AD-A134 158 METROPOLITAN WASHINGTON AREA WATER SUPPLY STUDY APPENDIX F STRUCTURAL ALTERNATIVES(U) CORPS OF ENGINEERS BALTIMORE MD BALTIMORE DISTRICT SEP 83

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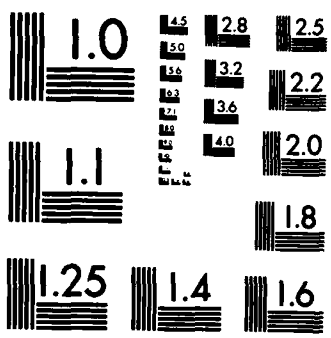
UNCLASSIFIED MWA-83-P-APP-F F/G 5/1 NL



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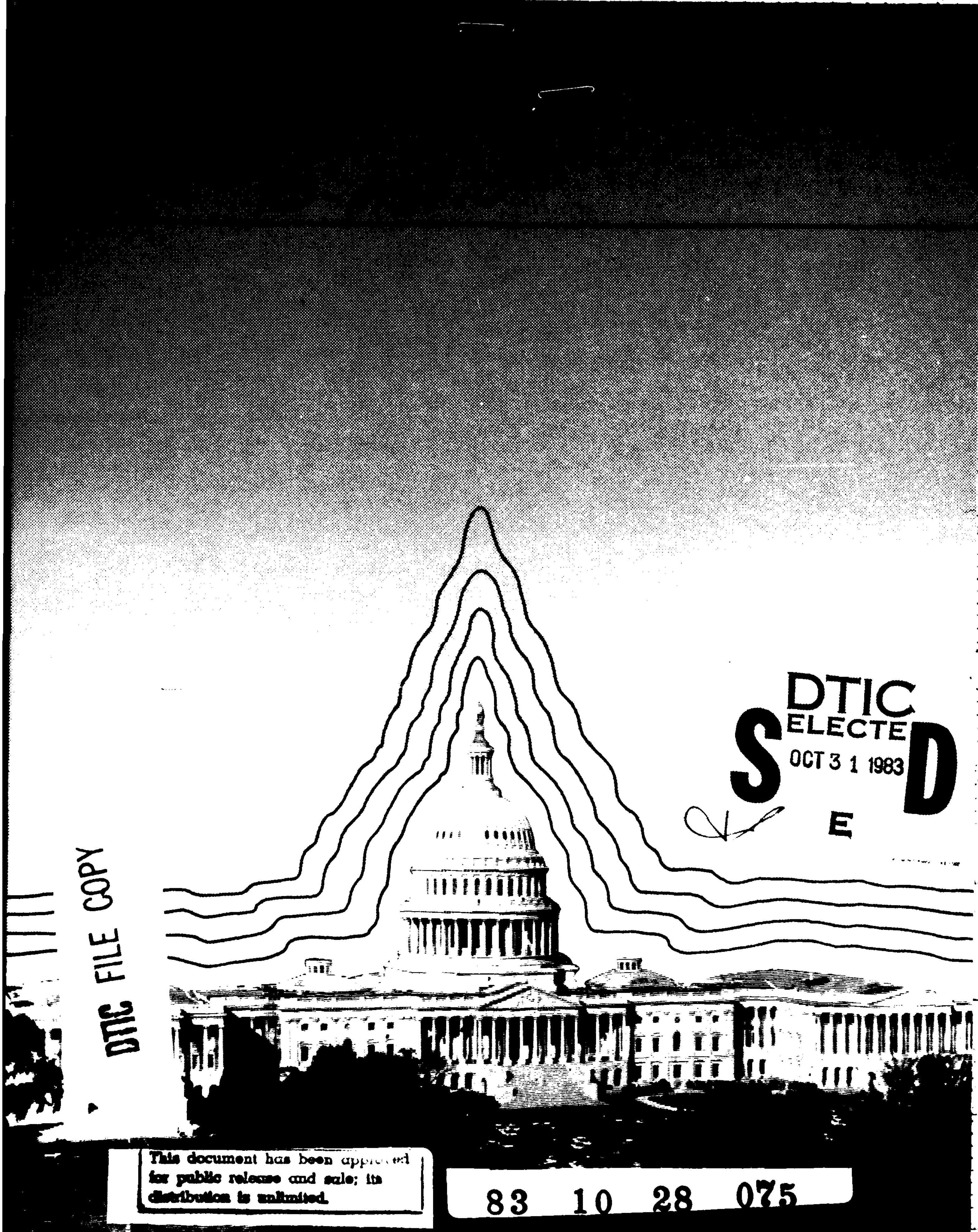

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SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE

READ INSTRUCTIONS BEFORE COMPLETING FORM

1. REPORT NUMBER: MWA-83-P

2. GOVT ACCESSION NO.: A134158

3. RECIPIENT'S CATALOG NUMBER:

4. TITLE (and Subtitle): METROPOLITAN WASHINGTON AREA WATER SUPPLY STUDY, [redacted] Appendix [redacted] F

5. TYPE OF REPORT & PERIOD COVERED: Final

6. PERFORMING ORG. REPORT NUMBER:

7. AUTHOR(s):

8. CONTRACT OR GRANT NUMBER(s):

9. PERFORMING ORGANIZATION NAME AND ADDRESS: Baltimore District, U.S. Army Corps of Engineers, ATTN: NABPL, P.O. Box 1715, Baltimore, Maryland 21203

10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS:

11. CONTROLLING OFFICE NAME AND ADDRESS: Baltimore District, U.S. Army Corps of Engineers, ATTN: NABPL, P.O. Box 1715, Baltimore, Maryland 21203

12. REPORT DATE: September 1983

13. NUMBER OF PAGES: ~~3,000 pages, 1 plate~~

14. MONITORING AGENCY NAME & ADDRESS (if different from Controlling Office):

15. SECURITY CLASS. (of this report): Unclassified

15a. DECLASSIFICATION/DOWNGRADING SCHEDULE:

16. DISTRIBUTION STATEMENT (of this Report): Approved for public release, distribution unlimited.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report):

18. SUPPLEMENTARY NOTES:

19. KEY WORDS (Continue on reverse side if necessary and identify by block number): water resources planning; water demand; water supply; Potomac River; groundwater; reservoir; raw water interconnection; Low Flow Allocation Agreement; Bloomington Lake; Little Seneca Lake; wastewater reuse; Potomac Estuary Experimental Water Treatment Plant; PRISM/COE; water pricing; water conservation; demand reduction; Cooperative Operations on the Potomac; flowby;

20. ABSTRACT (Continue on reverse side if necessary and identify by block number):

In response to the Water Resources Development Act of 1974, the Baltimore District of the U.S. Army Corps of Engineers conducted a comprehensive water supply analysis of the Metropolitan Washington Area (MWA). Severe water supply shortages had been forecast for the MWA and the study was undertaken to identify and evaluate alternative methods of alleviating future deficits. Initiated in 1976, the study was conducted in two phases over a 7-year period. The first, or early action phase, examined the most immediate water supply problems and proposed solutions that could be implemented locally. The second or long

19. KEY WORDS (continued)

water shortage; reregulation; finished water interconnection; Occoquan Reservoir; Patuxent Reservoir; Potomac Estuary; Water Supply Coordination Agreement; Verona Lake

20. ABSTRACT (continued)

range phase included an analysis of the full spectrum of structural and nonstructural water supply alternatives. In addition to such traditional water supply alternatives as upstream reservoir storage, groundwater and conservation, the study also considered such innovative measures as wastewater reuse, raw and finished water interconnections between the major suppliers, the use of the upper Potomac Estuary, reregulation and water pricing. A key tool in the study was the development and use of a basin-specific model that was used to simulate the operation of all the MWA water supply systems and sources under various drought scenarios. As the study progressed, local interests used the technical findings of the Corps' study to make great strides toward a regional solution to their water supply problems. The Corps' study concluded that with the implementation of a series of regional cooperative management agreements, contracts, selected conservation measures, and the construction of one local storage project to be shared by all, severe water supply shortages could effectively be eliminated for the next 50 years. The Final Report of the study is comprised of eleven volumes which provide documentation of both the study process and the results of all the technical analyses conducted as part of the study.

METROPOLITAN WASHINGTON AREA
WATER SUPPLY STUDY

APPENDIX F

STRUCTURAL ALTERNATIVES

Department of the Army
Baltimore District, Corps of Engineers
Baltimore, Maryland

September 1983

REPORT ORGANIZATION*

METROPOLITAN WASHINGTON AREA WATER SUPPLY STUDY

Appendix Letter	Appendix Title	Annex Number	Annex Title
	Main Report		
A	Background Information & Problem Identification		
B	Plan Formulation, Assessment, and Evaluation	B-I	Water Supply Coordination Agreement
		B-II	Little Seneca Lake Cost Sharing Agreement
		B-III	Savage Reservoir Operation and Maintenance Cost Sharing Agreement
C	Public Involvement	C-I	Metropolitan Washington Regional Water Supply Task Force
		C-II	Public Involvement Activities - Initial Study Phase
		C-III	Public Opinion Survey
		C-IV	Public Involvement Activities - Early Action Planning Phase
		C-V	Sample Water Forum Note
		C-VI	Public Involvement Activities - Long-Range Planning Phase
		C-VII	Citizens Task Force Resolutions
		C-VIII	Background Correspondence
		C-IX	Coordination with National Academy of Sciences - National Academy of Engineering
		C-X	Comments and Responses Concerning Draft Report
D	Supplies, Demands, and Deficits	D-I	Water Demand Growth Indicators by Service Areas
		D-II	Service Area Water Demand & Unit Use by Category (1976)
		D-III	Projected Baseline Water Demands (1980-2030)
		D-IV	Potomac River Low Flow Allocation Agreement
		D-V	Potomac River Environmental Flowby, Executive Summary
		D-VI	PRISM/COE Output, Long-Range Phase
E	Raw and Finished Water Interconnections and Reregulation	E-I	Special Investigation, Occoquan Interconnection Comparison
F	Structural Alternatives	F-I	Digital Simulation of Groundwater Flow in Part of Southern Maryland
G	Non-Structural Studies	G-I	Metropolitan Washington Water Supply Emergency Agreement
		G-II	The Role of Pricing in Water Supply Planning for the Metropolitan Washington Area
		G-III	Examination of Water Quality and Potability
H	Bloomington Lake Reformulation Study	H-I	Background Information
		H-II	Water Quality Investigations
		H-III	PRISM Development and Application
		H-IV	Flood Control Analysis
		H-V	US Geological Survey Flow Loss and Travel Time Studies
		H-VI	Environmental, Social, Cultural, and Recreational Resources
		H-VII	Design Details and Cost Estimates
		H-VIII	Drawdown Frequency and Yield Dependability Analyses
		H-IX	Bloomington Future Water Supply Storage Contract
		H-X	Novation Agreement
I	Outlying Service Areas		

*The Final Report for the Metropolitan Washington Area Water Supply Study consists of a Main Report, nine supporting appendices, and various annexes as outlined above. The Main Report provides an overall summary of the seven-year investigation as well as the findings, conclusions, and recommendations of the District Engineer. The appendices document the technical investigations and analyses which are summarized in the Main Report. The annexes provide detailed data or complete reports about individual topics contained in the respective appendices.

METROPOLITAN WASHINGTON AREA WATER SUPPLY STUDY

APPENDIX F - STRUCTURAL ALTERNATIVES

TABLE OF CONTENTS

TABLE OF CONTENTS (Cont'd)

TABLE OF CONTENTS (Cont'd)

TABLE OF CONTENTS (Cont'd)

LIST OF TABLES

LIST OF TABLES (Cont'd)

LIST OF FIGURES

LIST OF FIGURES (Cont'd)

LIST OF FIGURES (Cont'd)

LIST OF ANNEXES

APPENDIX F

STRUCTURAL ALTERNATIVES

INTRODUCTION

The Structural Alternatives Appendix presents a description of the structural alternatives investigated for the long-range planning needs of the Metropolitan Washington Area (MWA). The appendix is divided into three major sections. The first section discusses the use of the Potomac Estuary as a water supply alternative, in particular, the Emergency Estuary Water Pumping Station and the Potomac Estuary Experimental Water Treatment Plant. The second section describes the reservoir sites, both upstream and local, which were examined for potential storage. The last section addresses groundwater as an alternative for the MWA.

ESTUARY USE

POTOMAC ESTUARY STUDIES

INTRODUCTION

The Metropolitan Washington Area depends on the Potomac River as a major source of water supply with the river presently providing about 70 percent of the MWA needs. During droughts the ability of the river to meet both present and increasing future demands is becoming increasingly strained. Inflows to the estuarine portion of the river are decreasing with attendant impacts on the salinity and other water quality conditions in the upper estuary.

One of the alternatives which has long been advocated to alleviate projected water supply deficits in the MWA is the use of the Potomac Estuary. This segment of the report presents the findings of the estuary-related studies conducted as part of the overall MWA program. More specifically, included are: 1) a general description of the Potomac Estuary, 2) a discussion of the scope, and findings of the Potomac Estuary Experimental Water Treatment Plant studies, and 3) a presentation addressing the conduct and findings of the Potomac Estuary Hydraulic Model Testing Program.

THE POTOMAC ESTUARY

The following is a summary discussion of the more important physical properties of the Potomac Estuary. The information, as presented, was taken in large part from the Environmental Atlas of the Potomac Estuary, prepared under contract for the Maryland Department of Natural Resources by the Environmental Center, Martin Marietta Corporation. For more detail, particularly as it relates to the biota resources, the reader is referred to the above publication.

Before the Pleistocene glacial age, many great river systems drained the eastern slopes of the long mountain ridges along the North American continent. The greatest of these was the Susquehanna, which had a watershed of thousands of square miles with boundaries extending as far north as upstate New York and as far west as western Pennsylvania. As the Susquehanna meandered southward to the Atlantic, cutting through the Piedmont foothills, it was joined by waters from hundreds of streams, large and small. The largest of these tributaries was the Potomac River, which drained the southwestern slopes of the system. The Potomac, along with other southern tributaries (the York, James, and Rappahannock Rivers of Virginia), cut deep channels across the ancient coastal plain ledges. At the end of the last Pleistocene glacial age, from about 15,000 to about 9,000 years ago, sea levels rose with the melting retreat of the glaciers. Water inundated the valleys of the coastal plain rivers and eventually reached the base of the Piedmont hills at what is now called the fall line. These tidal waters flooded the lower portions of the Susquehanna River Basin, drowning the valleys inland for almost 180 statute miles (290 kilometers). As seawater intruded into the lower reaches of the Susquehanna Basin, the Chesapeake Bay and the estuarine portions of all its tributaries, including the Potomac River, were formed.

Today, the Potomac River flows through three physiographic provinces before it enters the Chesapeake Bay. The headwaters originate high in the Appalachian Mountains near the southwest corner of Maryland. The riverine segment ends approximately 300 miles downstream at Little Falls where the river comes under the influence of the tides. The estuary starts at Little Falls and extends approximately 113 miles to the southeast where it meets the Chesapeake Bay.

Draining a portion of four states (Maryland, Virginia, West Virginia, and Pennsylvania) and the District of Columbia, the Potomac watershed has a total drainage area of approximately 14,670 square miles. Of the above total, approximately 2,540 square miles constitute the drainage of the estuarine portion of the system, not including the estuary surface area of 430 square miles.

As shown on Figure F-1, the Potomac Estuary generally meanders in a south, southeastwardly direction except for a sharp bend which occurs near the midpoint of the estuary. The estuary is relatively broad, varying in width from about 200 feet near its head to nearly 10 miles at the confluence with the Chesapeake Bay. The total length of the estuarine shoreline is estimated to be 1,121 miles.

The Potomac is a relatively shallow estuary with depths ranging from 119 feet off Mathias Point to the shallow tidal marshs and mud flats that are exposed at low tide. The overall average depth of the estuary is 19.7 feet which compares with an average of 21.2 feet for the entire Chesapeake Bay system. The average total volume of the estuary at mean low water is 5,402,000 acre-feet or about 8.7 percent of the total volume of the Chesapeake Bay and tributaries.

The Potomac Estuary is carved out of the sediments of the Coastal Plain. These sediments are unconsolidated deposits of alternating layers of sand, silt, clay, diatomaceous earth, and gravel that form an eastwardly thickening sedimentary wedge. The sedimentary layers were formed over the ages from material of both marine and western slope origin.

On top of the ancient layers are the various sediments that have been brought into the estuary in more recent times from the upper river and the estuary drainage. Soft sediments occur mostly at the head of the estuary with some sandy areas found toward the mouth. Less compacted sediments are generally found in the channels and firmer muds and clays toward shore.

The quality of the water in the estuary is constantly varying with salinity, water temperature, nutrients, dissolved oxygen, and toxic concentrations changing as a function

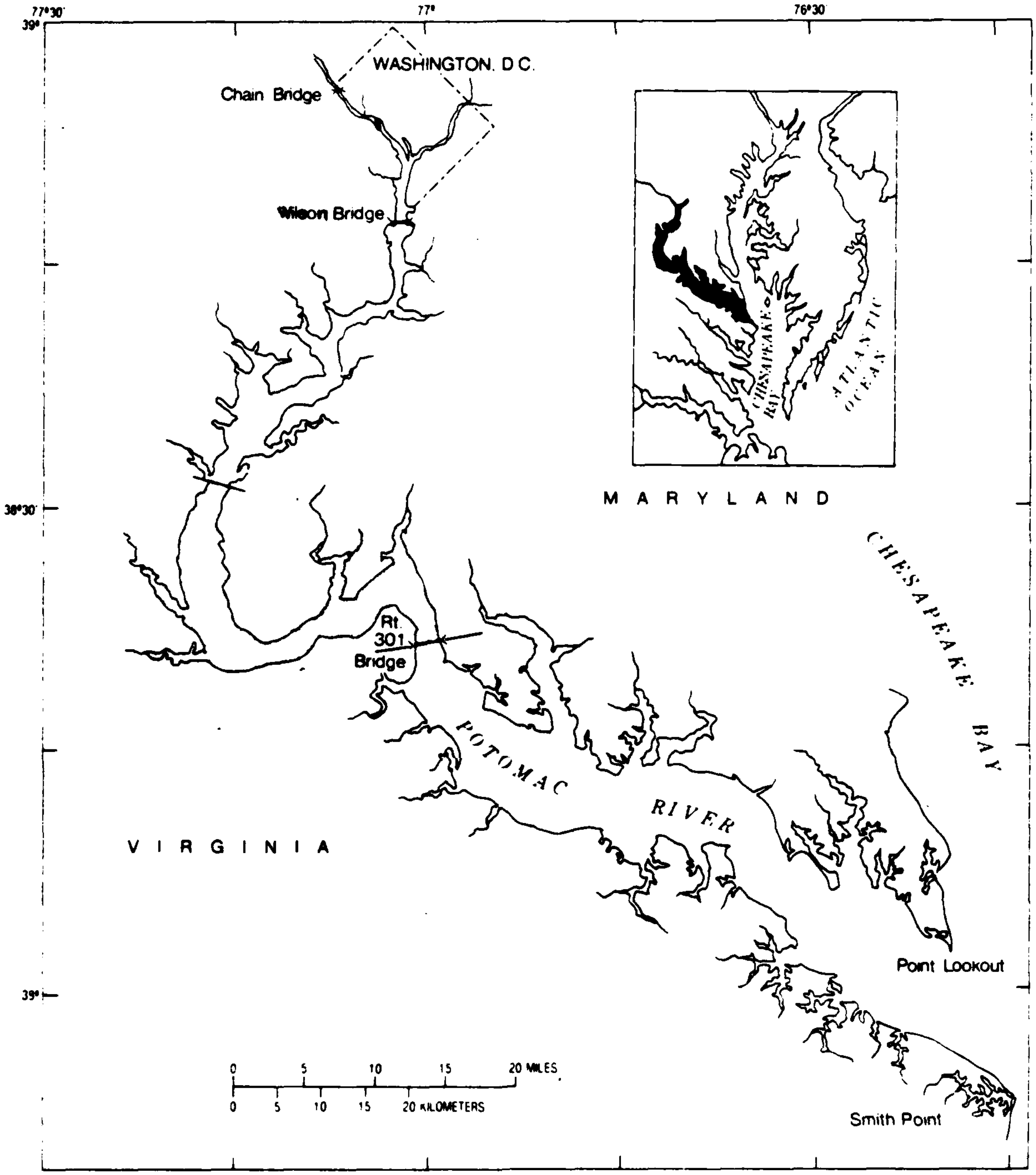


FIGURE F-1

THE POTOMAC ESTUARY

of freshwater inflow, tidal currents, and other physical processes. The main physical characteristics are summarized as follows:

a. Tidal influence extends all the way to the Fall Line.

b. For most of the year the estuary has only three (tidal fresh, oligohaline, and mesohaline) of the five possible salinity zones; however, during the fall or in extremely low flow periods, a fourth (polyhaline) zone occurs at the mouth.

c. Longitudinal salinity distributions and nontidal flows respond strongly to variations in river inflow.

d. Downstream from the tidal fresh region, the estuary develops two-layered net nontidal flow patterns.

e. Flow patterns affect salinity, temperature, and the distribution of nutrients and pollutants - all of which affect the general quality of the water.

Water transport in and out of estuaries is complex, with both salty and fresh nontidal flows taking part in flushing water through the estuarine basin. At any point in the tidal freshwater region of the Potomac, the entire nontidal flow is directed downstream and equals the cumulative freshwater input to that point. As fresh water meets denser salty water, a two-layered flow pattern develops, with the net nontidal flow in the upper layer directed downstream and the net nontidal flow in the lower layer directed upstream. In salty regions, the volume of water transport in both of these layers may at any point exceed the freshwater river input, but the difference between them is still equal to the cumulative freshwater input to that point.

Tides in the Potomac Estuary are semidiurnal, with a period of 12.4 hours. Tidal heights and currents bear a complex relationship to each other along the length of the estuary. The tidal motion is greatly affected by the shape of the entire basin, as well as by local features. The progression of the tidal wave in the Potomac is clearly seen when tidal heights along the estuary are compared to the rise and fall of the tide at Washington, D.C. When the tide is highest at Washington, D.C., the lowest tide occurs midway up the estuary near Morgantown, Maryland (nautical river mile 40). Conversely, during low tide at Washington D.C., the highest tide occurs near St. Catherine Island, Maryland (nautical river mile 28). The mean range of tides varies between 0.88 and 0.30 meters at Washington D.C. and Maryland Point, Maryland respectively.

During most of the year, the Potomac Estuary has only three salinity zones - tidal fresh, oligohaline, and mesohaline. The estuary is usually fresh above Indian Head and mesohaline from Colonial Beach downstream to the mouth, except during fall, when a short-lived tongue of polyhaline water may extend along the bottom near the mouth. During low flows, this zone may extend as far upstream as Piney Point. Maximum salinities at the mouth have been as high as 20 ppt (parts per thousand), but the long-term monthly averages reach only about 12 ppt.

There are marked salinity differences across the estuary. Gravity is the strongest influence in the vertical distribution, but other forces (e.g., the Coriolis force due to the earth's rotation and centrifugal force due to the curves of the estuary) affect the lateral distribution. The Coriolis force is important near the surface in broader portions of the estuary, whereas the centrifugal force may become more important in the bends of the estuary. For example, these forces may cause the surface salinities along the Maryland shore to be 2 or 3 ppt greater than those along the opposite Virginia shore. In the lower estuary, the saltier waters are channelized along the bottom by basin topography. This leads to cross-sectional salinity patterns that are different from those upstream.

Vertical salinity profiles show increasing salinities with depth. Typically, the most pronounced salinity stratification occurs in spring (May), and the least in fall (September and October). Depending on location and season, the differences in salt content between surface and bottom or from one shore to the other can be less than 1 ppt or as great as 6 or 7 ppt. Average monthly values do not necessarily reflect the magnitude of the difference at any one time.

There is a very direct relationship between freshwater inflows and longitudinal salinity distributions. During a consistently low flow year (1969), salinities remained fairly evenly distributed along the estuary throughout all seasons. In contrast, in a normal flow year (1970), each peak of discharge is reflected in the downstream movement of fresher waters.

Heavy storms and droughts cause substantial changes in salinity. In the early summer of 1972, Hurricane Agnes dropped the salinities in the estuary so suddenly and dramatically that virtually fresh water (less than 1 ppt) was brought downstream to within 11 nautical miles of the mouth. A return to normal season conditions was slow, and two months after the storm, the surface salinities at the mouth were 9 to 10 ppt, still below the seasonal norm of 13 to 15 ppt.

The occurrence of such extreme conditions is significant, because the single most important factor affecting aquatic life in the estuary is salinity. Each of the Potomac's three major salinity zones (plus a small temporary extension of a fourth) contain characteristic communities. The upper tidal fresh region of the Potomac is characterized by anadromous fish spawning and nursery areas and by areas of high phytoplankton and zooplankton productivity. The oligohaline region has major fish nursery areas, large forage fish populations, some anadromous fish spawning, and moderately large phytoplankton and zooplankton populations. The lower half, a typical mesohaline estuarine region, has large populations of forage fishes, predator fishes, and blue crabs and highly productive oyster and clam grounds. Any drastic changes in salinity, such as those caused by radical climatic events or large-scale water diversions, will probably affect the reproduction and growth of these populations.

The above information is provided as a brief overview of several of the more important physical characteristics of the Potomac Estuary. Subsequent sections of this appendix will provide the results of more detailed analyses of the estuary that were conducted as part of the study.

THE ESTUARY AS A SOURCE OF WATER SUPPLY

History

One of the alternatives which has long been advocated to alleviate projected water supply deficits in the MWA is the use of the Potomac Estuary. However, because of its uncertain composition, complex biological and chemical interactions, unknown environmental impacts of freshwater withdrawals on salinity regime, and other aquatic and biotic uncertainties, the use of estuary water was always considered questionable.

Because of these unknown factors, in many of the previous studies concerning the MWA water supply, the Potomac Estuary was not considered a viable water supply alternative. These studies did, however, indicate the need for further investigations and a full analysis of the estuary water. As a response to various reports and recommendations, the Congress in Section 85 of the Water Resources Development Act of 1974 (Public Law 93-251, 7 March 1974), directed an investigation of alternative solutions for the potential water supply problems in the MWA. Section 85b(2) authorized an investigation and study of the use of Potomac Estuary water and the feasibility of using estuary water as a source of water supply for the MWA.

Emergency Estuary Water Pumping Station

The temporary Emergency Estuary Water Pumping Station (EEWPS) was authorized by Supplemental Appropriation Act of 1971, Public Law 91-665, and is located on the Potomac Estuary about 1,000 feet upstream of Chain Bridge. The construction of the project began in December 1976, and was completed in spring of 1979 at a cost of $2.8 million.

The EEWPS is capable of withdrawing a maximum of 100 million gallons per day (mgd) of water from the Potomac Estuary during emergency low flow conditions in the freshwater portion of the Potomac River. The estuary water would be pumped to the Dalecarlia and McMillian Water Treatment facilities for mixing with freshwater and subsequent treatment. The purpose of EEWPS is to mitigate the severest impacts of a water deficit in the MWA, until such time as other means are available to supplement the water supply sources of the MWA.

It is important to note that the pumping station will operate only under emergency conditions when flow in the Potomac River is not sufficient to meet the needs of Washington Aqueduct Division's service area. Prior to operation of EEWPS, mandatory restrictions implemented by the Metropolitan Washington Council of Governments (MWCOG) would have to be in effect and the water supply would have to be acceptable for use by the Public Health Service. (To date, it has not been necessary to use the EEWTP, and it is presently planned to deactivate this facility after Little Seneca Lake is constructed).

POTOMAC ESTUARY EXPERIMENTAL WATER TREATMENT PLANT

As noted above, Section 85b(2) directed the Corps of Engineers to study the feasibility of using the Potomac Estuary as a source of water supply. The authorization further directed the construction, operation, and evaluation of an experimental project for the treatment of estuary water. The purpose of the plant was to determine the feasibility of producing potable water from the Potomac River Estuary. The experimental plant was located on a two-acre site at the District of Columbia Blue Plains Water Pollution Control Plant. The plant was designed for a 1.0 mgd maximum flow rate with unit processes that, based on the present knowledge and technology, could produce treated water for many uses. The following section of this appendix presents a more detailed discussion of the plant and testing program.

The information included relative to the Potomac Estuary Experimental Water Treatment Plant (EEWTP) was taken in whole or in part from reports prepared by the firm of James M. Montgomery (JMM), Consulting Engineers, Inc., which was hired in May 1980 to conduct the three-year program of operation, maintenance, and performance evaluation of the demonstration plant.

PROJECT OBJECTIVE

The overall objective of the EEWTP project was to determine the technical and economic feasibility of using the Potomac River Estuary as a supplemental source of potable water in the MWA. Achieving this objective required the answer to a number of key questions:

a. Using the best available analytical techniques, what quality of water could be produced by commonly used water treatment processes?

b. Was the water produced by the demonstration plant of potable quality?

c. What were the optimum process combinations which would ensure production of potable water at a minimum cost?

d. What was the operational feasibility and reliability of a water treatment plant that would be operated only intermittently?

e. Finally, what are the estimated costs of such a water treatment plant with hydraulic capacity of 200 mgd?

SCOPE OF PROJECT

The project was designed to provide answers to the above questions. Cost constraints limited project duration to three years, including approximately six months of plant start-up which was completed in March 1981, two years of plant operation between March 1981 and March 1983, and six months of plant deactivation. The final report for the EEWTP was submitted to a review committee designated by the National Academy of Sciences/National Academy of Engineering (NAS/NAE), in September 1983. This committee had the responsibility for evaluating the scientific and engineering validity of the study. At the same time, the EEWTP final report was submitted to the U.S. Congress as background data for use in evaluating proposed strategies for meeting the water supply programs in the MWA.

