$	$\frac{3XF'}{2Ch}$	$\frac{\lvert F\rvert}{H}$	new F
7	LATTERY				7.5						
		28,800	46.2	38.5		2.4	22×10^{-4}	45.9	.053	.0522	2.15
	W. N.Y.				7.70						
		28800	37.6	35.0		2.15		30.4	.072	.071	1.84
	G.W.B.				7.93						
		11800	36.6	34.5		1.84		70.4	.031	.026	1.74
	S.D.				8.02						
		22900	36.8	34.5		1.74		36.7	.060	.047	1.55
	YONK.				8.20						
		27000	31.5	31.5		1.55		22.8	.090	.068	1.30
	D.F.				8.44						
8	BAT				7.80						
		28800	48.6	39.6		2.6	13×10^{-4}	50.8	.029	.051	2.39
	W. N.Y.				8.00						
		28800	39.8	36.0		2.39		34.1	.039	.070	2.13
	G.W.B				8.22						
		11800	38.4	35.3		2.13		77.5	.017	.027	2.03
	S.D.				8.31						
		22900	38.5	35.4		2.03		40.1	.033	.051	1.86
	YONK				8.49						
		27000	32.8	32.5		1.86		24.7	.049	.075	1.63
	D.F				8.72						

5.84
1.30
8.14

8.14
1.63
9.77

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	REACH	X	A	C	$\frac{A}{X}$	F	F'	$\frac{H}{600\frac{R^2}{X}}$	$\frac{3XF'}{2CR}$	$\frac{\|F\|}{H}$	new E	
(9)	BATT.				8.10							
	W. N.Y.	28,800	51.2	40.7	8.30	2.0	3×10^{-4}	56.4	.0062	.035	1.92	
	G.W.B	28800	42.2	37.0	8.52	1.92		38.3	.0083	.050	1.81	
	S.D.	11800	40.5	36.5	9.61	1.81		86.2	.0036	.021	1.77	
	YONK	22900	40.6	36.5	8.78	1.77		44.6	.0070	.040	1.68	
	D.F.	27000	34.4	33.3	9.00	1.68		27.2	.011	.062	1.55	9.79
												1.55
												11.32
(10)	BATT				8.50							
	W.N.Y	28800	53.2	41.5	8.69	1.3	0	60.9	0	.021	1.27	
	G.W.B	28800	44.1	37.8	8.81	1.27		41.9		.030	1.23	
	S.D.	11800	42.3	37.0	8.90	1.23		94.0		.013	1.22	
	YONK	22900	42.3	37.0	9.07	1.22		48.4		.025	1.19	
	D.F.	27000	36.0	34.0	9.29	1.19		29.8		.040	1.14	11.32
												1.14
												12.46

	REACH	X	A	C	t	F	F'	H $620\frac{l^2}{x}$	$\frac{3xF'}{2CR}$	$\frac{|F|}{h}$	New F	
(1)	BATT				9.10							
		28800	53.2	41.5		-2.3	+20.4 $\times 10^{-4}$	60.9	+.040	.038	-2.12	
	W.N.Y				9.29							
		28900	44.1	37.8		-2.12		41.9	+.053	.051	-1.90	
	G.W.B.				9.51							
		11800	42.3	37.0		1.90		94.0	+.023	.020	-1.82	
	S.D.				9.60							
		22900	42.3	37.0		-1.82		48.4	+.045	.038	-1.67	
	YONK				9.77							
		27000	36.0	34.0		-1.67		29.8	+.068	.056	-1.46	
	D.F.				10.0							12.46 − 1.46 = 11.00
(2)	BATT				9.40							
		28800	50.9	40.5		-2.3	25.0 $\times 10^{-4}$	55.8	.052	.041	-2.09	
	W.N.Y.				9.60							
		28800	42.0	37.0		-2.09		38.0	.069	.055	-1.83	
	G.W.B.				9.81							
		11900	40.4	36.3		-1.83		85.8	.030	.048	-1.69	
	S.D.				9.90							
		22900	40.4	36.3		-1.69		44.2	.059	.038	-1.53	
	YONK				10.08							
		27000	34.5	33.3		-1.53		27.3	.088	.056	-1.31	
	D.F.				10.30							11.00 − 1.31 = 9.69

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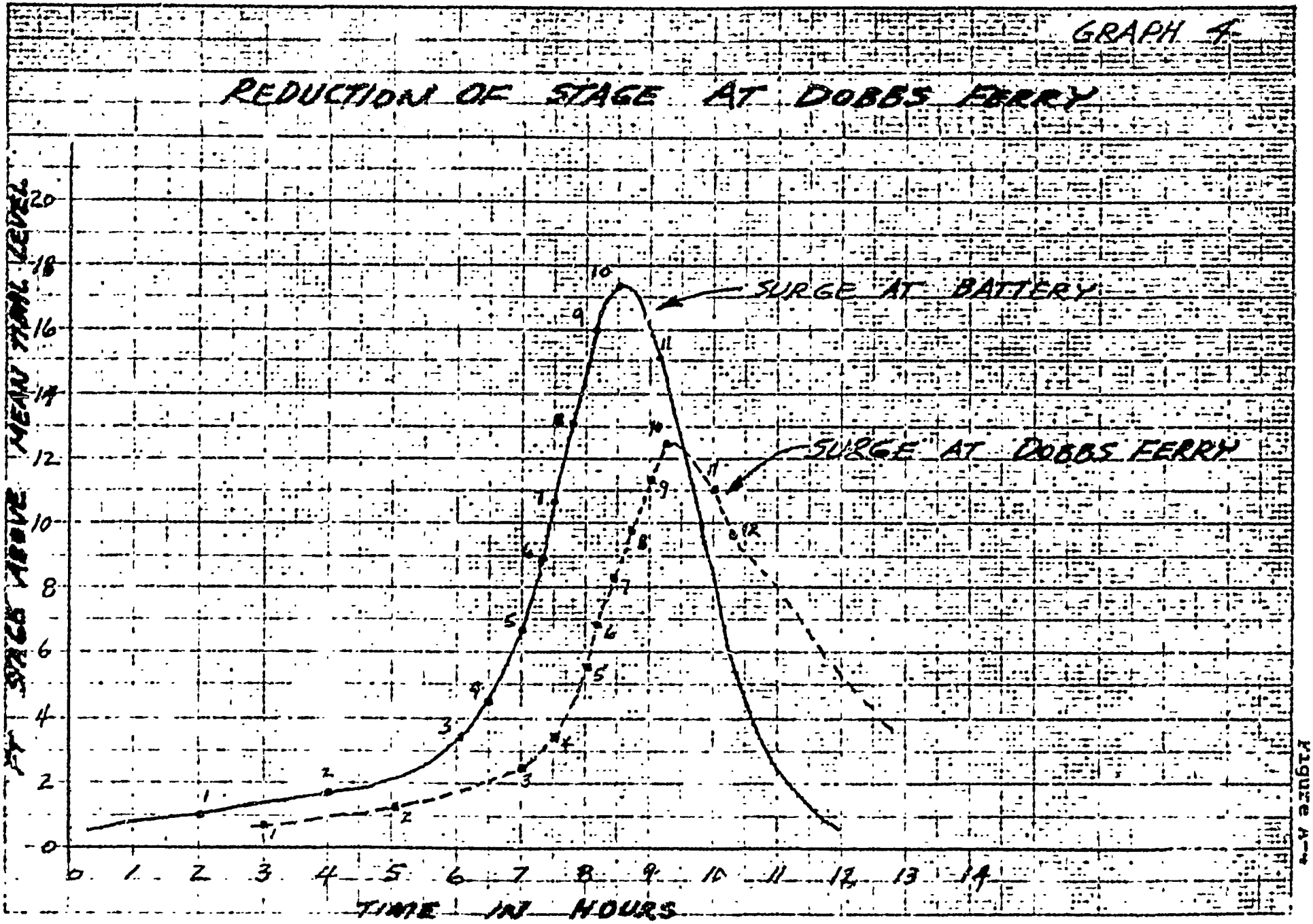

GRAPH 4
REDUCTION OF STAGE AT DOBBS FERRY
SURGE AT BATTERY
SURGE AT DOBBS FERRY
FT. STAGE ABOVE MEAN SEA LEVEL
TIME IN HOURS
0
2
4
6
8
10
12
14
16
18
20
1
2
3
4
5
6
7
8
9
10
11
12

Figure 4

EVALUATION OF SURGE STAGE FROM DOBBS FERRY TO INDIAN PT.

SEE PRODDANNI'S PAPER, EQ 31

$$\eta = PF\{1 - IF - JF' \cdots\}$$

HIGHER ORDER TERMS NEGATIVE SIGN

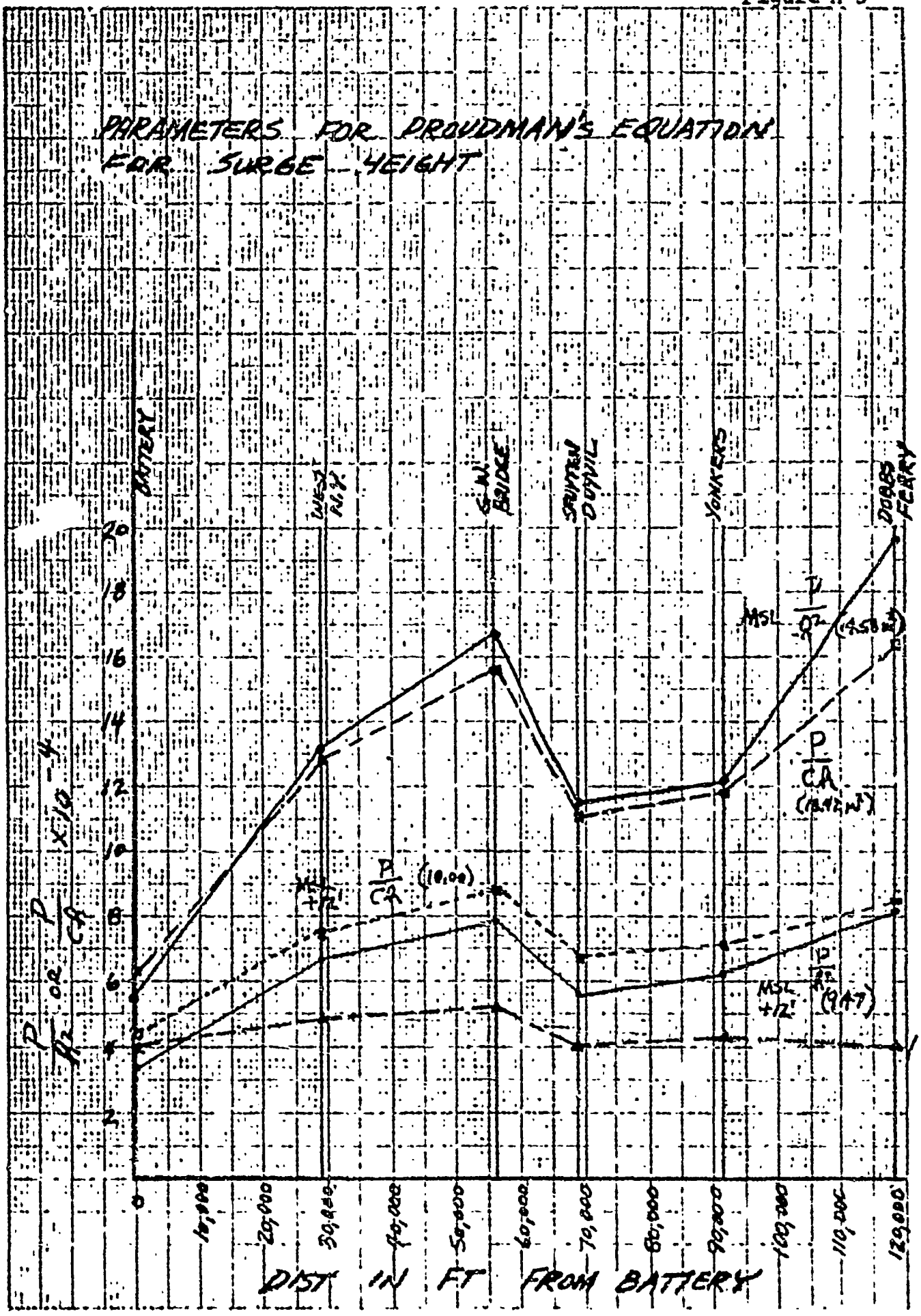
$$I = \frac{5}{6}\left[\frac{P}{R}\right]_0^x + \frac{1}{2}\int_0^x \frac{P'}{R}\,dx + \frac{K}{2}\int_0^x \frac{P}{R^2}\,dx$$

$$J = \frac{3}{2}\int_0^x \frac{P}{CR}\,dx$$

INTEGRALS MUST BE COMPUTED

REACH	S.C	$(\frac{17.5}{30})^{1/2}$ P	A FT	P/A	P/A^2	P/CA	X	C
BATTERY	17.5×10^4	1.00	42.5	.0235	.000553	.000632	0 FT	37.2 FPS
WEST N.Y.	11.8×10^6	1.21	30.2	.0400	.00132	.00129	28,800	31.0
G. W. BRIDGE	10.3	1.30	27.9	.0465	.00167	.00156	56,900	29.8
SOUTH DOCK	16.4	1.03	29.9	.0344	.00115	.00111	68,600	30.9
YONKERS	14.7	1.09	29.8	.0365	.00122	.00118	91,500	30.8
DOBBS FERRY	17.0	1.01	26.7	.0444	.00196	.00164	116,500	27.0
		$(\frac{19.3}{30})^{1/2}$	ABOVE @ 115 SL					
	$\times 10^4$							
BATTERY	19.8	1.00	54.5	.0183	.000336	.000436	ABOVE	42.0
WEST N.Y.	14.1	1.195	42.2	.0280	.000665	.000758		37.0
G.W. BRIDGE	12.4	1.263	39.9	.0316	.000793	.000879		36.0
SOUTH DOCK	19.1	1.013	41.9	.0242	.000579	.000674		37.0
YONKERS	17.1	1.076	41.8	.0257	.000615	.000715		36.9
DOBBS FERRY	21.1	0.969	34.7	.0279	.000804	.000933		33.5

Figure A-5



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EVALUATION OF PROUDMAN'S PARAMETERS
BATTERY TO DOBBS FERRY

$$I = \frac{5}{8}(.0444 - .0235) + \frac{1}{2}\int_0^x \frac{P'}{A}\,dx \pm \frac{K}{2}\int_0^x \frac{P}{A^2}\,dx$$

$$I = \frac{5}{8}(.0209) + \frac{K}{2}(156) = .0131 + \frac{.00322}{2}(156) = \underline{0.264}/\text{FT}$$

@ M.T.L.

$$J = \frac{3}{2}\int_0^x \frac{P}{CA}\,dx = \frac{3}{2}(147.2) = \underline{221.0}\ \frac{\text{SEC}}{\text{FT}^2}$$ @ M.T.L.

$$I = \frac{5}{8}(.0279 - .0183) + \frac{.00322}{2}(75.6) = .006 + .122$$

$$I = \underline{0.128}/\text{FT}$$ @ M.T.L. + 12'

$$J = \frac{3}{2}(85.5) = \underline{128}\ \frac{\text{SEC}}{\text{FT}}$$ @ M.T.L. + 12'

STEP	F	P	F'	I	J	IF	JF'	SPACE	ROUTING METHOD
1	1.0	1.01	.93	0.250	210	.250	.0195	.730	.798
2	.67	1.01	.93	0.245	205	.138	.0129	.570	.576
3	1.80	1.01	6.1	0.125	190	.340	.117	1.158	1.15
8	2.6	.969	13.0	0.105	115	.300	.150	1.440	1.63
9	2.0	.969	3.0	0.800	115	.190	.028	1.563	1.55
10	1.3	.969	0	0.700	95	.90	0	1.180	1.14

ABOVE VALUES
INTERPOLATED

CONCLUSION: ROUTING PROCEDURE & DIRECT USE OF PROUDMAN'S EQUATION GIVE CLOSE RESULTS.