PROJECT ORGANIZATION

Operation, maintenance, and performance evaluation of the Potomac Estuary Experimental Water Treatment Plant required both on-site and off-site personnel to complete the negotiated scope of work. Off-site analytical services were provided by the JMM Environmental Research Laboratory in Pasadena, California. In addition to laboratory services, off-site personnel provided technical direction on several key aspects of the project, including project management, plant operations, laboratory methods, engineering, data management, and procurement. Because of the size and broad scope of this project, a Technical Advisory Committee (TAC) was established, consisting of JMM personnel who were recognized experts in key project areas.

FACILITY DESCRIPTION

In this section, the EEWTP facilities are described as they were built and modified for project operation. Treatment processes and plant instrumentation are described, and plant flow schematics and design criteria are presented.

Process Schematics

Schematics of the unit processes, flow schemes, and instrumentation for the different aspects of the EEWTP are presented as Figures F-2 through F-4. These schematics were developed by JMM to provide updated and easily visible information regarding process flow capabilities, process instrumentation for continuous monitoring (location of flow meters, pH monitoring and sample points, and solids handling capabilities). Three schematics are provided representing the water treatment processes (both the main processes and the total dissolved solids (TDS) sidestream treatment processes), the sludge handling and chemical recovery processes, and the chemical handling systems, respectively.

The EEWTP consisted of a combination of physical-chemical unit operations designed to produce potable water from a highly contaminated source. Contaminants which could be removed included suspended material (particulates), heavy metals, organic contaminants, inorganic dissolved solids, and pathogenic microorganisms. A flow schematic of unit processes used to accomplish the treatment objectives is illustrated in Figure F-2.

Particulate matter degrades water quality from both aesthetic and health standpoints. Pathogens associated with particulates (such as S. typhosa and the hepatitus virus) and asbestos fibers pose potential health risks. Particulates also produce indirect health risks due to the adsorption of heavy metals, organics, and viruses, and the protection of pathogens from disinfection. In the EEWTP, particulates were removed by microscreens, coagulation-flocculation sedimentation, and filtration. In addition, some particulates remaining after filtration were captured on granular activated carbon.

Metals can pose both chronic and acute health risks depending upon concentration and specification. Some metals, such as iron and manganese, cause aesthetic and operational problems with finished water use. Metals which were absorbed on or entrapped by particulates were removed as previously described. Iron and manganese were removed by a combination of oxidation and subsequent filtration, with the success of this process dependent upon pH, oxidant reaction time, and other factors. Soluble metal species were partially removed by absorption on the granular activated carbon. Additional removals of soluble species were achieved by reverse osmosis, ion exchange, or electrodialysis.

A variety of organic compounds were present in the EEWTP influent. Two types of trace organic compounds of concern were synthetic organic chemicals and halogenated or oxidized by-products of disinfection. Some compounds in these groups are known animal or human carcinogens and have been the focus of considerable attention by water treatment authorities in recent years. Natural organics are not of direct health significance but are of concern because they act as precursors for the formation of trihalomethanes and other halogenated organics.

Removal of organics was dependent upon the nature of the organics compound. Of the natural organics, approximately 30-50 percent removal was achieved through coagulation-flocculation-sedimentation process, 5-15 percent through the filters, and about 20-30 percent (primarily dissolved organics) through the activated carbon. Synthetic organic compounds were removed by adsorption on activated carbon, although some removal was accomplished by adsorption on particulates and loss to the atmosphere by mechanical aeration.

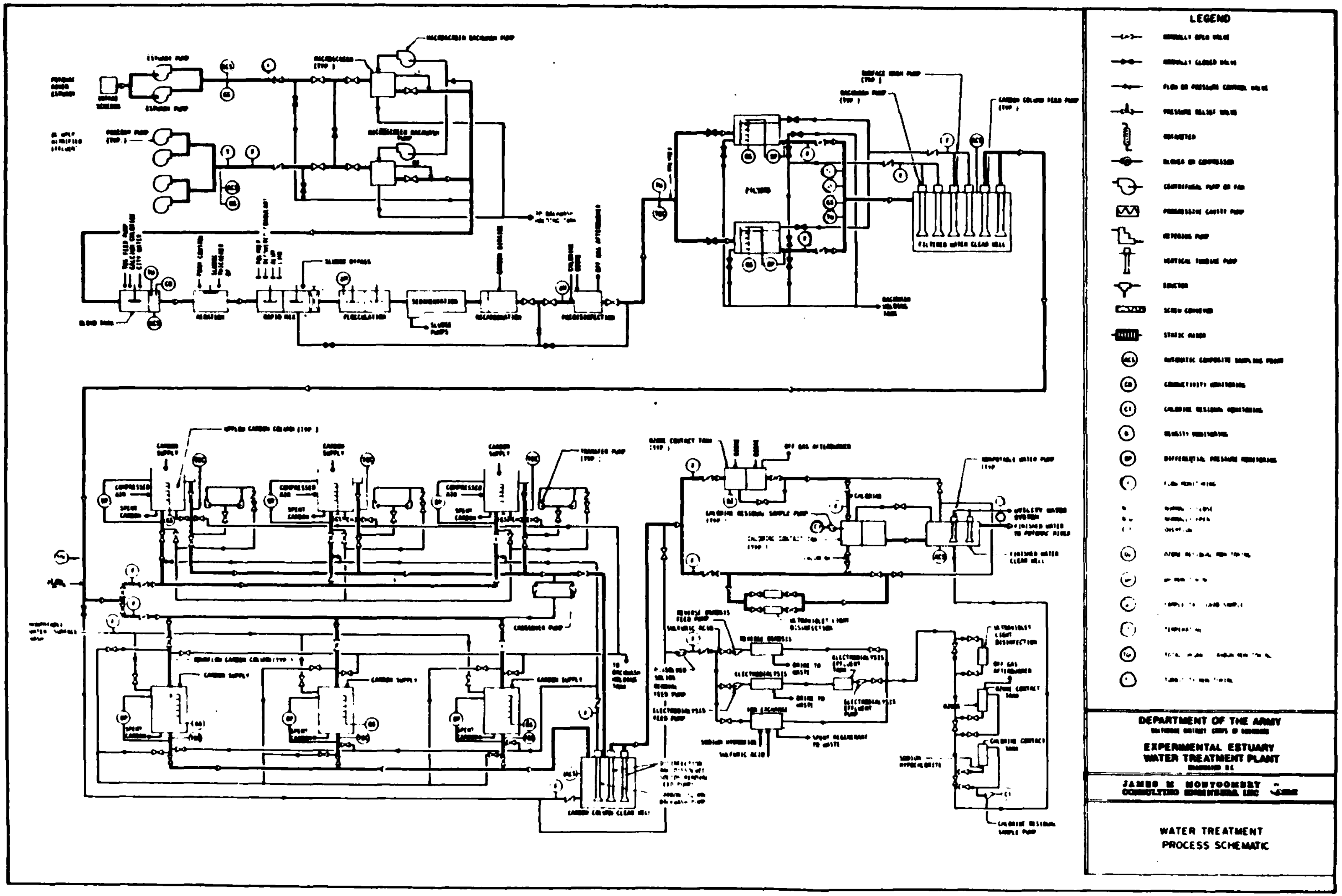


FIGURE F-2

SCHEMATIC OF THE UNIT PROCESSES

Microbial contaminants, including bacteria, viruses, and protozoa, were removed to some extent with the particulate removal process. Disinfection, however, was the principal barrier against microbial contaminants. Intermediate disinfection was provided through either chlorine or ozone, and final disinfection was accomplished by any one of five combinations of processes: chlorine, ozone, ultraviolet (UV), UV followed by chlorine, or ozone followed by chlorine.

Evaluation of dissolved solids removal was accomplished using a seven gallons per minute (gpm) sidestream from the carbon column effluent. Electrodialysis, reverse osmosis, or ion exchange were tested for removal of inorganic dissolved solids. Following demineralization, the sidestream was subjected to either the chlorine, ozone, or UV disinfection processes.

The unit processes for solids handling and chemical recovery are outlined in Figure F-3. The bold lines indicate the process flow scheme with operation for alum recovery, when alum was the primary coagulant. Basic unit processes included sludge thickening, sludge acidulation, centrifugation, and sludge disposal via truck transport to disposal or (possibly) incineration. Centrate was recirculated to the rapid mix tank or aeration basin, depending upon which centrifuge and sludge disposal technique was in use.

With an operation using lime and magnesium bicarbonate for coagulation, chemical recovery unit processes included sludge thickening, sludge carbonation, centrifugation, and sludge incineration. Lime and carbon dioxide (CO_2) were recovered from the lime furnace. Centrate containing recovered magnesium was clarified in the clarifying centrifuge (sludge to disposal via truck) and recirculated to the rapid mix tank.

Figure F-4 shows the chemical handling processes for the various dry and wet chemicals which were used at the EEWTP. The systems were relatively straight forward, although some modification and repair work was required during start-up to ensure proper operation of all systems.

Flow Control

Flow control valves were located at the following points in the plant: Blue Plains effluent line, estuary water line, filter effluent lines, filter backwash line, upflow carbon column influent line, downflow carbon column influent line, carbon column backwash line, carbon transfer lines, carbon column by-pass line, disinfection influent lines, and the TDS sidestream influent line.

During the first six months of plant start-up, JMM experienced considerable difficulty with several of the flow control systems; however, modifications were made such that adequate flow control was established for all processes. Essentially, all automatic control valves (except upflow carbon transfer valves) were set to maintain constant flow. This was accomplished by means of set point controllers in the main or local control panels.

Parameter Monitoring

In addition to the flow monitoring noted above, there was provision for continuous monitoring of several additional parameters. These various monitoring points are indicated with circles and abbreviated symbols in Figures F-2 through F-4. Table F-1 provides a more complete list, including a description of recording capabilities.

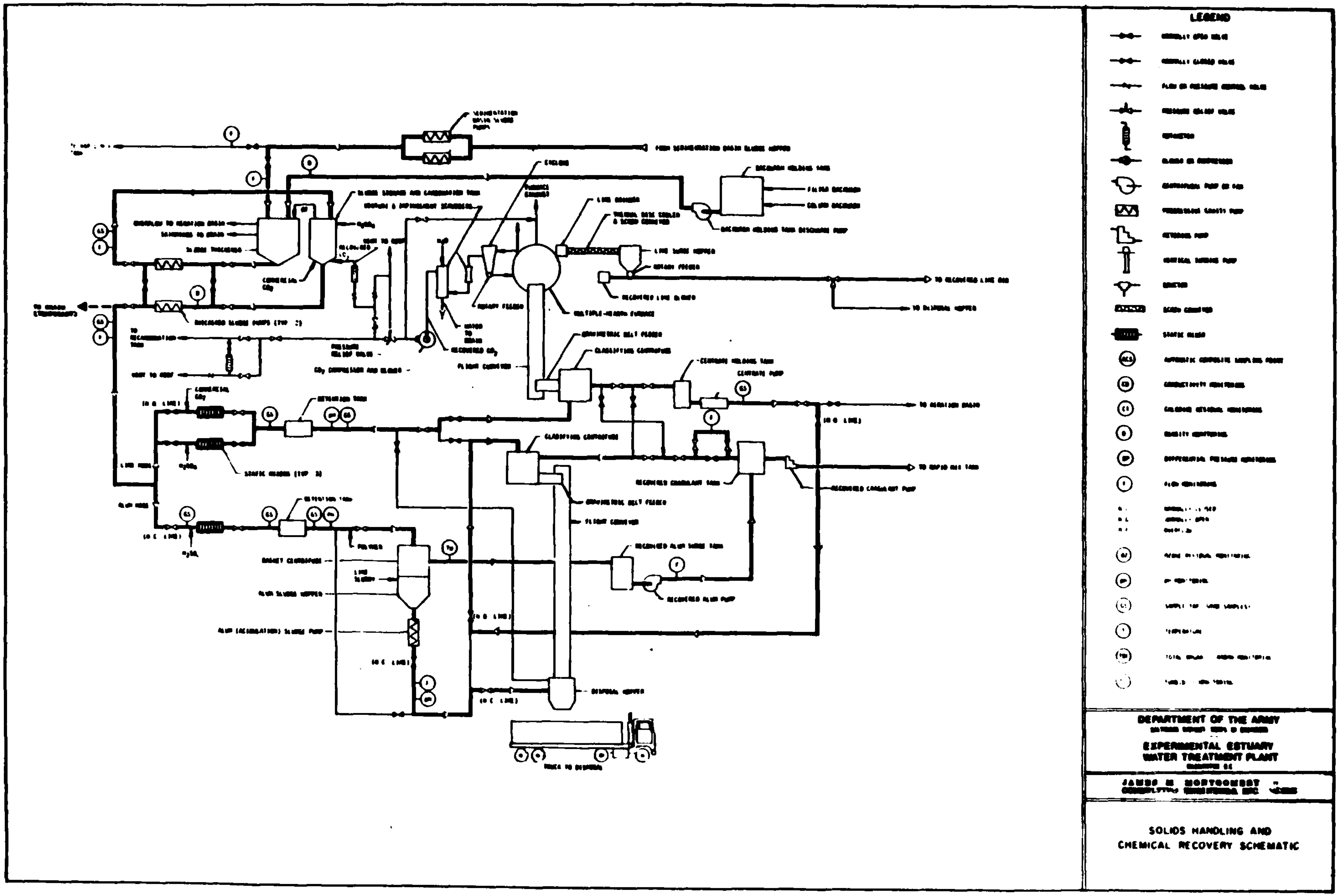


FIGURE F-3

SCHEMATIC OF THE FLOW SCHEMES

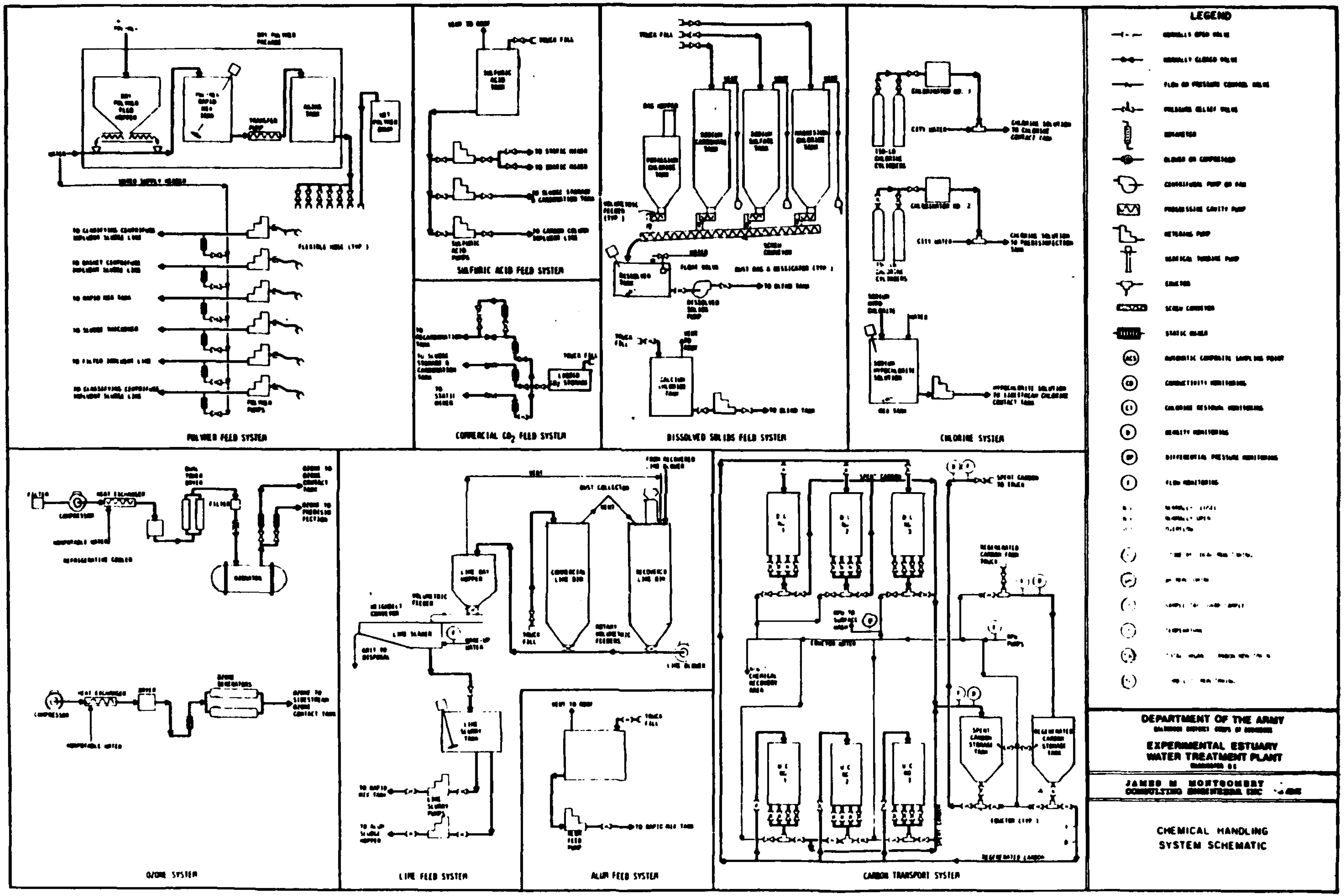


FIGURE F-4

SCHEMATIC OF THE INSTRUMENTATION

TABLE F-1

PROCESS MONITORING

	Indicating	Recording	Totalizing
pH			
Rapid Mix Tank	C		
Recarbonation Basin Effluent	C	C	
Sludge Storage Tank Effluent (to Basket Centrifuge)	L		
Sludge Storage Tank Effluent (to Bowl Centrifuges)	L		
Basket Centrifuge Cake	L		
Turbidity			
Blend Tank Effluent	L	C	
Filter Influent	L	C	
Filter No. 1 Effluent	L	C	
Filter No. 2 Effluent	L	C	
Recovered Alum	L		
Density			
Backwash Holding Tank Pump Discharge	L		
Thickened Sludge	L		
Mass Rate-Spent Carbon to Storage	L		L
Mass Rate-Spent Carbon from Storage	L		L
Mass Rate-Regenerated Carbon to Storage	L		L
Mass Rate-Regenerated Carbon from Storage	L		L
Temperature			
Estuary Water	L	C	
Blue Plains Water	L		
Ambient Air	L	C	
Furnace Hearths	L	L	
Differential Pressures			
Each Carbon Column (6 total)	L		
Filter No. 1	L	C	
Filter No. 2	L	C	
Electrical Conductivity			
Blend Tank	L		
Weight			
Sludge Cake-Classifying Centrifuge	L	L	
Sludge Cake-Clarifying Centrifuge	L	L	

TABLE F-1

PROCESS MONITORING (Continued)

	Indicating	Recording	Totalizing
Level			
Estuary Water Surface		C	
Microscreen No. 1 Influent Chamber	L,C		
Microscreen No. 2 Influent Chamber	L,C		
Filtered Water Clearwell	L,C		
Backwash Water Holding Tank	L,C		
Carbon Column Effluent Clearwell	L,C		
Finished Water Clearwell	C		
Recovered Chemical Tanks	L		
Commercial Chemical Tanks	L		
Regenerated Carbon Storage Tank		C	
Spent Carbon Storage Tank		C	
TOC			
Microscreen No. 1 Effluent	L	L	
Microscreen No. 2 Effluent	L	L	
Blend	L	L	
Filter Influent	L	L	
Filter Effluent	L	L	
First Stage Carbon Column Effluent	L	L	
Final Carbon Column Effluent	L	L	
Finished Water	L	L	

NOTES:
- L: Local
- C: Central Operations Area

WATER QUALITY GOALS

Water quality standards currently used to regulate the quality of drinking water in the U.S. are based on the assumption that the raw water source is the best one available. In many cases, these sources originate in the upper portions of watersheds and are subject to minimal human contamination. Such would probably not be the case for the influent sources to the future water treatment plant using the tidal fresh portion of the estuary. The EEWTP treated a 50:50 blend of estuary water plus nitrified effluent from the Blue Plains Wastewater Treatment Plant to simulate expected water quality under drought conditions. Given the quality of the influent to the EEWTP, meeting the U.S. Environmental Protection Agency (EPA) primary and secondary drinking standards was not considered sufficient evidence that the water produced by the EEWTP would be acceptable for human consumption.

As a consequence, criteria for plant operation and control were based on a set of water quality goals which included parameters not currently regulated by the EPA or by state health departments.

In this section, the basis for selecting water quality goals for the finished water from the EEWTP is discussed. The goals presented are not intended to imply that a water meeting these goals is potable. Rather, the goals were used as a guide for control of plant operations.

Basis for Goals

In developing water quality goals, a number of factors were considered. Literature was reviewed for up-to-date regulations or standards, both in the United States and other countries.

As a minimum, the goals proposed for the EEWTP must match existing drinking water regulations. Current United States regulations, promulgated by EPA, include the National Interim Primary Drinking Water Regulations (EPA, 1975) and the EPA Secondary Drinking Water Regulations (July, 1979). The primary regulations are based on health effects and are mandatory, while the secondary regulations are based on consumer acceptance and are recommended. EPA has also published regulations for the control of trihalomethanes (THM's) (November, 1979) and for monitoring of sodium and corrosion in distribution systems (August, 1980). In the District of Columbia, the EPA regulations govern, since there are no distinct District regulations.

Other sources of information in developing water quality goals were international drinking water standards. In particular, the European standards were helpful since they tended to consider previous contamination of sources. The World Health Organization (WHO) European standards (1970) were consulted. WHO also has international standards, but these are prepared for situations in developing countries, where these standards would be less stringent. The European community (Knoppert, 1980) has recently developed new drinking water standards which are fairly comprehensive.

More information was recently developed on water quality critria based on health effects. Some of this data will eventually be incorporated into the EPA primary regulations. A considerable amount of data on health effects was available from the National Academy of Sciences (1977 and 1980). The Office of Water Planning and

Standards of the EPA (November 1980 and August 1981) has also developed health effects criteria for the priority pollutants.

The information on health effects can generally be divided into three categories: (1) carcinogens, (2) non-carcinogens, and (3) compounds of unknown health effects. By far, the largest number of constituents which can be analyzed in drinking water fall into the latter category. Carcinogens, or substances which cause cancer, are generally believed to have health effects which are proportional to the dosage; that is, there is no "safe" level or threshold level below which no risk exists. For carcinogens where risk estimates can be extrapolated, a given concentration of the compound is associated with an additional or incremental lifetime cancer risk. These incremental risks are typically on the order of one in one hundred thousand to one in ten million. For an incremental lifetime cancer risk of one in one million, it could be estimated that one person in a population of one million drinking the water over a lifetime would get cancer who otherwise would not have. On the individual level, a single person's risk from drinking this water over a lifetime would increase by one in one million. For non-carcinogens with known health effects, the health effects of drinking the water are from toxic properties and the response is an immediate or acute one. Generally, an acceptable daily intake is calculated, then divided by a safety or uncertainty factor to obtain a drinking water criteria. Typically, the safety factors range from 10 to 1,000, depending upon how much is known about the compound and its health effects.

A final factor to consider in setting goals is consumer satisfaction. This refers to the aesthetic properties of the water, such as taste, odor, and clarity. Goals for consumer satisfaction have been established by the American Water Works Association (AWWA, 1968 and Bean, 1974) as well as the EPA (July, 1979). Meeting aesthetic criteria would be particularly important for a plant such as the proposed full-scale estuary water treatment facility, since consumers are likely to know that a portion of the water originated from treated wastewater. Therefore, the treated water quality should be at least as good as currently accepted limits in other drinking water facilities in the region.

Selection Process

The following decision steps were followed in developing water quality goals for the finished water from the EEWTP:

a. The goal was to at least comply with current applicable EPA regulations.

b. The water quality was to be acceptable to consumers.

c. Treated water quality for regulated parameters was to be equivalent in quality to levels achieved by current practice in well-operated water treatment plants.

d. Health risks were to be minimal or negligible. Where data was available on health effects, negligible health risks were defined as follows: (1) for carcinogens, a one in one million incremental lifetime cancer risk and (b) for non-carcinogens, a safety factor of 100 using the acceptable daily intake.

Goals

Water quality goals were established for the majority of parameters tested at the EEWTP. These goals for water quality are summarized in Table F-2. This table

TABLE F-2

LISTING OF SELECTED WATER QUALITY GOALS

Parameter	Units	EPA(a) MGL	NAC(b)	EPA(c) WGC	AWWA(d) Goal	State(e) Stds	WHO(l) Eur	EC(g)	Survey of Industrial Practice(b)	Proposed Goal	Comment
Microbiological -											
Total Coliform	org/100 ml	1	-	-	0.1	1(e)	1	1	-	0.1	achievable
S.P.C.	org/ml	500(u)	-	-	-	-	-	100	-	50	achievable
Ent. Virus	pfu/l	-	-	-	-	0.025(w)	1	-	-	0.01	achievable
Inorganics -											
Arsenic	mg/l	0.05	0.05	2.2(l)	0.01-0.1	0.05(e)	0.05	0.05	-	0.01	MCL high
Lead	mg/l	0.05	0.025	0.05	0.05	0.05(e)	0.1	0.05	-	0.025	MCL high
Cadmium	mg/l	0.01	0.005	0.01	0.01	0.01	0.01	0.005	-	0.01	achievable
Mercury	mg/l	0.002	0.002	144(y)	0.002	0.002	-	0.001	-	0.002	achievable
Nitrate	mg/l	10	10	-	-	10(e)	11-22	6	-	5	MCL high
Asbestos	Mf/l	-	-	0.03(j)	-	-	-	-	-	0.1	MDI
Chromium	mg/l	0.05	-	170(III) 0.05(VI)	0.05	0.05(e)	0.05(VI)	0.05	-	0.05(VI)	only issue
Physical -											
Turbidity	T.U.	1	-	-	0.1	0.5(e)	-	0.4	0.2(n)	0.2	achievable
Particulates (22.5 microns)	No./ml	-	-	-	-	-	-	-	50(n)	50	achievable
Corrosivity (gen'l)		non-corr	-	-	control	rel. low(e)	-	-	-	-	Undefined
Lang. Indx (LI)	pH U.	monitor	-	-	-	-	-	-	-	-0.1 LI 0.3	A/C pipe
Cooper	mpy	-	-	-	-	-	-	-	0.2(x)	0.2	achievable
Galv	mpy	-	-	-	-	-	-	-	2(x)	2.0	achievable
Iron	mpy	-	-	-	-	-	-	-	6(x)	6.0	achievable
Color	a.c.u.	15	-	-	3	15(e)	-	1	-	3	achievable
Organics -											
TOC	mg/l	-	-	-	-	-	-	(s)	-	3.0(r)	THM control
TOX	ug/l	-	-	-	-	-	-	1.0(p)	150-200(Q)	150	achievable
TTHM's	ug/l	100	0.29(j)	0.19(j)	-	100(e)	-	(l)	-	50	most impt SOC
PCE	ug/l	3.5(k)	3.6(j)	0.8(j)	-	4(l)	-	-	-	4	low health risk
TCE	ug/l	-	4.5(k)	2.7(j)	-	4.51	-	-	-	4.5	low health risk
DDT	ug/l	50,000(v)	42	0.024	-	-	-	100(m)	-	50	achievable
Lindane	ug/l	4	0.054(j)	0.0186(j)	-	4(e)	-	0.1(m)	-	0.05	low health risk

TABLE F-2 (continued)

LISTING OF SELECTED WATER QUALITY GOALS

a. Primary or secondary MCL in National Interim Primary DWS.
b. Value derived from NAS report, vols I thru III.
c. Value from EPA "Water Quality Criteria", Fed. Reg.
d. AWWA Water Quality Goals 1968.
e. California Dept of Health Services 1977.
f. World Health Organization, Europena stds for Dr. Wir, 1970.
g. European Economic Community, draft standards, 1980.
h. Information on industrial practice from various sources.
i. This As value is in ng/l for 1/1,000,000 risk.
j. Values correspond to 1/1,000,000 increment risk.
k. Values correspond to 1/1,000,000 risk in EPA SNARL.
l. "Action Level" recommended by Calif. Dept of Health Serv.
m. Pesticides and related product considered separately.
m. JMM 1976 Survey of WTP's in the SW U.S.
p. Maximum sum of non-pesticide organlo-chlorine compounds.
q. W.F. 21 measurements by Jekel and Roberts ES&T 1980.
r. Based on achievability of THM goal.
s. Increases above background must be investigated.
t. As low as possible.
u. Proposed by EPA 3/13/75, not adopted.
v. Described by EPA 3/14/75, recommended by NAS, 1972.
w. Arizona std for wastewater discharge to effluent-dominated streams.
x. JMM 1980 report for Contra Costa CWD.
y. This value in ug/l.

describes the various physical, microbiological, inorganic, organic, and radiological parameters for treated water quality. Where data was available, columns show regulations from the EPA, International Standards from the World Health Organization and European Community, suggested criteria for minimizing health effects, and levels that were achieved at well-operating water treatment plants. Finally, the actual goals for the EEWTP are listed in the last column.

SELECTION OF INFLUENT CONDITIONS FOR THE EEWTP

Introduction

The principal objective of the EEWTP was to determine the technical and economic feasibility of using the Potomac River Estuary as a supplemental water supply source for the Metropolitan Washington Area during periods of severe and sustained drought. Thus, the EEWTP was designed with facilities for using two influent sources to simulate projected water quality levels which might be reached at Chain Bridge (one possible site of the future water treatment plant) during a drought. One source is the Estuary, the other, nitrified, unchlorinated effluent from the Blue Plains Wastewater Treatment Plant.

According to the "Special Study Report," February 1979, prepared by Malcolm Pirnie, Inc. (MPI), a mixture of one part nitrified effluent and two parts estuary water was originally recommended to simulate future water quality characteristics. This particular mix was recommended on the basis of the following assumptions:

a. 100 mgd minimum flowby in the river during the drought,
b. hydraulic conditions equivalent to the worst drought in the hydrologic record (1930-1931),
c. 90-day drought duration,
d. 2030 monthly average water demands,
e. estuary withdrawal rate of 200 mgd, and
f. future estuary pump station located close to Chain Bridge.

The specified mix was considered conservative, and consistent with the results of estuary modeling studies of total dissolved solids using the EPA Dynamic Estuary Management (DEM) model. It was also stressed that a "perfect simulation" of the water quality in the intake of a future estuary water treatment plant was unlikely.

At the time of preparation of the Special Study, selection of the appropriate influent mix was hampered by lack of monitoring data on the two available influents, especially with respect to parameters of potential health significance including synthetic organic chemicals, total organic halogen (TOX), metals, asbestos fibers, viruses, and other potentially pathogenic microorganisms. Since February 1979, questions were also raised on the suitability of the hydraulic boundary conditions, and of the water use increments applied in the DEM model. As a consequence of these concerns, the rationale for the specified influent mix was reevaluated by the JMM during the contract start-up period, using more recent influent water quality monitoring data, and revised boundary conditions for the DEM. No attempt was made to recalibrate the DEM model, however.

Limitations on Selection Procedure

The objectives of the contractor modeling and monitoring efforts were as follows:

a. To estimate the worst water quality levels for key parameters at Chain Bridge under 1930 drought conditions and 2030 demands.

b. To estimate the expected water quality levels in the estuary intake and the Blue Plains nitrified effluent during the operation of the EEWTP.

c. To recommend a blend of estuary water and nitrified effluent that simulated the worst expected water quality levels during operation of the full-scale emergency water treatment plant.

Selection of the appropriate blend of the two influent streams to the EEWTP was complicated by inherent limitations in the available data base. An ideal strategy would be to first predict the frequency distribution of all water quality parameters in the estuary with any health significance under 2030 drought conditions, and then compare these distributions with the expected distribution of each parameter in the two influent streams to the EEWTP. Unfortunately, pursuit of this strategy was hindered by: 1) the inability of estuary water quality models to predict the fate of parameters of health significance; 2) limitations in the selection of appropriate model boundary conditions; and 3) the limited amount of water quality data obtained from monitoring of the EEWTP influents.

Although prediction of all parameters with significant health effects was not possible with the presently available models, it was possible to estimate conservative pollutant transport in the Estuary under drought conditions with a revised implementation of the DEM model.

Previous Modelling Efforts

The available methods for estimating future estuary water quality range from simple steady-state reactor models to highly complex diffusion and flow simulations. Two substantially different models were applied to the Potomac Estuary in previous studies. Each served to estimate the potential worst water quality at a hypothetical estuary water treatment plant.

The Hydroscience Model

In 1973, Hydroscience, Inc. completed a feasibility study for the Corps of Engineers on the use of water from the Potomac Estuary to supplement the water supply of the Metropolitan Washington Area under drought conditions. At the time of the study, the design capacity of the proposed water supply facility was not fixed. The Hydroscience analysis estimated the worst water quality at the head of the estuary near Chain Bridge for a number of minimum streamflows and plant recycle rates, dependent on three alternative reservoir construction programs.

The following pollutants were modeled: total dissolved solids, chlorides, coliforms, total phosphorus, 5-day biochemical oxygen demand, organic nitrogen, nitrate nitrogen, nitrite nitrogen, ammonia nigtrogen, algal nitrogen, and stream-dissolved oxygen. The model used was a steady-state, one-dimensional segmented estuary model, with a single dispersion constant to represent tidal influences. Nonpoint sources were not included.

The increase in pollutant concentrations due to municipal use was predicted by assuming a fixed increase in the concentration of each pollutant during each cycle through the municipal system.

The Hydroscience analysis supported the viability of recycling estuary water for municipal use, under a limited set of conditions. Because the model assumed steady-state conditions, used 1960 drought flow conditions rather than 1930 flows, and utilized a small number of channel segments to represent the estuary, the Hydroscience results were an insufficient basis for selecting the blend ratio.

GKY Model

As part of the development of a testing program for the operation of the EEWTP, Malcolm Pirnie, Inc. (MPI) contracted with GKY Associates to perform a more detailed analysis of the estuary system, with a new model and revised boundary conditions. The GKY modeling study addressed two primary issues:

a. What was the present variability of water quality parameters in the estuary near the EEWTP, and how did this variability influence the necessary frequency of sample collection?

b. What were the expected levels of TDS and other conservative constituents at the site of the future intake point under various potential drought conditions?

GKY employed the EPA's DEM model to simulate estuary water quality under both future drought conditions and present average flow conditions. For the drought simulation, only conservative parameters were considered, including TDS, total phosphorus, major anions and cations, nitrate, ammonia, and several trace metals. The model consisted of a one-dimensional, segmented flow simulation with a 90-second time step. Inflow from the Potomac River was varied monthly over a 90-day drought interval. Upstream flows in the Potomac River varied from 600 to 1,300 mgd, with water demands decreasing from 750 to 600 mgd during the simulated period. The operating capacity of the recycle plant was fixed at 200 mgd for the simulation. Additionally, the model was used to estimate present average summer and winter water quality in the estuary near the site of the EEWTP, in order to judge the variability in water quality in plant influent from the estuary during operation.

Based on the simulation results, with consideration of the probable quality of unfiltered, nitrified effluent from Blue Plains, a blend ratio of one part nitrified effluent and two parts estuary water was recommended.

Revised Modelling Efforts

The major limitation of the MPI study was the use of monthly average inflows to the estuary, which restricted the ability of the model to respond to severe low flow sequences. It was decided that a reevaluation of the estuary system was necessary given the limitations of the MPI approach. After consideration of the previous model applications and the data available for the present study, the DEM model was chosen as the most suitable for further use in estuary modeling. The major advantage of the DEM was that variable inflows and wasteloads to the estuary could be considered. Also, the DEM represents the estuary in smaller segments than do simpler steady-state models, and was able to better predict water and conservative pollutant transport in the estuary.

Accurate characterization of non-conservative pollutants in the estuary was not as certain with any model, given the complex nature of non-conservative pollutant interactions, especially under drought conditions. The present study was therefore limited to those pollutants which may be considered conservative. This also included some non-conservative parameters which undergo slow degradation or transformation in the Estuary, in which case the model predictions served as an estimate of the maximum attainable level of the pollutant in the Estuary.

The DEM model, although accurate, was highly consumptive of computer time. The 90-second hydrologic time step required by the model results in over 86,000 separate computations of water flow in each of the 139 estuary segments during a single 90-day simulation. Fortunately, the same hydrologic run may be used as a basis for many runs of the associated water quality model, which requires only a 30-minute time step to preserve the accuracy of results. Available resources for this study therefore limited consideration of alternative scenarios to only a few runs of the hydrologic model.

The principal objectives of the revised modeling efforts were to verify previous estimates of future water quality levels for conservative parameters, and to determine the sensitivity of those predictions to alternative boundary conditions, including variable use increments, alternative withdrawal sites for the future water treatment plant, and extreme value situations. These alternative conditions included:

a. the effect on conditions in the Estuary of varying upstream flow daily instead of using monthly averages;

b. the effect of relocation of the proposed plant intake to a point below the head of the Estuary;

c. the potential for exceedingly high pollutant levels during an extended period of zero flowby from the Potomac River to the Estuary;

d. the effect of a short or long-term breakdown in the treatment efficiency of the Blue Plains Wastewater Treatment Plant on conservative pollutants; and,

e. the variation in pollutant loading observed with changes in municipal-use increments and background pollutant levels.

Further additions to the modeling analysis were considered, but were excluded, primarily for technical reasons. The inclusion of nonpoint source (NPS) and combined sewer overflow (CSO) loadings was considered unnecessary, as these effects would likely contribute little during an extended drought. Non-conservative pollutants were also excluded from the analysis, because of the questionable accuracy of the kinetic parameters used in the water quality model. The DEM model was previously calibrated for channel and flow parameters in the Potomac Estuary.

The version of the DEM used to predict the impact of the above-listed conditions on future water quality was the same as the model used by GKY Associates in their study of the Estuary. The following changes were made to the model boundary conditions and use parameters for the alternative conditions modeled:

a. The inflow to the estuary from the Potomac River was varied daily instead of monthly, as in the GKY runs. The 1930 observed flows in Table F-3 are the observed flows entering the estuary at Chain Bridge, from July to December, 1930, after the removal of approximately 300 cubic feet per second (cfs) by the Dalecarlia plant at that time. The simulated flows at Chain Bridge in Table F-3 are derived from the 1930 observed flows by subtracting 450 cfs from the observed flows, which is assumed to be the amount of additional water withdrawn upstream of Chain Bridge by other water treatment plants under 2030 demand conditions. By this method, flowby reaches zero on one day during the simulation (25 August), and remains over 100 mgd during most of the simulated period. Inflow from the Anacostia River is small compared to the Potomac flow, and was therefore held constant at 20 cfs.

b. Total municipal demand was held constant at 1,550 cfs (1,000 mgd) during the six month simulation period for all alternatives modeled.

c. The five-month simulation was initiated with moderately low flow conditions on 16 June 1930. The estuary water treatment plant was assumed to begin operation on 14 August. Water withdrawal was maintained constant at 200 mgd (310 cfs) for the remainder of the simulation in all model runs, ending on December 15.

d. The water deficit between the fixed demand and the river plus estuary supplies was assumed to be satisfied by neighboring supplies (including the Dalecarlia plant). Before the estuary plant began operation, outside supply equaled the fixed demand of 1,550 cfs. After the plant began operation on August 14, the outside supply equaled the demand (1,550 cfs) minus the estuary plant contribution of 310 cfs, or 1,240 cfs.

e. Rather than performing a separate run to determine the initial dissolved solids concentrations in the estuary at the beginning of the simulation, the initial conditions provided in the GKY report were used. Flowby from the Potomac River was then held constant from June 16 to July 20 at 550 cfs in all runs, allowing the model to reach a steady-state condition before the drought period began. This situation is a reasonable approximation to the actual flow conditions in the river and estuary in 1930.

f. Wastewater pollutant flux was computed by assuming that ten percent of water withdrawn from the river, estuary, and external sources was lost to consumption, and that the remaining flow contained a TDS-use increment of 400 milligrams per liter (mg/l) (as used in the GKY report). As will be pointed out later, model results for any other desired use increment or conservative pollutant may be simply derived from the results for the 400 mg/l TDS.

g. Potomac River water and all outside source water was given a background TDS level of 180 mg/l. Results for other background concentrations may be directly derived from the results for the 180 mg/l TDS level, as discussed subsequently. Municipal water was delayed four days between withdrawal from the estuary and disposal through the various waste treatment plants. The percentage of waste flow leaving through each waste treatment plant was equivalent to those percentages used in the GKY model runs (the Blue Plains plant handled over 70% of waste flow).

TABLE F-3

FLOWBY AT CHAIN BRIDGE DURING 183-DAY SIMULATION

Day	JULY 1930 obs. (cfs)	JULY Chain Bridge (cfs)	JULY Chain Bridge (mgd)	AUGUST 1930 obs. (cfs)	AUGUST Chain Bridge (cfs)	AUGUST Chain Bridge (mgd)	SEPTEMBER 1930 obs. (cfs)	SEPTEMBER Chain Bridge (cfs)	SEPTEMBER Chain Bridge (mgd)
1	-	550.	355.	602.	152.	98.	665.	215.	139.
2	-	550.	355.	594.	144.	93.	594.	144.	93.
3	-	550.	355.	570.	120.	77.	665.	215.	139.
4	-	550.	355.	594.	144.	93.	654.	204.	132.
5	-	550.	355.	621.	171.	110.	594.	144.	93.
6	-	550.	355.	632.	182.	117.	578.	128.	83.
7	-	550.	355.	594.	144.	93.	562.	112.	72.
8	-	550.	355.	654.	204.	132.	546.	96.	62.
9	-	550.	355.	698.	248.	160.	578.	128.	83.
10	-	550.	355.	610.	160.	103.	752.	302.	195.
11	-	550.	355.	610.	160.	103.	1020.	570.	368.
12	-	550.	355.	610.	160.	103.	736.	286.	185.
13	-	550.	355.	578.	128.	83.	632.	182	117.
14	-	550.	355.	562.	112.	72.	654.	204.	132.
15	-	550.	355.	562.	112.	72.	832.	382.	246.
16	-	550.	355.	516.	66.	43.	784.	334.	215.
17	-	550.	355.	495.	45.	29.	832.	382.	246.
18	-	550.	355.	481.	31.	20.	832.	382.	246.
19	-	550.	355.	554.	104.	67.	736.	286.	185.
20	934.	484.	312.	594.	144.	93.	676.	226.	146.
21	880.	430.	277.	546.	96.	62.	621.	171.	110.
22	768.	318.	205.	509.	59.	38.	602.	152.	98.
23	768.	318.	205.	495.	45.	29.	621.	171.	110.
24	1120.	670.	432.	474.	21.	15.	643.	193.	125.
25	898.	448.	289.	450.	0.	0.	676.	226.	146.
26	709.	259.	167.	502.	52.	34.	709.	259.	167.
27	632.	182.	117.	516.	66.	43.	698.	248.	160.
28	768.	318.	205.	621.	171.	110.	594.	144.	93.
29	736.	286.	185.	632.	182.	117.	676.	226.	146.
30	709.	259.	167.	602.	152.	98.	602.	152.	98.
31	632.	182.	117.	578.	128.	98.	-	-	-

* June 16 - July 19: 550 cfs (355 mgd) at Chain Bridge.

TABLE F-3 (Continued)

FLOWBY AT CHAIN BRIDGE DURING 183-DAY SIMULATION

Day	OCTOBER 1930 obs. (cfs)	OCTOBER Chain Bridge (cfs)	OCTOBER Chain Bridge (mgd)	NOVEMBER 1930 obs. (cfs)	NOVEMBER Chain Bridge (cfs)	NOVEMBER Chain Bridge (mgd)	DECEMBER 1930 obs. (cfs)	DECEMBER Chain Bridge (cfs)	DECEMBER Chain Bridge (mgd)
1	632.	182.	117.	523.	73.	47.	1040.	590.	381.
2	632.	182.	117.	481.	31.	20.	829.	379.	245.
3	602.	152.	98.	530.	80.	52.	632.	182.	117.
4	621.	171.	110.	610.	160.	103.	698.	248.	160.
5	594.	144.	93.	709.	259.	167.	1140.	690.	445.
6	523.	73.	47.	720.	270.	174.	1120.	670.	432.
7	488.	38.	25.	594.	144.	93.	1020.	570.	368.
8	555.	105.	68.	594.	144.	93.	988.	538.	347.
9	530.	80.	52.	676.	226.	146.	848.	398.	257.
10	516.	66.	43.	698.	248.	160.	1100.	650.	419.
11	516.	96.	62.	610.	160.	103.	1280.	830.	535.
12	516.	66.	43.	698.	248.	160.	1180.	730.	471.
13	586.	136.	88.	698.	248.	160.	1490.	1040.	671.
14	538.	88.	57.	665.	215.	139.	1830.	1380.	890.
15	530.	80.	52.	610.	160.	103.	2020.	1570.	1013.
16	530.	80.	52.	676.	226.	146.	-	-	-
17	495.	45.	29.	784.	334.	215.	-	-	-
18	562.	112.	72.	768.	318.	205.	-	-	-
19	546.	96.	62.	1010.	560.	361.	-	-	-
20	698.	248.	160.	1360.	910.	587.	-	-	-
21	643.	193.	125.	880.	430.	277.	-	-	-
22	687.	237.	153.	709.	259.	167.	-	-	-
23	709.	259.	167.	768.	318.	205.	-	-	-
24	698.	248.	160.	709.	259.	167.	-	-	-
25	736.	286.	185.	736.	286.	185.	-	-	-
26	643.	193.	125.	610.	160.	103.	-	-	-
27	488.	38.	25.	988.	538.	347.	-	-	-
28	516.	66.	43.	546.	96.	62.	-	-	-
29	562.	112.	72.	467.	17.	11.	-	-	-
30	562.	112.	72.	562.	112.	72.	-	-	-
31	594.	144.	93.	-	-	-	-	-	-

Model Results - Base Conditions

Each of the modeling runs performed were distinguished by the type of flowby, the location of the water treatment plant intake, and the treatment efficiency of the Blue Plains Wastewater Treatment Plant. The base condition run assumed 1930 flows in the Potomac River (minus the 300 mgd diversion), as listed in Table F-3. The plant intake was assumed to be located at Chain Bridge, the highest point in the estuary, and the Blue Plains plant was assumed to operate at zero removal for TDS. It should be noted that any conservative parameter could be used in the model, and the results manipulated to estimate concentrations of other conservative pollutants. However, TDS was selected for the base condition run in order to determine the level of TDS supplied by tidal action from the seaward boundary. It was found that there was a negligible contribution of TDS from tidal action under drought conditions which means that any conservative parameter could be used in the model irrespective of its concentration at the seaward boundary.

The projected daily variability in TDS levels at Chain Bridge and at Potomac Park (located intermediate between Chain Bridge and the Blue Plains plant) are plotted in Figure F-5 for the last ninety days of the base condition simulation. TDS levels range from the background concentration of 180 mg/l to a high of 447 mg/l for the 400 mg/l municipal use increment. This level is about 10 percent lower than the maximum projected by GKY, primarily due to the variability in flowby.

An additional run of the model with a use increment of 200 mg/l showed that the additional TDS over background levels at any time was proportional to the use increment used in the run. This validates an equation first presented in the GKY study, vis:

$$C_{max} = C_{back} + (TDS_{max} - TDS_{back}) * C_{UI} / TDS_{UI}$$

where C_{max} = maximum concentration of a conservative pollutant
C_{back} = background concentration of the pollutant
C_{UI} = municipal use increment for each pollutant

This formula assumes that all pollutants are conservative, and that the contribution of pollutants from downstream in the estuary is negligible, as has been demonstrated. From this formula, it is possible to predict critical concentrations for any conservative pollutant given its background concentration and municipal use increment.

Table F-4 presents the estimated critical concentrations for a number of pollutants, for the GDY and JMM base condition model runs. Background concentrations for the JMM results were derived from the observed water quality in the Potomac River, and use increments were taken as the difference between the background concentration and the average concentration observed in the Blue Plains secondary effluent.

The JMM maximum projected values for all constituents shown in Table F-4 are generally lower than the GKY estimates due to the lower maximum projected TDS levels. JMM estimates exceed GKY estimates for nitrate because of substantially different assumed background levels and use increments. Values shown for most heavy metals are expected to be higher than likely future levels because of metal adsorption on particulates and subsequent loss to the sediments under actual conditions.

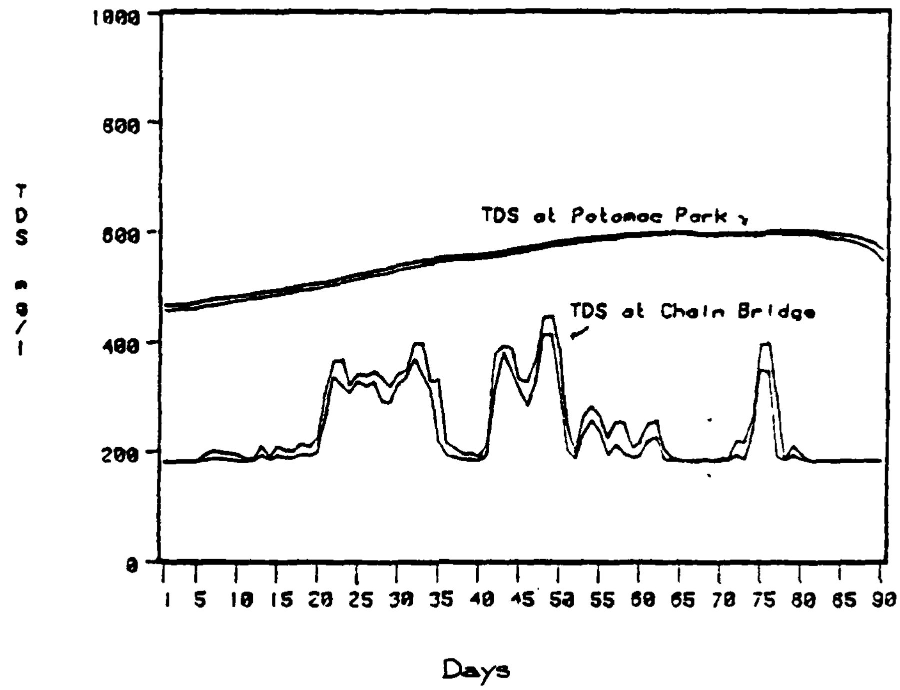


FIGURE F-5

DAILY AVERAGE AND MAXIMUM TDS,
1930 FLOWBY, CHAIN BRIDGE INTAKE
LAST 90 DAYS OF PERIOD

TABLE F-4

MAXIMUM PROJECTED POLLUTANT LEVELS AT CHAIN BRIDGE INTAKE
1930 POTOMAC RIVER FLOWBY, 2030 WATER DEMANDS
(ALL VALUES IN MG/L)

	BACKGROUND		USE INCREMENT		MAXIMUM PROJECTED LEVELS	
Parameter	GKY	JMM	GKY	JMM	GKY	JMM
TDS	281	180	400	400	488	447
Total P	.04	.09	.80	.31	.58	.30
Alkalinity	85	63	100	*	152	130
Hardness	133	91	100	*	201	158
Cl	18.5	9.4	107	*	91	81
Na	19.5	8.0	85	*	77	65
SO_4	31.6	27.2	43	*	75	56
Ca	41.4	26.9	30	*	62	47
Mg	7.0	5.9	6.0	*	11	9.9
K	3.12	2.3	7.0	*	7.8	7.0
Ni	.004	.010	.2	.05	.14	.04
Mn	.142	.096	.2	*	.28	.23
Zn	.038	.026	.2	.04	.17	.05
Al	.61	.83	.2	*	.74	.96
Cr	.002	.012	.2	.17	.14	.13
Sr	.22	*	.2	*	.36	.35
Cu	.014	.006	.1	.10	.08	.07
Fe	1.67	1.36	.1	*	1.7	1.4
Pb	.002	.002	.1	.10	.07	.07
Cd	0	0	.1	*	.07	.07
Ag	0	0	.02	*	.01	.01
NO_3-N	.62	1.27	1.2	6.7	1.4	5.7
NH_3-N	1.0	.06	2.0	1.9	2.4	1.3
Hg	NR	0	NR	.0014	NR	.001

* - Use increments and/or background concentrations were not available from recent water quality data for these parameters; GKY estimates were used in combination with available data to determine critical levels.

NR - Not reported in GKY study.

The nitrified effluent supplied to the EEWTP was substantially different in quality from the secondary effluent reflected in the "Projected Levels" column of Table F-4. The projected maximum pollutant concentrations in Table F-4 were based on the assumption that secondary effluent from Blue Plains characterized the average effluent quality entering the estuary in the year 2030. If we accept the alternative assumption that waste treatment plants will be discharging primarily nitrified effluent in the year 2030, Table F-4 could be reconstructed using use increments derived from the observed nitrified effluent quality minus the background concentrations in Table F-4. Based on the limited data available, estimated maximum concentrations of TDS and metals would be lower than in Table F-4, since nitrified effluent is lower in TDS and metals than secondary effluent.

Model Results - Relocation of Plant Intake

For this alternative scenario, the model was run in essentially the same way as for the base condition run, with the exception that water for the recycle plant was removed at Potomac Park (estuary segment no. 8) instead of at Chain Bridge. Flowby and waste treatment plant efficiency were unchanged from the base condition run. In effect, this scenario places the plant intake closer to the Blue Plains outfall, and creates a zone of "fresh" water at the head of the estuary.

Results for TDS are presented in Figure F-6 for the last 90 days of the simulated drought period. It can be seen from Figure F-6 that TDS levels rise gradually to a maximum near the end of the period, and then decline due to the rapidly increasing flowby at Chain Bridge. This scenario more closely resembles a simple recycle system, due to the reduced influence of flowby on estuary concentrations. Predicted conservative pollutant levels as a function of maximum TDS for this run are presented in Table F-5.

Model Results - Zero Flowby

In this run, the plant intake was relocated to Potomac Park, as in the previous run, and flowby at Chain Bridge was artifically set to zero beginning on 25 August of the model year (the day of minimum river flow). This scenario may represent a sudden interruption in outside water supply which requires overuse of the Potomac River, or any other situation in which flow at Chain Bridge is restricted for a substantial period of time. The resulting TDS levels shown in Figure F-7 indicate the extreme buildup of TDS during a zero flowby period, both at Chain Bridge and at the plant intake near Potomac Park. The estimates of maximum concentrations for other conservative pollutants for this run are presented in Table F-5, along with the results from the previous model runs.

Model Results - Waste Treatment Plant Breakdown

The possibility of a substantial release of a pollutant to the estuary from one or more waste treatment plants due to breakdowns during a drought period is a possible scenario. Since TDS is unaffected during treatment (the plant operates at zero removal efficiency), TDS is not a suitable parameter to use for this analysis. A better example is NH_3-N, with a background concentration of 1 mg/l and a use increment of 10 mg/l, which is assumed to be removed by a functioning plant. To test the variation of pollutant

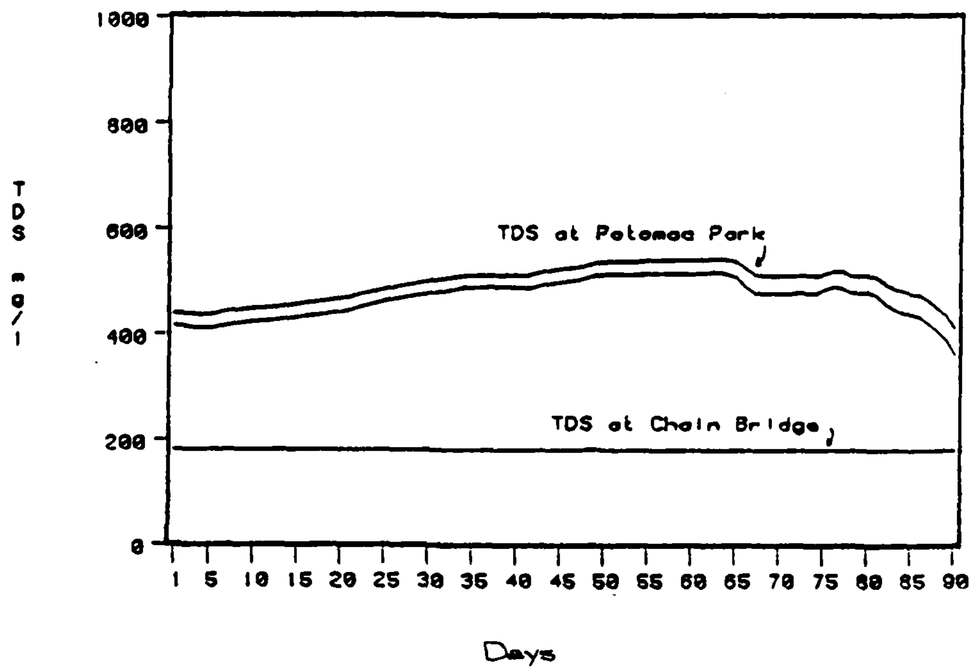


FIGURE F-6

DAILY AVERAGE AND MAXIMUM TDS,
1930 FLOWBY, POTOMAC PARK INTAKE
LAST 90 DAYS OF PERIOD

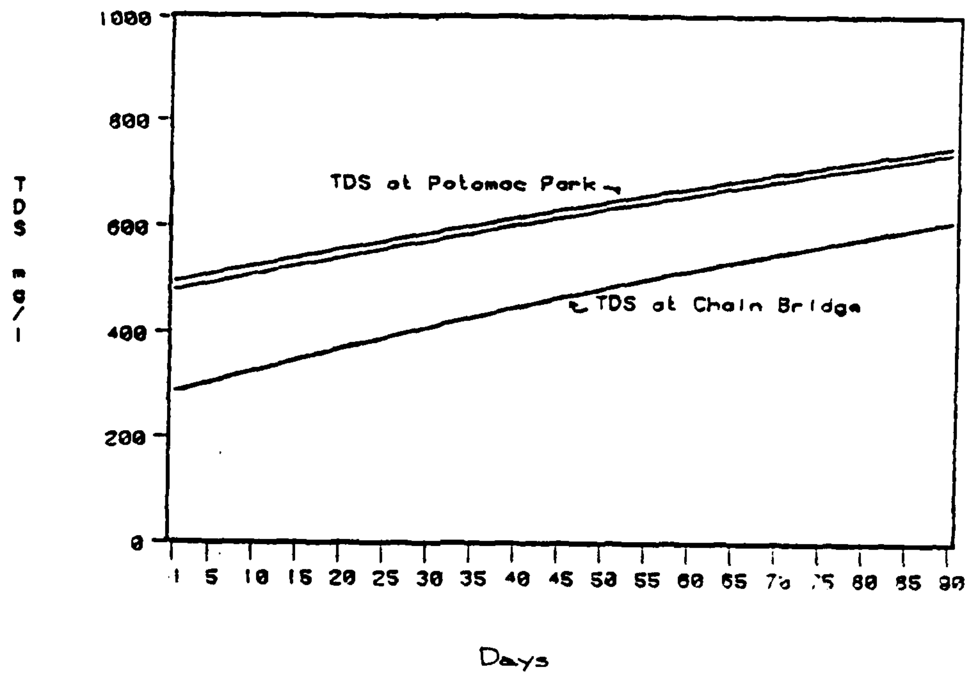


FIGURE F-7

DAILY AVERAGE AND MAXIMUM TDS,
ZERO FLOWBY, POTOMAC PARK INTAKE
LAST 90 DAYS OF PERIOD

TABLE F-5

MAXIMUM OBSERVED CONSERVATIVE POLLUTANT CONCENTRATIONS
AT RECYCLE PLANT INTAKE FOR ALL SCENARIOS
(ALL VALUES IN MG/L)

PARAMETER	Intake at Chain Bridge, 1930 Flowby	Intake at Potomac Park, 1930 Flowby	Intake at Potomac Park, Zero Flowby
TDS	447.	542.	749.
Total P	0.30	0.37	0.53
Alkalinity	130.	154.	205.
Hardness	158.	182.	233.
Cl	81.	106.	162.
Na	65.	85.	129.
SO_4	56.	66.	88.
Ca	47.	54.	70.
Mg	9.9	11.1	14.
K	7.0	8.6	12.
Ni	0.04	0.06	0.08
Mn	0.23	0.28	0.38
Zn	0.05	0.06	0.08
Al	0.96	1.0	1.1
Cr	0.13	0.17	0.25
Sr	0.35	0.40	0.50
Cu	0.07	0.10	0.15
Fe	1.4	1.5	1.5
Pb	0.07	0.09	0.14
Cd	0.07	0.09	0.14
Ag	0.01	0.02	0.03
NO_3-N	5.7	7.3	11.
NH_3-N	1.3	1.8	2.8
Hg	0.001	0.001	0.002

loading to the estuary with changes in the length of plant breakdown, the model was run repeatedly for breakdown intervals ranging from 5 to 110 days. Flowby was set equal to that used in the base condition run, and separate runs were completed for the two alternative locations of the recycle plant intake. The breakdown period was assumed to end at the end of the simulated period, for all breakdown intervals. For example, for a breakdown period of 60 days, effluent NH_3-N was set to 1 mg/l from the beginning of the simulation to 60 days before the end of the simulation. On that day, the use increment was increased from zero to 10 mg/l, and held at 10 mg/l to the end of the simulation.