HAVERSTRAW BAY PARAMETERS

REACH	AREA SURFACE BETWEEN SECTIONS	SURFACE WIDTH	HYDRAULIC DEPTH M.L.W.	DISTANCE BETWEEN SECTIONS
DOBBS FERRY	324×10^6 FT^2	6300 FT	21'	3.8 M ≈ 20,300 FT
TARRYTOWN	312×10^6	14700 FT	14'	4.5 M ≈ 23,800
SLEEPY HOLLOW	429×10^6	10,900	16'	6.1 M ≈ 32,200 FT
BOWLINE	147×10^6	13600	16'	2.2 M ≈ 11,600
MONTROSE	67.0×10^6	5950	26'	2.3 ≈ 12,100
TRAP ROCK	33.9×10^6	4000 FT	38'	1.4 ≈ 7,400
INDIAN PT.		4050 FT	40'	

$\leq 1313 \times 10^6$ SQ FT

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REACH	$AC \times 10^4$	P	A_{FT}	P/A^2 $1/FT^2$	P/CA sec/ft	X_{FT}	C_{FPS}
[illegible] FERRY	19.03	1.00	28.7	.00121	.00115	ABOVE	30.2
TRAPYKEN	38.50	.703	21.6	.00151	.00124		26.2
SCARBOROUGH	29.76	.800	23.5	.00145	.00125		27.3
CROTON PT	20.45	.965	31.5	.000972	.000966		31.7
BOWLINE	36.99	.717	23.4	.00131	.001126		27.2
[illegible]	19.52	.987	33.5	.000879	.000898		32.8
TRAP ROCK	15.40	1.11	45.6	.000536	.000630		38.5
IND. PT.	15.84	1.10	47.6	.000484	.000587		39.1

Figure A-6

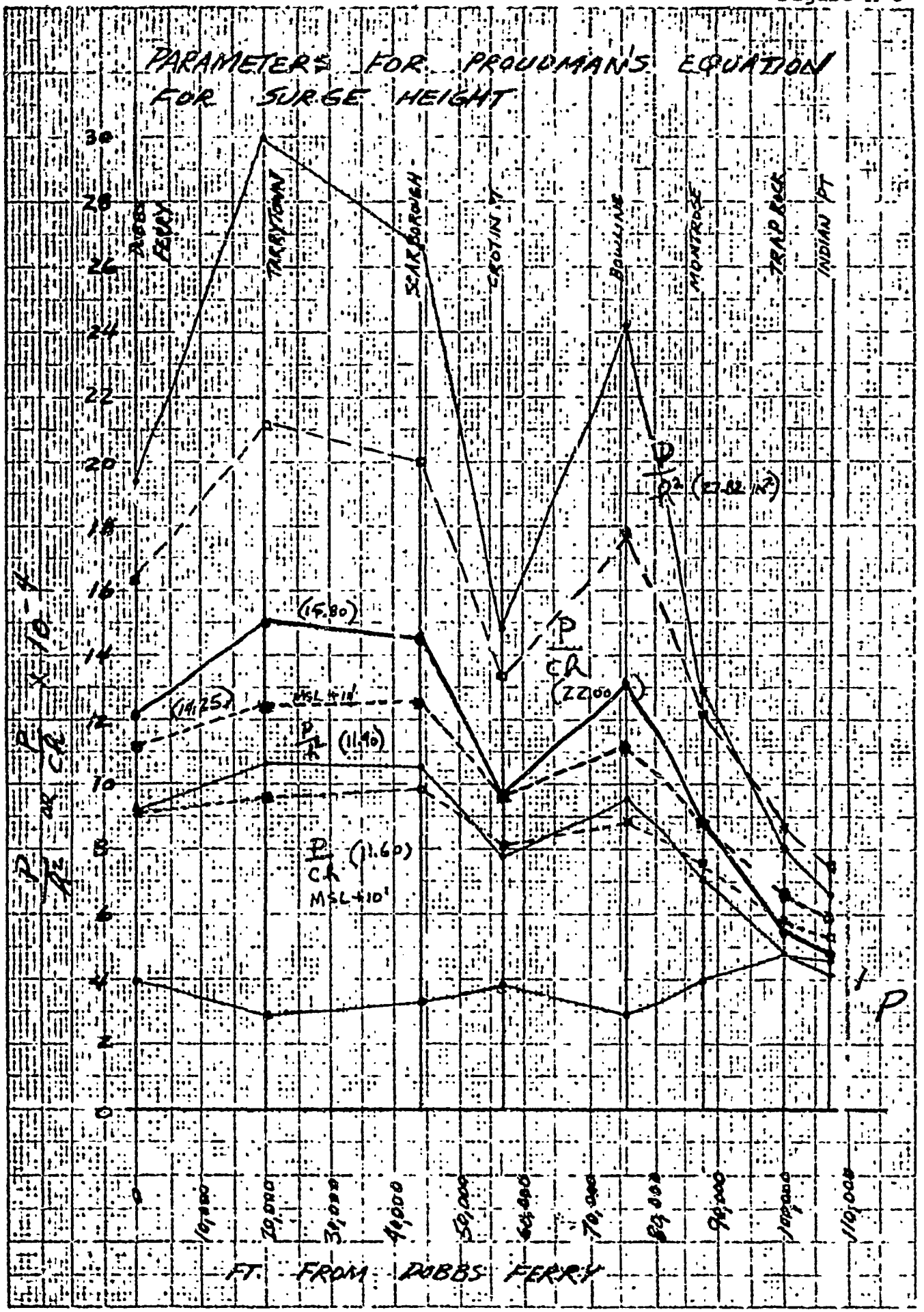
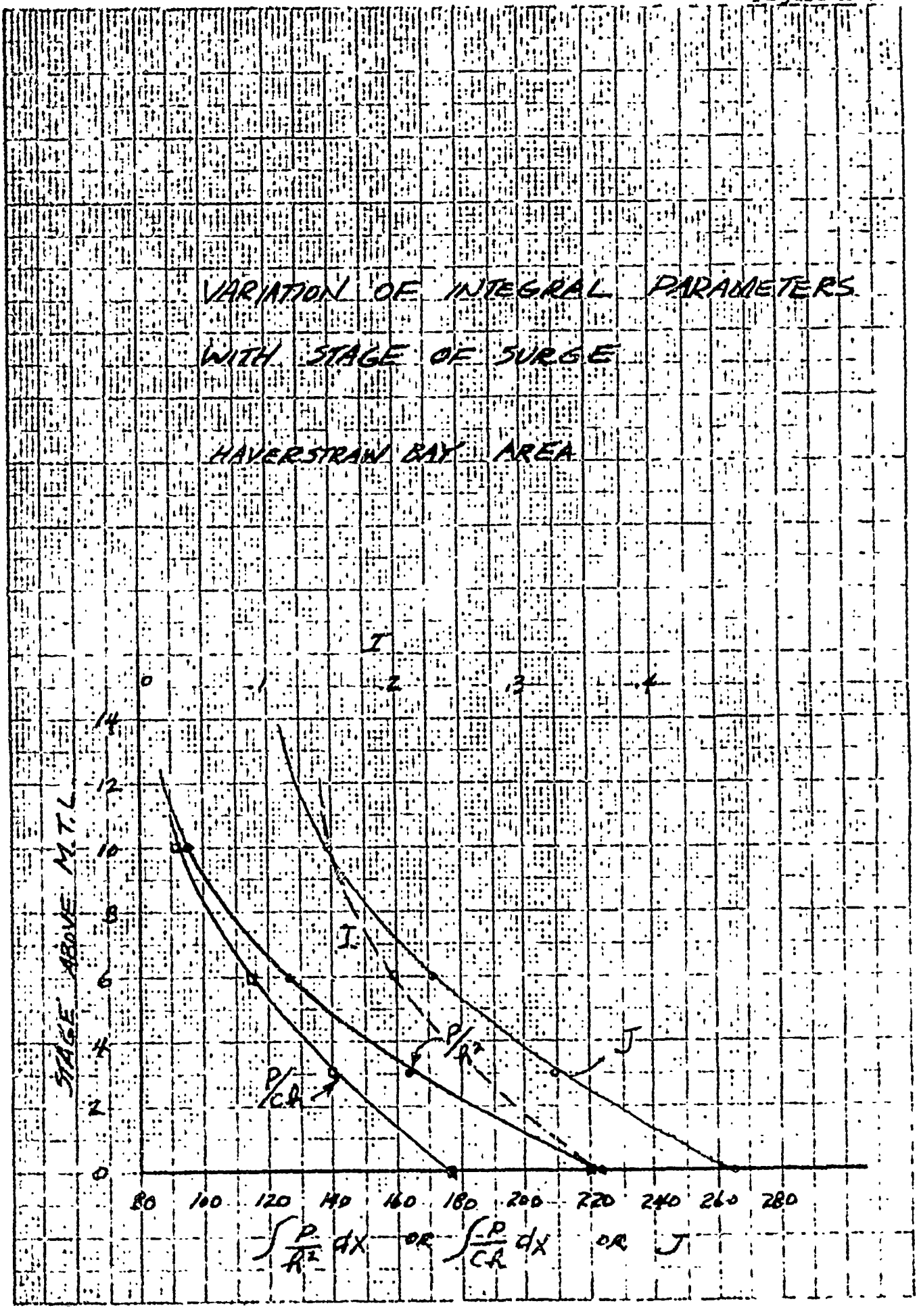

PARAMETERS FOR PROUDMAN'S EQUATION FOR SURGE HEIGHT
ROSS FERRY
TARRYTOWN
SCARBOROUGH
CROTON P.
BOWLINE
MONTROSE
TRAP ROCK
INDIAN PT
(14.80)
(14.25)
MSL+10'
(11.90)
(11.60)
MSL+10'
(22.00)
(17.82)
FT. FROM DOBBS FERRY

Figure A-1


VARIATION OF INTEGRAL PARAMETERS
WITH STAGE OF SURGE
HAVERSTRAW BAY AREA
I
J
P/R²
P/CR
STAGE ABOVE M.T.L.
0
1
2
3
4
14
12
10
8
6
4
2
0
80
100
120
140
160
180
200
220
240
260
280
∫ P/R² dx OR ∫ P/CR dx OR J

10 X 10 TO ½ INCH 46 1322
7 X 10 INCHES
KEUFFEL & ESSER CO

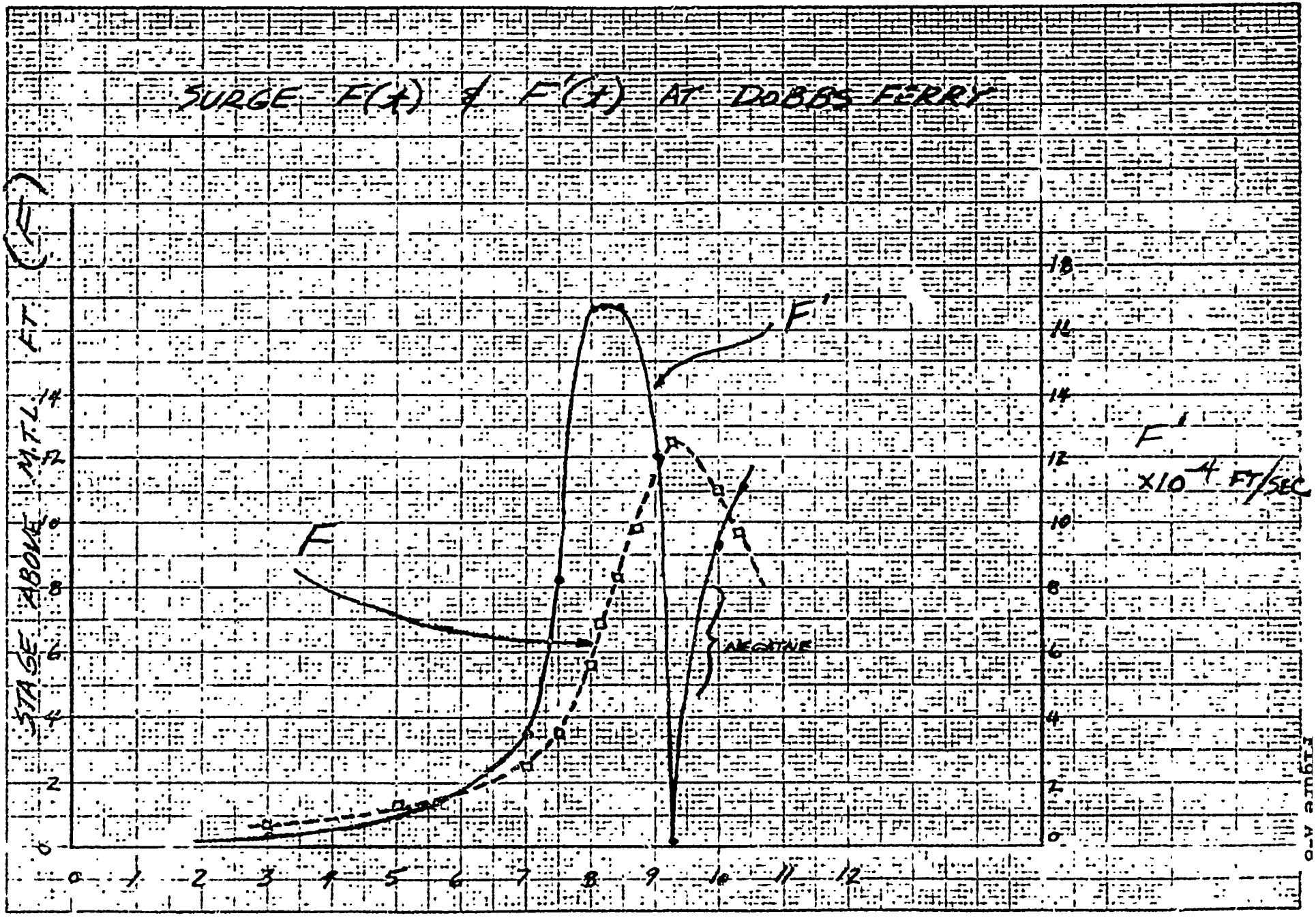


Figure A-8

COMPUTATION FOR SURGE REDUCTION AT INDIAN PT.