Results for this set of runs are presented in Figures F-8 and F-9, for two locations of the recycle plant intake. It may be observed from the figures that the time of maximum concentration varies with the length of the breakdown, due primarily to the variation in flowby. The level of other conservative pollutants that would be observed during a plant breakdown depends on their background concentrations and the difference between the use increments before and after breakdown. In Figure F-10, the maximum predicted ammonia concentration for each breakdown run is plotted versus the length of the breakdown, for each location of the plant intake. This figure indicates that the location of the intake at Chain Bridge serves to delay the effects of a breakdown for about 40 days, and that ammonia buildup after that time proceeds at about the same rate for both locations of the plant intake.

Future Considerations

It was apparent that a precise match of expected and projected water quality levels was not feasible. At best, the actual frequency distributions of various water quality parameters obtained from influent monitoring data collected during the first year of operation could be compared to projected values of these parameters. To facilitate this process, the modeling results presented in Figures F-5, F-6, and F-7 were reorganized into a single graph, shown in Figure F-11. The curves offered a simple method for determining the maximum concentration of any conservative pollutant under each scenario, given the background (water supply) concentration and municipal use increment of the pollutant. Any concentration unit may be used with Figure F-11. As an example, if further monitoring of Blue Plains nitrified effluent indicates that the use increment for chloride averages 80 mg/l, we can determine the maximum chloride concentration the pilot plant must treat by finding 80 mg/l on the x-axis, and reading the y-axis value for each curve. For the zero-flowby situation, the y-axis value equals 115 mg/l, which is equal to the maximum concentration minus the background concentration. The background concentration for chloride is given in Table F-4 as 9.4 mg/l. Then, the maximum chloride concentration under a zero-flowby situation for a plant intake at Potomac Park would be 124 mg/l (115 + 9.4). Similar curves for plant breakdown scenarios are complicated by the relationship between the background concentration and the change in use increment before and after breakdown, and as such are not plotted. However, Figure F-10 may be used to estimate maximum pollutant loading for any parameter under plant breakdown conditions if the background concentation is small compared to the use increment during breakdown. In this case, Figure F-10 may be normalized such that the y-axis ranges from the background level to background plus use increment for the parameter.

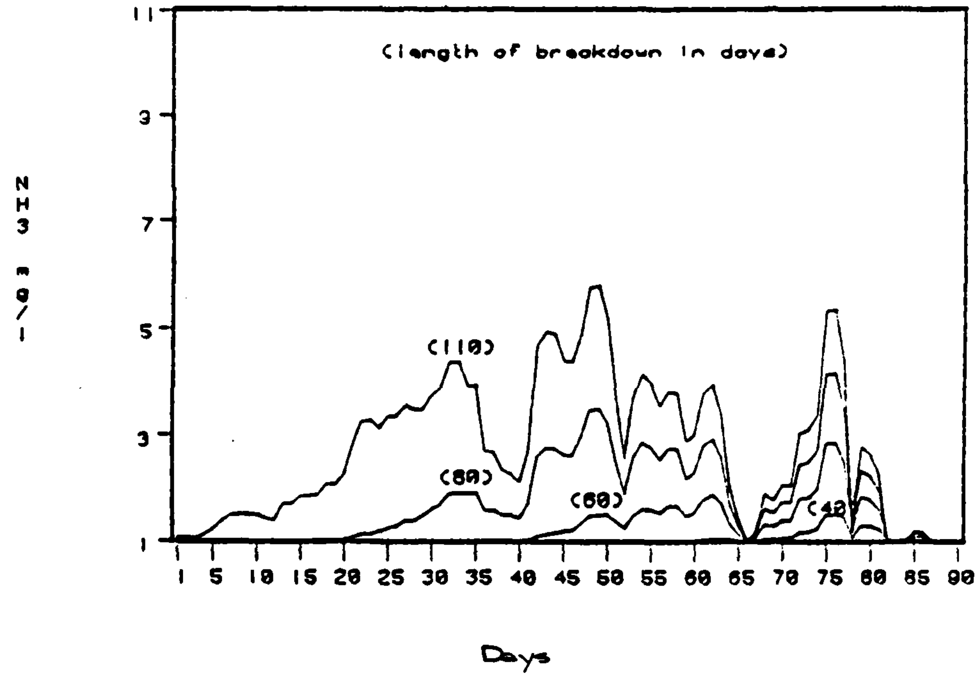


FIGURE F-8

MAXIMUM DAILY NH_3 DURING PLANT BREAKDOWN CHAIN BRIDGE INTAKE

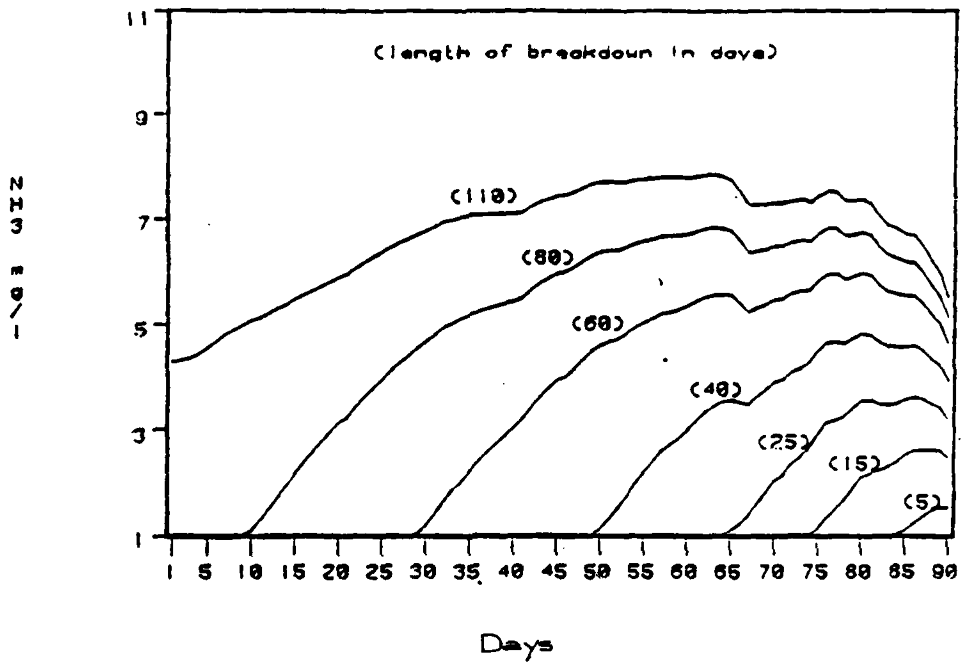


FIGURE F-9

MAXIMUM DAILY NH_3 DURING PLANT BREAKDOWN, POTOMAC PARK INTAKE

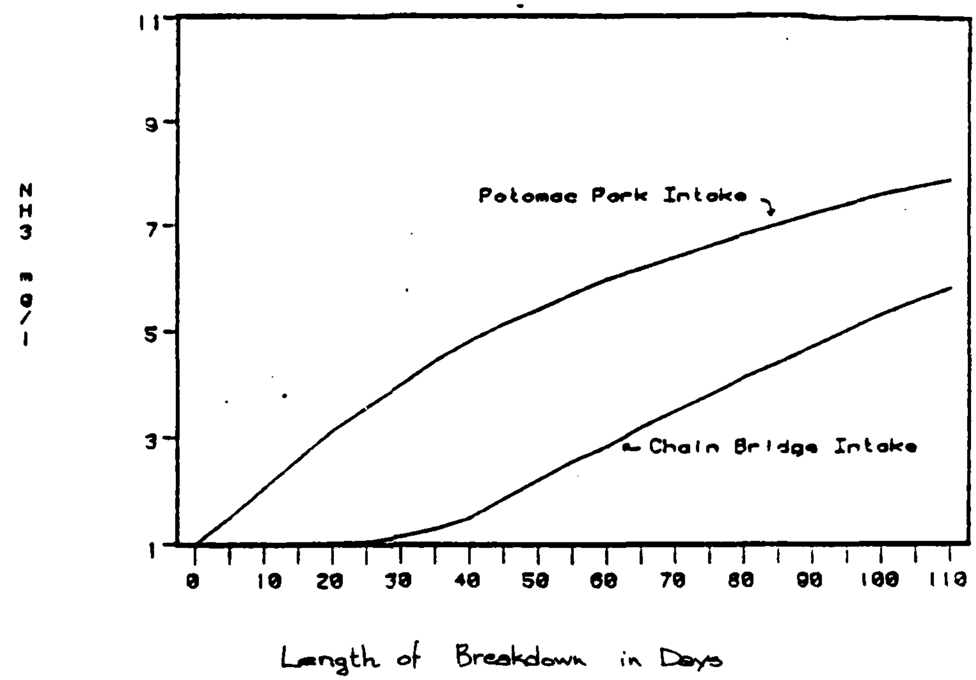


FIGURE F-10

NH_3 BUILDUP VERSUS LENGTH OF PLANT BREAKDOWN

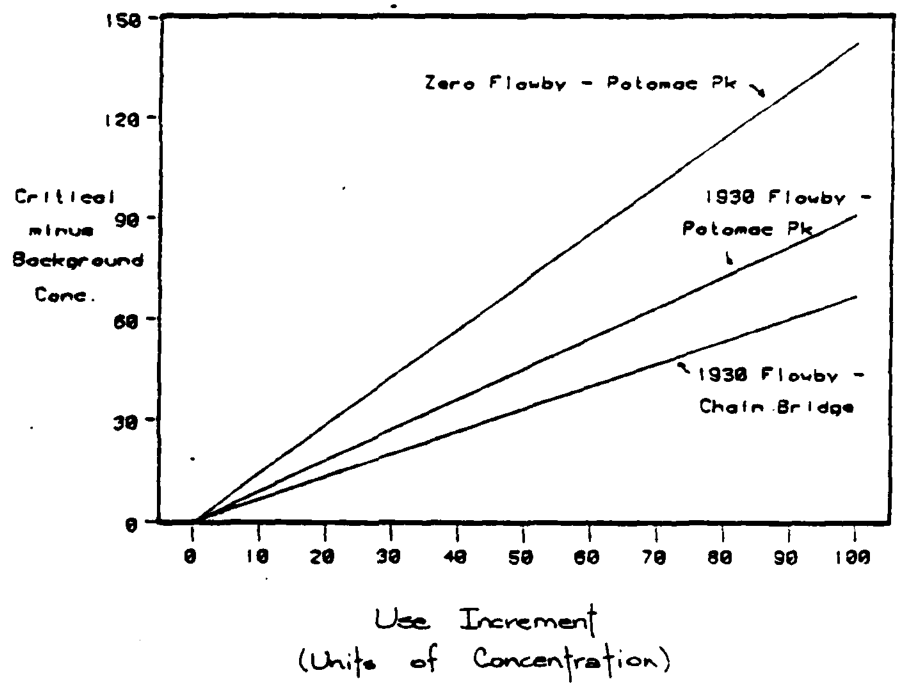


FIGURE F-11

CRITICAL CONCENTRATION MINUS BACKGROUND VERSUS USE INCREMENT

THE WATER QUALITY TESTING PROGRAM

The overall water quality testing program for the EEWTP was subdivided into three parts:

a. The Routine Water Quality Testing Program,
b. The Testing Program for Process Adjustment and Modification, and
c. The Operator Data Collection System.

The Routine Water Quality Testing Program (RWQTP)

The RWQTP was by far the largest program, involving the most sophisticated analyses and the largest number of samples. The purposes of the RWQTP were several. First, baseline information was collected on the influent quality from the Blue Plains STP, the Potomac Estuary, and the blended mixture of these two sources. Second, water quality data were collected from three MWA water treatment plants for comparison to the finished water produced by the EEWTP. (As discussed earlier, none of the current drinking water standards address water quality issues that arise when a drinking water source is unprotected and subjected to high levels of contamination. Because of the lack of objective standards for defining potability, an alternative procedure was to compare the finished water quality from the EEWTP to the finished water quality produced by existing water treatment plants). And third, water quality data were collected from the EEWTP effluent. With this broad catalog of water quality data, evaluations as to the effectiveness of the EEWTP could be made.

At the outset of the testing program, the RWQTP was operated with a sampling and analytical schedule designed as closely as possible to the program outlined in the "Statement of Objectives" as prepared by Malcom Pirnie Inc. (Special Study Report, Feb 1979). As the testing program progressed, however, the results were reviewed for relevancy and additional or reduced monitoring was implemented for certain parameters.

The Testing Program for Process Adjustment and Modification (TPPAM)

The performance of the standard treatment process train was primarily evaluated based on results generated under the RWQTP. Several questions, however, were not resolved by the first-year testing program. These included the following:

a. Were the design criteria used in the design of the EEWTP optimum values for the design of the full-scale plant?

b. Was the standard process combination the most cost-effective process train for producing "potable" water from the Potomac River Estuary?

c. What was the reliability of the standard process sequence when stressed with unusually high loadings of contaminants of health concern?

In order to answer these principal questions, a series of engineering studies were being conducted to complement the results of the RWQTP. These studies were specified as part of the Testing Program for Process Adjustment and Modification (TPPAM).

The principal objective of the TPPAM was to determine the optimum process combination and design criteria for a future 200 mgd emergency estuary water treatment plant. This plant should be capable of producing a potable water at minimum cost, consistent with requirements for process reliability. It was the purpose of the TPPAM to optimize the design criteria, as well as to evaluate alternative process combinations. A series of individual studies were undertaken to evaluate the full-scale plant, to optimize the unit processes, and to investigate possible process alternatives. Table F-6 is a summary of the engineering studies that were conducted.

Operator Data Collection System (ODCS)

Primary Data Collection

The operation of the plant was monitored using two primary log sheets, the process control logs and the analytical logs.

The process control logs required the operator to make a tour of the plant every two hours and record certain critical operational data. Some of the data, for example, pump discharge pressures, mixer speeds, and chlorine injector suction pressure were used to check on the operation of the process equipment. The majority of the data, however, were essential to the operational data base and served as the recorded value on process parameters. Data in this category included the following:

a. Water temperatures,
b. Microscreen levels,
c. Microscreen backwash rates,
d. Process flow rates,
e. Head loss in filters and carbon columns,
f. Clearwell levels,
g. Chemical dosage rates,
h. Filter and GAC backwash times,
i. On-line turbidity and chlorine residual readings,
j. Sludge pumping frequency and time.

Chemical dosage rates were checked every two hours by measuring the actual flow pumped into a graduate cylinder using a stopwatch. These numbers were then averaged to give a daily dosage rate calculated in mg/l. A second check on chemical dose rates was available by logging in batch make-up times and quantities of chemical used per batch, such as polymer, lime, and potassium permanganate.

The analytical log was a single data sheet on which certain measured parameters necessary for process control were recorded. These parameters, the number of sites, and their frequency of collection, were as follows:

Parameter	No. of Sites	Frequency
pH 6	Every 4 hours	
Electrical conductivity	1	Every 4 hours
Turbidity	6	Every 2-4 hours
Chlorine residual (free/total)	2	Every 4 hours
Dissolved oxygen	5	Every 24 hours
Dissolved oxygen	3	Every 12 hours

TABLE F-6

SUMMARY OF ENGINEERING STUDIES

	STUDY	OBJECTIVES
1.	Individual Processes Performance Evaluation	Determine performance of each process; optimize operation for maximum removals, minimum cost.
2.	Optimization of Granular Activated Carbon (GAC)	Determine optimum process design criteria for GAC (pretreatment, carbon type, bed depth, superficial velocity).
3.	Optimization of Coagulation Chemistry	Evaluate effectiveness of alum/polymer combinations and coagulants other than alum for removal of particulates and total organic carbon (TOC).
4.	Optimization of Disinfection	Evaluate relative efficiency of disinfectants with respect to microbial control, formation of by-products.
5.	Control of Volatile Organics	Determine feasibility of air stripping for control of volatile organics.
6.	By-Product Formation during Chlorination	Determine expected levels of THM/TOX due to chlorination under varying water quality conditions.
7.	Process Reliability	Determine response of various treatment process combinations to variable influent water quality (spiking studies).
8.	Cost Projection - 200 mgd Plant	Determine capital and O&M costs for 200 mgd plant.

Both pH and turbidity were measured with instruments on the operator's lab bench in the plant. These instruments were calibrated prior to each set of runs and were routinely checked for compatability with the laboratory instruments every several days. The electrical conductivity, chlorine residual, and dissolved oxygen readings were measured in the laboratory.

All data on the analytical log were entered into the computer directly from the log sheet. For ODCS purposes, daily averages were computed and entered onto a monthly summary sheet.

Special Logs

The following series of special logs were also maintained as part of the plant operation process:

a. Flow totalizer logs - once per day, at a specified time, the main process flow totalizers were read and the flow for the previous 24 hours computed. These numbers served as the basis for computing all daily flow averages.

b. Equipment run time log - this sheet recorded any process equipment down-time and tabulated the number of hours that a particular piece of equipment had been run during the past 24 hours. Once per week all of the equipment hour-meters were read as a second check on equipment run-times.

c. Jar test log - every day, jar tests were run once or twice to determine optimal coagulant dose. Chemical dosages were changed based on the jar test results for turbidity and total organic carbon (TOC) removal.

d. Engineering tests - as required for special tests or special data, additional log sheets were sometimes used for short periods.

e. Operator's log book - all unusual occurrences, special duties, and other important events were recorded in a daily log book.

Data Summary Log

Each morning the previous day's data were reviewed and recorded on a summary data sheet. Important data in this log included the following:

a. Clearwell levels,
b. Flows through each unit processes—recorded as gallons per 24 hours,
c. Flow losses—through backwashing, sludge withdrawal,
d. Chemical dosages in mg/l and lbs/day,
e. Filter and carbon column backwash data,
f. Microscreen operational data,
g. Temperature, and
h. Pounds of sludge pumped.

A remarks section was provided so any unusual operational occurrences, such as disruptions or down-time could be recorded. The summary log served as the basis for nearly all of the engineering review of the processes on-line. This log was eventually incorporated into the computer data base.

Overview

A schematic representation of the ODCS is shown in Figure F-12. This chart shows the interaction of the data system and illustrates the degree of redundance available. For the important parameters, there were at least two independent checks on the actual numbers recorded in the data file.

RESULTS OF THE TESTING PROGRAM

The overall objective of the EEWTP project, as specified by the U.S. Congress in the Water Resources Development Act of 1974, PL. 93-251, was to evaluate the technical feasibility of using the Potomac River Estuary as a supplemental water supply source for the Metropolitan Washington Area to meet potential water shortages that might occur under severe drought conditions.

This section presents the results of this evaluation, based on the two years of water quality monitoring at the EEWTP. Three treatment process combinations were investigated during the two-year monitoring program as summarized in Table F-7. The key issues addressed by the EEWTP project were as follows:

1. Selection of appropriate blend of treated wastewater and Potomac River Estuary water for the EEWTP influent to simulate the estuary water quality expected under drought conditions.

2. Suitability for human consumption of the finished water produced by the treatment combinations monitored at the EEWTP.

3. Process performance and process reliability of the selected treatment combinations monitored during the two-year program with respect to control of those water quality parameters known to affect the aesthetic quality of the finished water and known or suspected to pose health risks to consumers.

Issue 1: Selection of Influent Water Quality

An equal blend of nitrified treated wastewater and Potomac River Estuary water was selected to simulate the expected water quality conditions in the Potomac River Estuary at Chain Bridge, under 1930 drought conditions with projected water supply demands for the year 2030. The 50:50 blend was found to be a conservative simulation of expected water quality in the estuary at Chain Bridge, based on a comparison of water quality predictions developed by the Dynamic Estuary Model (DEM), and the water quality observed in the blended influent.

Issue 2: Evaluation of Finished Water Quality

Within the limits of analytical techniques used on this project, the process combinations tested in the EEWTP (see Table F-7) were shown to be capable of producing a finished water of quality suitable for human consumption. For several water quality parameters, levels in the EEWTP exceeded the highest levels observed in the finished waters of three major local water treatment plants in the MWA. For most of these parameters the potential increase in health risks was judged to the negligible. Conclusions regarding the suitability of finished water quality for each of the key water quality parameters are presented below by parameter group.

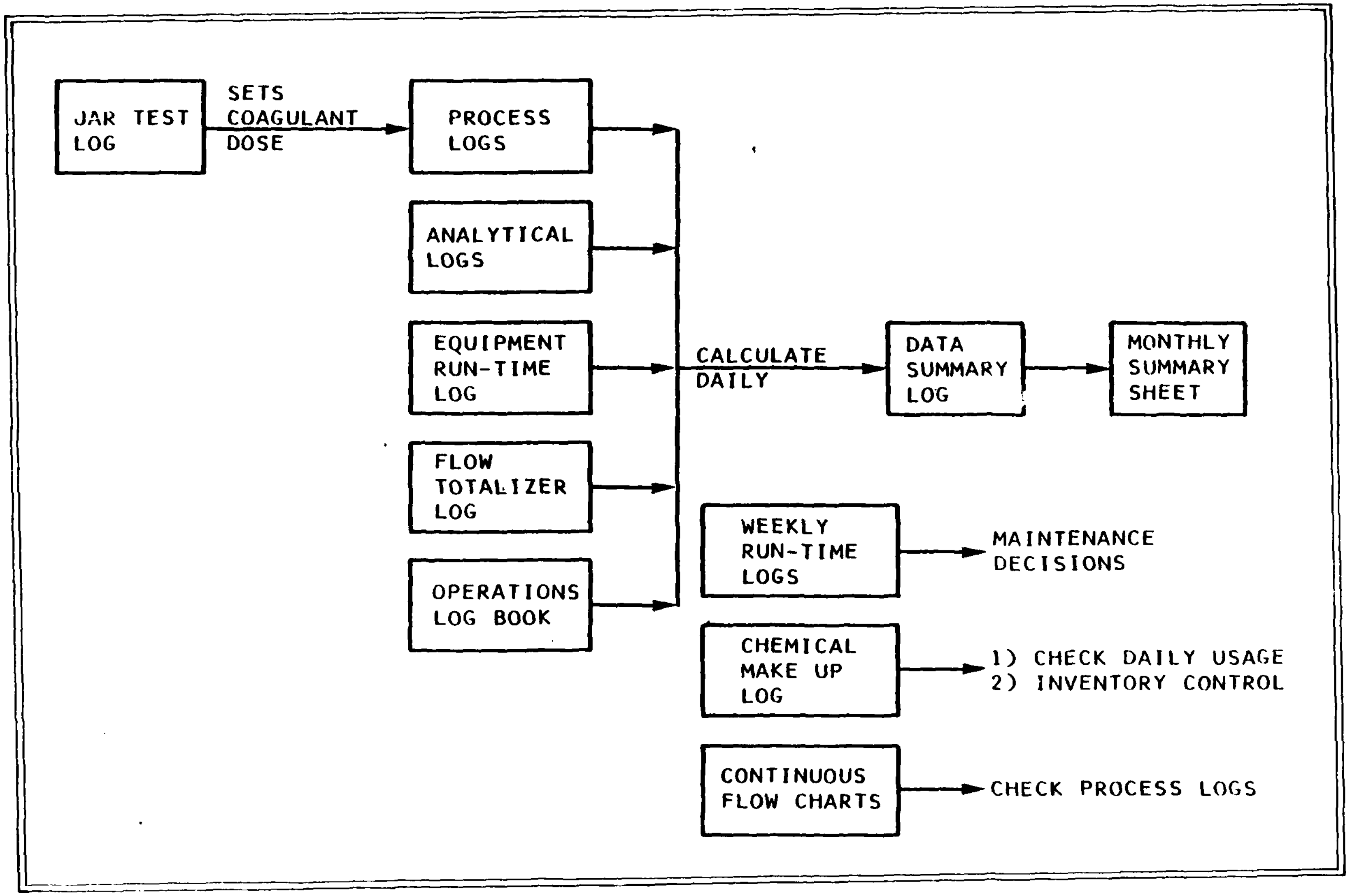


FIGURE F-12

OPERATIONAL DATA COLLECTION SYSTEM LOGS

TABLE F-7

SUMMARY OF EEWTP PROCESS COMBINATIONS

PHASE	PROCESSES	DURATION
Phase IA	Alum coagulation, flocculation, sedimentation, intermediate disinfection with chlorine, dual-media gravity filtration, granular activated carbon (lignite based with 15 minute empty-bed contact time), free chlorine disinfection.	16 March 1981 to 16 March 1982 (52 weeks)
Phase IB	As above, with ozone in place of chlorine as intermediate oxidant/disinfectant	17 March 1982 to 7 July 1982 (15 weeks)
Phase IIA	Lime coagulation, flocculation, sedimentation recarbonation, dual-media gravity filtration, granular activated carbon (bituminous based carbon, 30 minute empty-bed contact time), ozone disinfection, chloramine disinfection.	17 July 1982 to 1 February 1983 (28 weeks)

Physical-Aesthetic Parameters

The key physical-aesthetic water quality parameters included turbidity, color, odor, and pH, parameters that have been included in either the federal primary or secondary drinking water regulations.

The three treatment process combinations monitored (Phases IA, IB and IIA) produced a finished water quality that rarely exceeded the maximum contaminant levels (MCL) for turbidity and color, but frequently exceeded the MCL for odor. Levels of pH were lower than the standard during the first few months of operation during Phase IA.

Geometric mean values of turbidity in the finished waters were significantly less than the highest turbidity level in one of the local water treatment plants.

Odor levels during the alum phase of operation exceeded the secondary MCL threshold odor number of 3 in more than 95 percent of the samples. Odor values were generally comparable to levels observed in the local water treatment plants, however, although the mean value exceeded the highest mean odor level in the local plant.

The Phase IIA process reduced the odor levels considerably, with the mean value during this phase of operation being significantly less than the highest off-site plant. Odor levels during this phase did not exceed levels as the local plants at any percentile of the sample population.

Anions, Major Cations and Nutrients

This parameter group included eighteen inorganic parameters, one of which is included in the primary drinking water regulations (nitrate), and three of which are included in the secondary regulations (chloride, sulfate, total dissolved solids (TDS)). Sodium and cyanide are also incuded in this group, two parameters currently being considered for inclusion in the regulations because of potential adverse health effects.

Hypothesis testing was used to determine if the geometric mean values of the parameters from the EEWTP compared to the highest value observed in the local plants were significantly different at a five percent (0.05) level of significance.

In general, the finished water quality from the EEWTP during all phases of operation exhibited higher levels of all the parameters included in this group, a consequence of the contribution of the treated wastewater to the quality of the blended influent, and the inability of the process combinations tested to remove these dissolved salts.

The levels of nitrate in the EEWTP finished waters occasionally exceeded the primary MCL of 10 mg/l-N (3 percent of the samples). In all cases, this occurred when the blended influent consisted of nitrified effluent only.

Nitrate levels in the EEWTP finished waters were significantly higher than values observed in the local water treatment plants. The 90th percentile values of nitrate observed during the three phases of operation reached levels of 9 mg/l-N, compared to the primary standard of 10 mg/l-N. The 90th percentile values observed also match the maximum projected value of nitrate expected in the estuary under drought conditions.

Because these high values would provide almost no safety factor for this parameter compared to the MCL, the observed levels of nitrate represent a potential health issue should an estuary plant be constructed.

The mean values of those parameters of health or aesthetic significance in this parameter group were significantly greater than the highest mean value observed in the local water treatment plants. These parameters included total dissolved solids, sulfate, chloride, and sodium. Cyanide levels in the EEWTP were low (< 0.003 mg/l) and not significantly different from the local water treatment plants. The levels of sodium exceeded the suggested criteria of 20 mg/l, but the observed levels were similar to median values observed in water systems in the U.S. Thus, no adverse health effects are anticipated.

Trace Metals

Twenty-four individual metals were included in this parameter group, eight of these being included in the primary regulations (arsenic, barium, cadmium, chromium, lead, mercury, selenium, and silver) and four in the secondary regulations (copper, iron, manganese, and zinc).

For those metals of health or aesthetic significance, the mean values in the EEWTP finished waters during one or more of the operational phases exceeded the highest mean value observed in the local plants for the following metals: mercury, manganese, nickel, and zinc. The observed concentration levels for mercury were below MCLs, however, and not considered to pose increased health risks. Mean mercury levels during Phase IIA operations were reduced below the highest mean observed in the local water treatment plant.

With the exception of mercury, concentrations of metals in the EEWTP finished waters never exceeded the specified maximum contaminant levels. Only during Phase IA operation did the mercury levels exceed the MCL (three samples or about 1 percent of the total sample taken). Following application of revised pH control within the process combination, the mercury levels were maintained well below the MCL. Potential health concerns because of the presence of mercury are considered to be negligible.

During the Phase IA operation, the secondary MCL for manganese was exceeded in 34 percent of the samples. Oxidant addition (permanganate and ozone) adjustments to pH control were successful in reducing manganese levels below the MCL.

Radiological Parameters

The monitored radiological parameters included gross alpha, gross beta, tritium and strontium-90, all of which are included in the NIPDWR. Levels of these parameters in the finished waters from the EEWTP never exceeded the federal MCLs. Gross beta radionuclides in the finished waters from the EEWTP were significantly greater than the levels observed in the local water treatment plants. Levels of strontium-90 and tritium were well below the MCLs, however, and presumed not to cause any measurable increase in health risks.

Microbiological Parameters

This parameter group consisted of seven parameters (viruses, parasites, salmonella bacteria, endotoxin, standard plate count, and fecal and total coliform), with only the total coliforms included in the primary drinking water regulations. All of these parameters, however, have known or potential acute health effects when ingested in drinking water.

Although detected in the blended influent, no viruses, parasites or salmonella bacteria were detected in the finished waters produced by the EEWTP. Standard plate count levels were generally low in the EEWTP finished waters (media value less than 1 colony/-ml), during all phases of operation. Levels were significantly lower than the highest observed values in the local water plants, and well below the National Research Council recommended level of less than 100 colonies/ml for treated waters obtained from heavily contaminated sources.

During Phase IA operation, fecal and total coliform levels in the EEWTP finished waters exceeded the levels observed in the local water plants. Although total coliform levels never exceeded the MCL of MPN/100 ml, positive coliform counts were observed in over 70 percent of the samples. These results were due primarily to the presence of high ammonia concentrations and the use of insufficient quantities of chlorine during the first four months of the alum phase of operation. Improved process performance after the first four months of operation reduced the coliform levels below the NRC recommendation for acceptable coliform levels in water taken from a contaminated source. The Phase IIA operation reduced the EEWTP fecal and total coliform levels below the highest values observed in the local water plants.

Trace Organics Parameters

Of the 14 targeted organic compounds specifically monitored in this parameter group, only seven compounds (4 pesticides, 2 herbicides and total trihalomethanes) are included in the primary drinking water regulations. Another six volatile organic chemicals are currently under consideration for inclusion in the regulations. Organics analyses conducted in this project were conveniently organized into three categories: (1) surrogate parameters, (total organic carbon, and total organic halide), (2) targeted organic compounds (compounds targeted for analysis using standards for confirmed identification and quantitation) and (3) secondary compounds (tentative identification, approximate quantitation).

Generally, the observed levels of all monitored organic compounds in the EEWTP finished waters were comparable to or lower than values observed in the finished water from the local water treatment plants. The MCLs for pesticides and herbicides were never exceeded in any of the finished waters. Three of the six regulated pesticides and herbicides were detected in the EEWTP finished waters, but only on three occasions, and at levels below the method detection limit. Total trihalomethanes in the EEWTP finished waters never exceeded the values observed in the local water treatment plants with mean values significantly less than at all three plants.

For all other targeted organic compounds, only 13 compounds were quantified frequently enough to permit quantitative estimates of sample population statistics. With the exception of total trihalomethanes, the estimated geometric means of the other quantified compounds were less than 1 ug/L (1 part per billion). The number of targeted and non-targeted (secondary) organic compounds detected at least once in the finished waters were observed to be lower in the EEWTP finished water than in the local water treatment plants. Total organic halide, a measure of the total quantity of halogenated organic compounds in the finished waters, was lower in the EEWTP finished waters than in the local finshed waters by a factor of three to ten. Lowest values were observed during the Phase IIA process, due to the elimination of free chlorine from the process, and the lower loading of organics on the GAC process.

Based on the observed concentration levels of the targeted trace organic compounds in the finished waters from the EEWTP, it was concluded that the water quality produced by all three process combinations would pose negligible health risks with respect to those organic compounds which could be detected and quantified by the techniques used on this project, and for which health risk data were available. Compounds in this category include chloroform, tetrachloroethene, trichlorethene, benzene toluene, 1, 2 - dichloroethane, carbon tetrochloride, vinyl chloride, and 1, 1, 1, - trichlorethane.

Because only a small fraction of the organic compounds included in total organic carbon and total organic halide measurement can be detected by currently available analytical instrumentation, it is not possible to evaluate the absolute risks associated with ingestion of the water produced by the EEWTP.

Toxicological Parameters

The two toxicological parameters monitored in the EEWTP project included: The Ames Salmonella microsome test and the mammalian cell transformation test. These tests represented two of the four tests recommended by the NRC committee on Water Quality Criteria for Reuse for determination of the relative safety of a drinking water for human consumption, regardless of the source water quality. Neither of these parameters are currently regulated. In addition, the absolute values of the test results cannot currently be used to estimate potential health risks. In addition, it was difficult to compare results observed on this project with values reported in other finished drinking waters because of variabilities in sampling and analytical protocals. Thus, only results based on comparisons between sampling sites specific to this project can be discussed.

Positive Ames assay results, as measured by either the specific activity or the mutagenic ratio, were observed in the finished waters from both the EEWTP and the local water treatment plants. However, the number of positive assays in both Salmonella tester strains (TA 98 and TA 100) was lower in all of the EEWTP finished waters than in the local water plants, based on more than twenty-five assays during the Phase IA process and more than twenty assays during the Phase IIA process.

Although positive assay results were observed in the EEWTP finished waters the health implication of these results are unknown. However, because the observed levels of mutagenic activity were so low, it was concluded that the water quality produced by the EEWTP during all phases of operation was relatively safe and therefore suitable for human consumption based on the Ames test.

Median values for the specific activities (revertants/l) in the EEWTP during all phases of operation were slightly lower than values observed in the local plants for both Salmonella tester strains. These results again indicate the relative suitability of the finished waters for human consumption.

Of the 23 to 25 mammalian cell transformation assays completed at each finished water site, three samples in the EEWTP finished waters and one to three samples in each of the local water plants (six total) were positive for transformation activity. Where positive assays were observed, the number of transformed cells were always low, indicating that the waters contained few compounds capable of causing a carcinogenic response in the mouse cell line tested based on the concentration and assay protocals used in this project. Based on the comparative results of the mammalian assay, it was concluded that the EEWTP finished water would be relatively safe for human consumption, compared to drinking water currently provided to the residents of the MWA.

Issue 3: Process Performance

Three distinct treatment process combinations (see Table F-7) were evaluated during the two-year operation of the EEWTP as to their technical feasibility for producing a water suitable for human consumption. Each process combination was monitored extensively, to determine the capabilities of individual processes for controlling water quality parameters with known or suspected health effects.

The Phase IA process combination was demonstrated to be a technically feasible process combination for producing a finished water with acceptable quality, provided that appropriate levels of process chemicals were added to maintain target pH levels, following coagulation and target free chlorine residual levels following final disinfection. The finished water from the Phase IA process combination exhibited three water quality problems, compared to the finished water quality in the local water treatment plants; high odor levels, high manganese levels, and high total coliform levels. To reduce total coliform to acceptable levels free chlorine residual greater than 3 mg/l, following 60 minute contact, was required. To control soluble manganese levels below the secondary maximum contaminant level, pH control combined with an oxidant (potassium permanganate or ozone) was required. High odor levels in Phase IA were reduced by maintaining the pH of the finished water above 7.

Defining process reliability as the ability of any process combination to produce a water quality in which the ratio of the maximum contaminant level to the 90 percentile value is at least greater than two, the Phase IA process combination was demonstrated to have acceptable process reliability for most parameters included in the primary or secondary standards with the exception of odor, manganese, nitrate, and total dissolved solids (TDS). For nitrate and TDS, demineralization processes such as reverse osmosis would be required to improve process reliability if additional safety factors were deemed necessary for these parameters because of the contaminated nature of the source.

In the second process combination tested (Phase IB), ozone in place of chlorine following sedimentation was able to improve process reliability for manganese. The reliabillity ratio increased above two compared to 0.5 during Phase IA. Use of the target free chlorine residual also significantly improved the process reliability for reduction of total coliforms. The Phase IB process was demonstrated to be a technically feasible process when treating an influent water of the quality observed. However, the process combination would not be capable of controlling ammonia prior to final disinfection which could inhibit the efficiency of final free chlorine disinfection and could lead to unacceptable levels of total coliform in the finished water. Thus, this process was not considered to be sufficiently reliable for producing a water quality acceptable for human consumption under influent water quality condictions similar to that observed during a full year of operation.

The Phase IIA process combination was demonstrated to be a technically feasible process combination for producing a finished water with acceptable water quality, under all observed influent water quality conditions, and all operating conditions tested. Process reliability for the Phase IIA process was superior to that demonstrated for the Phase IA and IB processes with respect to total coliforms and manganese. Odor levels in Phae IIA were also lower than observed in the alum processes, but levels still exceeded the secondary MCL threshold odor number of 3. The high odor levels were attributed to the conditions of analytical test, expecially with respect to the sensitivity of the odor panel. The odor levels in the finished water from Phase IIA were lower than the highest levels in the local water plants, for most percentiles of the sample distribution.

DESIGN AND COST OF FULL SCALE ESTUARY PLANT

In the original concept for an estuary water treatment plant presented in the NEWS Study, an estuary treatment plant was proposed to be located near Chain Bridge, which is the point in the Potomac River where the estuary begins. The estuary treatment plant was considered to be a separate water treatment plant using the estuary as a raw water source and with two alternatives for use of the finished water: 1) pumping the finished water upstream along the Potomac River and releasing into the river several miles above current water intakes; and, 2) pumping the finished water directly to distribution points within the MWA utilizing existing distribution systems.

The details of this proposed estuary water treatment plan are uncertain at this time. Because of the current water situation, which indicates that no serious shortages are likely to occur until well into the next century, the proposed estuary water treatment plant remains hypothetical in many respects. Based upon these uncertainties, it was necessary in designing and estimating the cost of the full-scale plant to make certain reasonable assumptions. These assumptions are discussed below. The assumptions pertain to the water source, plant capacity, operational strategy, and facilities included in the cost estimates presented. The assumptions do not affect on the actual estimated costs of the treatment processes themselves.

As mentioned, design details and cost estimates for the treatment processes were based on EEWTP results and accepted engineering practice. The estimated costs presented for the treatment configurations, therefore, adequately reflect the full-scale treatment costs of the treatment processes associated with producing the water qualities previously discussed.

Water Source

For the purposes of the cost estimate, it was assumed that the intake for the estuary treatment plant would be at Chain Bridge, the point in the river exhibiting the highest water quality in the estuary under drought conditions. The predicted water quality under drought conditions at Chain Bridge was also the basis for selection of the equal mix of nitrified wastewater effluent and river water used as the influent to the EEWTP.

Plant Capacity and Operational Strategy

The cost estimates presented in this section are based upon the two major treatment process configurations (Phase IA and IIA) which were examined in detail the two years of operation of the EEWTP. It is assumed that a 200 mgd hypothetical plant would be operated at 100 percent capacity for 365 days a year. Under normal cost estimating procedures, operational levels for a water treatment plant would be selected at 70 percent of the maximum hydraulic capacity. In this case, however, costs were developed primarily based on conservative estimates of the costs of such a proposed estuary water treatment plant. The cost estimates also did not account for the possibility that either the entire water treatment plant or portions of the water treatment plant would not be operated every day of the year, as opposed to operating only under drought conditions. Operationally, it was assumed that treatment processes were used and operated to produce a finished water that could be distributed directly to consumers via existing distribution systems. This finished water produced at the EEWTP was compared with drinking water standards and with the finished waters of other water treatment plants. If an estuary water treatment plant were ever built and operated, the water from the estuary plant could be blended with the existing finished waters from water treatment plants in the area or could serve as the raw water supply for other water treatment plants. Either situation could change the quality of the water from the estuary water treatment plant before it reached a consumer.

Facilities Included in Cost Estimates

The cost estimates presented for a 200 mgd estuary water treatment plant were based on the design, construction, and operation of a water treatment plant consisting of the treatment processes demonstrated at the EEWTP. Two cost estimates were provided, one for each of the major process configurations demonstrated. Because of the hypothetical nature of the full-scale estuary water treatment plant, it was not practical to include estimated costs for several parts of a complex estuary water treatment plant should it ever be built. Due to the unknown exact location of the full-scale plant and the unknown operating philosophy, the following facilities were excluded from the cost estimates: intake structure, intake pumping station, finished water pumping station, finished water reservoirs, finished water distribution piping, land purchase, and site preparation other than basic clearing and grading.

The costs for these facilities would substantially increase the cost of a complete estuary water treatment plant. However, the estimated costs presented were reflective of the treatment costs necessary to produce a finished water of the quality discussed earlier using a raw water source similar to that anticipated from the estuary under drought conditions.

Alum/GAC/Chlorine Process (Phase IA)

The first of two process combinations evaluated at the EEWTP to be the basis for full-scale 200 mgd cost estimation was the process combination described as Phase IA in Table F-7. The process combination included the following treatment processes: coagulation with alum polymers, sedimentation, gravity filtration, adsorption on granular activated carbon, chlorination, and solids handling and disposal. Although surface aeration and microscreening were also studied during Phase IA, they were not included in the design of the full-scale plant because operating results did not indicate sufficient benefit to warrant inclusion.

Also included in a full-scale plant would be two additonal chemical feed systems. Permanganate would be added as a chemical oxidant at the front end of the plant and lime would be added for pH and corrosion control. Based upon limited bench-scale testing and on review of accepted practice in sludge handling and disposal, processes were included in the full-scale plant design for sludge thickening and dewatering. Final sludge disposal would be by landfill.

This process combination is not unique to the water treatment industry. Alum/polymer coagulation is widely used for chemical clarification. Permanganate addition is often used to oxidize a variety of substances in water, including manganese which was of concern in the operation of the EEWTP. Chlorination is the most commonly used disinfectant in the U.S. The intent of this process combination was to have a free chlorine residual. Even when ammonia is present in the raw water, breakpoint chlorination can yield the desired free chlorine residual. Although granular activated carbon (GAC) is not widely used in water treatment in the U.S., it has been successfully used in plants where organic compounds are of concern. The GAC process included a lignite based carbon and had an EBCT of fifteen minutes. Gravity thickening and centrifugation were proven and accepted means of slude handling. In summary, this process combination employed proven water treatment processes.

Lime/GAC/0_3/Chloramine Process (Phase IIA)

The second of two prcess combinations evaluated at the EEWTP to be the basis for full-scale 200 mgd cost estimation was the process combination described as Phase IIA in Table F-7. The process combination included the following processes: coagulation with lime and ferric salts, sedimentation, recarbonation, gravity filtration, adsorption on granular activated carbon, ozonation, chloramination, and solids handling and disposal. This process combination was chosen for evaluation at the EEWTP because it was believed there might be possibilities for producing better quality finished water than the alum/GAC/chlorine process.

Although lime coagulation is not often used in water treatment except when softening is required, it was chosen to investigate any changes in TOC removal to provide for better heavy metals removal, and to provide for some form of disinfection prior to filtration that would not require chlorination. Several wastewater reclamation plants use lime as a primary coagulant and the high lime process is usually part of the magnesium carbonate process proposed for use. Ferric salts are often used with lime in water and wastewater treatment as a coagulant aid.

Two disadvantages of lime coagulation are the need for lowering the pH after coagulation to produce a stable water and the handling of the large amounts of solids produced. With a 200 MGD plant, it may become practical to recalcine the lime sludge produced to be able to recover the coagulatant and to produce carbon dioxide for use in recarbonation. Recalcination would also greatly reduce any problems with final disposal of residual solids produced in the process. For these reasons, a recalcination furnace was proposed as part of the design of the full-scale plant.

The GAC adsorption process included was different in two ways from that included in the alum/chlorine process combination. Based upon what was used during Phase IIA, the full-scale plant would use a bituminous based carbon and longer EBCT of thirty minutes. It was believed that these changes would result in a longer lifetime of the carbon with resulting cost savings and would provide a greater barrier for preventing the occurence of synthetic organic compounds in the GAC treated water.

The other major difference between this process combination and the alum/chlorine process was in disinfection. The primary disinfectant in this process was ozone with chloramination providing for a residual disinfecting compound. It was believed that ozonation would provide for better disinfection and that the use of ozone and chloramines would eliminate some of the chlorinous odors reported in Phase IA.

Cost Estimates of Process Combinations

Using the facilities just described, cost estimates were prepared for a 200 mgd estuary water treatment plant. Two cost estimates were generated: one for the alum/chlorine process combination studied during Phase IA of the EEWTP; and, one for the lime/ozone process combination studied during Phase IIA. The purpose of these estimates was to provide some reference for comparing the alternative of constructing and operating an estuary water treatment plant as a means of solving potential water supply shortages in the Metropolitan Washington Area. The cost estimates could also be used in comparing the two process combinations.

It is again emphasized that the cost estimates provided did not include all the facilites that would be required in constructing a full-scale estuary water treatment plant. As previously discussed, because of the hypotethical nature of the full-scale plant, cost estimates were only provided for the water treatment process involved.

Methods Used to Estimate Costs

The methods used to generate cost estimates were a hybrid of two types of cost estimates: study estimates and preliminary estimates. Study estimates require flow diagrams, material and energy balances, and knowledge of types and sizes of equipment. They are intended for generalized evaluations, guidance for further investigation, or as a basis for process selection. Their usual reliability is ± 30 percent. Preliminary estimates require a bit more detail. They require some engineering of the structures and facilities and are often the basis for budget authorizations. Their usual reliability is ± 20 percent.

The cost estimates presented were a hybrid in that while most of the estimates are study type estimates, some processes were subjected to a preliminary cost estimate. This was the situation with the GAC absorption process. Because this process was central to both process combinations considered and because this process is not usually employed in water treatment because of its high cost, it was believed that a more detailed cost estimate was warranted.

The cost estimates of the full scale plant was first made using a FORTRAN computer program ("WATER") prepared for the EPA. This program was previously obtained from the EPA and placed on JMM's VAX 11/780 interactive computer in Pasadena. The computer program determined costs by retrieving stored coefficients for a least squared polymonial fit of cost curves which had been generated for 72 unit processes used in water treatment. The cost curves were based on conceptual designs of water supply systems with capacities between 1 and 200 mgd. Process capital and O&M costs are plotted versus an appropriate design parameter, such as pounds per day for chemical feed systems or square feet of surface area for filters. For a further description of the development of the individual cost curves, reference can be made to the original EPA documentation.

The costs generated by the program were broken down into capital costs and operation and maintenance costs. Capital costs consisted of the construction costs for each unit process, together with additional capital costs for sitework and interface piping, subsurface consideration, standby power, contractor overhead and profit, engineering, legal, fiscal and administration services, and interest during construction. Operation and maintenance costs included those for electricity, labor, maintenance materials, diesel fuel, natural gas, and chemicals.

These costs were also compared with water treatment plant cost information generated by JMM which was also on the computer. This was done to determine the reasonableness of the generated cost estimates. As mentioned, in some cases it was necessary to do more preliminary engineering to arrive at reasonable estimates.

All costs were updated to April, 1983 dollars. Various cost indexes were used to update original cost data. Table F-8 lists the various cost criteria that were used to generate the costs, including the cost indexes that were used to update cost data. The indexes reflect the current economic climate and construction costs in the Baltimore area and were believed to be a sound basis for the estuary water treatment plant costs. Eight different indexes were used for different portions of construction and O&M costs. Use of the eight separate indexes was compared with using the all-encompassing ENR Constructon Cost Index (CCI) and was found to generate costs eleven to thirteen percent lower than using the CCI. This better fit the data in JMM's data on water treatment plant costs and therefore was the basis for cost up-dating.

Estimated Costs

Estimated costs for a 200 mgd estuary water treatment plant for the alum/chlorine and lime/ozone process combinations are shown in Tables F-9 and F-10, respectively. Shown for the individual processes and groupings of processes are the capital costs, the annual operation and maintenance costs, and the cost in dollars per $1000m^3$ (cents/1000 gal). This unit cost was based on amortizing the capital costs over a twenty year period at 8 percent interest.

Cost Comparison with Conventional Water Treatment

For purposes of comparison, the costs for the two process combinations monitored were compared with a more conventional water treatment plant that was not treating a contaminated raw water source. A realistic conventional water treatment plant would be the same as described in the alum/chlorine process combination if the GAC adsorption process was removed. The cost comparison is shown in Table F-11.

TABLE F-8

COST CRITERIA FOR FULL-SCALE ESTUARY PLANT

I. Capital Cost Factors (% of Construction Costs):

1.	Engineering (%)	=	7.5
2.	Sitework, interface piping (%)	=	5.5
3.	Subsurface considerations (%)	=	1.0
4.	Standby power (%)	=	1.0
5.	Interest rate (%)	=	8.0
6.	Number of years for capital cost amortization	=	20.0

II. Unit Cost Factors:

1.	Electricity ($/KWH)	=	0.060
2.	Labor ($/hr)	=	12.000
3.	Diesel fuel ($/gal)	=	1.180
4.	Natural gas ($/ft^3)	=	0.007[a]
5.	Building energy use (KWH/ft^2/yr)	=	102.600

[a] Based on use of Number 2 fuel oil @ $1.00/gal to obtain equivalent BTU's. Assumes 1,000 BTU/ft^3 of natural gas versus 141,000 BTU/gal fuel oil.

III. Cost Indexes (April, 1983, Baltimore):

1.	Excavation (ENR skilled labor)	=	332.8
2.	Manufactured equipment (BLS #114)	=	307.5
3.	Concrete (BLS #132)	=	309.6
4.	Steel (BLS #101.3)	=	340.3
5.	Labor (ENR skilled labor)	=	332.8
6.	Pipes & valves (BLS #114.901)	=	323.5
7.	Electrical and instrumentation (BLS #117)	=	235.8
8.	Housing (ENR Building Cost)	=	339.9
9.	Producer Price Index	=	285.7

IV. Chemical Costs:

1.	Chlorine ($/ton)	=	230
2.	Alum ($/ton)	=	147
3.	Polymer ($/ton)	=	6,000
4.	Potassium permanganate ($/ton)	=	2,056
5.	Quicklime ($/ton)	=	68
6.	Ferric sulfate ($/ton)	=	140
7.	Androus ammonia ($/ton)	=	240

TABLE F-9

ESTIMATED COST
200 MGD ESTUARY WATER TREATMENT PLANT
ALUM/CHLORINE PROCESS

	($ Million) Capital Costs	($Million) Annual O & M Costs	Cost in $/1000 m^3 (Cents/1000 Gal)
Alum feed	0.46	2.25	8.43 (3.19)
Polymer Feed	0.08	0.19	0.74 (0.28)
Permanganate Feed	0.03	0.64	2.32 (0.88)
Lime Feed	0.23	0.35	1.40 (0.53)
Rapid Mix	1.13	0.85	3.75 (1.42)
Flocculation	2.04	0.84	4.28 (1.62)
Sedimentation	11.43	0.23	7.74 (2.93)
Subtotal	15.40	5.35	28.66 (10.85)
Polymer Feed	0.03	0.06	0.24 (0.09)
Gravity Filters & Media	12.54	0.85	10.65 (4.03)
Air/Water Backwash	1.18	0.08	1.00 (0.38)
Surface Wash	1.84	0.06	1.32 (0.50)
Filter Clearwell	0.58	0.00	0.34 (0.13)
Subtotal	16.17	1.05	13.55 (5.13)
GAC Feed Pumping	1.40	0.95	4.28 (1.62)
Contactors & Carbon	27.54	0.04	16.75 (6.34)
Backwash Pumping	0.50	0.07	0.55 (0.21)
Regeneration & Make-up Carbon	6.70	4.10	18.90 (7.15)
Subtotal	36.14	5.16	40.48 (15.32)
Chlorination (Intermediate) and Final	0.57	0.34	1.59 (0.60)
Final Clearwell/Chlorine Contact	2.31	0.00	1.40 (0.53)
Washwater Storage	0.86	0.00	0.53 (0.20)
Washwater Pumping	0.06	0.01	0.08 (0.03)
Subtotal	0.92	0.01	0.61 (0.23)
Sludge Pumping	0.17	0.03	0.21 (0.08)
Gravity Thickening	0.78	0.02	0.55 (0.21)
Sludge Pumping	0.06	0.02	0.11 (0.04)
Polymer Feed	0.03	0.05	0.21 (0.08)
Centrifugation	1.34	0.07	1.06 (0.40)
Sludge Hauling	0.31	0.17	0.79 (0.30)
Subtotal	2.69	0.36	2.93 (1.11)

TABLE F-9 (Continued)

ESTIMATED COST
200 MGD ESTUARY WATER TREATMENT PLANT
ALUM/CHLORINE PROCESS

	($ Million) Capital Costs	($ Million) Annual O & M Costs	Cost in $/1000 m^3 (Cents/1000 Gal)
Admin, Lab & Maintenance Building	0.61	0.30	1.45 (0.55)
TOTAL PROCESS	74.81	12.57	90.67 (34.32)
Sitework@ 7.5%	5.61		
Contractor OH & P @ 15%	12.06		
TOTAL Construction	92.48		
Engineering @ 7%	6.47		
Legal, Fiscal, Admin	0.21		
Int During Construction	12.02		
Contingency @ 10%	11.12		
TOTAL Capital	122.30		

TABLE F-10

ESTIMATED COSTS
200 MGD ESTUARY WATER TREATMENT PLANT
LIME/OZONE PROCESS

	($ Million) Capital Costs	($ Million) Annual O & M Costs	Cost in $/1000 m^3 (Cents/1000 Gal)
Lime Feed	0.24	0.22	0.98 (0.37)
Ferric Sulfate Feed	0.12	0.14	0.61 (0.23)
Rapid Mix	1.13	0.85	3.86 (1.46)
Flocculation	2.04	0.84	4.39 (1.66)
Sedimentation	11.43	0.23	7.93 (3.00)
Recarbonation	2.60	0.21	2.38 (0.90)
Subtotal	17.56	2.49	20.15 (7.62)
Polymer Feed	0.03	0.06	0.24 (0.09)
Gravity Filters & Media	12.54	0.85	10.91 (4.13)
Air/Water Backwash	1.18	0.08	1.03 (0.39)
Surface Wash	1.84	0.06	1.37 (0.52)
Filter Clearwell	0.58	0.00	0.37 (0.14)
Subtotal	16.17	1.03	13.92 (5.29)
GAC Feed Pumping	1.40	0.95	4.41 (1.67)
Contactors & Carbon	39.80	0.04	24.81 (9.39)
Backwash Pumping	0.53	0.08	0.63 (0.24)
Regeneration & Make-up Carbon	7.70	5.90	26.69 (10.10)
Subtotal	49.43	6.97	56.54 (21.40)
Ozone Generation	6.31	0.69	6.47 (2.45)
Ozone Contact	1.09	0.00	0.66 (0.25)
Subtotal	7.40	0.69	7.13 (2.70)
Ammonia Feed	0.13	0.09	0.42 (0.16)
Chlorination	0.33	0.24	1.11 (0.42)
Subtotal	0.46	0.33	1.53 (0.58)
Final Clearwell	2.31	0.00	1.43 (0.54)
Washwater Storage	0.86	0.00	0.53 (0.20)
Washwater Pumping	0.06	0.01	0.08 (0.03)
Subtotal	0.92	0.01	0.61 (0.23)

TABLE F-10 (Continued)

ESTIMATED COSTS
200 MGD ESTUARY WATER TREATMENT PLANT
LIME/OZONE PROCESS

	($ Million Capital Costs	($ Million) Annual O & M Costs	Cost in $/1000 m^3 (Cents/1000 Gal)
Sludge Pumping	0.14	0.02	0.16 (0.06)
Gravity Thickening	0.43	0.01	0.29 (0.11)
Sludge Pumping	0.06	0.02	0.11 (0.04)
Polymer Feed	0.03	0.05	0.21 (0.08)
Centrifugation	1.39	0.07	1.11 (0.42)
Recalcination	10.00	4.00	21.06 (7.97)
Subtotal	12.05	4.12	22.94 (8.68)
Admin, Lab & Maintenance Building	0.61	0.30	1.51 (0.57)
TOTAL PROCESS	106.91	15.94	125.76 (47.62)
Sitework @ 7.5%	8.02		
Contractor OH & P @ 15%	17.24		
TOTAL Construction	132.17		
Engineering @ 7%	9.25		
Legal, Fiscal, Admin	0.26		
Int During Construction	16.63		
Contingency @ 10%	15.83		
TOTAL Capital	174.14		

TABLE F-11

COST COMPARISON OF CONVENTIONAL PROCESS
WITH PROCESSES MONITORED

Treatment Configuration	Cost in \$/1000 m^3 (Cents/1000 Gal)
Conventional Water Treatment Plant	50.48 (19.00)
Alum/GAC/Chlorine Process Combination	90.67 (34.32)
Lime/GAC/Ozone Process Combination	125.76 (47.62)

CHESAPEAKE BAY HYDRAULIC MODEL TESTING

INTRODUCTION

As noted in the earlier description of some of the physical characteristics of the Potomac Estuary, the hydrodynamics of the estuary are very complex and in some ways little understood. To determine the potential of the estuary to serve as a source of water supply not only required the results of the Potomac Estuary Experimental Water Treatment Plant Program, but an understanding of the estuary's hydrodynamics under various flow regimes. The Corps' Chesapeake Bay Model provided a means of reproducing to a manageable scale both natural events and man-made changes, and thereby allowed the collection of some of the data required to assess the consequences of these happenings. The following paragraphs provide a description of the hydraulic physical model and the testing program that was conducted relative to the Potomac Estuary.