STEP	F	P	F'	I	J	IF	JF'	1-IF-JF'	ROOTED STAGG
1	1.0	1.14	0.7×10^{-4}	.320	245	.320	.0173	.663	.756
2	1.0	1.13	2.4×10^{-4}	.290	228	290	.055	.655	.740
3	1.0	1.13	5.4	.265	210	.265	.113	.622	.701
4	1.0	1.12	11.0	.240	196	.240	.216	.544	.607
5	1.0	1.12	15.6	.215	183	.215	.286	.501	.561
6	1.0	1.11	16.8	.196	171	.196	.286	.518	.575
7	1.0	1.11	16.8	.180	160	.18?	.270	.550	.610
8	1.0	1.11	16.8	.167	152	.167	.256	.577	.640
9	1.0	1.11	16.8	.155	145	.155	.244	.601	.667
10	1.0	1.10	16.0	.144	139	.144	.243	.613	.679
11	1.0	1.10	13.0	.140	133	.140	.173	.687	.750
12	1.0	1.10	4.0	.140	129	.140	.052	.808	.893
13	0.5	1.10	0.0	.140	127	.070	.000	.930	.513

$\Sigma = 8.651$

$I_{@\,HTL} = \frac{5}{8}(.0163) + .0016(224) = .350$

$I_{@\,HTL+6} = \frac{5}{6}(.0118) + .0016(126) = .146$

$I_{@\,HTL+10} = \frac{5}{3}(-.001) + .153 = .144$

$J_{@\,HTL} = \frac{3}{2}(176) = 265$

$J_{@\,HTL+6} = \frac{3}{2}(114) = 171$

$J_{@\,HTL+10} = \frac{3}{2}(92.6) = 139$

NONE H.T.L. 8.687 [illegible]

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COMPUTATION FOR AVERAGE FETCH @ VERPLANCK PT

LINE	(A) DIST	ANGLE FROM MAIN WIND α	(B) ARC	(C) $\cos\alpha$	(A)×(B)×(C)
(1)	5.4 MILES	10°	10°	.984	53.2
(2)	11.5	3°	5.5°	.999	63.0
(3)	10.2	2°	4.5°	.999	46.0
(4)	6.0	9°	8.5°	.987	50.5
(5)	5.7	19.5°	11.5°	.942	61.6
			40.0°		Σ = 274.3

$$\frac{274.3^\circ \text{ MILES}}{40^\circ} = 6.85 \text{ MILES}$$

AVERAGE FETCH LENGTH

36,000 FT

WAVE HEIGHT COMPUTATION

HURRICANE WIND MAX ≈ 124-136 MPH

FACTOR OF REDUCTION INLAND ≈ .7

FORWARD SPEED OF HURRICANE 8 TO 48 KNOTS

WAVE HEIGHT COMPUTATION

EFFECTIVE FETCH LENGTH = 6.85 MILES

FETCH LENGTH / FORWARD SPEED = 6.85 MILES / 8×1.15 MPH TO 6.85 MILES / 48×1.15 MPH

= .743 HRS TO .124 HRS
WIND ACTION OVER FETCH

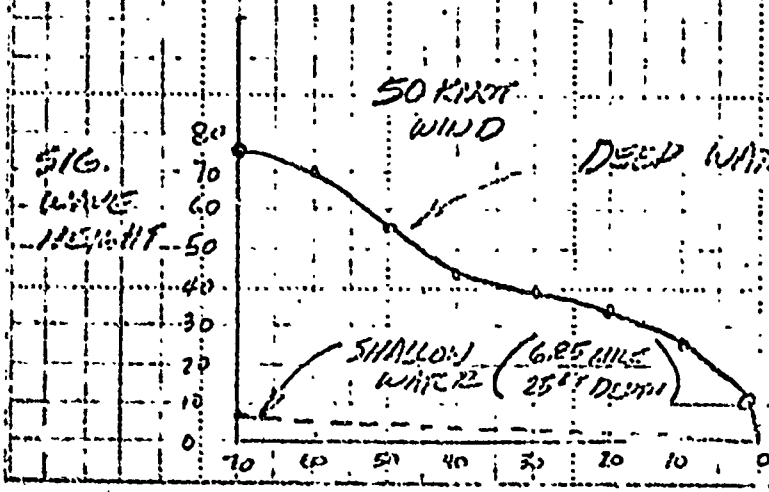
DURATION OF THE HIGH SPEED WINDS WILL BE TOO SHORT TO TRANSFER MAXIMUM AMOUNT OF TOTAL ENERGY TO FROM WIND TO WATER WAVE

FOR EXAMPLE A 65 MPH WIND WOULD REQUIRE 88 HOURS TO FULLY DEVELOP WAVE STATE

A 11 MPH WIND = 2.4 HRS

AS AN EXAMPLE OF MAGNITUDE:

A 50 KNOT/HR WIND IN THE OPEN SEA FULLY DEVELOPS A 77 FT SIGNIFICANT WAVE IN 70 HRS THIS APPROACHES A SIGNIFICANT WAVE OF 10 FT FOR SHORT TIME DURATIONS



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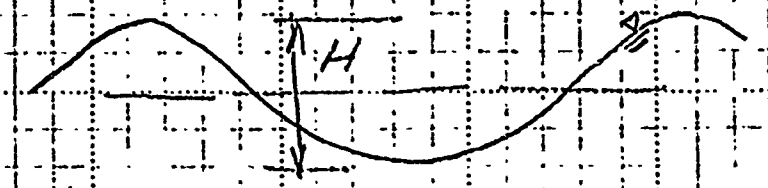
REACH	θC $\times 10^4$	P	h ft	P/h^2 $1/ft^2$	P/Ch sec/ft^2	X ft	C fps
DOBBS FERRY	17.0	1.00	22.7	.00194	.00163	0	27.0
TARRYTOWN	32.3	.725	15.6	.00298	.00211	20,3000	22.0
SCARBOROUGH	25.6	.815	17.5	.00266	.00198	44,100	23.5
CROTON PT.	18.3	.964	25.5	.00148	.00133	56,800	28.3
BOWLINE	31.9	.730	17.4	.00241	.00178	76,300	23.5
[illegible]	17.6	.983	27.5	.00129	.00121	87,900	29.5
TRAP ROCK	14.3	1.20	39.6	.000790	.000848	100,000	35.8
INDIAN PT	14.9	1.140	41.6	.000658	.000746	107,400	36.7
D.F.	20.5	1.00	32.7	.000935	.000940	ABOVE	32.5
T.T.	41.9	.699	25.6	.00106	.000958		28.5
SCAR	32.3	.797	27.5	.00105	.000978		29.6
C. PT	21.9	.968	35.5	.000767	.000801		34.0
BOW	40.0	.716	27.4	.000953	.000885		29.5
MONT.	20.8	.993	37.5	.000705	.000756		35.0
T.R.	15.9	1.135	49.6	.000460	.000573		40.0
IND. PT.	16.5	1.114	51.6	.000418	.000529		40.8

CONCLUSION ABOUT WAVES AT INDIAN PT IS THAT THEY SHOULD BE MINOR ON THE ORDER OF A 2 FOOT DEPTH

SAY $H_{\frac{1}{3}} \approx 2.0$ FT



∴ WAVE HEIGHT ≈ 1.0 FT

BY MD DATE ENVIRONMENTAL SCIENCE & ENGINEERING CONSULTANTS SHEET NO.

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SUBJECT

SUMMARY OF HURRICANE ROUTING

CRITICAL SURGE STAGE @ BATTERY	15.6 FT	
STAGE INCREASE DUE TO PRESSURE REDUCTION	+ 1.9 FT	
	17.5 FT	ABOVE M.S.L. or M.T.L. @ BATTERY
	ROUTED	
SURGE @ INDIAN PT. M.T.L.	8.8 FT	
ADDITIONAL MAGNITUDE OF WIND SURGE UP HUDSON ESTUARY	+ 1.0 FT	
ESTIMATE OF SPRING HIGH TIDE @ INDIAN PT ABOVE MTL	+ 3.0	
	12.8 FT. M.T.L.	
ADDITIONAL ESTIMATE FOR WIND WAVES (PERIOD ≈ 7–9 SEC $H_{1/3}$ ≈ 2')	1.0 FT	
ESTIMATED MAX STAGE	13.8 FT M.T.L. @ I.P.	
	OR	
	13.8	
	4.7	
	14.5 FT M.S.L.	

FROM P.J.L. 1-6-51 COMP

1/3 2/3 = 3.3

.57 / .43 = 2.8

6.1

3.05

APPROXIMATE CHECK — ESTUARY ASSUMED UNIFORM FROM BATTERY TO INDIAN POINT

ASSUME $P \approx 1$. $h \approx 30$ FT INITIAL

$F' \approx 7 \times 10^{-4}$ FT/SEC $X = 42(5280)$ FT

$$\eta = F \left\{ 1 - \frac{3XF'}{2ch} - \frac{|F|}{H} \right\}$$

$$H = 620 \frac{h^2}{X} = \frac{620}{42(5280)} h^2 = .00279 h^2$$

$$\frac{3XF'}{2ch} = \frac{3(42)(5280)(7 \times 10^{-4})}{2\sqrt{g}\ \sqrt{h}\ h} = \frac{40.9}{\sqrt{h}\ h}$$

USING 17.5 INCREMENTS OF 1.0 FT ΔF

$$\sum_{0}^{17.5} \frac{1.0}{.00279 h^2} = 4.52$$

$$\sum_{0}^{17.5} \frac{40.9}{\sqrt{h}\ h} = 3.12$$

h is incremented by + 1.0 ft in each step

$\eta = 9.9$ FT BASED ON $h \approx 30'$
$P \approx 1$
$X = 42(5280)$ FT

ok 8.8 is therefore a good value

COMPUTATION FOR FLOOD FLOW AT INDIAN POINT

RAINFALL COMPUTED BY U.P.L. TO BE 14 IN/72 HRS FROM STUDY OF SUSQUEHANNA RIVER DRAINAGE BASIN

$\frac{14 \text{ IN}}{72 \text{ HRS}} = 0.195 \text{ IN/HR}$

$0.195 \text{ IN/HR} \times 10,500 \text{ SQ MILES} \approx 1,320,000 \text{ CFS}$

DRAINAGE AREA

USING RUNOFF COEFFICIENT

$C \approx 0.1 \rightarrow 0.3$

GOOD — HIGH — WOODED AREAS & FARM LANDS

$1,320,000 \text{ CFS} \times 0.3 \approx 397,000 \text{ CFS}$

ESTIMATED MAXIMUM RUNOFF

OR 37.8 CFS/SQ MILE

GREATEST RECORDED FLOOD @ GAUGE STATION

131,000 CFS Dec 31, 1948 — 22.4 CFS/SQ MI

215,000 CFS March 19, 1936 — 26.6 CFS/SQ MI

DRAINAGE AREA 8090 SQ MI

ASSUMING MAXIMUM EBB TIDE & DAM FAILURE FLOWS ARE O.K.

275,000 CFS EBB TIDE FLOW
785,000 CFS DAM FAILURE ESTIMATE
397,000 CFS MAXIMUM RAINFALL

$\Sigma = 1{,}457{,}000$ CFS TOTAL FLOW

$n \approx 0.03$ WIDTH OF CHANNEL $\approx$ 4000 FT
$S \approx 0.0001$

$$Q = \frac{1.49}{n} A (R_H)^{2/3} S^{1/2}$$

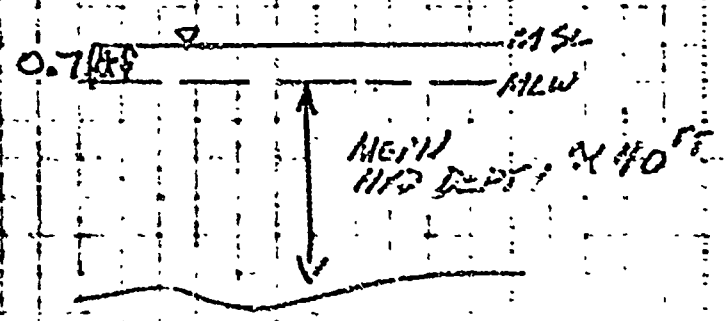
$R_H \rightarrow D$ IN RECT. CHANNEL

$$1.457 \times 10^6 \text{ CFS} = \frac{1.49}{0.03}(4000) D (D)^{2/3} (.0001)^{1/2}$$

$$732 = D^{5/3}$$

$(53)^{5/3} \approx 750$
$(52.5)^{5/3} \approx 740$
$(52.3)^{5/3} \approx 730$

$D \approx 52.4$ FT.

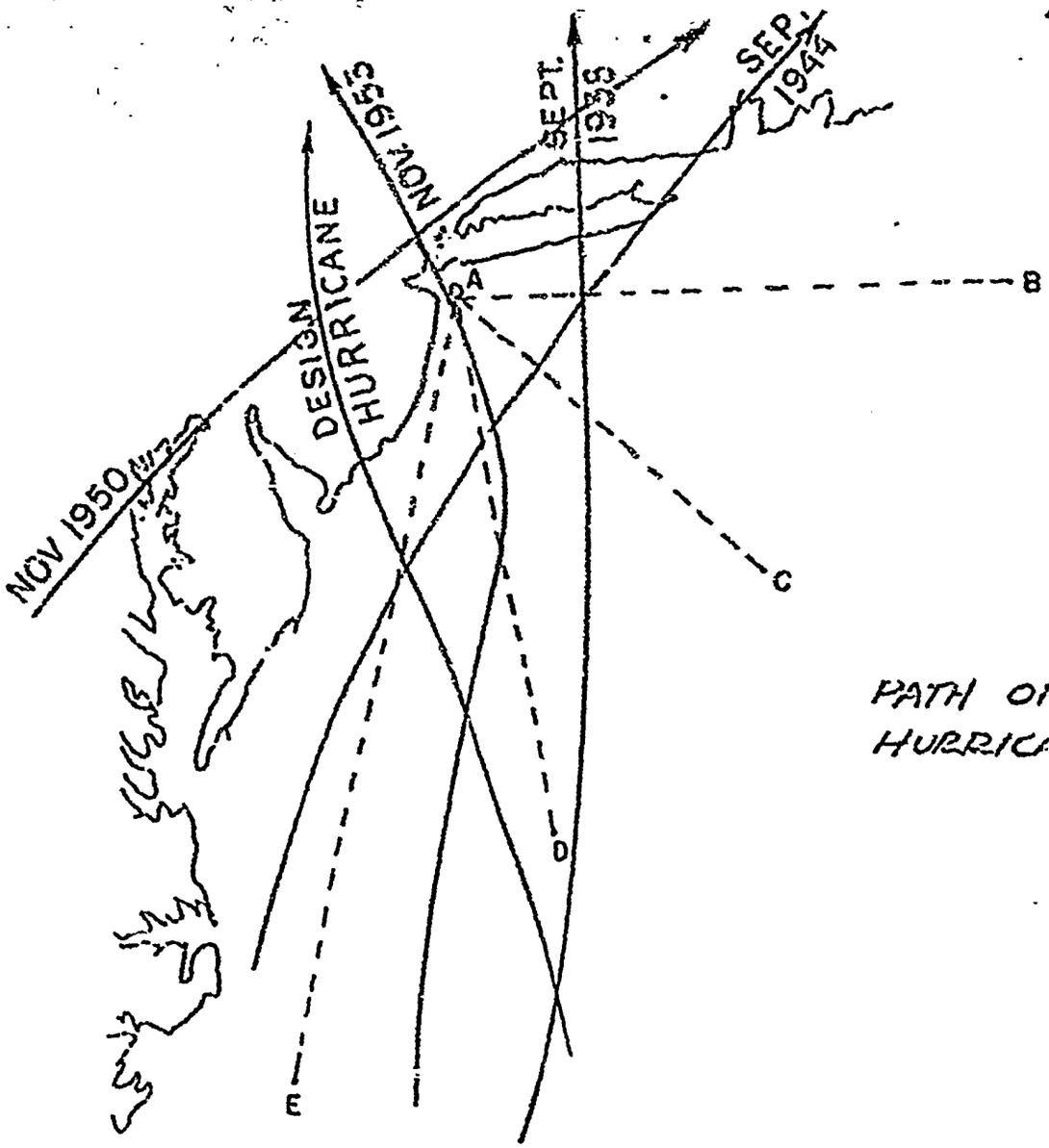


52.4 FT
− 40.7
11.7 FT ABOVE M.S.L

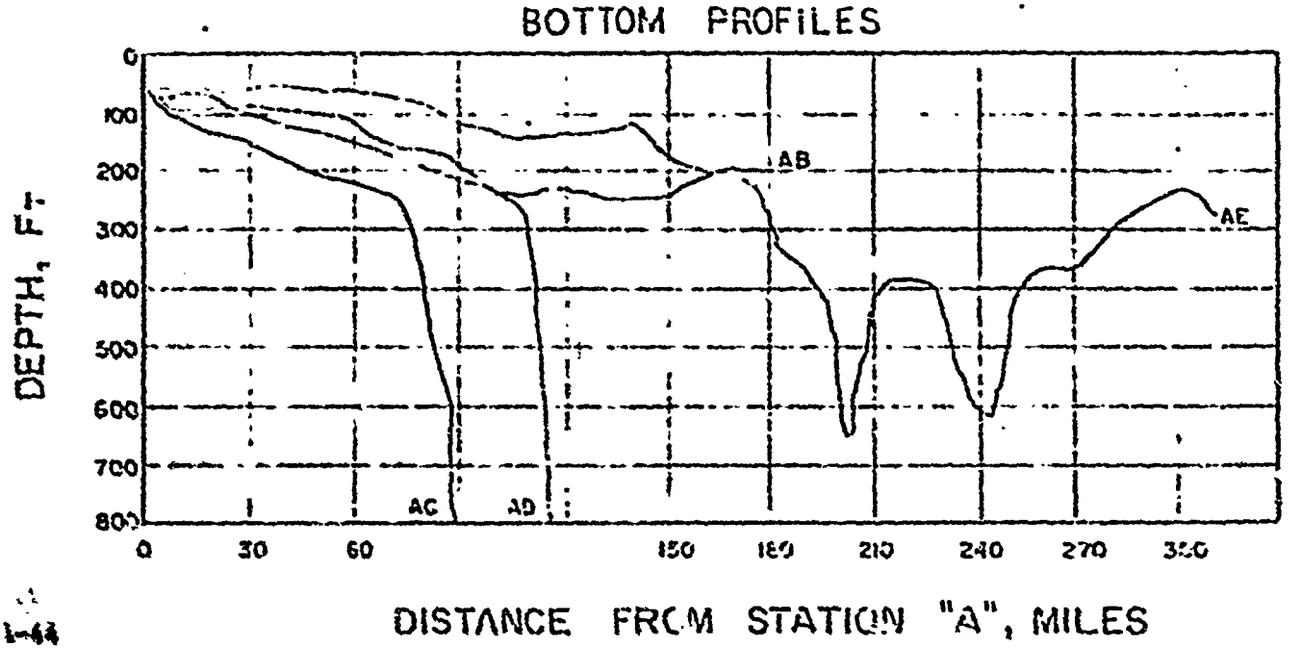
CONCLUSION FLOOD + EBB TIDE + DAM FAILURE
11.7 FT ABOVE M.S.L.