MODEL LOCATION

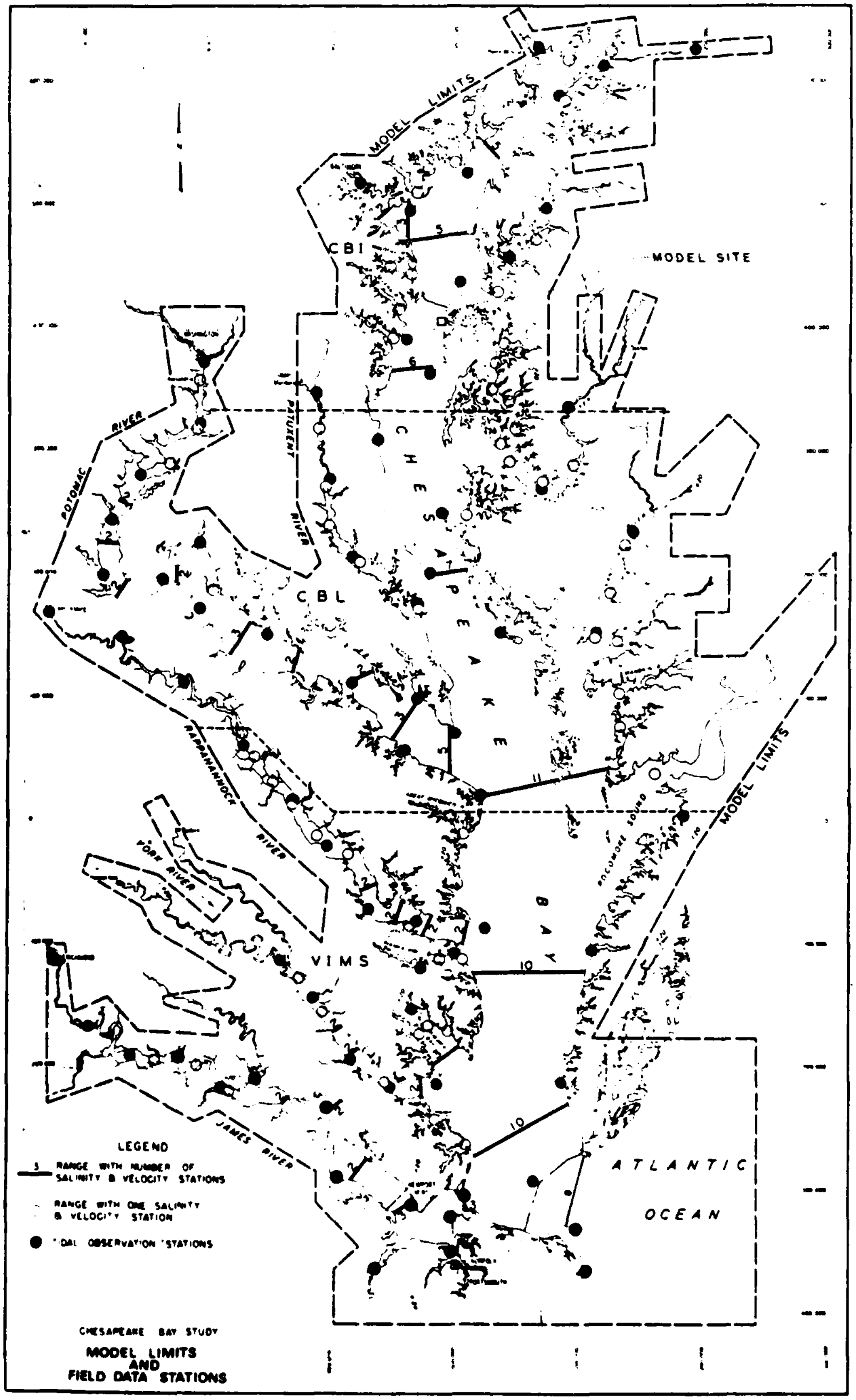
The hydraulic model of the Chesapeake Bay is located at Matapeake, Maryland, on a 65-acre tract of land donated by the State of Maryland. The site is on the Delmarva Peninsula, lies along Maryland Route 8, and is approximately three miles south of the eastern terminus of the William Preston Lane Memorial Bridge (Chesapeake Bay Bridge). It is within commuting distance of over 3,000,000 people, being less than 50 miles from both Washington, D.C. and Baltimore, Maryland.

MODEL DESCRIPTION

The hydraulic model of the Chesapeake Bay is the largest estuarine model in the world. It is a fixed-bed, geometrically distorted scale model, hand-molded in concrete. The model is nine acres in area, and encompasses the Bay proper, all of its tributaries up to the head of tidal effects, and the adjacent overbank areas to the contour of 20 feet above mean sea level. The model is enclosed in a 14-acre, prefabricated steel truss building in order to protect it from such elements as wind, rain, and debris. Figure F-13 is a map showing the model limits.

FIGURE F-13

CHESAPEAKE BAY MODEL LIMITS



The hydraulic model was designed based on the equality of Froude numbers, model to prototype, reflecting similitude of gravitational effects. The Froude number, IF, is defined as:

$$IF = \frac{V}{gd}$$

where V = velocity
g = gravitational acceleration
d = characteristic length

For distorted-scale models, the characteristic length, d, is taken to be the vertical dimension or depth. Geometric scales of the model are 1:1000 horizontally and 1:100 vertically, reflecting a distortion ratio of 10:1. These dimensions and Froudian model laws defined the following model-to-prototype ratios:

Characteristic	Model: Prototype Ratio
Vertical length	1:100
Horizontal length	1:1000
Slope	1:10
Time	1:100
Velocity	1:10
Volume	1:100,000,000
Discharge	1:1,000,000

The model-to-prototype ratio for salinity is 1:1 which is the general practice for distorted-scale models. The model was designed and equipped so that selected prototype boundary conditions could be simulated, and the model response to these conditions recorded.

The hydraulic model of the Chesapeake Bay is a facility of the Baltimore District, Corps of Engineers, and all aspects of the model-related program are the responsibility of the Baltimore District Engineer. As personnel of the Waterways Experiment Station, (WES) are the Corps of Engineers' recognized experts in hydraulic modeling, WES has been responsible for the design, construction, operation, and maintenance of the hydraulic model.

SCOPE AND OBJECTIVES OF HYDRAULIC MODEL TESTING

As mentioned above, the quality of the water in the Potomac Estuary is a complex combination of several factors to include the Potomac River inflow, the tidal and salinity conditions in the Chesapeake Bay, wastewater discharges from area sewage treatment plants, and withdrawals from the estuary. The objectives of the Chesapeake Bay Hydraulic Model testing were to define the salinity regime and wastewater dispersion patterns in the Potomac Estuary under several freshwater inflow conditions, and to determine the impact of pumping water out of the upper Estuary at Washington, D.C. on both salinities and wastewater dispersion patterns.

The testing was to be conducted in two phases. The objective of the Base or Phase I testing was to define the salinity and wastewater dispension patterns for freshwater inflows under present (1980) conditions assuming no estuary withdrawal. The objective of the Futures or Phase 2 testing was to define salinity and dispersion patterns for four freshwater inflows under future (2020) conditions assuming estuary withdrawals ranging between 0 and 200 mgd. Table F-12 lists the inflow and withdrawal conditions to be reproduced during both phases of the testing.

In response to the above objectives, three components of the estuary were examined in detail:

a. The salinity regime of the entire Estuary over the test period was measured to establish the variation in salinity as a function of Potomac River low flows and wastewater discharges. An intensive sampling program was conducted at the start of each test to examine the salinity variations in the Estuary during spring and neap tides. The time interval between the start of the drought period and the first appearance of salinity at the Washington Aqueduct Emergency Estuary Water Pumping Station was determined as a function of Potomac discharge.

b. The dispersion of the conservative wastewater constituents (dye injected into the model) from the Washington Area Sewage Treatment Plants (STP's) as affected by Potomac discharge for Base and Future test conditions was investigated. An intensive sampling program covering both the spring and neap tide period was conducted at the start of each test to examine the dispersion of the dye and the possible transport of dye upstream. Similar to the salinity test, the arrival of the waste water at the Washington Aqueduct Emergency Water Pumping Station was determined as a function of discharge.

c. The mixing of the Potomac River flow as it entered the estuary as a function of Potomac River low flow was also studied.

Based on the availability of both personnel and the model facilities, together with the level of funding for the Chesapeake Bay Study Program, it was decided that the estuary testing as shown on Table F-12 would be conducted in two parts. The first part, which was comprised of eight tests, was conducted in March - June 1979. The testing conducted included all of Phase 1 (Tests 1-4) and Tests 5, 6, 8 and 9 from Phase 2. Further testing on the second part of program is contingent upon future funding of the Chesapeake Bay Program by Congress. The remainder of this section of the report addresses the conduct and results of the first part of the testing.

MODEL TEST CONDITIONS AND PROCEDURES

Model Geometry

The model geometry was maintained as constructed and verified with the addition of the proposed 50-foot Baltimore Harbor and approach channels and several minor modifications in the Potomac Estuary.

Tidal Conditions

For each test the model was filled by introducing freshwater in the upstream reaches of the rivers and salt water from the return sump. As the model was filled, a repetitive cosine tide was generated. After a short period of time, tide control was switched to a computer-controlled cosine tide. The tide had a range of 4.25 feet and a mean water

TABLE F-12

POTOMAC ESTUARY WATER SUPPLY AND WASTEWATER DISPERSION TESTING
SUMMARY OF INFLOW AND WITHDRAWAL CONDITIONS

TEST	INFLOW POTOMAC RIVER (flow-by) (mgd)	INFLOW ALL OTHER TRIBUTARIES	ESTUARY WITHDRAWAL RATE (mgd)	WASTEWATER[2] TREATMENT PLANT CONDITIONS
PHASE 1 - BASE[1]				
1	0	1960's Drought Flows	0	Present (1980)
2	100	1960's Drought Flows	0	Present (1980)
3	500	1960's Drought Flows	0	Present (1980)
4	900	1960's Drought Flows	0	Present (1980)
PHASE 2 - FUTURE				
5	0	1960's Drought Flows	0	Projected Future (2020)
6	100	1960's Drought Flows	0	Projected Future (2020)
7	500	1960's Drought Flows	0	Projected Future (2020)
8	900	1960's Drought Flows	0	Projected Future (2020)
9	0	1960's Drought Flows	100	Projected Future (2020)
10	100	1960's Drought Flows	100	Projected Future (2020)
11	500	1960's Drought Flows	100	Projected Future (2020)
12	900	1960's Drought Flows	100	Projected Future (2020)
13	0	1960's Drought Flows	200	Projected Future (2020)
14	100	1960's Drought Flows	200	Projected Future (2020)
15	500	1960's Drought Flows	200	Projected Future (2020)
16	900	1960's Drought Flows	200	Projected Future (2020)

[1]Test to be conducted using a second dye which would be representative of the water quality of the Potomac River over Little falls.

[2]Present Conditions - 418 mgd; Future Conditions = 705 mgd.

level of +0.18 feet. This tide was representative of the maximum spring tide of the 28-day lunar month tide. The tide was repeated until the model reached stability. At a specified time, after both tide and salinity stability had been achieved, the tide was changed to a 28-day lunar month tide which was maintained during the hydrograph and steady-state low flow conditions.

Chesapeake and Delaware Canal

The Chesapeake and Delaware (C & D) Canal was not operated during the Potomac Estuary Testing. The associated boundary control conditions of C & D tides, source salinity, and net flows were not applicable. The C & D Head Bay weir was raised to its maximum position so that there was no tidal or salinity input to the canal from the Delaware River.

Freshwater Inflows

The model was stabilized at a discharge of 100,000 cfs using a repeatable cosine tide. After stabilization, the model was stepped through 4-3/4 months of weekly hydrographs with a 28-day variable tide, simulating the period April - August 1964, to dynamically bring the model to drought conditions. Drought conditions were maintained for a 6-month test period with all inflows set at the average August - October 1964, steady-state flows. The Potomac River discharge into the upper Potomac Estuary was varied from 0 to 900 MGD during the drought, according to the test requirements.

Wastewater Inflows

Wastewater discharge for the Washington Area STP's in the upper Potomac Estuary was simulated by constant discharges of a conservative dye (Rhodamine WT). Table F-13 lists the MWA STP's and their respective wastewater flows for the Present and Future tests (the Future tests represent projected 2020 wastewater flows).

The wastewater inflows during lead-in steady-state flows were supplemented into the Potomac River at the base flow rate of 418 mgd. At the start of the hydrograph, the wastewater flow was transferred to the respective outfall locations. Freshwater was used to simulate the wastewater until Lunar months - 0 and Tidal Cycle - 36, when the dye release started. During a brief period prior to dye release, the outfalls were disconnected and dye was run through the lines. The wastewater flow rate was measured volumetrically; and at slack after flood at station PO 01-03 (the mouth of the Potomac River), the outfalls were connected and dye was released into the model. Outfall lines were modeled to discharge "Q" and exit velocity "V" of the wastewater. Outfall area "A" was not modeled due to the distortion ratio of the model. Outfalls were set at prototype location and depth. Wastewater specific gravity was set at 1.0.

Ocean Source Salinity

The model ocean source salinity was maintained at 31 gm/l for all of the Potomac Estuary tests. Sumps were monitored hourly and salinity adjusted as necessary by increasing or decreasing lixate flow. The return sump level was kept at 3.0 $\pm$ 0.1 feet during normal operation to avoid changing the flow characteristics of the return gate valve. To prevent the sumps from gaining head and/or changing salinity, surface water of lower salinity was drawn from the ocean through the skimming weirs. The skimming weir control valve was adjusted so that the rate of withdrawal through the skimming weirs equalled the freshwater inflow to the model.

TABLE F-13

POTOMAC ESTUARY TESTING
WASTEWATER TREATMENT FACILITIES

DESIGNATED INFLOW POINT (FIGURE F-14)	LOCATION OF FACILITY	PRESENT[1] (mgd)	FUTURE[2] (mgd)
A	Blue Plains	305	450
B	Piscataway	22	60
C	Arlington	20	30
D	Alexandria	33	40
E	Lower Potomac	38[3]	60
F	Mattawoman	-	65[4]

[1]Based on projected 1980 discharges.

[2]Based on projected capacity requirements for 2020 from current 208 planning documents.

[3]Combined Lower Potomac and Mooney during Base Test.

[4]Combined Mattawoman and Mooney during Futures Test.

Data Collection

Since the major objectives of the study were salinity changes and overall wastewater dispersion characteristics, salinity and dye sampling were emphasized. In order to provide additional data for numerical modeling, tidal heights and velocities were also collected at several stations. A more detailed description of data collection procedures is provided in the following paragraphs.

Salinity Data

Salinity-dye samples were collected at the stations shown in Figure F-14. Samples were taken at Slack After Flood (SAF) and Slack After Ebb (SAE). When water depth exceeded 60 feet, samples were taken at the surface, one-quarter depth, mid-depth, three-quarters depth, and bottom. When depths ranged from 20 to 60 feet, samples were collected at the surface, mid-depth, and bottom. At depths between 10 and 20 feet, samples were collected at surface and bottom. At depths less than 10 feet, samples were collected at mid-depth only.

Sampling commenced at lunar month 1 and tide 42. The upper estuary (PO-06 to PO-16) was sampled at SAF and SAE for the first 15 consecutive tides. The lower estuary (PO-01 to PO-05) was sampled on lunar month 1, tides 42 and 48. Starting with lunar month 2, all stations were sampled on tides 1, 10, 28, and 48 of the 56 cycle - 28-lunar day tide except at the end of the test when tide 42 of lunar month 7 was sampled in lieu of tide 48.

During each test, a synoptic salinity sample was taken of the entire Potomac Estuary on lunar month 3, tide 38, at high water slack at the mouth of the Potomac River. A series of synoptic samples were also taken during Test 9 from the beginning of the dye release at lunar month 0 and tidal cycle 36 until the test start, at 8 tidal cycle increments to establish the initial dye dispersion pattern.

Salinities were monitored at the Chesapeake Bay salinity monitoring stations on the same tides, 1, 10, 28, and 48 to establish a representative salinity for all tests. Salinities were continuously monitored using Balshbaugh-Conductance probes and meters. Salinities were recorded at mid or near bottom depth on a strip chart recorder, and monitoring notes were made approximately every three hours.

Dye Concentrations

Fluorescent dyes were used to trace wastewater effluent and the Potomac River inflow. The wastewater of the Washington Area STP's was labeled using a conservative fluorescent dye (Rhodamine WT). The concentration of Rhodamine WT was 1000 parts per billion (ppb) for all tests. The Potomac River freshwater was labeled using a conservative fluorescent dye, Fluorescene, for all Base tests and the Future tests. The Fluorescene was injected at a concentration of 1000 ppb.

Dye was released and allowed to achieve a background equilibrium prior to the start of the test. Dye was released at both the Washington Area STP's and in the Potomac River at a constant flow for the duration of the test. Dye-salinity samples were collected at stations in the upper estuary (PO-9 to PO-16). Dye concentration was also analyzed

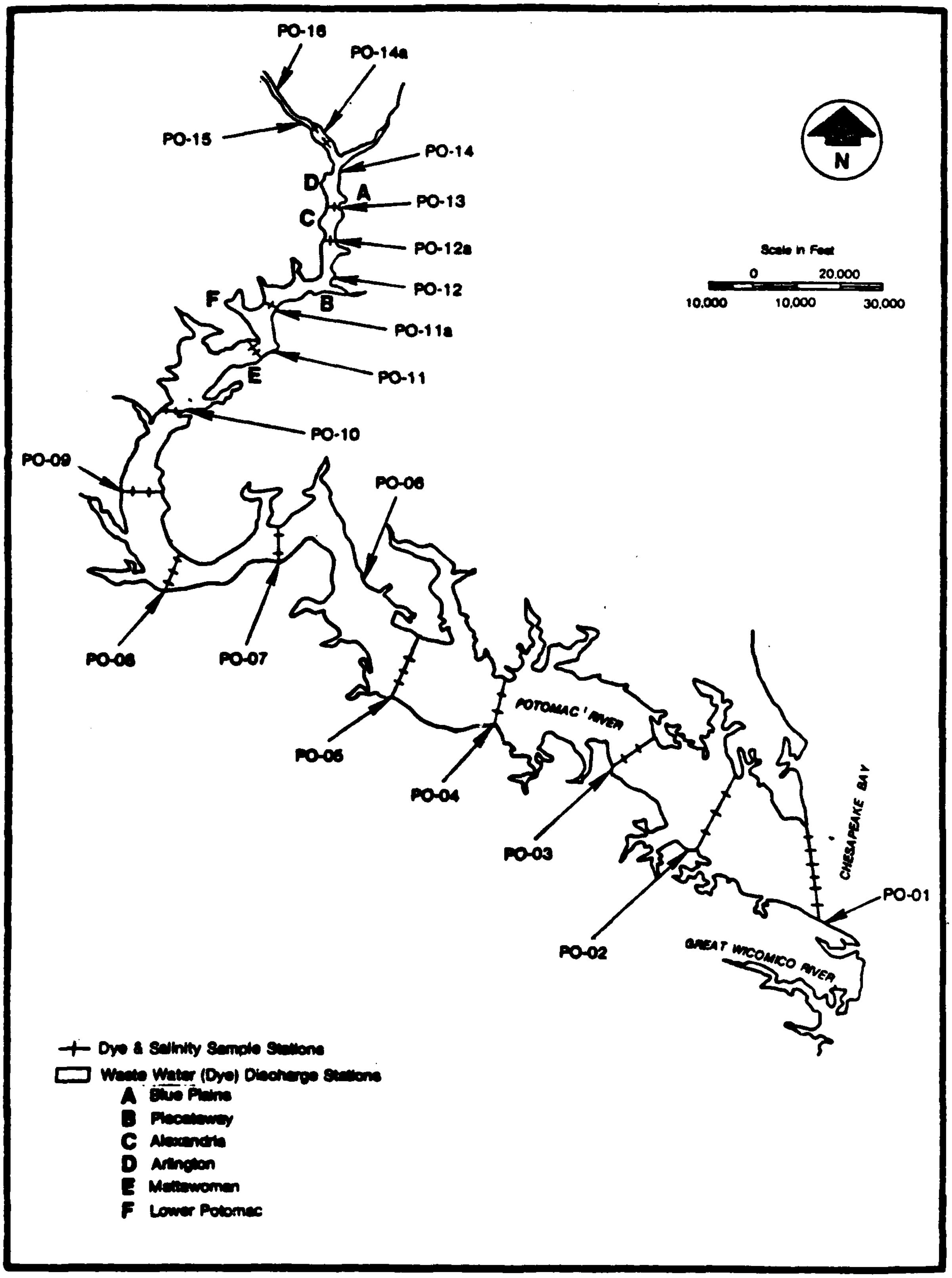


FIGURE F-14

LOCATION OF DATA COLLECTION STATIONS

during the synoptic sampling lunar month 3, tide 38, and during the pre-test of Test 9. Sampling procedures have been described in the preceding discussion of the salinity data.

Tidal Elevations

Tides were monitored throughout the model using nine automatic water level detectors (Tide Data Acquisition (TDA)). Four of the TDA's were positioned in the Potomac Estuary at Cornfield Harbor, Dalgren, Quantico, and Washington D.C. Manual tide measurements were also taken at these locations to give comparison tide values, and to check for TDA error or drift. Manual tide measurements were taken three times per test at the four TDA locations. Tides were measured during lunar months 2, 4, and 6, starting at low water (LW) on tide 53 and continuing on a lunar hour basis to LW of tide 55. Tide 55 was representative of a maximum spring tide.

Current Velocities

Current velocities were collected in two base tests (1 and 4) and two futures tests (5 and 9). Velocities were taken at stations PO-1, PO-6, PO-11, and PO-14 at the same depths as the salinity samples.

Velocities were measured on lunar month 5, between low water of tides 21-22 for the bottom depth, between low water 23-24 for the mid-depth, and between low water 25-26 for the surface. Readings were taken every lunar hour. Tides 22, 24, and 26 were representative of an average tide for the 28-day lunar tide.

Test Results

As noted in the preceding sections, salinity, dye concentrations, tidal elevation, and current velocity data were collected during the eight tests which were conducted as part of the first phase of testing. Unfortunately, without the data from the eight remaining tests it was not possible to satisfy the objectives of the testing program as originally stated; however, some representative data from the initial tests are presented in the following paragraphs, as well as some generalized statements regarding the significance of these data.

Based on a cursory examination of the salinity data, the tests results confirmed expected hydrodynamic conditions in the Potomac Estuary. Salinity declined with the distance from the mouth of the Potomac and varied with the level of Potomac inflows, waste-water discharges, and withdrawal at the Emergency Estuary Water Pumping Station. Further, the longitudinal salinity distribution generally followed observed data with salinity increasing with water depth throughout the Potomac Estuary.

As it related to salinity, the area of greatest interest was the degree of salinity intrusion that occurred under various inflow conditions. Table F-14 provides an overview of the salinity intrusion by showing the estimated time of arrival of various salinities at station PO-16-01 (Emergency Pumping Station Upstream from Chain Bridge). For example, this table indicates that with a recurrence of the 1964 flows (April-August 1964) as a lead-in for a drought condition, it would take approximately 13 weeks for the salinity to reach 1 ppt at PO-16 assuming a Potomac inflow of 100 mgd. Given the nature and duration of both the 1960's and 1930's droughts it is not unreasonable to assume that salinity intrusion could occur and may present a potential treatment problem for an estuary

TABLE F-14

POTOMAC ESTUARY MODEL TESTING
SALINITY TIME OF ARRIVAL AT EMERGENCY PUMPING STATION (PO-16)

Test	Potomac Inflow (Flow-by) MGD	Inflow Other Tributaries	Wastewater Treatment Plant Conditions	Estuary Withdrawal (MGD)	Salinity Time of Arrival (in Weeks) at Emergency Pumping Station[1] Salinity Level in ppt[2]			
					1.0	2.0	3.0	4.0
Phase 1 - Base								
1	0	1960's Drought Flows	Present (1980 - 418 mgd)	0	9	13	18	22
2	100	1960's Drought Flows	Present (1980 - 418 mgd)	0	13	20	22	24
3	500	1960's Drought Flows	Present (1980 - 418 mgd)	0	-	-	-	-
4	900	1960's Drought Flows	Present (1980 - 418 mgd)	0	-	-	-	-
Phase 2 - Future								
5	0	1960's Drought Flows	Future (2020 - 705 mgd)	0	10	20	22	-
6	100	1960's Drought Flows	Future (2020 - 705 mgd)	0	14	17	21	-
8	900	1960's Drought Flows	Future (2020 - 705 mgd)	0	-	-	-	-
9	0	1960's Drought Flows	Future (2020 - 705 mgd)	100	9	13	20	25

1. Given that the model was brought to drought conditions by simulating the period April-August 1964, this represents the number of weeks it would take the salinity to reach the designated value with a steady-state Potomac inflow as noted.

2. Salinity samples taken at the bottom of the section.

treatment facility. It should be recognized, however, that the severe salinity intrusion occurred at only the lowest flowby values and during the latter part of the drought period.

As a further example of some of the salinity results, included as Figures F-15 and F-16, are a longitudinal salinity distribution for the entire Potomac River Estuary and a salinity time history for several stations, respectively. Both of these figures are based on salinity data from Test 2 of the Phase I testing which reflects the base conditions and a Potomac inflow of 100 mgd. These two figures also supported the conclusion that during a severe, prolonged drought, nearly the entire Potomac Estuary to Little Falls is subject to saline water intrusion for Potomac flowbys of 100 mgd or less. This conclusion was further supported by the results of the Low Freshwater Inflow Study model testing which also demonstrated rather extensive salinity intrusion under prolonged drought conditions.

It should be noted that a more refined estimate of the extent and duration of the salinity intrusion plus the impacts of varying levels of estuary pumping could not be developed until the remainder of the hydraulic model testing is conducted.

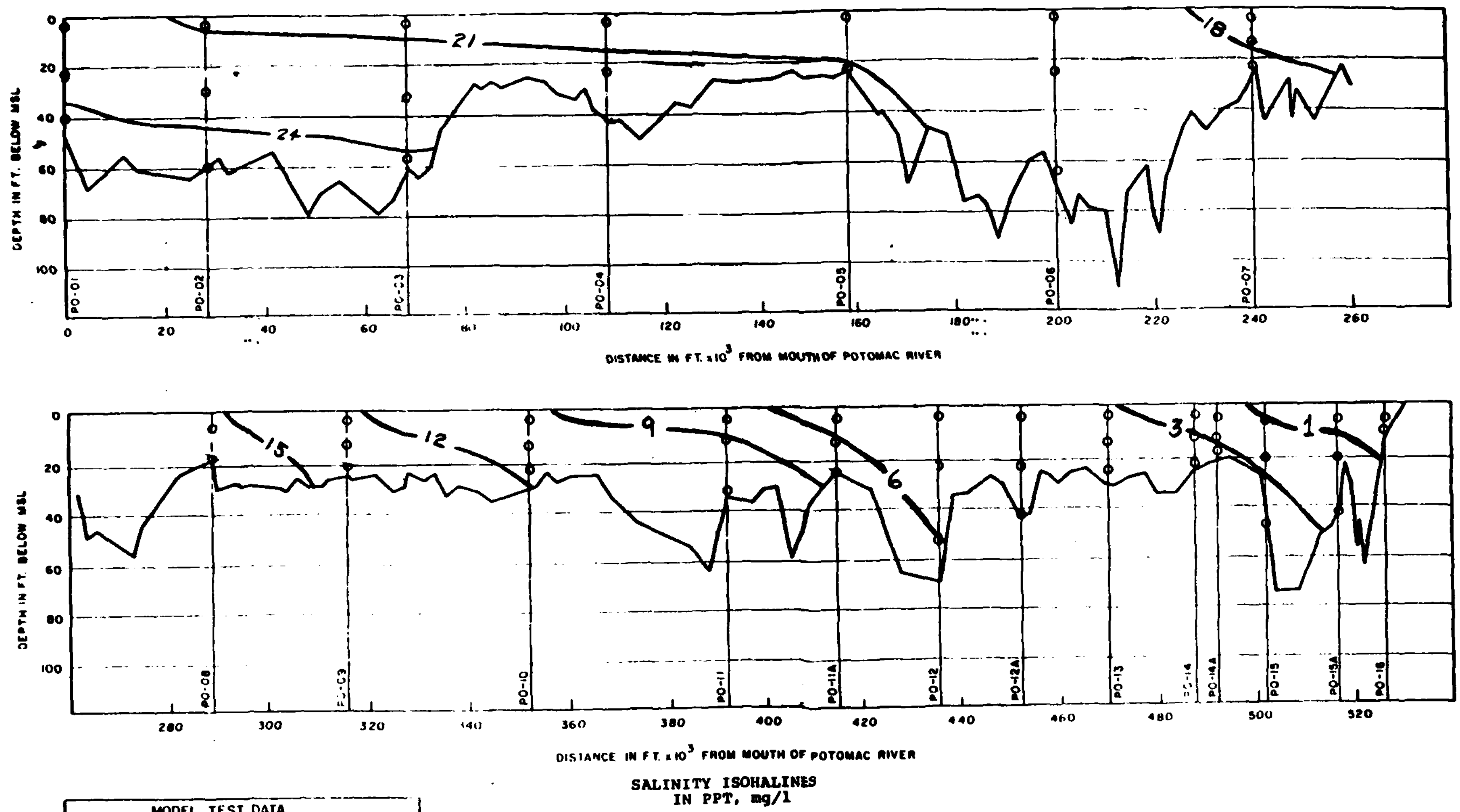
It was also difficult to draw any conclusions relative to wastewater dispersion patterns using the results of only the initial dye dispersion testing. As noted in Figure F-17, concentrations of dye on the order of 200 ppb do reach a point midway between station PO-15A (Georgetown Reservoir) and Station PO-16 (Chain Bridge) under base conditions (Test 2) and a 100 mgd Potomac River inflow. The source dye for this test was Rhodamine WT which was released at the Washington area sewage treatment plants noted on Figure F-17 and in Table F-13. It should be noted that the hydraulic model testing provided only the dispersion characteristics of a conservative dye and not the level of pollutants that could be expected at any given point in the model. It was originally intended that following completion of the second phase of the physical model testing that the physical model data would be used as input to the Environmental Protection Agency's Dynamic Estuary Model (numerical) which would then be run to provide estimates of the levels of pollutants under the various conditions tested. Unfortunately, the second phase of the hydraulic model testing has not been funded, and there is insufficient data to conduct the numerical modeling. No conclusions relative to the level of pollutants at any proposed estuary treatment plant locations can be provided at this time.

Generally, it would appear that the suitability and treatability of the estuary water would be more of a function of the levels of salinity that could occur under drought conditions rather than degraded water quality from the sewage treatment plants in the MWA. Further hydraulic and numerical modeling should be conducted prior to any recommendation for use of the estuary as a future source of supply.

RESERVOIRS

INTRODUCTION

The Metropolitan Washington Area (MWA) depends on the Potomac River for its water supply with the Potomac River providing more than two-thirds of the water supply for the three major Potomac users - the Washington Aqueduct (WAD), Fairfax County Water Authority (FCWA), and Washington Suburban Sanitary Commission (WSSC). There are



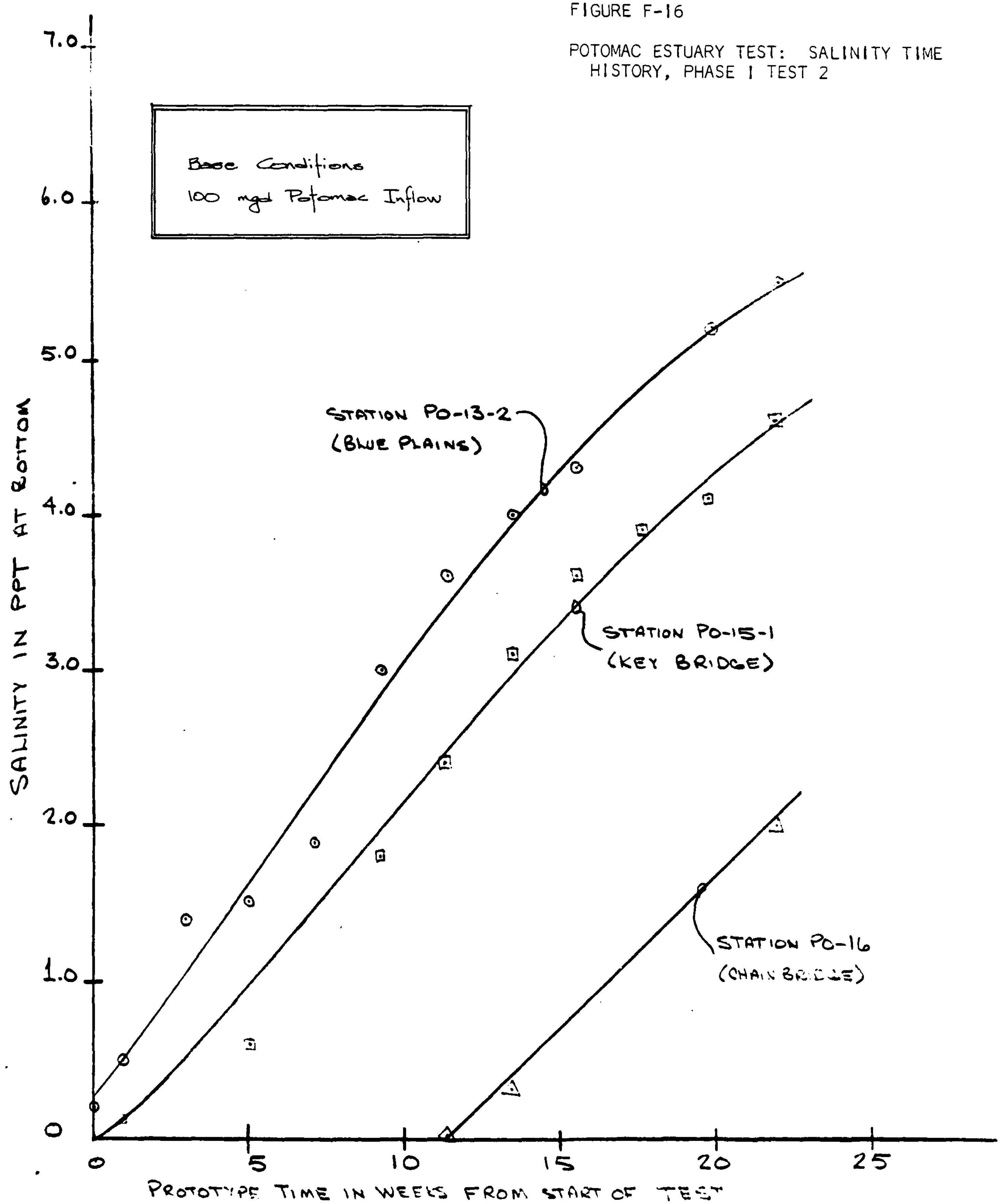
MODEL TEST DATA	
TEST NO	03PO5A
TIDE	28 LUNAR-DAY
LUNAR MONTH/TIDE CYCLE	S/1
OCEAN SOURCE SALINITY	30 PPT
WASTEWATER DISCHARGE	418 MGD
POTOMAC DISCHARGE	100 MGD
POTOMAC WITHDRAWAL	0 MGD

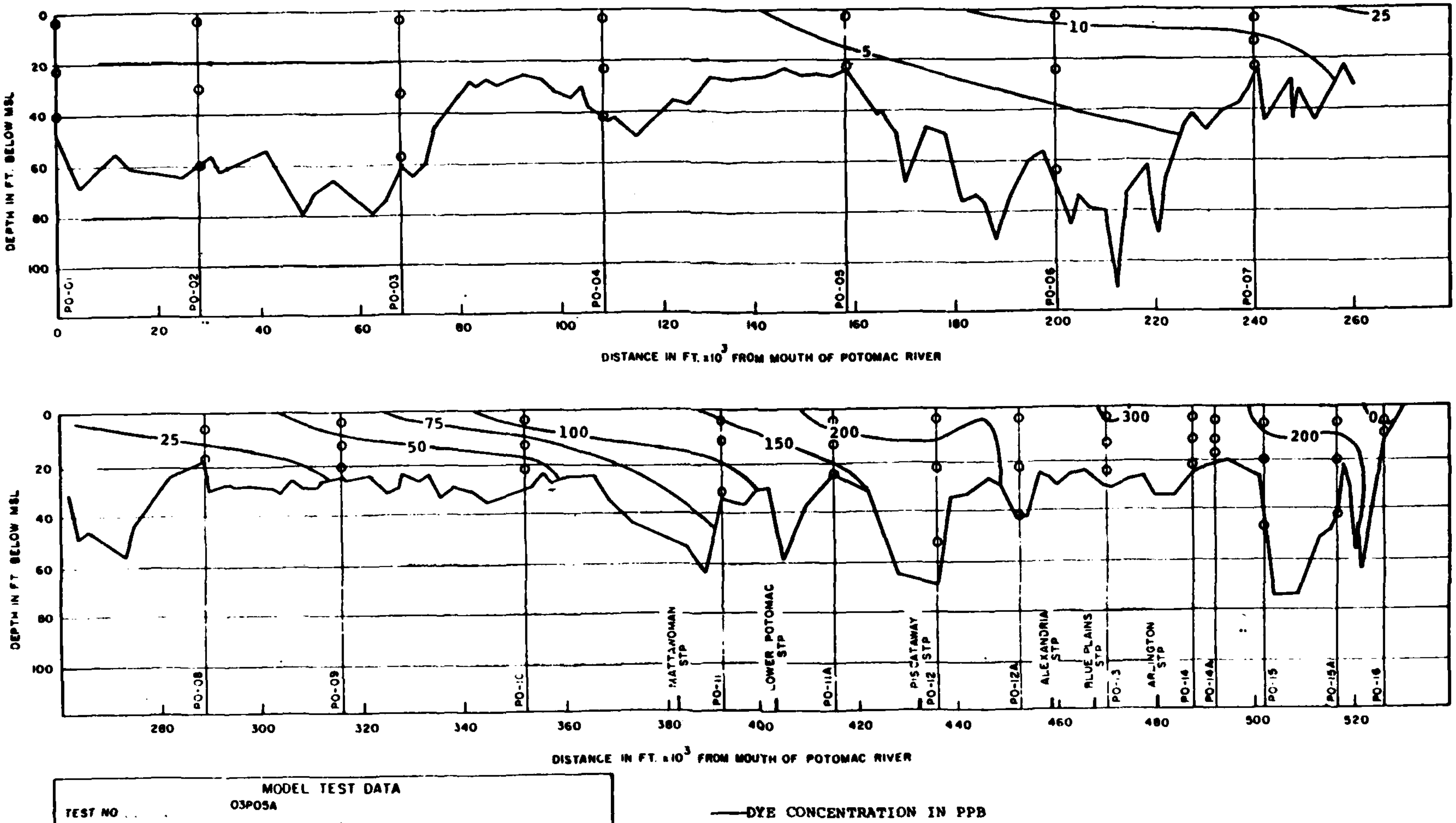
FIGURE F-15

POTOMAC ESTUARY MODEL STUDY:
LONGITUDINAL SALINITY DISTRIBUTION,
POTOMAC RIVER

FIGURE F-16

POTOMAC ESTUARY TEST: SALINITY TIME HISTORY, PHASE I TEST 2





MODEL TEST DATA	
TEST NO.	03P05A
TIDE	28-LUNAR-DAY
LUNAR MONTH/TIDE CYCLE	4/48
DYE RELEASE POINTS	WASHINGTON AREA SEWAGE TREATMENT PLANTS
SOURCE DYE	RHODAMINE WT
DYE INJECTION RATE	418 MGD-BASE
SOURCE DYE CONCENTRATION	1000 PPB
WASTEWATER DISCHARGE	418 MGD
POTOMAC DISCHARGE	100 MGD
POTOMAC WITHDRAWAL	0 MGD

FIGURE F-17

POTOMAC ESTUARY MODEL STUDY:
LONGITUDINAL DYE DISTRIBUTION,
POTOMAC RIVER

several impoundments within the MWA, on the Patuxent River in Maryland and Occoquan Creek in Virginia, which together provide about one-third of the MWA water supply. With the completion of FCWA's Potomac River intake, all three major users within the MWA have direct access to the Potomac River.

The Potomac River Basin has approximately 14,500 square miles of drainage area, and spreads into several states including West Virginia, Virginia, Pennsylvania, Maryland, and the District of Columbia. The Potoma River Basin is substantially uncontrolled with the exception of the Corps of Engineers' recently completed Bloomington Lake Project on the North Branch. From the Potomac's source to its outfall into the Chesapeake Bay, there is no other major reservoir project. While there are some other storage projects, such as the Savage River Reservoir and Stony River Reservoir, they are relatively small.

The MWA Water Supply Study was authorized to investigate and evaluate all alternative means to provide water supply for the growing MWA population. Several alternatives, which are described in other sections of this report, were investigated. The objectives of this section are to identify the reservoir sites which have been investigated in previous studies or proposed by others for investigation, and to provide pertinent data and updated costs for these sites. This information is intended to show only that reservoir storage is one of several means to provide additional flow into the Potomac River during low flow periods. It is neither the purpose nor the objective of this section to recommend construction of any reservoir project. The information presented in this section simply reports on those reservoir sites that have been investigated by various studies, and could provide additional flow for water supply or flowby.

LOCAL RESERVOIRS

RESERVOIRS EXAMINED IN THE AUGUST 1979 PROGRESS REPORT

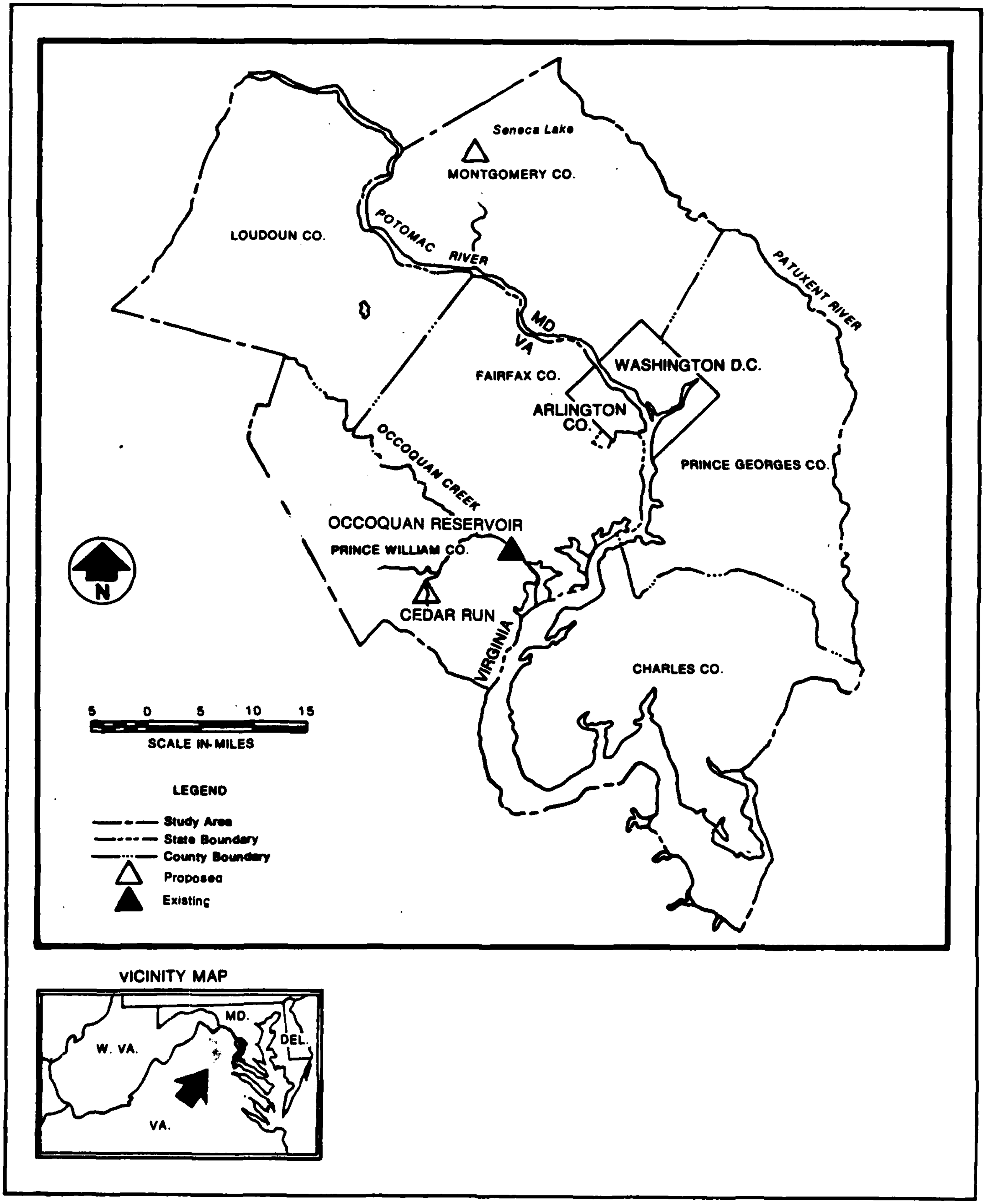
The Metropolitan Washington Area Water Supply Study for the Potomac Water Users, Local Storage Speciality Appendix, August 1979, identified three local reservoir sites - Little Seneca Creek, raising the existing Occoquan Project, and Cedar Run. Figure F-18 shows the locations of these three local sites. The Little Seneca Creek Project which is being actively pursued by WSSC, will be constructed on Little Seneca Creek in Montgomery County, Maryland, as a regional project to benefit all three Potomac users. Regarding the second local site, FCWA has completed raising the Occoquan Creek Reservoir by two feet. The third project examined in the 1979 Progress Report is located in Prince William County, Virginia. The Cedar Run Reservoir would provide additional storage capacity needed for the projected growth in Prince William County. Details of these three projects are given in the following sections.

Little Seneca Lake

In November 1968, the Maryland-National Capital Park and Planning Commission (M-NCPPC), Montgomery Soil Conservation District, and the Montgomery County Council submitted an application to the Maryland State Soil Conservation Service for Public Law 83-566 assistance in developing a multi-purpose, flood control-recreation project in the Seneca Creek Watershed. P.L. 83-566 authorizes the United States Department of Agriculture Soil Conservation Service (SCS) under authority of the Watershed Protection and Flood Protection Act of 1954, as amended, and in accordance with the National Environmental Policy Act of 1969, Section 102 (2)(c), to construct

FIGURE F-18

LOCATION OF STORAGE SITES EXAMINED IN 1979 PROGRESS REPORT



headwater dams and other engineering works in upstream watersheds for a variety of purposes including flood control. In response to the above request, the SCS prepared a preliminary investigation report for the Seneca Creek Watershed, which identified six impoundment projects. As a result of this report, SCS Site #3 (hereafter referred to as Little Seneca Lake) was recommended as the most feasible of the six projects. Following the recommendation, the Montgomery County Planning Board, under Resolution MCPB 75-35, allocated funds for the planning and design of the project. As envisioned, the project purposes would be to reduce flood damages within the watercourse of the Seneca Creek stream system, to reduce erosion and sediment damages throughout the watershed, to provide some relief to water-based recreational deficiencies in Montgomery County, to improve surface water quality, to maintain fish and wildlife resources, to provide a potential emergency water supply for the Metropolitan Washington Area, and to provide a data base to supplement existing tools for planning and managing watershed resources.

Subsequent to the report's release in December 1975, the Bi-County Water Supply Task Force, comprised of representatives from the Washington Suburban Sanitary Commission (WSSC), and the Montgomery and Prince Georges County Councils, commissioned the consulting engineering firm of Henningson, Durham, and Richardson (HDR) to reevaluate the original SCS design and to determine the project's ability to serve as a water supply facility in satisfying projected water supply shortages in the WSSC water service area.

In December 1977, HDR was also requested to investigate the feasibility of an alternate design for the Little Seneca Lake project that would increase the normal conservation pool level of the reservoir, thus maximizing water supply capacity at the site. The consultant's report, entitled <u>Bi-County Water Supply Study Evaluation of Alternatives</u> and published in March 1978, stated that the project, with minor modifications to the original design could operate as a water supply facility. After this determination, the Little Seneca Lake project was then included as part of the <u>Montgomery County Ten-Year Capital Improvement Program, 1978-1987</u>.

Several independent investigations have demonstrated the utility of Little Seneca Lake as a regional water supply project. The August 1979 MWA Water Supply Study Progress Report identified Little Seneca Lake as part of several regional schemes. The Interstate Commission on the Potomac River Basin through its CO-OP model had demonstrated that Little Seneca Lake should be a part of the MWA regional water supply solution. The Washington Metropolitan Regional Water Supply Task Force also recommended Little Seneca Lake be developed as a regional water supply facility.

In 1979, WSSC awarded a contract for the preparation of an environmental assessment (EA) and an advance engineering and design study for the construction of Little Seneca Lake. On 4 March 1980, WSSC made a formal request to the Baltimore District for a Department of Army permit pursuant to Section 404 of the Clean Water Act. The Environmental Assessment (EA) prepared by WSSC in support of their permit application, was coordinated with the public by WSSC to obtain comments concerning the effects of the proposed project on the environment. This EA was reviewed by the Corps of Engineers as part of the permit process. Additionally, the Corps of Engineers prepared an Environmental Assessment to discuss the issues of concern that were identified in the permit review process, in order to make a determination as to the need for the preparation of an Environmental Impact Statement (EIS). The Corps' Environmental Assessment determined that the construction of Little Seneca Lake was not expected to have significant impact on the environment, and therefore, the preparation of an EIS was not warranted. A permit for the project was issued on 31 March 1982.

Project Description

Little Seneca Lake will be located in the Seneca Creek watershed, just northeast of the community of Boyds in Montgomery County, Maryland (Figure F-19). As presently designed, the dam will be constructed from rock and soil excavated as part of the spillway construction. The upstream face of the dam will be covered with rip-rap to prevent wave damage to the dam. Pertinent data for Little Seneca Lake are given in Table F-15. The impoundment will control a drainage area of approximately 21 square miles.

The reservoir at its normal conservation pool level elevation of 385 feet msl will have a total storage capacity of 4.25 billion gallons of water (13,050 acre-feet) with 4.02 billion gallons (12,350 acre-feet) allocated for water supply. Incidental to the water supply storage, the reservoir will also provide flood control storage for approximately 1.28 billion gallons (3,920 acre-feet) of water.

The normal conservation pool will create a water surface area of 505 acres, increasing to 607 acres at the flood pool elevation of 392 feet msl. The mainstem of the reservoir is primarily concentrated on Little Seneca Creek. Two arms of the lake will extend upstream along Tenmile Creek and Cabin Branch. Figure F-20 shows a plan view of the proposed reservoir area indicating the maximum water surface and normal conservation pool perimeters.

Details of the Little Seneca Lake project are given in the report titled, Project Development Report on Little Seneca Lake for the Washington Suburban Sanitary Commission, by Black and Veatch, Consulting Engineers, Bethesda, Maryland, Project No. 8452-1980.

Project Cost

Preliminary cost estimates for the Little Seneca Lake project are presented in Table F-16. These costs were prepared by the consulting engineers based on January 1980 prices and updated to October 1981 prices using the appropriate ENR Construction Cost Indices.

As part of the regional concept of Little Seneca Lake's use, WSSC has also negotiated a formula to share the project costs with the other Potomac water users - the District of Columbia and the Fairfax County Water Authority (FCWA). Recently, with the help of the Washington Metropolitan Water Supply Task Force and its Citizens Advisory Group, the three users have consummated a cost-sharing agreement. Under the terms of the agreement, WSSC will pay 50 percent, FCWA 10 percent, and the District of Columbia, the remaining 40 percent of the water supply portion of the project costs of Little Seneca Lake. WSSC will also be responsible for paying the project land cost, as well as building and maintaining the regional park around Little Seneca Lake. The construction of Little Seneca Lake is expected to begin in the fall of 1982 and is scheduled for completion within the next three to four years.

Environmental Impacts

On 14 August 1980, WSSC prepared and submitted an Environmental Assessment which enunciates the impact of the Little Seneca Lake project on the area's environment, fishery, water quality, flood protection, water supply, recreation, and economy. The WSSC Environmental Assessment was coordinated with the public to obtain local reaction and comments on the proposed Little Seneca Lake. The Environmental Assessment was

TABLE F-15

PERTINENT DATA FOR LITTLE SENECA LAKE

Dam		
Type		Zoned Earth and Rockfill
Length		Approximately 600 feet
Crest Elevation		408 feet msl
Height Above Stream Bed		91 feet
Reservoir Pool Elevations		
Normal Pool		385 feet msl
Top of Sediment Pool		340 feet msl
Flood Crest, 100-Year Flood		392 feet msl
Maximum Flood Crest, MPF*		403 feet msl
Reservoir Surface Area		
Normal Pool (Elevation 385)		505 acres
Minimum Water Supply Pool (Elevation 340)		94 acres
Lowest Drawdown Pool (Elevation 330)		31 acres
Flood Pool, 100-Year Flood (Elevation 392)		607 acres
Maximum Flood Pool, MPF* (Elevation 403)		767 acres
Gross Reservoir Storage	Acre-Feet	Billion Gallons
Normal Pool	13,050	4.25
Minimum Water Supply Pool	700	0.23
Lowest Drawdown Pool	98	0.03
Flood Pool, 100-Year Flood	16,970	5.53
Maximum Flood Pool, MPF*	24,185	7.88
Net Reservoir Storage (usable storage of water supply)		
	Acre-Feet	Billion Gallons
Normal Pool	12,350	4.02
Service Spillway		
Type		Chute Spillway with Ogee Crest
Width at Crest		70 feet
Crest Elevation		385 feet msl
Peak Discharge, 100-Year Flood		4,210 cfs
Peak Discharge, MPF*		17,400 cfs
Auxiliary Spillway		
Type		Open Channel in Rock with Concrete Control Station
Width at Crest		270 feet
Crest Elevation		392 feet msl
Peak Discharge, MPF*		32,100 cfs
Outlet Works		
Maximum Release Capacity		425 cfs (275 mgd)

*MPF = Maximum Probable Flood

SOURCE: Project Development Report on Little Seneca Lake for the Washington Suburban Sanitary Commission, Black and Veatch, Consulting Engineers, Bethesda, Maryland, 1980.

FIGURE F-19

PROJECT SITE LOCATION FOR LITTLE SENECA LAKE

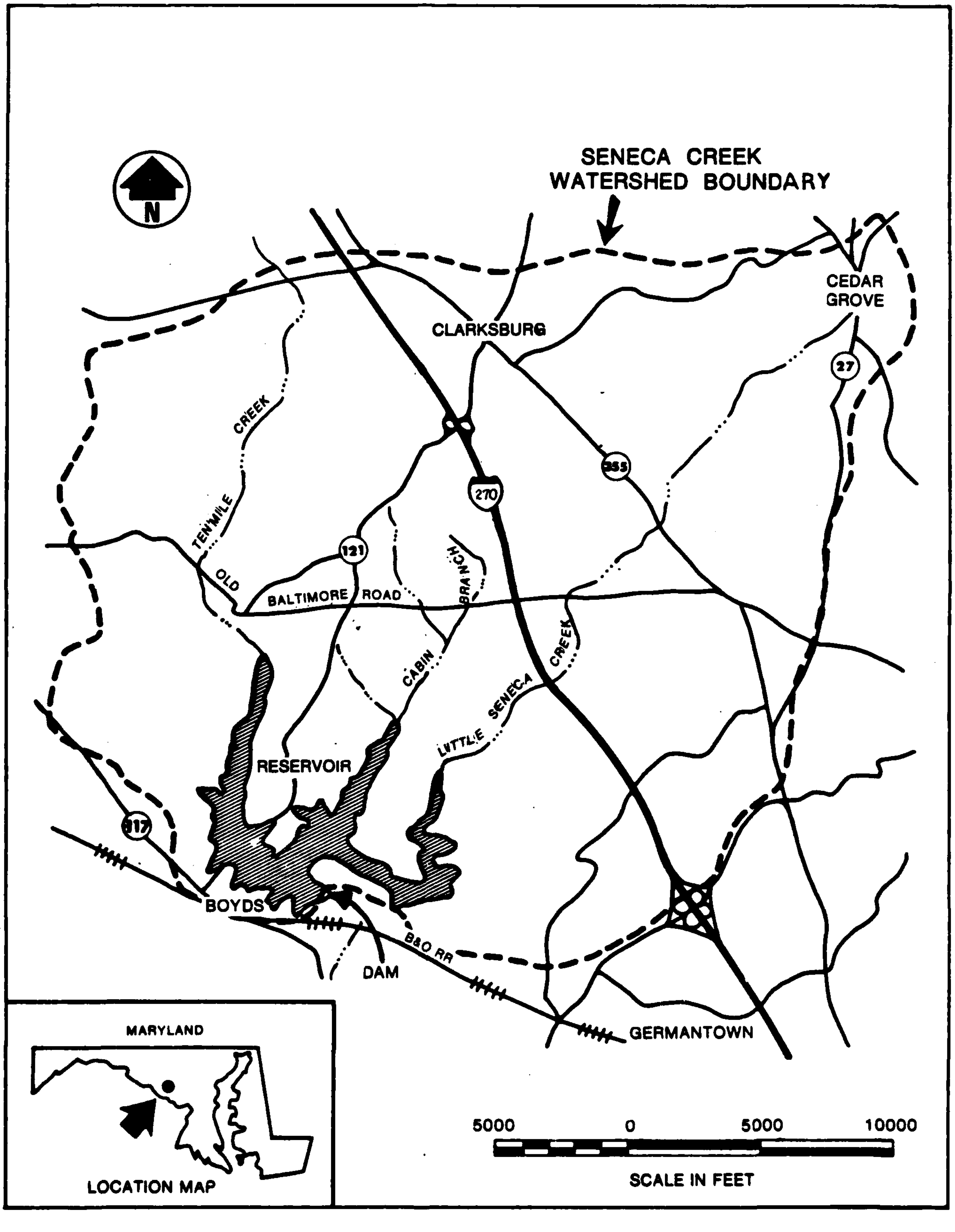


FIGURE F-20

PLAN VIEW OF RESERVOIR AREA FOR LITTLE SENECA LAKE

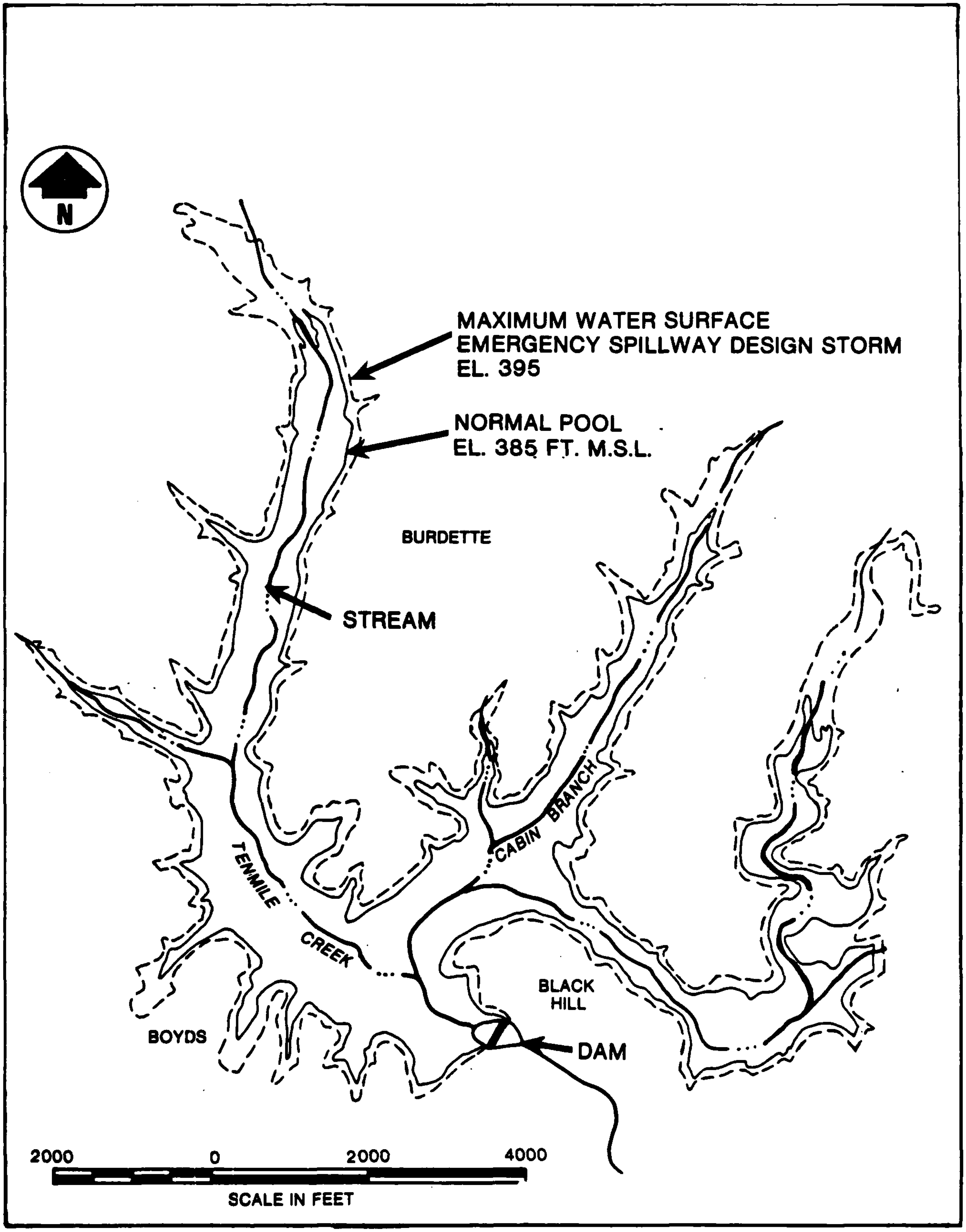


TABLE F-16

SUMMARY OF PROJECT COSTS*
FOR LITTLE SENECA LAKE

		Cost
A.	Water Supply Cost	
	Dam and Spillway	$11,399,000
	Route #121 Relocation	5,320,000
	Utility Relocations	3,127,000
	Protection for Churchill Dam	56,000
	Underwater Treatment and Sediment Control	612,000
	Modifications of Wells and Septic Systems	76,000
	Land Acquisition	2,721,000
	Water and Service to Recreational Areas	874,000
	Subtotal	$24,185,000
	Engineering, Legal, and Administration Costs (15%)	3,628,000
	Total Water Supply Costs	$27,813,000
B.	Recreational Cost	
	Park Land Acquisition	$ 5,114,000
	Development of Planned Recreation Facilities	4,106,000
	Total Recreational Cost	$ 9,220,000
C.	Total Project Cost (A+B)	$37,033,000

*Updated to October 1981 prices.