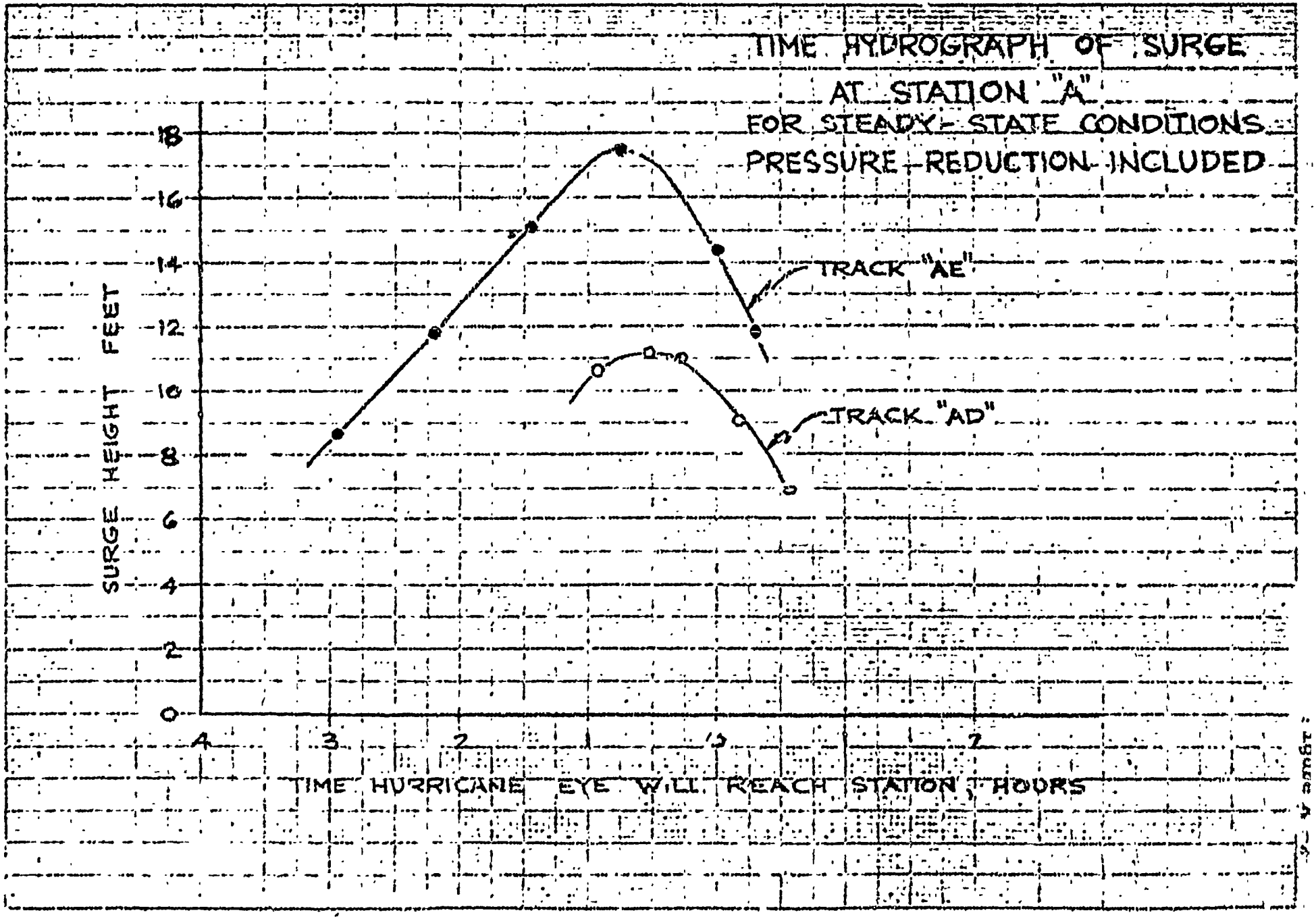
APPENDIX A'

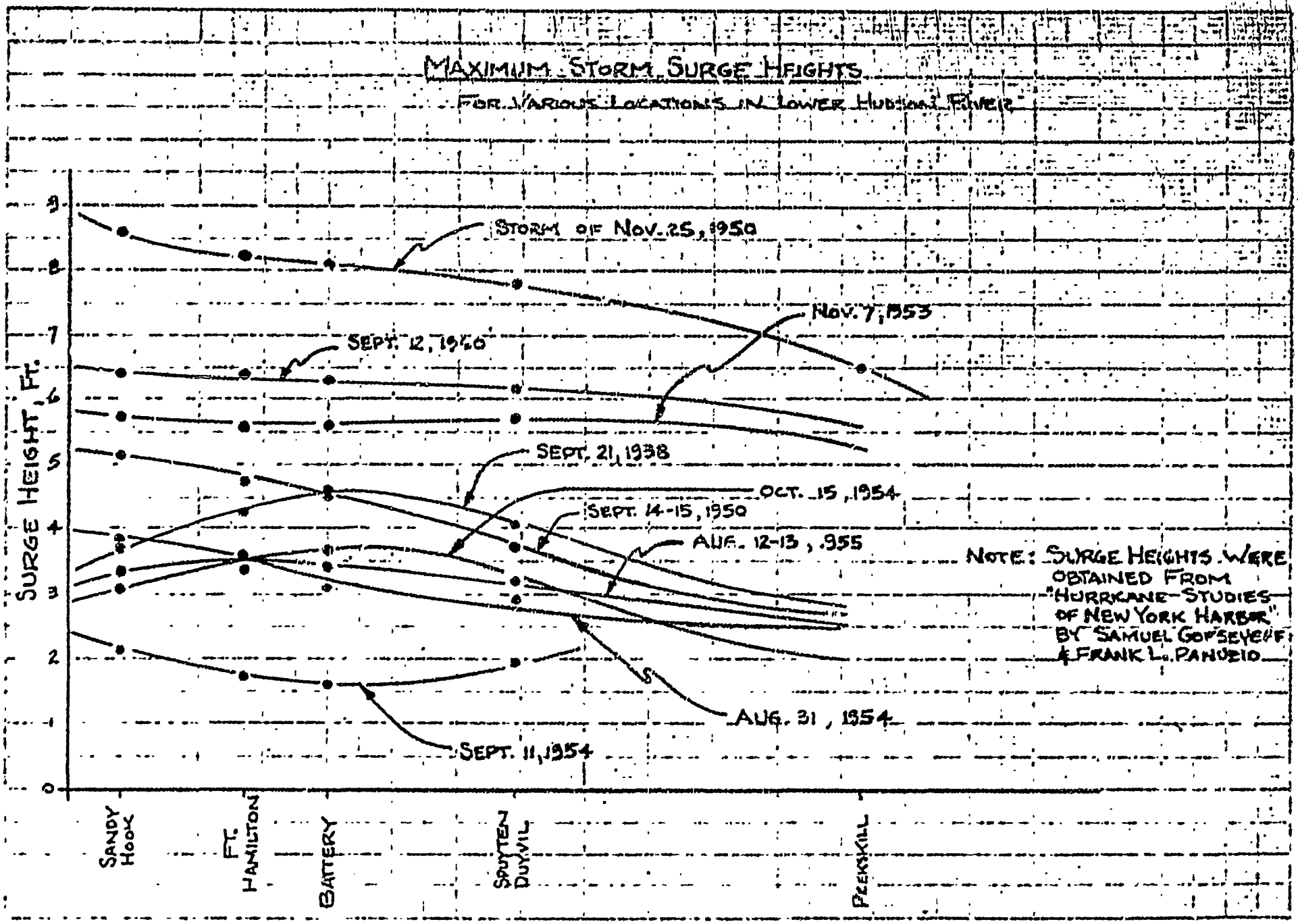
Selected Calculations and Figures from the Preliminary Report



PATH OF DESIGN HURRICANE




TIME HYDROGRAPH OF SURGE
AT STATION "A"
FOR STEADY-STATE CONDITIONS
PRESSURE-REDUCTION INCLUDED
TRACK "AE"
TRACK "AD"
SURGE HEIGHT FEET
0
2
4
6
8
10
12
14
16
18
4
3
2
1
0
1
2
TIME HURRICANE EYE WILL REACH STATION, HOURS


MAXIMUM STORM SURGE HEIGHTS
FOR VARIOUS LOCATIONS IN LOWER HUDSON RIVER
SURGE HEIGHT, FT.
0
1
2
3
4
5
6
7
8
9
STORM OF NOV. 25, 1950
NOV. 7, 1953
SEPT. 12, 1960
SEPT. 21, 1938
OCT. 15, 1954
SEPT. 14-15, 1950
AUG. 12-13, 1955
AUG. 31, 1954
SEPT. 11, 1954
NOTE: SURGE HEIGHTS WERE OBTAINED FROM "HURRICANE STUDIES OF NEW YORK HARBOR" BY SAMUEL GOFSEYEFF & FRANK L. PANUZIO
SANDY HOOK
FT. HAMILTON
BATTERY
SPUYTEN DUYVIL
PEEKSKILL

APPENDIX B

FLOOD STUDY

OF

UPPER HUDSON RIVER BASIN

MARCH 21, 1969

STONE & WEBSTER ENGINEERING CORPORATION

BOSTON, MASS.

REPORT ON
FLOOD STUDY
UPPER HUDSON RIVER BASIN

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Fig. No. 2 - Profile - Hudson River Above Troy
Fig. No. 3 - Probable Maximum Flood Hydrograph
Fig. No. 4 - Stage Discharge Curve
Fig. No. 5 - Discharge Frequency Curves

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 for the two conditions is as follows:

1. The probable maximum flood would have a peak flow of 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 about 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. 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 with reasonable 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 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 Fort 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 in. of precipitation which fell as a combination of rain, snow and sleet. The estimated maximum mean daily flow at Stillwater was 61,500 cfs. The significantly lower flood flow at the Stillwater in comparison

with the 1913 storm is due in 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, the discharge of the Sacandaga during March 1913 reached a peak of 35,500 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 the watershed are shown on Table 2.

PROBABLE MAXIMUM FLOOD

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 floods. 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 definiton 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 hydrologic 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.

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.

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 Hydrometeorological 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.

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 th se 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.

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
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 I - Principles and Determinations,"[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 Determinations."

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 and Schwarz,(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.

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 each 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 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:

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	48,000	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.

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 stage 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 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

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 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. Stage-discharge-volume relationships for the various reaches of the river

d. X and K, which are constants used to determine routing coefficients

The methods used to determine the above parameters will be described briefly.

Hydraulic Characteristics

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 1938. 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 Champlain Canal. Use was made of all available

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. 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.

Manning'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 stage-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 data 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.

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 judgment 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 furthered assumed that the volume below the fixed dam crest under consideration could be ignored and that only the volume above the crest was usable.

X and K

The interrelated constants X and K, used to determine the coefficicents 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

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 Mechanicville Dam, because it was not known initially where river

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 1/2 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.

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 Sacandaga Reservoir behind the Conklingville Dam, with a volume greatly in excess of all other reservoirs combined. 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.

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 El. 752.2 at the time. 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 two 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 overtopped 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 Conklingville 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

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.

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 additonal 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], Schoklitsch [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 by 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 the hypothetical failure of Conklingville Dam, the channel cross section at the Dam was approximated by three rectangles with a total area equaling that

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

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 2½ 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 make 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 Sacandaga would divide when it reached the Hudson River and flow

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 complexity 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 riverbank runoff from the probable maximum precipitation were combined with the dam break wave in the proper time sequence as the wave was routed.

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.

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 event, 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 data 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 to 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 intervals[28].

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
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 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[11]. 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 at 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 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 stage 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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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
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
Fort Edward Dam	Niagara Mohawk Power Corporation
Crockers Reef	State of New York

Dam	Owner
Fort Miller Dam	Fort Miller Pulp & Paper Company
Thomson Dam	United Paperboard Corporation, Thomson Paper Company
Stillwater Dam	Niagara Mohawk Power Corporation
Upper Mechanicville Dam	State of New York, West Virginia Pulp and Paper Company
Winnies Reef	Niagara Mohawk Power Corporation

Table No. 1

REPRESENTATIVE STREAM GAGING STATIONS
UPPER HUDSON RIVER

Location	Years of Record	Drainage Area, Square Miles	Zero of Gage	Annual Runoff					
				Maximum		Minimum		Average	
				Cfs	In.	Cfs	In.	Cfs	In.
Hudson River									
At Gooley	49	419	1,033.02	1,311	42.47	465	15.06	822	26.63
At North Creek	58	792	987.52	2,301	39.44	862	14.77	1,522	26.08
At Hadley	44	1,664	563.99	4,153	33.88	1,408	11.49	2,816	22.97
At Mechanicville	69	4,500	66.63	10,498	31.67	4,240	12.79	7,431	22.42
Indian River									
Near Indian Lake	51	132	1,604.27	415	42.68	106	10.90	285	29.31
Schroon River									
At Riverbank	58	527	699.31	1,178	30.34	319	8.22	787	20.27
Sacandaga River									
Near Hope	54	491	801.31	1,465	40.50	611	16.89	1,083	29.95
At Stewarts Bridge	58	1,055	582.00	3,020	38.86	1,122	14.44	2,090	26.89
Batten Kill									
At Battenville	43	394	369.09	980	33.76	335	11.54	696	23.97
Kayaderosseras Creek									
Near West Milton	38	90±	376.06	198	29.86	59.3	8.94	132	19.91

Table No. 2

PERTINENT FLOOD DATA FOR KEY GAGING STATIONS
UPPER HUDSON RIVER

Location	Drainage Area, Sq Mi	Maximum of Record				Period of Record
		Stage, Ft	Date	Discharge, Cfs	Date	
Hudson River						
Near Newcomb	192	11.40	1/01/49	7,440	1/01/49	1925-1965
At Cooley	419	11.20	2/18/54	15,000	1/01/49	1916-1965
At North Creek	792	12.14	12/31/48	28,900 28,400	12/31/48 3/27/13	1907-1965
At Thurman	1,533	12.5	3/27/13	46,000	3/27/13	1907-1920
At Hadley	1,664	21.21	1/01/49	42,700	1/01/49	1921-1965
At Spier Falls	2,779	18.59	3/28/13	89,100	3/28/13	1912-1922
At Mechanicville	4,500	-	-	120,000 118,000	3/28/13 1/01/49	1887-1956
Indian River						
Indian Lake Reservoir	131	1,656.75 (a)	3/28/13	-	-	1900-1965
Near Indian Lake (b)	132	7.8	3/28/13	3,460	3/28/13	1912-1913 1915-1965
Schroon River						
At Riverbank	527	12.18	3/21/36	12,100	3/21/36	1907-1965
Sacandaga River						
Near Hope	491	11.0	3/27/13	32,000 31,400	3/17/13 1/01/49	1911-1965
Sacandaga Reservoir	1,044	769.43 (c)	4/19/54	-	-	1930-1965
At Stewarts Bridge (d)	1,055	12.36	3/28/13	35,500 8,560	3/28/13 3/30/53	1907-1929 1929-1965
Batten Kill						
At Battenville	394	17.7	11/04/27	21,300	11/04/27	1922-1965
Kayaderosseras Creek						
Near West Milton	90±	10.78	3/18/36	4,710	3/18/36	1927-1965

(a) Elevation above mean sea level. Spillway crest is at El. 1651.35.
(b) Flow almost completely regulated by Indian Lake Reservoir.
(c) Elevation above mean sea level. Spillway crest is at El. 771.00.
(d) Flow completely regulated by Sacandaga Reservoir since March 1930.

Table No. 3

OBSERVED UNIT HYDROGRAPH DATA
UPPER HUDSON RIVER

Location	Storm Period	Area Sq Mi	L Mi	L_{ca} Mi	$S_s t$ Ft/Ft	Unit Duration, Hr	q_p Cfs/Sq Mi	t_p Hr	640 Cp	C_t	Infiltration Curve No.
Hudson River near Newcomb	Sept. 19-27, 1938	192	24.3	11.4	0.0049	6	10.1	33	333	6.11	87
Cedar River below Chain Lakes	Sept. 19-27, 1938	160	33.5	15.4	0.00417	6	17.9	15	269	2.30	78
Hudson River at Gooley	Sept. 18-30, 1938	419	35.0	5.0	0.0040	6	10.4	17	177	3.61	84
Indian River at Indian Lake Dam	Sept. 26-Oct. 2, 1942	131	28.0	11.2	-	6	26.3	15	395	2.67	81
Schroon River at Riverbank	Sept. 19-Oct. 15, 1938	527	55.2	21.1	0.000423	6	3.2	51	163	7.02	70
Sacandaga River near Hope	Sept. 19-27, 1938	491	32.9	7.2	0.0068	6	24.1	9	217	1.75	76
Sacandaga River at Conklingville Dam	Sept. 17-30, 1938	1,044	67.1	34.1	-	6	27.4	9	247	0.88	74
Batten Kill at Battenville	Nov. 25-Dec. 30, 1950	394	47.2	22.3	0.00161	6	15.0	21	255	2.60	75
Kayaderosseras Creek near West Milton	Sept. 19-27, 1938	90	20.4	6.4	0.0039	6	21.3	10	213	2.32	55

Table No. 4

SUBBASIN UNIT HYDROGRAPH DATA
UPPER HUDSON RIVER

Subbasin	Area, Sq Mi	L Mi	L_{ca} Mi	$S_s t$ Ft/Ft	t_p Hr	640 C_p	C_t	q_p Cfs/Sq Mi	Base Flow, Cfs	Infiltration Curve No.
1	419.0	35.0	5.0	0.0040	17	177	3.61	10.4	400	85
2A	131.0	28.0	11.2	-	15	395	2.67	26.3	100	80
2B	73.0	18.0	8.7	0.0062	9	419	1.98	46.5	80	80
3	78.7	-	-	-	3	-	-	95.0	80	85
4	90.3	24.4	13.1	0.0083	9	350	1.60	38.8	100	85
5	565.3	70.3	31.0	0.00059	59	160	5.88	2.7	200	85
6	176.7	-	-	-	3	-	-	90.4	200	75
7	130.0	-	-	-	3	-	-	80.6	150	75
8	1,044.0	67.1	34.1		9	247	0.88	27.4	800	75
9C	36.2	-	-	-	1	-	-	95.2	0	85
9E	21.3	-	-		1	-	-	125.2	0	85
9G	17.0	-	-	-	1	-	-	128.2	0	85
9L	80.0	20.7	10.6	0.0022	10	820	3.57	45.5	0	70
9M	81.5	22.4	13.6	0.0026	15	609	2.70	40.6	0	70
9N	6.6	-	-	-	3	-	-	106.1	0	70
9P	19.0	-	-	-	2	-	-	84.1	0	70
10	254.7	44.1	17.0	0.0016	15	182	2.06	12.1	100	70
11	440.6	59.0	32.2	0.0020	27	358	2.80	13.3	200	75
12	78.9	-	-	-	1	-	-	114.0	0	70

Rainfall run-off was considered instantaneous for subbasins 9A, B, D, F, H, J and K. Infiltration curve No. 90 was used to compute losses.

SUB BASIN NO.	1	2A	4	8
DRAINAGE AREA, SQ MI	419.0	131.0	90.3	1044.0
Time, Hr				
0	0	0	0	0
6	790	250	800	13,000
12	3,070	2,240	3,500	28,600
18	4,300	3,440	2,800	14,500
24	3,580	2,300	1,100	10,000
30	4,150	1,690	670	7,600
36	3,730	1,280	410	6,200
42	3,820	950	240	5,100
48	3,210	700	140	4,300
54	2,600	510	50	3,600
60	2,180	340	0	3,100
66	1,770	220		2,700
72	1,620	120		2,400
78	1,470	50		2,100
84	1,250	0		1,900
90	1,040			1,700
96	930			1,400
102	830			1,200
108	720			1,000
114	620			700
120	520			500
126	480			200
132	430			100
138	380			0
144	330			
150	280			
156	230			
162	180			
168	150			
174	120			
180	90			
186	60			
192	30			
198	0			
204				
210				
216				

SIX HOUR UNIT HYDROGRAPHS
UPPER HUDSON RIVER

%	10	11	5
40.0	254.7	440.6	565.3

Time, Hr				
0	0	0	0	0
300	520	170	24	650
1,260	1,900	1,860	48	1,400
3,640	3,085	3,490	72	1,530
1,400	2,690	4,040	96	1,480
620	2,400	4,160	120	1,370
410	2,140	4,290	144	1,200
300	1,890	3,520	168	1,030
210	1,650	2,920	192	880
150	1,440	2,320	216	770
100	1,240	2,020	240	690
70	1,050	1,710	264	620
50	890	1,410	288	560
40	740	1,110	312	520
30	630	1,020	336	470
20	550	930	360	420
10	490	840	384	370
0	435	750	408	320
	390	670	432	270
	350	580	456	220
	320	490	480	170
	290	460	504	120
	260	430	528	70
	240	410	552	20
	220	380	576	0
	200	350		
	185	320		
	170	290		
	155	270		
	140	240		
	130	210		
	120	180		
	100	150		
	80	150		
	60	140		
	35	120		
	0	0		

SUB BASIN NO.	2B	3	6	7
DRAINAGE AREA, SQ MI	73.0	78.7	176.7	130.0
Time, Hr				
0	0	0	0	0
3	300	4,000	9,000	4,400
6	750	7,500	16,000	10,500
9	1,800	3,100	7,700	6,700
12	3,400	1,500	2,950	3,250
15	2,600	500	1,300	1,650
18	1,700	250	600	790
21	1,100	100	300	380
24	900	0	100	190
27	680		50	90
30	530		0	40
33	450			20
36	380			0
39	300			
42	240			
45	180			
48	130			
51	90			
54	70			
57	50			
60	30			
63	20			
66	0			
69				
72				
75				
78				
81				
84				

SIX HOUR UNIT HYDROGRAPHS
UPPER HUDSON RIVER

9C	9E	9G	9M	9N	9P	12
36.2	21.3	17.0	81.5	6.6	19.0	78.9
0	0	0	0	0	0	0
3,450	2,690	2,180	10	170	620	9,000
2,980	1,310	1,020	40	700	1,600	4,700
880	400	320	120	250	950	2,180
290	140	110	480	120	460	700
100	40	30	1,600	70	240	250
30	0	0	3,340	50	120	100
15			2,960	30	60	40
0			2,280	20	30	0
			1,640	10	10	
			1,160	0	0	
			900			
			700			
			550			
			430			
			330			
			250			
			190			
			150			
			110			
			90			
			60			
			50			
			40			
			30			
			20			
			10			
			10			
			0			

Table No. 6

MAJOR DAMS ABOVE Stillwater
UPPER HUDSON RIVER

Dam	Type	River Miles from .	Height of Dam - Ft.	Total Storage, Million Cu Ft.
Thomson	Concrete Gravity	13.5	13.5	58
Port Miller	Timber Crib	16.5	8	50
Crockers Reef	Concrete Gravity	18.5	15	80
Fort Edward	Timber Crib Concrete Gravity	25.5	35 10	90
Bakers Falls	Concrete Gravity	28.5	23	18
Glens Falls	Multiple Arch Buttress	30.5	10	46
Feeder	Concrete Gravity	32.5	20	87
Sherman Island	Concrete Gravity Multiple Arch Buttress	40.5	52 75	177
Spier Falls	Masonry Gravity	44.5	150	287
Palmer Falls	Rubbusen	49.5	38	12
Curtis Falls	Concrete Gravity	50.5	34	98
Stewarts Bridge	Earth, Concrete Gravity	58.9	107	810
Conklingville	Earth, Concrete Gravity	60.5	96	37,800
Indian River	Masonry Gravity, Earth	116.5	43	5,000

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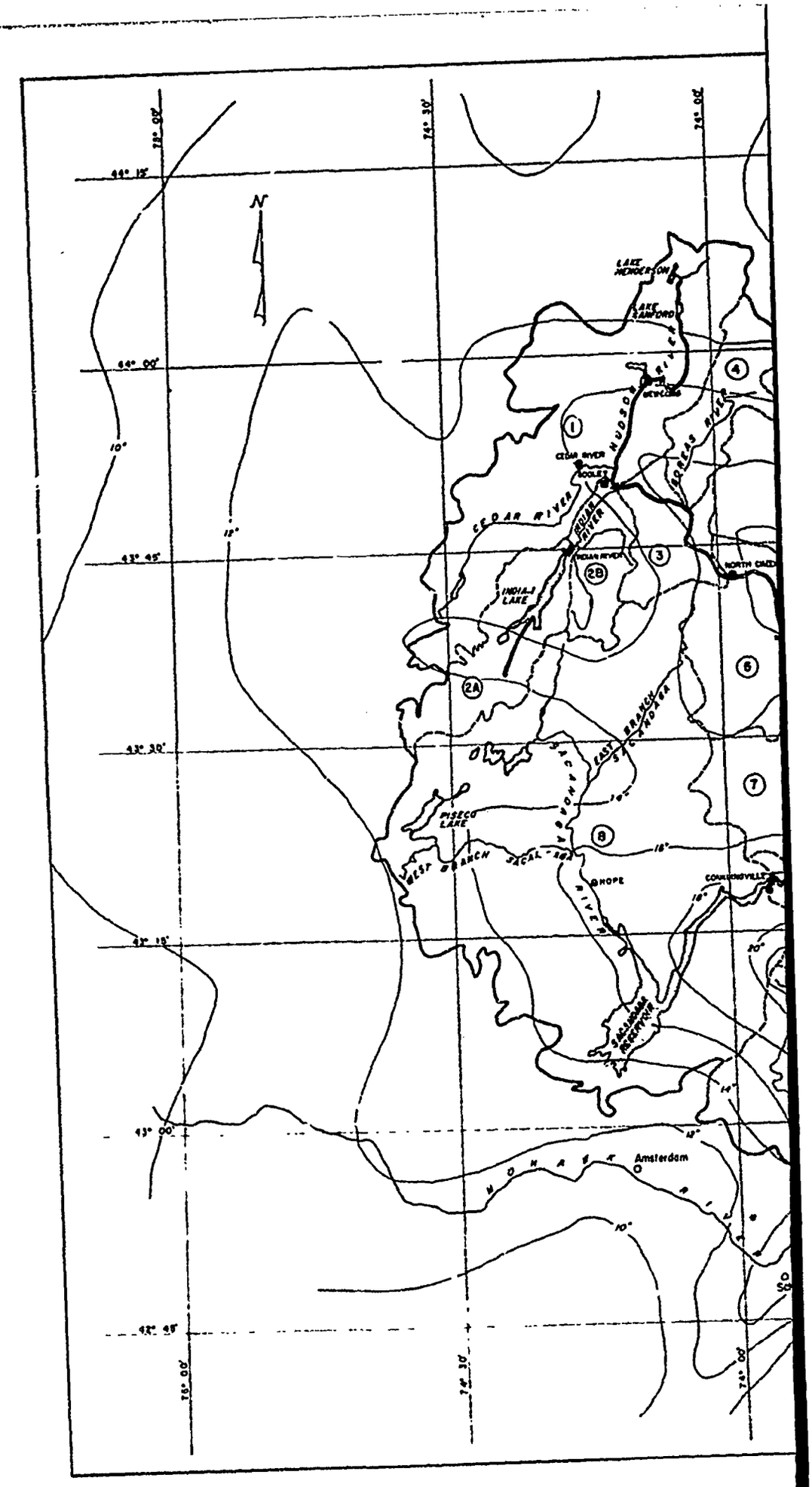
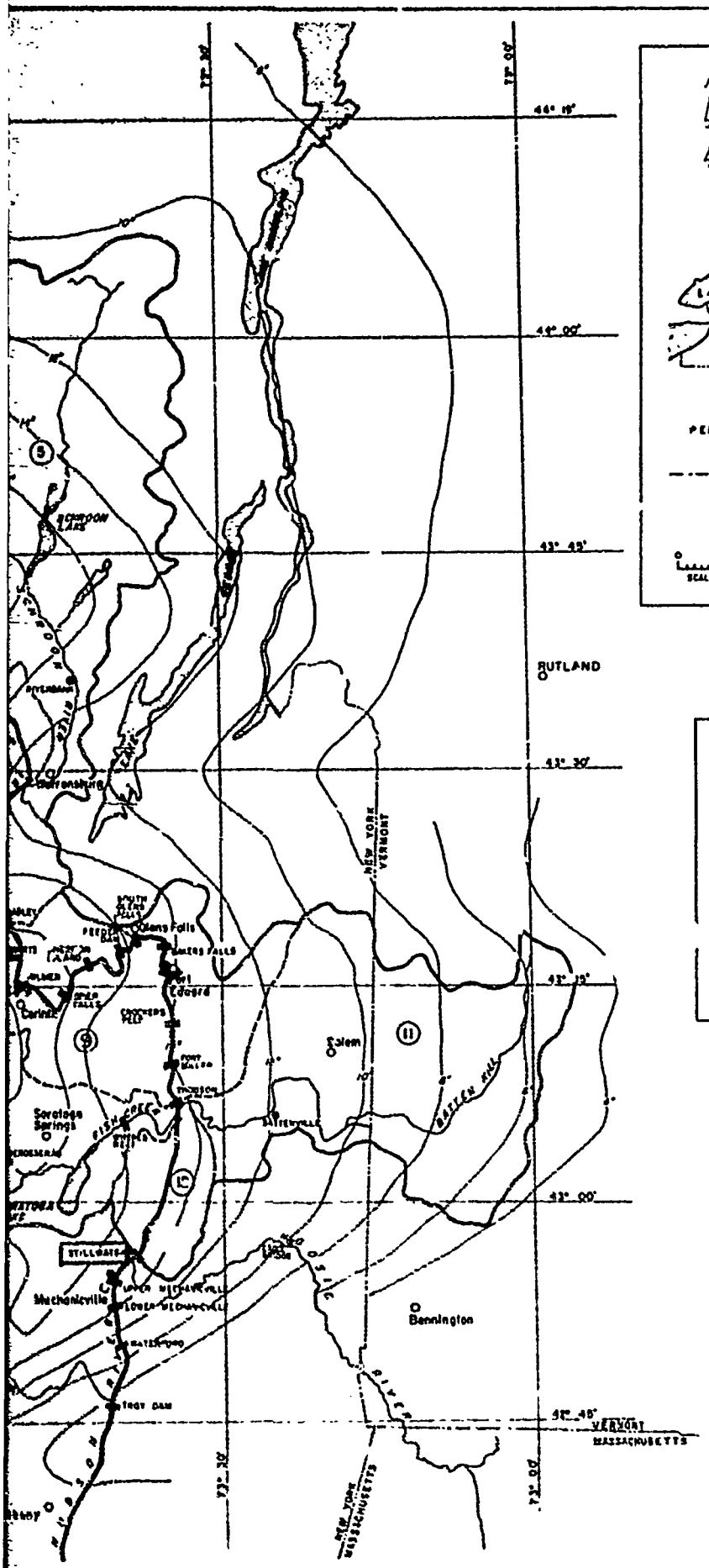
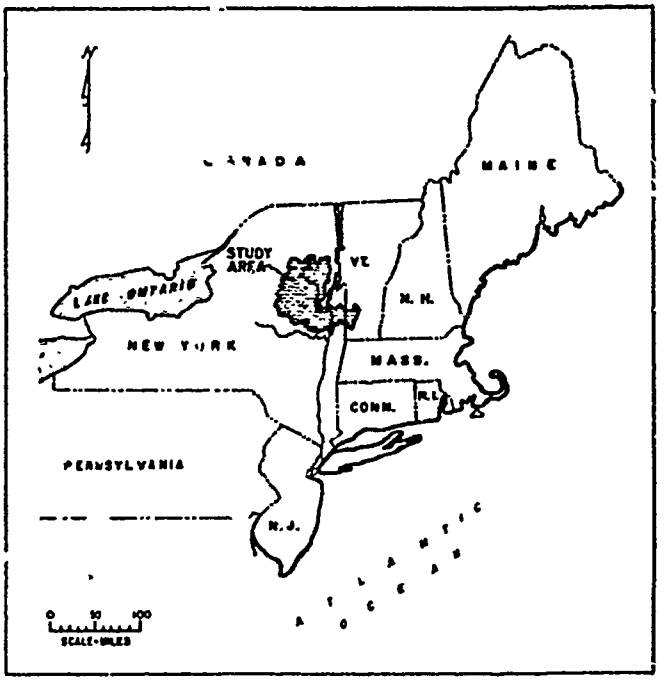


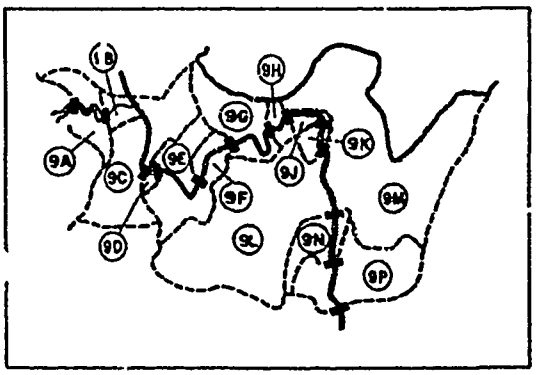
N
LAKE HENDERSON
LAKE SANFORD
HUDSON RIVER
CEDAR RIVER
INDIAN RIVER
INDIAN LAKE
EAST BRANCH SACANDAGA
PISECO LAKE
WEST BRANCH SACANDAGA RIVER
HOPE
SACANDAGA RESERVOIR
Amsterdam
MOHAWK RIVER
1
2A
2B
3
4
6
7
8
44° 15'
44° 00'
43° 45'
43° 30'
43° 15'
43° 00'
42° 45'
75° 00'
74° 30'
74° 00'
10°
12°
14°
16°
18°
20°

FIG. 1A





LOCATION MAP



SUB DIVISIONS OF SUB BASIN (9)

LEGEND

(7) – SUB BASIN

HOPE – GAGING STATION

SHERMAN ISLAND – DAM

Glens Falls – CITY

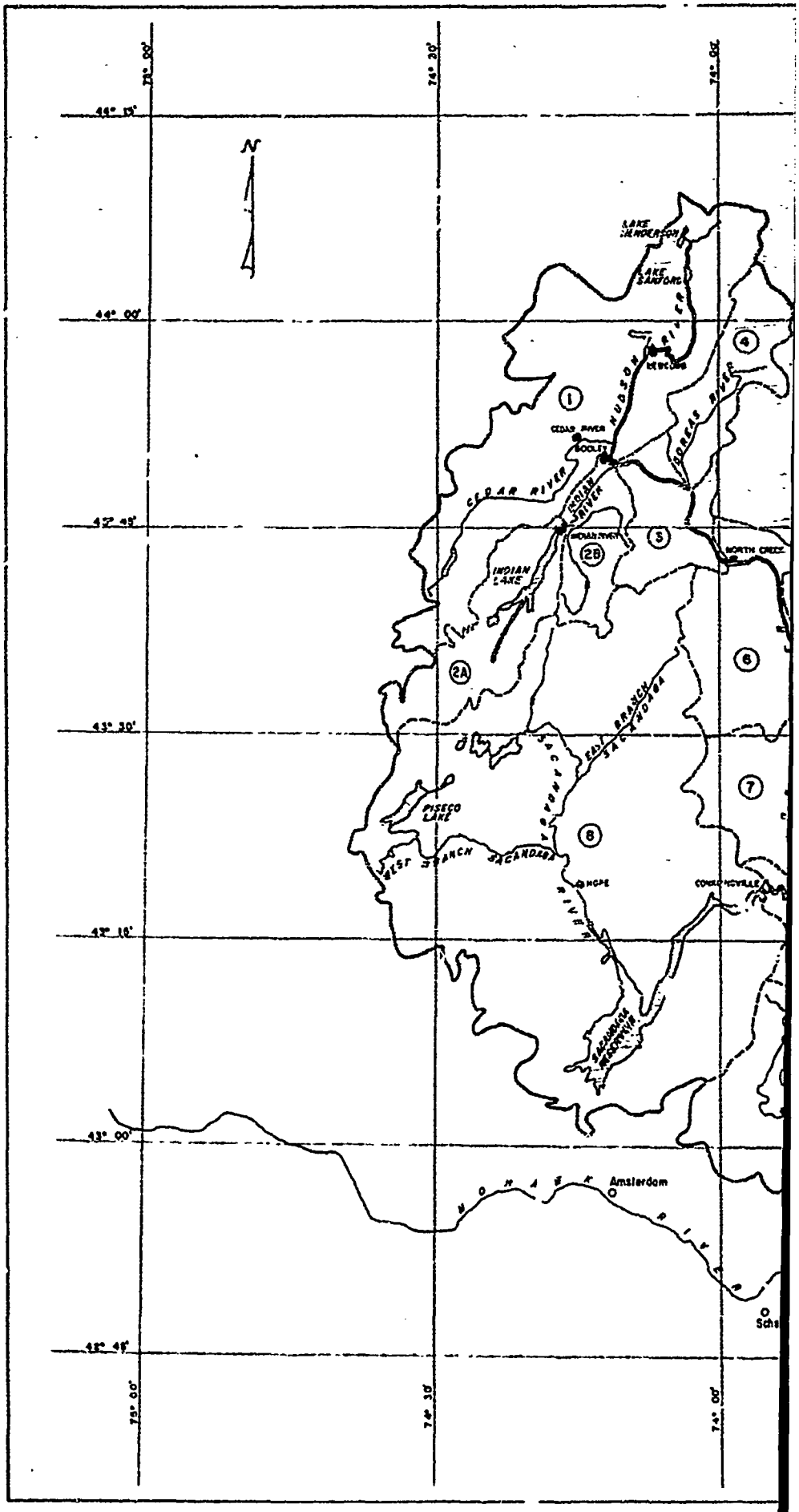
—12"— – ISOHYETS FOR 72 HOUR PMP

0 5 10 15 20

SCALE–MILES

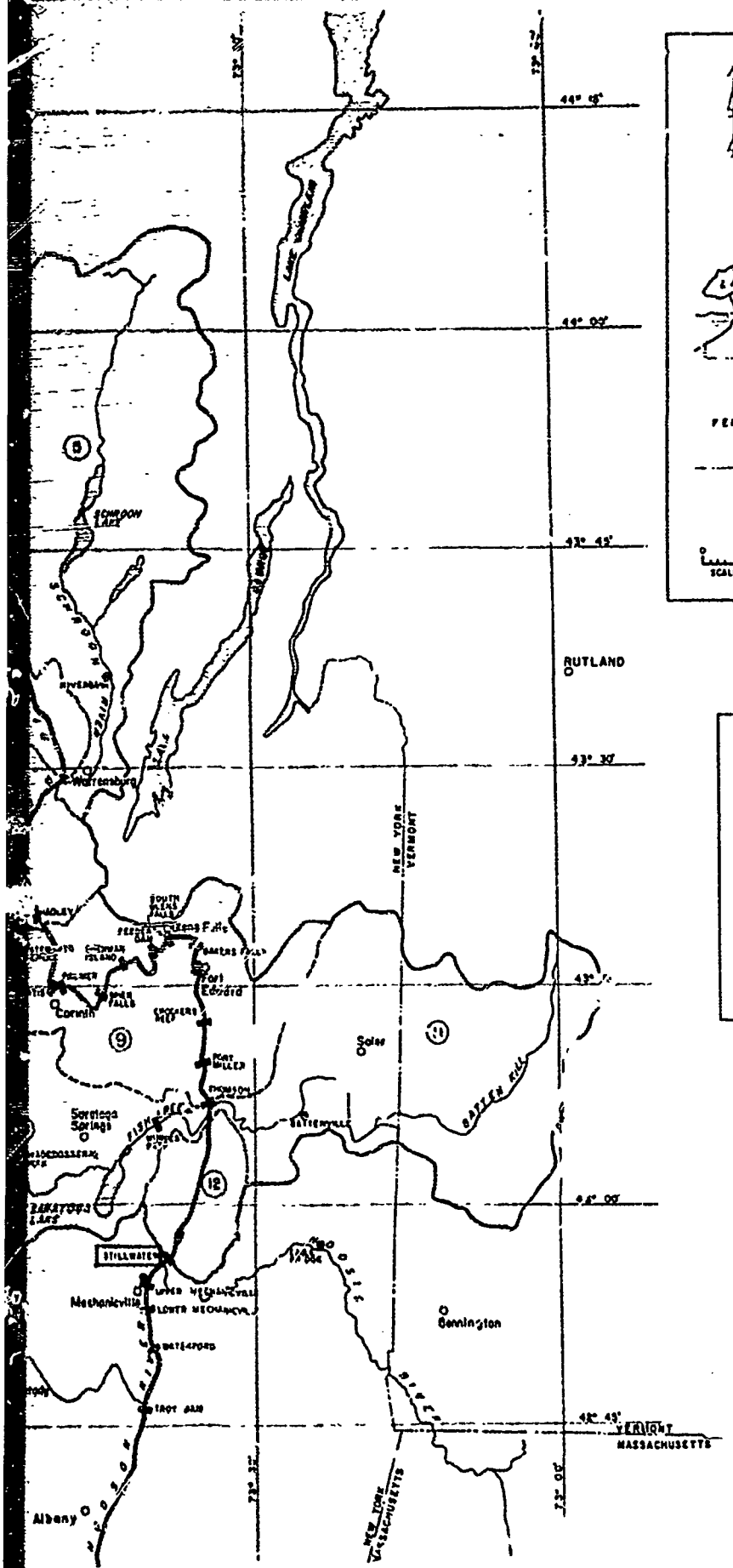
PLAN–UPPER HUDSON RIVER BASIN
FLOOD STUDY

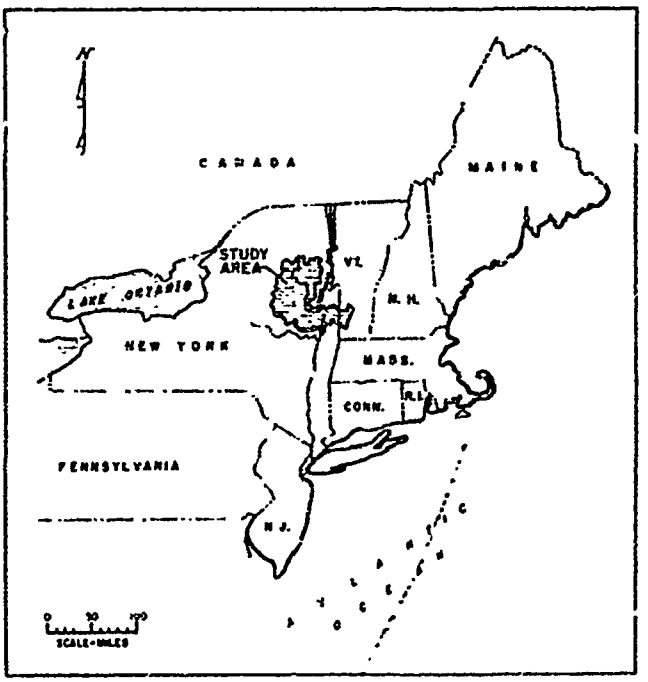
STONE & WEBSTER ENGINEERING CORPORATION



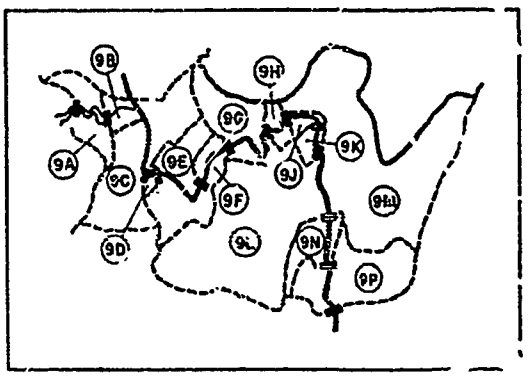
N
LAKE HENDERSON
LAKE SANFORD
HUDSON RIVER
GOREAS RIVER
CEDAR RIVER
INDIAN RIVER
INDIAN LAKE
NORTH CREEK
EAST BRANCH SACANDAGA
SACANDAGA RIVER
PISECO LAKE
WEST BRANCH SACANDAGA
SACANDAGA RESERVOIR
MOHAWK RIVER
Amsterdam
1
2A
2B
3
4
6
7
8
44° 15'
44° 00'
43° 45'
43° 30'
43° 15'
43° 00'
42° 45'
75° 00'
74° 30'
74° 00'

FIG. 1





LOCATION MAP



SUB DIVISIONS OF SUB BASIN (9)

LEGEND

(7) – SUB BASIN

(9) HOPE – GAGING STATION

SHERMAN ISLAND – DAM

O Glens Falls – CITY

0 5 10 15 20

SCALE—MILES

PLAN-UPPER HUDSON RIVER BASIN
FLOOD STUDY

STONE & WEBSTER ENGINEERING CORPORATION

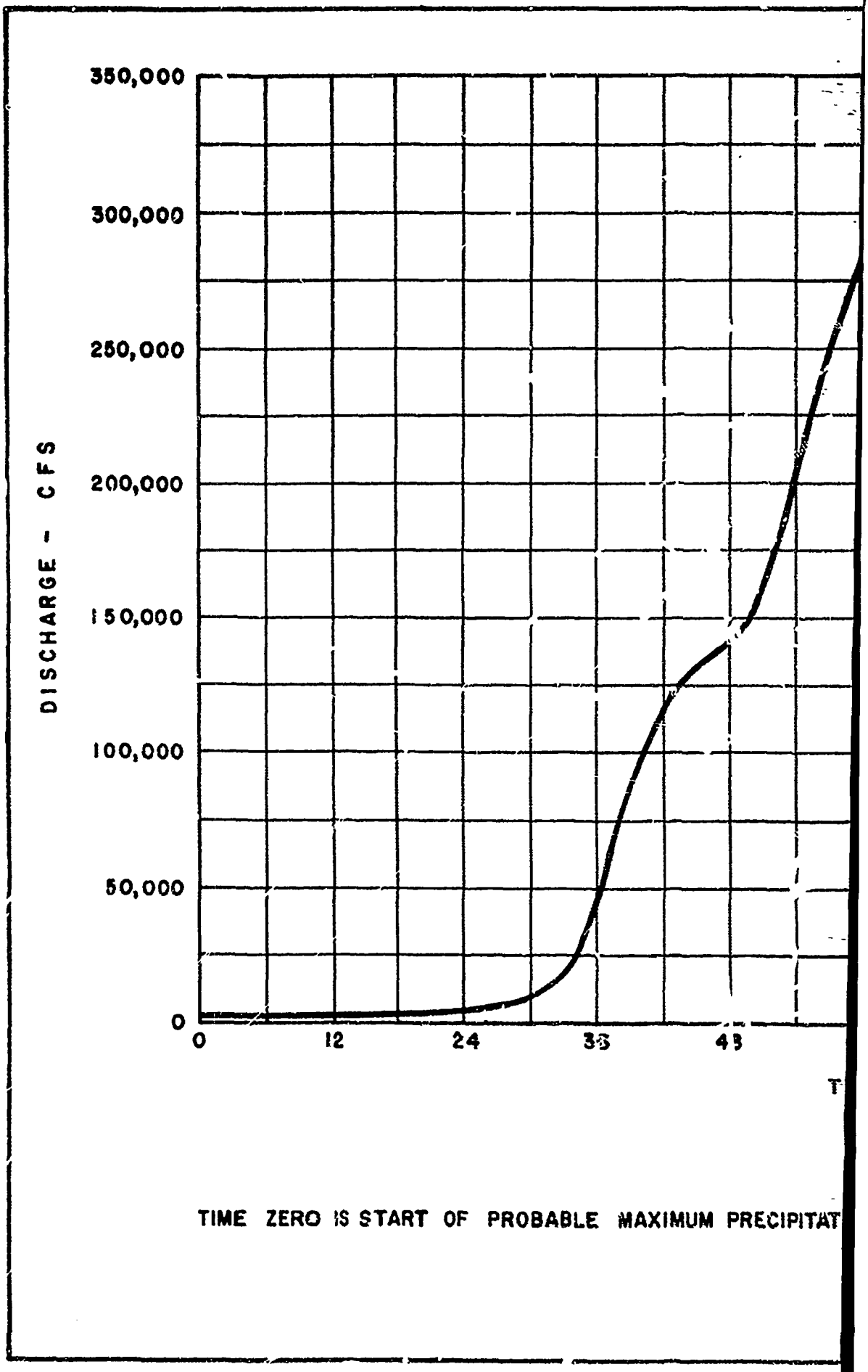
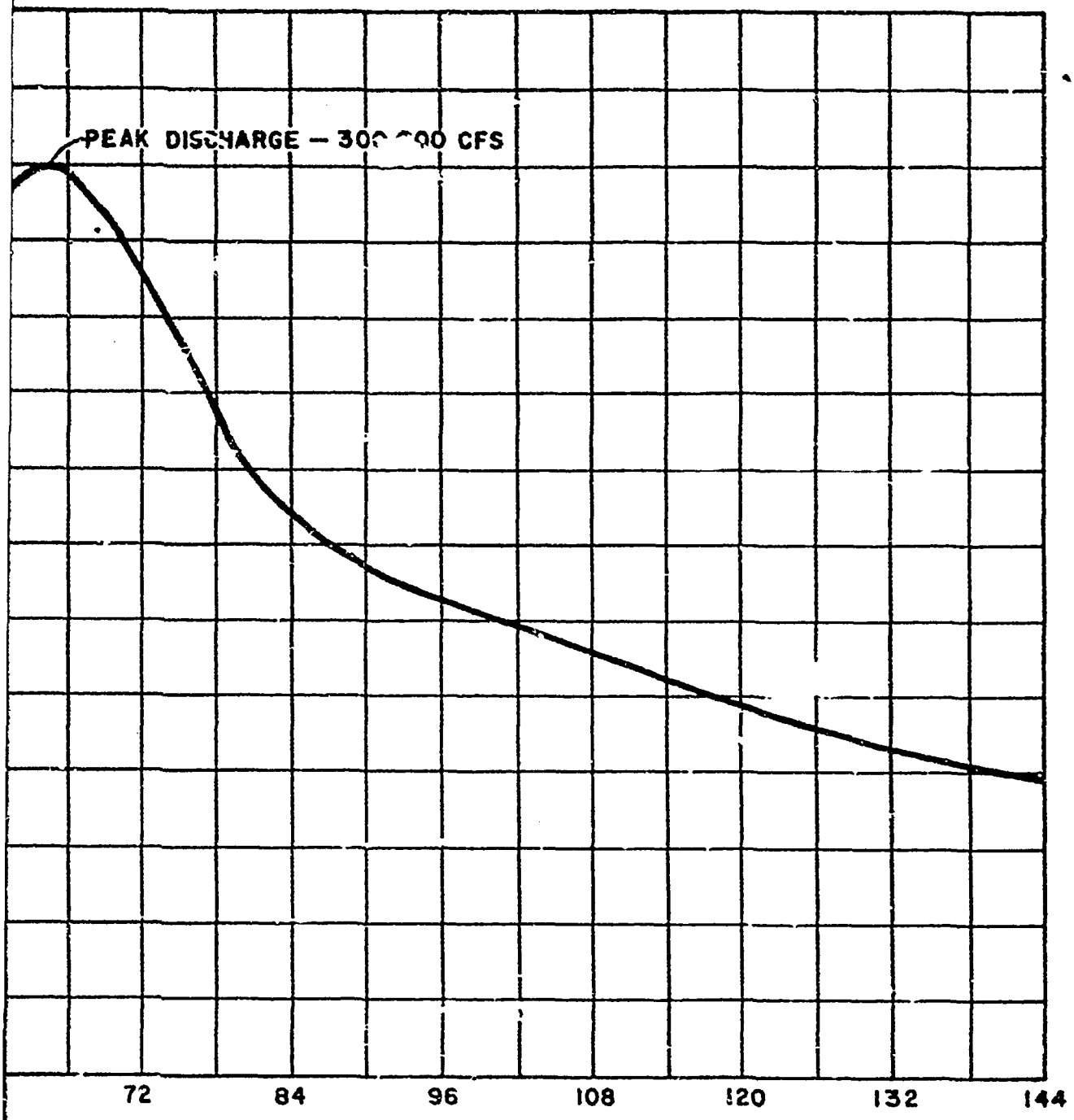
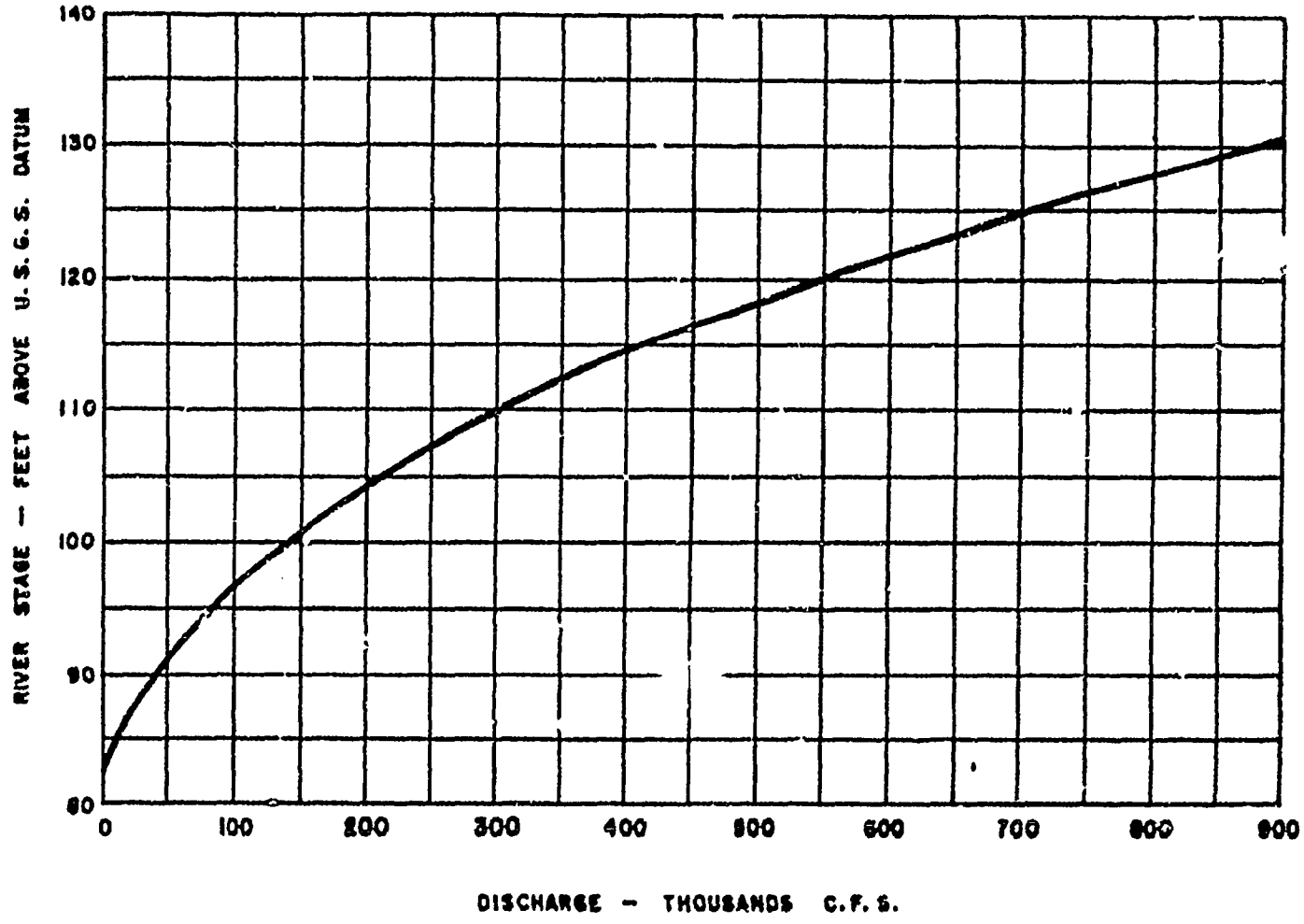

350,000
300,000
250,000
200,000
150,000
100,000
50,000
0
DISCHARGE - CFS
0
12
24
36
48
T
TIME ZERO IS START OF PROBABLE MAXIMUM PRECIPITAT

FIG. 3



PROBABLE MAXIMUM FLOOD HYDROGRAPH
HUDSON RIVER AT STILLWATER
FLOOD STUDY

STONE & WEBSTER ENGINEERING CORPORATION

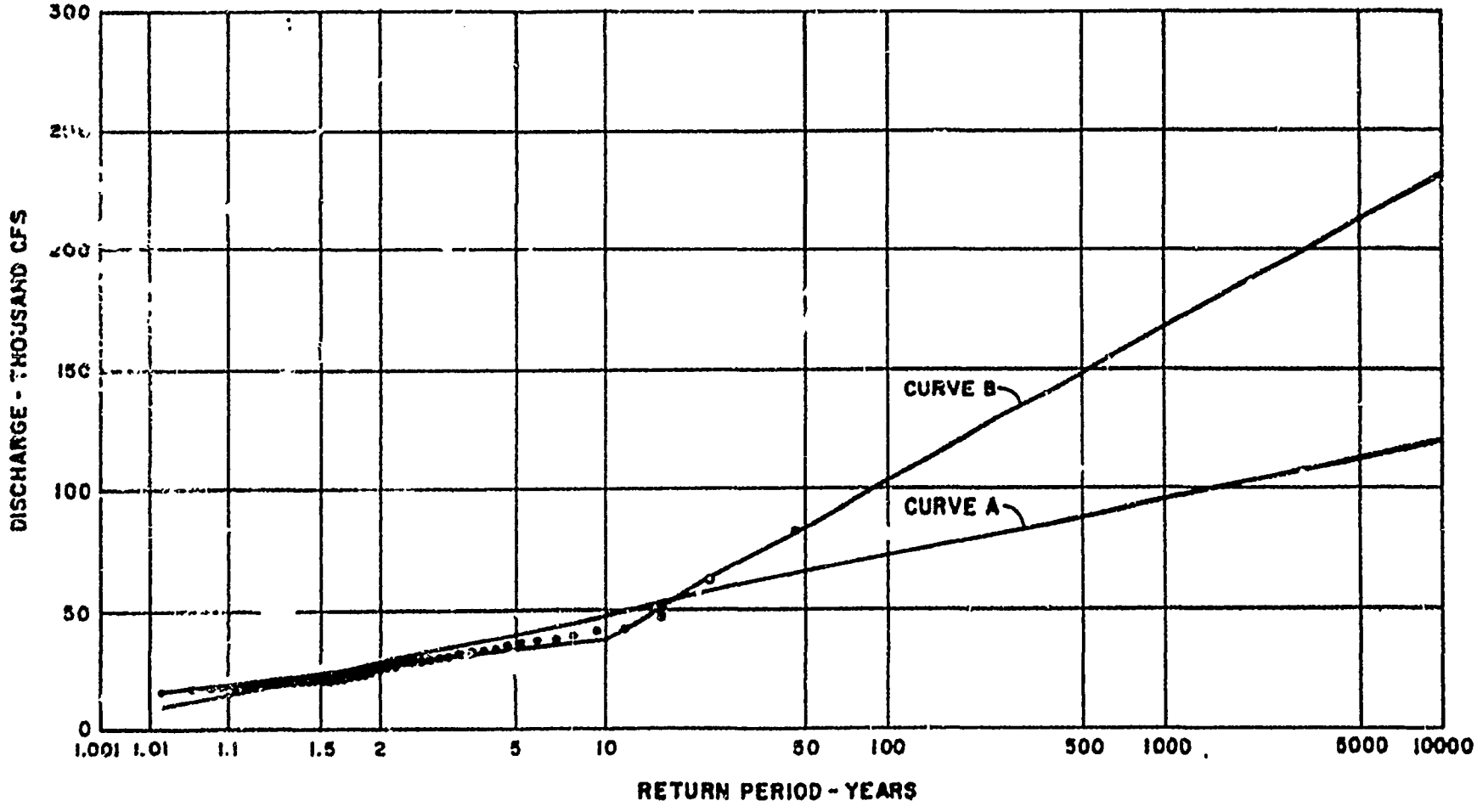


STAGE-DISCHARGE CURVE

HUDSON RIVER AT STILLWATER
FLOOD STUDY

STONE & WEBSTER ENGINEERING CORPORATION

FIG. 5



DISCHARGE FREQUENCY CURVES
HUDSON RIVER AT STILLWATER
FLOOD STUDY

PERIOD OF RECORD 1911 - 1956
MAX. MEAN DAILY FLOWS - ADJUSTED

STONE & WEBSTER ENGINEERING CORPORATION

APPENDIX C

NOTATIONS & SYMBOLS USED IN THE REPORT

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$^{\frac{1}{2}}$/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. $\frac{1}{2}\gamma H^2$, where γ = 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 $\frac{1}{2}\gamma HB$, 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.

NOTATIONS & SYMBOLS USED IN THE REPORT (Continued)

G	=	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 succesive intervals of time of length $t_{n+1} - t_n$
O	=	Rate of outflow from the reservoir

NOTATIONS & SYMBOLS USED IN THE REPORT (Continued)

O_1, O_2 = Outflow from reach 1 and 2

P_n, 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, $R_H = F_H + F_\Sigma$ 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

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

S_o = 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

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

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

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^o = Angle between horizontal line and upstream face of the dam.

Δt = Length of the routing period having a maximum value of $2K_s$ 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

APPENDIX D

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2.6 METEOROLOGY

2.6.1 GENERAL

The meteorology of the Indian Point site has been thoroughly studied by two years of observations at the site itself and surrounding locations. This meteorological program was carried out by the Research Division of New York University and the ensuing reports were submitted to the AEC in their entirety as Exhibits L-1, L-5 and L-6 of Docket 50-3, Indian Point Unit No.1.

The findings of this meteorological program and the precipitation information gathered by the National Weather Records Center from the nearby Bear Mountain Weather Station, constitute the solid basis upon which the safety analysis of the Loss-of-Coolant Accident can be made.

The most important meteorological characteristic of the Indian Point site is the prevalent north-south wind direction. The orientation of the ridges channel the air flow in the direction of the Hudson Valley. This predominant wind direction holds at different altitudes and for lapse, neutral and inversion conditions. There is no thickly populated area north and south of the site and wind trajectories towards the center of the largest city in the area, Peekskill, are infrequent.

New York University under a contract with Consolidated Edison Company made extensive tests on the meteorological conditions at the Indian Point site. The testing program, begun in 1955 and completed in 1957, was described in three technical reports prepared by the New York University staff under the immediate direction of Prof. Benjamin Davidson. Prof. Davidson has on several occasions in the past 12 years acted as consultant to Consolidated Edison on meteorological matters and thus has continued an association with the Indian Point site. The original New York University reports or applicable excerpts

therefrom which have been reviewed by Pro. Davidson and the Consolidated Edison staff are included in this section.

The present section contains the following material:

1. NYU Technical Report 372.1, "A Micrometeorological Survey of the Buchanan, N. Y. Area" dated November 1955, which was Exhibit L-1, Docket 50-3, given in its entirety. The topography of the area surrounding the site is described and the effects of the topography on meteorological conditions are discussed. The types of data collected, the methods and frequencies of collection, the description and location of the equipment and the general scope of the meteorological program are indicated in this report. Seasonal wind characteristics, including speeds, directions and frequencies are tabulated.

2. Section 2 and 3 of NYU Technical Report 372.3, "Evaluation of Potential Radiation Hazard Resulting From Assumed Release of Radioactive Wastes to the Atmosphere From Proposed Buchanan Nuclear Power Plant," dated April 1957. This report was submitted to the AEC in its entirety as Exhibit L-5, Docket 50-3. These section discuss the diffusive conditions and the climatological data of the site. The basis for evaluating the diffusion parameters selected for the safety analysis is given on Pages 19 to 21. Section 3 contains tables of frequency distribution of diffusion classes and wind directions, and also wind roses.

3. NYU Technical Report 372.4, "Summary of Climatological Data at Buchanan, N. Y., 1956-1957," dated March 1958, was Exhibit L-6, Docket 50-3. This report summarizes the final meteorological testing at Indian Point.

4. Report by the National Weather Records Center of the Department of Commerce giving precipitation information and the prevalent

wind directions associated with precipitation. A table of wind directions during thunderstorms is also included. This information is based on data collected at the Bear Mountain Weather Station, three miles north of the site. The Consolidated Edison staff has drawn wind roses from the wind frequencies shown in the tables; these wind roses are inserted in the NWRC report.

2.6.2 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 Section 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 direction 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.

The most frequent wind flow at low heights under inversion conditions is down the axis of the valley. This direction, roughly 010-030°, 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 assumes 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 005-020° wind direction.

The valley wind is diurnal in nature, i.e., up-valley during unstable hours and down-valley during stable hours. In general, these local winds are most frequent under clear sky and relatively light prevailing

wind conditions such as occur mostly in the fall of the year. The diurnal variation of the vector mean wind as measured 70 ft. above river during Sept.-Oct. 1955 is shown in Figure 2.6-1 for conditions in which the large scale flow was virtually zero (12 days) and in Figure 2.6-2 for conditions in which the large scale flow (geostropic 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 2.6-3 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 about 20-06 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 about 08 hours and thereafter becomes quite unreliable. In short, these data indicate that a consecutive 24 hour down-valley flow with light wind speeds and inversion conditions is extremely improbable.

The Safety Analysis of the Loss-of-Coolant Accident assumes that the accident occurs during down-valley inversion flow conditions and this condition persists for 24 hours with average wind speeds slightly less than 2m/sec. Figures 2.6-1 & 2.6-2 indicate that the duration of the down-valley flow is about 12 hours rather that 24 hours, and that the vector mean wind speeds are on the order of 2.9m/sec.

In view of the discussion above, it must be concluded that the safety analysis for the first 24 hours is conservative to within a factor of about two.

The remainder of the safety analysis assumes that for the next 30 days, 35% of the winds are in the 20° sector corresponding to the nocturnal down-valley flow and that wind speed and thermal stability are 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 005-020° winds could be expected to occur. The analysis assumes 276 hours of 005-020° winds occur in the first 31 days after the accident, and that about 130 of these hours are charcterized by inversion conditions. Approximately 35 weak pressure gradient days were observed in Sept.-Oct. 1955 or about 430 hours per month. From Figure 2.6-3 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 in fact about 0.9 or less), the number of down-valley inversion winds per month during Sept. and Oct. 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 is 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 20° sector for 35% of the time

This is equivalent to assuming that in the model 20° sector, the inversion frequency is 14.8 percent for the 30 day period. The observed annual maximum inversion frequency for a 20° sector is 6.2% (p. 29, Table 3-3, MIT 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 inversions in the model 20° sector are

consi[illegible]ed to occur in the first 31 day month after the accident. The assumption of uniform spread of inversion frequency over the year is examined above where an attempt is made to isolate those local meteorological conditions at Indian Point which might yield the highest 30 day dose. It is concluded that the "worst" meteorological conditions are associated with the nocturnal down-valley flow which is most frequent during September and October.

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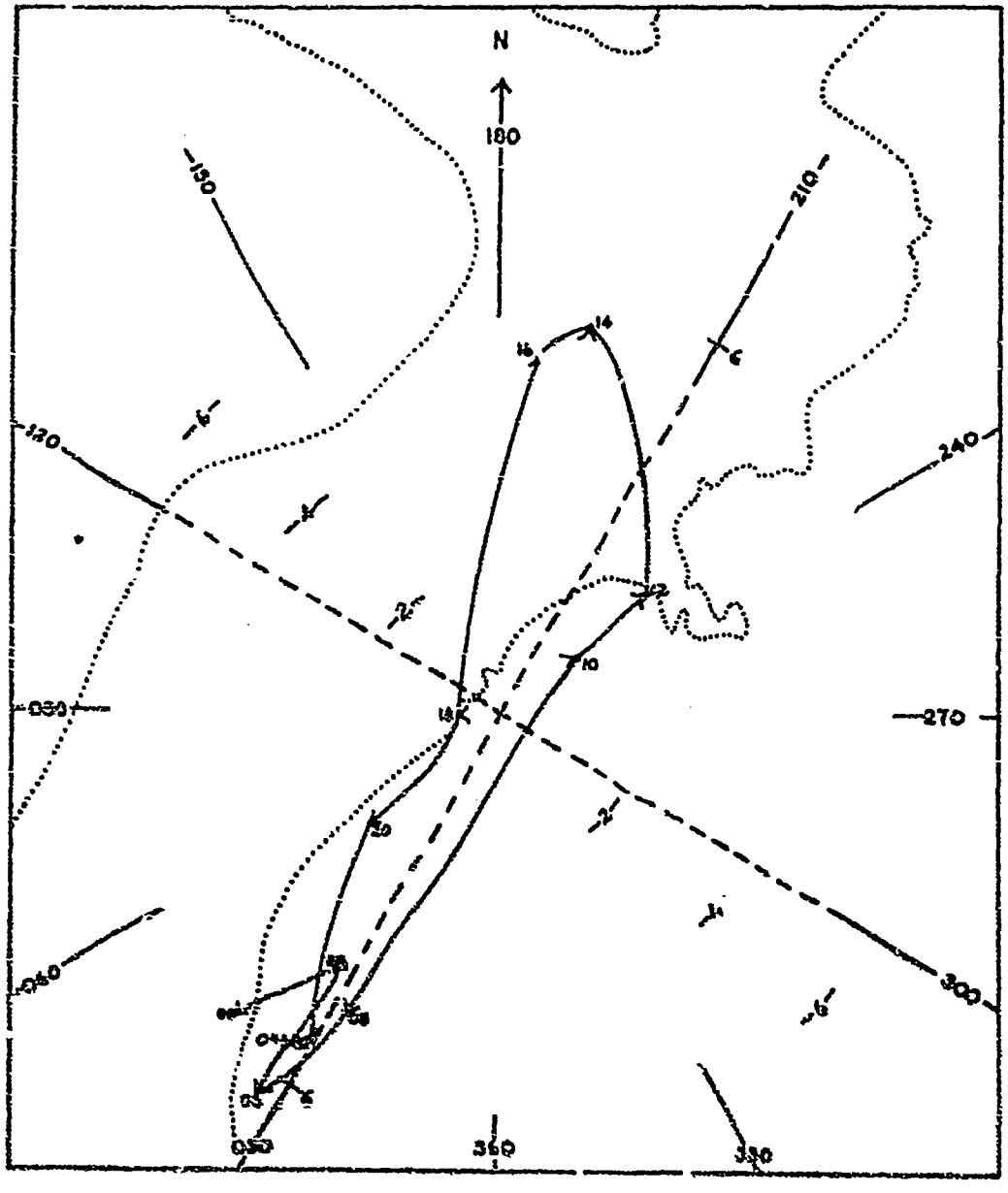

Figure 2.6-1

Diurnal variation of mean vector wind for virtually zero pressure gradient conditions, Sept.-Oct., 1955, 70 ft. above river.

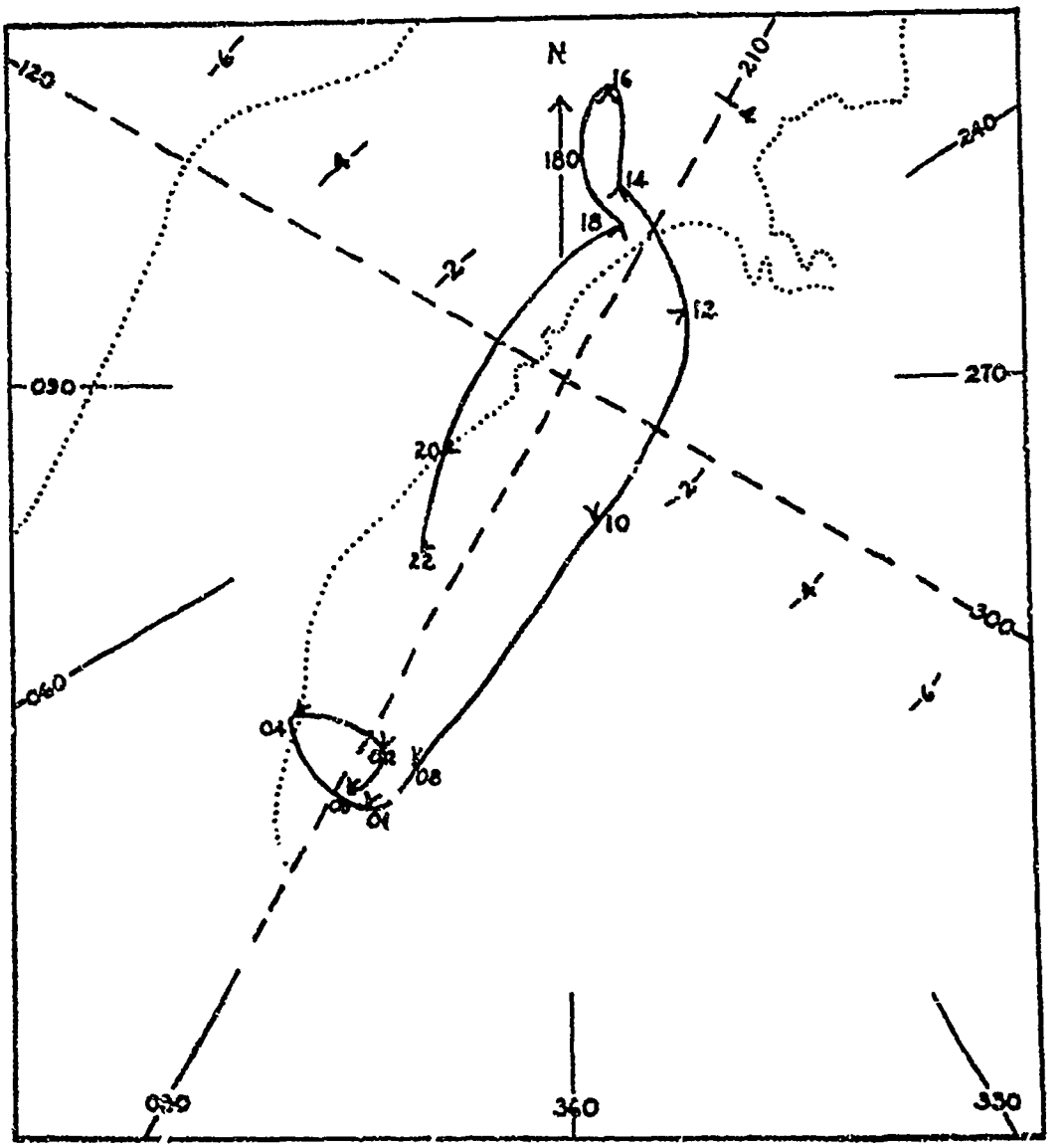

Figure 2.6-2

Diurnal variation of mean vector wind for 24 hr. periods of weak pressure gradient condi... (Geostrophic winds less than 16 mph, Sept.-Oct., 1945, 78 ft. above river).

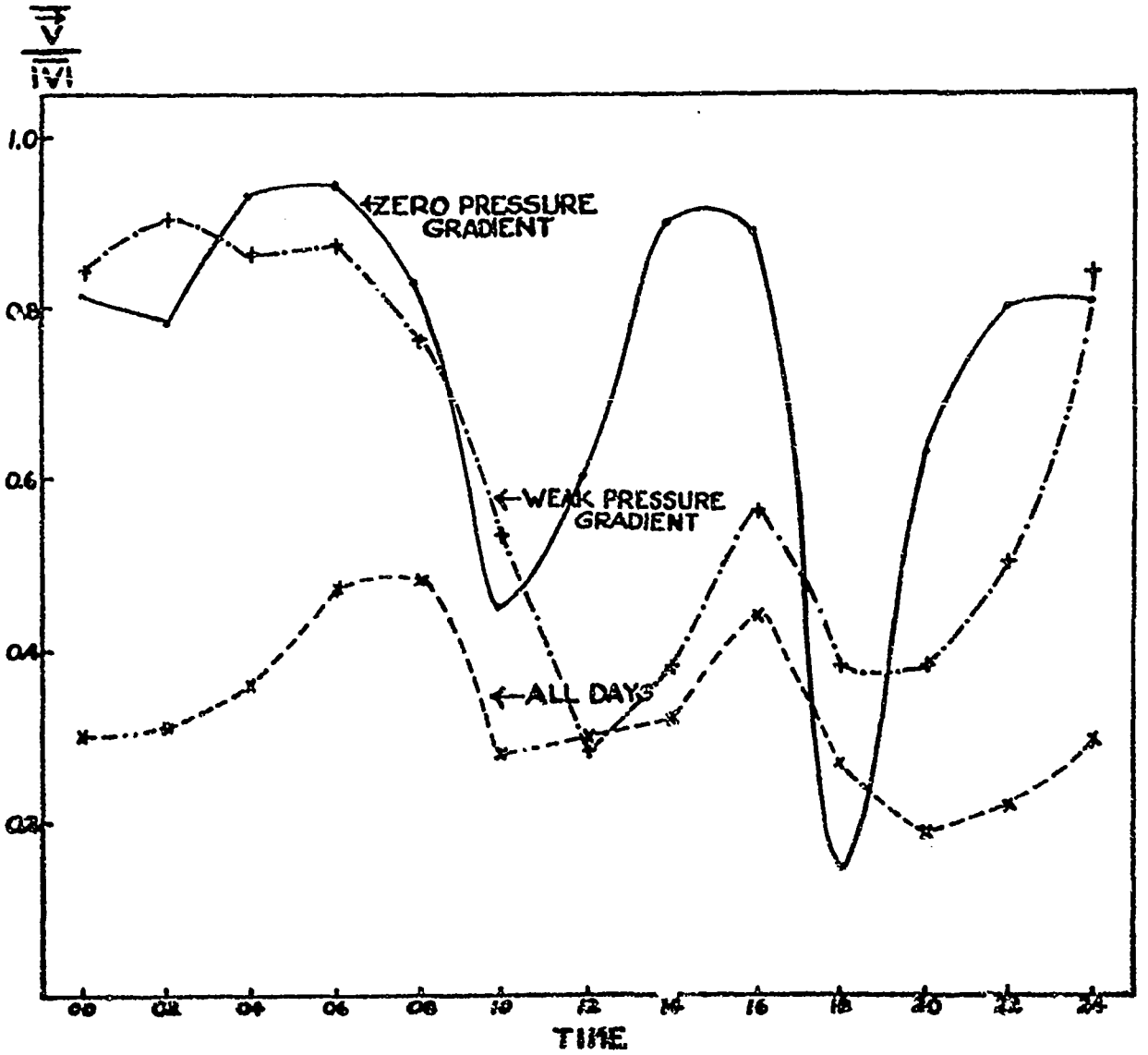

Figure 2.6-3

Steadiness of wind as a function of time of day for indicated pressure gradient conditions.

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Technical Report No. 372.1

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Prepared for
Consolidated Edison Co. of N. Y., Inc.
November, 1955

Q 1