SOURCE: Project Development Report on Little Seneca Lake for the Washington Suburban Sanitary Commission, by Black and Veatch, Consulting Engineers, Bethesda, Maryland, 1980.

reviewed by the Corps of Engineers as part of the Section 404 permit application. The results of this review indicated that the primary environmental concerns were the water quality projections and the quality of fishery in the lake and downstream area. The Corps of Engineers prepared an Environmental Assessment on the Little Seneca Lake project to discuss the issues of concern that were identified in the review of the Assessment prepared by the WSSC. The additional analysis indicated that fair to good water quality will exist in the lake and that water quality will improve in Seneca Creek downstream from the dam. A moderate warm-water fishery can be expected in the lake with potential for a high quality fishery with proper management of the lake. The dam will tend to decrease habitat diversity, and thus diminish the quality of the fishery for approximately six miles downstream from the dam, at which point substantial recovery of the fishery would be expected. Overall, the diversity of the recreation fishery opportunities within the region will increase with the construction of the Little Seneca Lake project due to the continuation of the downstream fishery and the addition of the warm-water lake fishery.

In summary, the construction of Little Seneca Lake project is not expected to have a significant impact on the environment. The project will be a valuable addition for alleviating future regional water supply shortages and will contribute to a more efficient management of the water resources of the region.

Project Status

As mentioned before, the required permit was issued and construction began in the fall of 1982 with completion expected in late 1984. The advanced status of the project merited its inclusion as part of the baseline conditions for the MWA Water Supply Study.

Occoquan Dam Modifications

In recognizing the need to accommodate continuing urban development, FCWA commissioned the consulting engineering firm of Greely and Hansen in 1970 to prepare a comprehensive report that would: (1) evaluate additional water supply needs; (2) investigate feasible alternative means for satisfying these needs; and (3) recommend a program of progressive water supply improvements to continuously maintain an adequate system for the county. Among the potential water supply alternatives identified by this report for further investigation was a proposal to increase the existing storage capacity of the Occoquan Reservoir through construction of a five-foot structural addition to the existing spillway on the Upper Dam. The rationale behind the five-foot raising can be traced back to the original design of the Upper Dam and Reservoir. In planning for the future and recognizing the fact that the present water supply storage capacity of the reservoir (9.1 billion gallons) would one day be inadequate to satisfy the county's growing needs, the Alexandria Water Company (later reorganized as the Virginia American Water Company) designed and constructed the Upper Dam with provisions in the spillway section to allow for a maximum height increase of five feet; while still insuring the structural stability of the dam under the increased pressures.

In further pursuing the five-foot increase, FCWA, in November 1971, commissioned the consulting engineering firm of Harza Engineering to inspect the Upper Dam in order to determine its condition and to make a feasibility study of increasing the height of the dam. The draft report presenting Harza's findings was submitted to the FCWA in April 1973.

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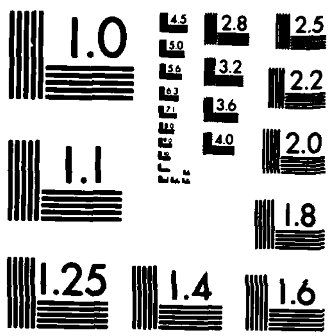


MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

Recognizing that the present storage capacity of the Occoquan Reservoir could be severely tested and could leave serious doubt as to its adequacy for providing sufficient storage in the event of a prolonged drought (1930-31, mid-60's, summer of 1977), the Fairfax County Board of Supervisors created a special water supply committee in the fall of 1977. The major work efforts of the committee were directed toward conducting a thorough investigation of the county's water supply situation and recommending viable alternatives that would be responsive to the present and projected water needs of the county. The major recommendation resulting from the committee's study was a reinforcement of the previous proposal to increase the height of the Upper Dam. The committee's recommendation was further echoed by the Virginia State Water Control Board and the State Water Supply Commission in their joint report published in 1977.

Therefore, after several years of studies FCWA in January 1978 authorized the Harza Engineering Company to review and update the 1973 report. In the revised document, special emphasis was placed on presenting a detailed engineering study, complete with designs and cost data on the effect of raising the spillway structure and subsequent pool level elevation by five feet and of its gate operations on water levels both upstream and downstream of the dam. Subsequent to this, in July 1978, FCWA requested Harza, as part of the same study, to prepare a brief evaluation complete with engineering designs and cost on the effect of raising the spillway height two feet via temporary flashboards. The two-foot alternative was evaluated based on backwater curve analyses, an analysis of upstream flooding rights, preliminary economic considerations, and ease of implementation studies performed by FCWA. In December 1978, Harza forwarded its report, entitled Increase in Height of the Upper Occoquan Dam, which investigated the five-foot extension, and submitted its findings on the two-foot extension as well. In January 1979, after a thorough evaluation of the Harza findings with regard to both the five-foot and two-foot extensions, FCWA announced that as an alternative to the five-foot extension, it would proceed, after approval of the Water Authority Board, with raising the spillway structure by two feet. This decision was based on economics, ease of implementation, and existing flood easement rights owned by the Authority. In November 1980, FCWA completed the installation of a two-foot addition to the height of the Upper Occoquan Dam.

Project Description

The Occoquan Reservoir system consists of an upper and lower dam. Both structures are located on the Occoquan River just upstream of the Town of Occoquan, Virginia. The Lower Dam, constructed in 1950, is a concrete, gravity-type structure approximately 30 feet high and 436 feet wide. The spillway is ogee-shaped and 387 feet wide, with a crest elevation of 52 feet mean sea level (msl). This dam impounds a relatively small reservoir containing about 55 millions gallons of water (168.8 acre-feet). A 350-kilowatt (kw) hydroelectric generator also is in use below this dam.

The Upper Dam, located approximately 3,000 feet upstream from the Lower Dam, was constructed and placed in service by the Alexandria Water Company (now Virginia American Water Company) in 1957. It is a concrete, gravity-type structure with a height of 65 feet and an overall width of 735 feet. The dam has a maximum height of 70 feet above the foundation except for the intake structure, which is at elevation 130 feet msl or 80 feet above the foundation. The impoundment controls a drainage area of 570 square miles. At its normal conservation pool elevation of 122 feet msl which includes the two-foot addition, the reservoir, as presently operated, has a total storage capacity of 11.1 billion gallons of water (33,900 acre-feet) with 10.3 billion gallons (31,600 acre-feet) allocated for water supply. There is no incidental flood control storage provided for by the Upper Dam. The normal conservation pool creates a water surface area covering 1,900 acres. FCWA owns flooding rights up to elevation 130 feet msl. The mainstem of the reservoir extends about 16 miles upstream along the Occoquan River to Lake Jackson. A branch stem extends about six miles upstream along Bull Run to the Southern Railroad (refer to Figure F-21 for a geographic location of the system).

The raw water intake structure is located on the north end of the dam, 80 feet above the foundation. This structure serves raw water pipelines that lead to the water treatment plant located less than a mile downstream from the dam. In 1966, three additional intakes were installed in the south or non-overflow section of the dam.

A small hydroelectric facility with two 500 kw generating units adjoins the intake structure on the downstream side of the dam. This powerhouse is reported capable of generating 17,500 kilowatt-hours per day. According to FCWA's reservoir management plan, power is generated only when there is surplus water available. Figure F-21 shows the location of these existing reservoirs.

The spillway structure for the Upper Dam is 523 feet long, has an ogee shape, acts in a free overflow manner, and has a crest elevation of 122 feet msl, including the two-foot extension. It should be noted here that in 1972, Tropical Storm Agnes produced flood flows that overtopped the spillway and all of the non-overflow portions of the dam for the full 730 feet of its length and more. Since the reservoir is used for both water supply and hydropower generation, the outlet works relate primarily to these functions. The combined maximum one-day treatment capacity of the two Occoquan Water Treatment Plants (Lorton and Occoquan) is approximately 112 mgd. On a maximum 30- and 7-day operation, the combined rated capacities of the plants are 84 mgd and 95 mgd, respectively. In addition, when the hydroelectric generators are in full operation, approximately 150 mgd passes through them. Two 24-inch gate valves and one 36-inch gate valve are available to pass water through the dam bypassing the generators. The safe yield of the reservoirs at its present operating pool elevation of 122 feet msl is 67.5 mgd. Table F-17 presents the pertinent data for the Upper Dam.

Spillway Extension

As mentioned before, FCWA released a detailed report in December 1978, entitled Increase in Height of Upper Occoquan Dam, which investigated the feasibility of raising the then conservation pool level elevation of the reservoir from 120 feet msl to 125 feet msl. Since that report, FCWA has raised the spillway two feet.

A remaining option would be to raise the spillway three more feet to achieve the full five foot addition originally considered. While the FCWA now considers this option to be undesirable (primarily due to the additional flooding easements which would be required), the three-foot addition was carried through the formulation process as a potential site for future water supply storage.

In the 1978 report, five alternatives were considered, and all included the installation of gates. The alternative which involved installation of eight tainter gates, 5 feet high and 62 feet long, was recommended for implementation based on construction cost alone. This alternative, (Alternative 2 in the above-mentioned report) would not change the crest elevation and would require structural modifications necessary for installation of

FIGURE F-21

LOCATION OF THE OCCOQUAN DAM

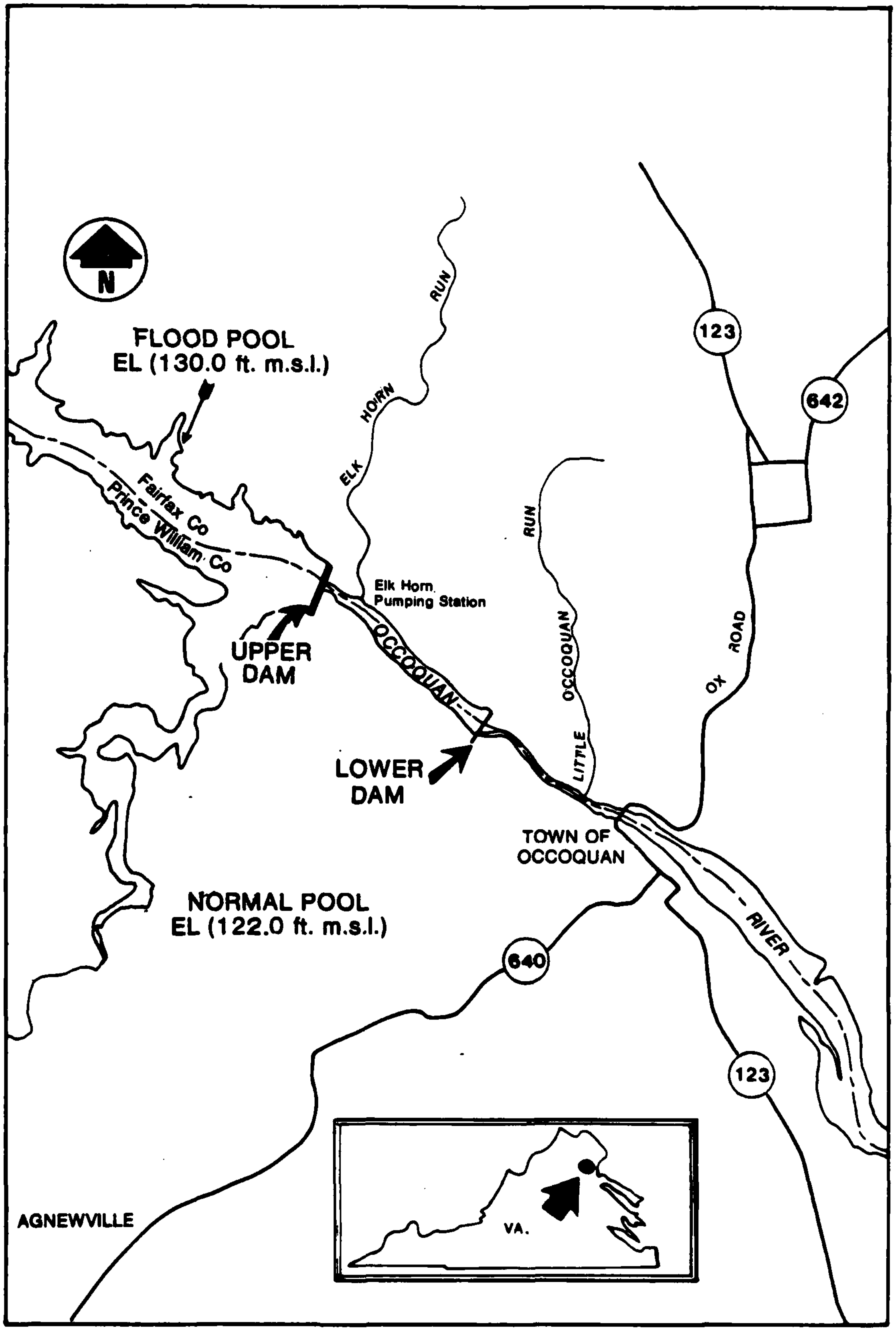


TABLE F-17

PERTINENT DATA FOR THE
UPPER OCCOQUAN DAM AND RESERVOIR

Bottom of Dam Elevation, (feet msl)	50.0
Normal Conservation Pool Elevation, (feet msl)	122.0
Total Storage at Normal Conservation Pool Level (billion gallons/acre-feet)	11.1/33,900
Usable Storage at Normal Conservation Pool Level (billion gallons/acre-feet)	10.3/31,600
Conservation Storage (billion gallons/acre-feet)	10.3/31,600
Flood Control Storage above Normal Conservation Pool level (billion gallons/acre-feet)	0/0
Buffer and Inactive Storage (billion gallons/acre-feet)	0.76/2,300
Drainage Area to Reservoir (square miles)	570.0
Normal Conservation Pool Surface area, (acres)	1,900.0
Flood Control Pool Surface Area, (square miles)	Flood easement rights up to elevation 130 feet msl at the dam and to slightly varying higher levels upstream
Safe Yield, Normal Conservation Pool Elevation, (mgd)	67.5

the gates. This alternative would increase the water supply storage capacity of the reservoir 12.5 billion gallons and the safe yield to 80 mgd. Table F-18 presents the changes in the project data with the three-foot extension.

Project Cost

The estimated cost of providing tainter gates to increase the conservation pool elevation by three feet would be $1,981,000 (October 1981 prices). This reflects updating the cost estimated by the FCWA consulting engineer ($1,547,000, December 1978 price level) using the ENR Construction Index. This estimate does not include the costs associated with the acquisition of additional land for flooding easements.

Environmental Impacts

The impact of raising the impoundment level of the reservoir by three feet would have varying effects on the ecology of the reservoir and surrounding areas. Loss of wildlife habitat would be marginal. Vegetated fringe areas inundated by water would result in a temporary loss of nesting and borrowing habitat, since the affected terrestrial species would relocate to higher elevations, thereby mitigating any long-term impacts. Enhancement of the aquatic habitat in the reservoir might result from increasing the pool size and depth, both in terms of submerged areas and total shoreline.

The Occoquan watershed (including Cedar Run) presently has more than 50 percent of the main streams encompassed in the existing or authorized impoundment. Flooding of a portion of the remaining headwater streams, particularly in the Bull Run watershed, would result in the alteration of typical headwater habitat and would reduce the amount of free-flowing stream habitat required by a wide range of fish species and benthic organisms for propagation. However, no significant or sensitive habitat would be affected.

Socio-Economic Concerns

At present, there are two regional parks (Bull Run Marina and Fountainhead) that offer a wide variety of water-oriented activities at the Occoquan. The raising of the Occoquan's normal pool three feet would have a minimal impact on these activities and would result in an overall positive contribution, due to the increase in the surface area of the lake and an increase in the amount of shoreline available for recreational use.

Land use within the Occoquan Basin is variable, ranging from densely populated residential areas to open parks and woodlands. Most of the land immediately adjacent to the reservoir is relatively undeveloped and used primarily for public parks, open space, and private recreational areas. Some additional areas along the 100-year floodplain of Bull Run in Prince William County and Fairfax County is restricted in its use. Activities at these areas should only be temporarily affected by an increase in the reservoir level; however, some acreage would be lost permanently to the new reservoir levels.

TABLE F-18

COMPARISON OF PERTINENT DATA FOR THE
OCCOQUAN DAM AND EXTENSION

Item	Existing	Five-Foot Extension
Bottom of Dam Elevation, (feet msl)	50.0	50.0
Normal Conservation Pool Elevation, (feet msl)	122.0*	125.0
Total Storage at Normal Conservation Pool Level (billion gallons/acre-feet)	11.1/33.900	13.2/40,500
Usable Storage at Normal Conservation Pool Level (billion gallons/acre-feet)	10.3/31,600	12.5/38,400
Conservation Storage (billion gallons/acre-feet)	10.3/31,600	12.5/38,400
Buffer and Inactive Storage (billion gallons/acre-feet)	0.76/2,300	0.76/2,300
Drainage Area to Reservoir (square miles)	570.0	570.0
Normal Conservation Pool Surface Area, (acres)	1,900.0	2,200
Flood Control Pool Surface Area, (acre)	Flood easement rights up to elevation 130.0 feet msl at the dam and to slightly varying higher levels upstream	
Safe Yield, mgd	67.5	80.0

*Elevation of Conservation Pool as a result of the recent change to the project.

Twenty-two archaeological sites were located at the Occoquan Reservoir on broad terraces. This indicates a high potential exists for additional archaeological sites to be present along the stream valleys and shorelines of the existing reservoir.

Again, it is important to note that the FCWA considers the implementation of the three-foot addition at the Occoquan Dam to be highly unlikely because of the probable difficulty in acquiring the necessary flooding easements (see letter from the FCWA regarding this matter in Annex C-VIII-Background Correspondence).

Cedar Run Dam and Reservoir

In August 1965, Prince William County commissioned the consulting engineering firm of Wiley and Wilson to prepare a comprehensive water supply report that would identify potential reservoir sites capable of providing sufficient storage capacity to satisfy the projected water needs of the entire county through the year 2000. During the course of the study, several reservoir sites were investigated. Based on a comparative analysis of field cross-sections, preliminary designs of dams and spillway lengths, and geological investigations of each site, it was concluded that the Cedar Run Dam and Reservoir project located at Brentsville, Virginia, was the most feasible source of water supply for meeting the county's future needs.

The 1965 report was then updated both in 1968 and 1976, reflecting changes in projected water supply requirements and project costs. With these revisions, the Cedar Run project was still recommended as the most feasible water supply source for Prince William County.

In June 1977, Prince William County pursued the Cedar Run project to the point of submitting an application for a Department of the Army permit pursuant to Section 404 of the Federal Water Pollution Act Amendments of 1972 (P.L. 92-500).

On 26 September 1978, the Baltimore District Corps of Engineers informed Prince William County that the issuance of a Department of the Army permit would not be possible at that time for the following reasons: (1) lack of sufficient information as required by the permit application; and (2) objections to the project from the United States Marine Corps Quantico Marine Base, which would be partially inundated in some areas if the project were constructed. It was also stated that should the Marine Corps express its approval of this project, Prince William County could resubmit its permit application to the Baltimore District.

Other water supply projects which have been examined and rejected by the County as alternative to the Cedar Run project include impounding other streams, development of groundwater resources, construction of Soil Conservation Service multi-purpose water supply and flood control impoundments, and use of the Potomac Estuary waters.

Project Description

The Cedar Run Project would be located about 1,200 feet upstream of the confluence of Broad Run and Cedar Run, less than five miles above the Occoquan Reservoir in Fairfax County, Virginia. The project would be in Prince William County, Virginia, near the Community of Brentsville. Figure F-22 shows the location of the project. The dam would be a clay-core, earthfill structure with rockfill ballast and a separate spillway. The dam would be 40 feet high and 800 feet long. The top of dam elevation would be 190 feet msl and the project would control a drainage area of 197 square miles.

The reservoir at its normal conservation pool level elevation of 180 feet msl would have a total storage capacity of 8.96 billion gallons of water (27,500 acre-feet) with 8.22 billion gallons (25,222 acre-feet) allocated for water supply. Incidental to the water supply storage, the reservoir would also provide storage for approximately 3.5 billion gallons of water (10,739 acre-feet) for flood control. At the normal conservation pool level, 2,550 acres of water surface area would be created while a surface area of 3,240 acres would be created at the maximum flood pool elevation of 187 feet msl.

The design of the principal spillway structure was based on stream gage records for a stream gage located on Bull Run near the City of Manassas. Review of this stream gage records indicates that a spillway width of 450 to 500 feet would be required.

Although the outlet works required to handle non-flood flows have not been designed, preliminary analyses have indicated that these structures must incorporate fish handling and protection devices. In addition, it must be capable of providing for a minimum release of 20 cubic feet per second (12.9 million gallons per day).

The project's safe yield, based on a maximum flood pool elevation of 187.0 feet msl and a normal conservation pool elevation of 180 feet msl, is estimated to be 44.5 mgd; however, the additional yield to the Occoquan system is only 25 mgd. Higher yields would have detrimental effects on the water supply capability of FCWA's Occoquan Reservoir. Table F-19 presents the pertinent project data.

Project Cost

Table F-20 presents a breakdown of the construction cost as available from the firm of Wiley and Wilson, who performed the original work. Real estate costs were developed by the Corps of Engineers and are presented in October 1981 dollars. All construction costs have been updated to reflect October 1981 dollars using the ENR indices.

Environmental Impacts

This project site is located on Cedar Run, several hundred feet above the confluence of Cedar Run and Broad Run in Prince William County, Virginia. The impoundment area would inundate approximately 4,030 acres of land at the normal pool level. Approximately 53 miles of shoreline would be created along with a storage capacity of 8.9 billion gallons of water.

FIGURE F-22

PROJECT SITE LOCATION FOR THE CEDAR RUN DAM AND RESERVOIR

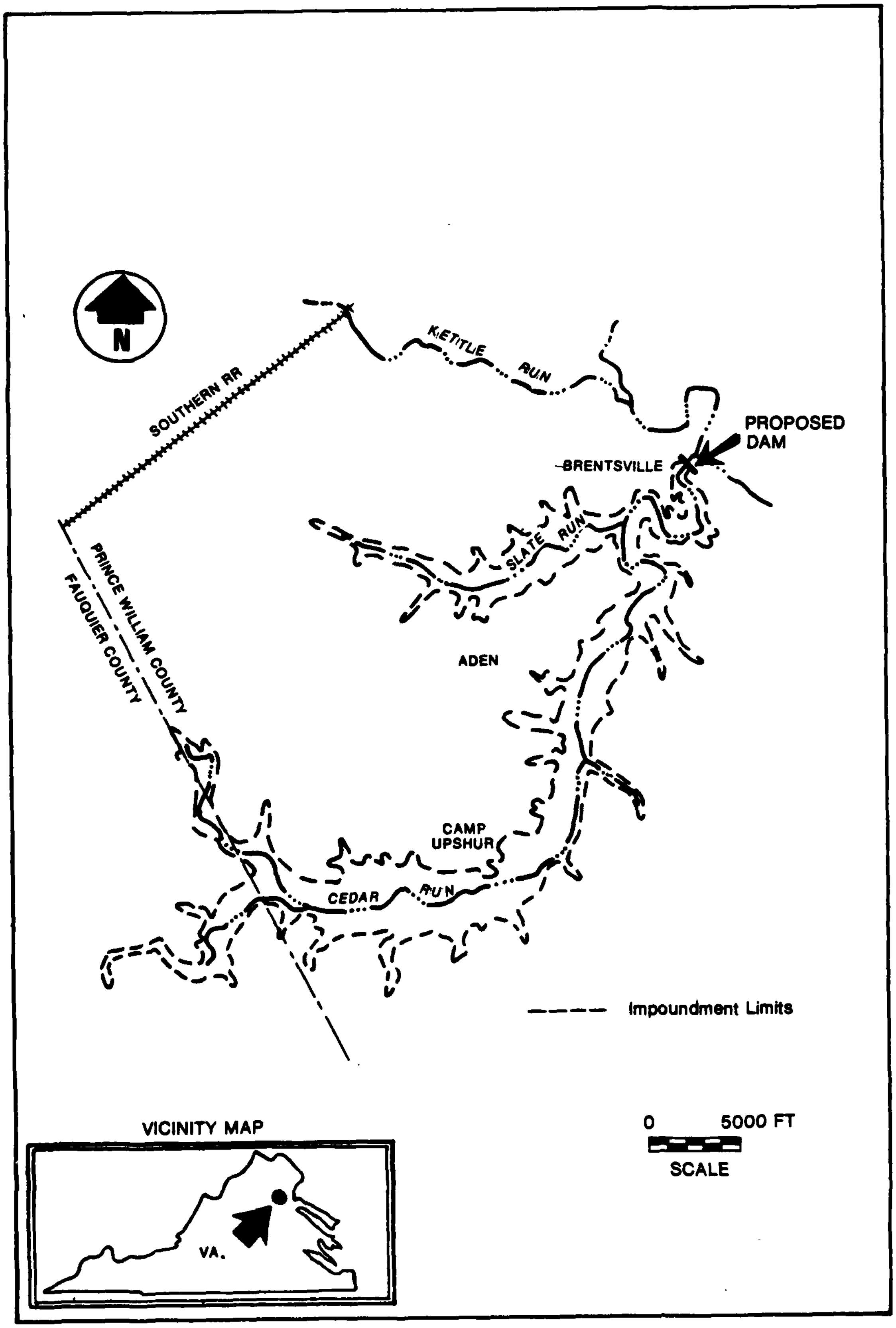


TABLE F-19

PERTINENT DATA FOR CEDAR RUN DAM AND RESERVOIR

Bottom of Dam Elevation, (feet msl)	148.0
Normal Conservation Pool Elevation, (feet msl)	180.0
Total Storage at Normal Conservation Pool Level (billion gallons/acre-feet)	8.96/27,500
Usable Storage at Normal Conservation Pool Level (billion gallons/acre-feet)	8.22/25,237
Flood Control Storage above Normal Conservation Pool level (billion gallons/acre-feet)	3.50/10,750
Buffer and Inactive Storage (billion gallons/acre-feet)	0.73/2,227
Drainage Area to Reservoir (square miles)	197.0
Normal Conservation Pool Elevation surface area, (acres)	2,550
Flood Control Pool Elevation surface area, (acres)	3,240
Safe yield, (mgd)*	25.0

* Represents yield to the Occoquan system, without detrimental effects to downstream reservoirs.

TABLE F-20

PROJECT CONSTRUCTION COSTS FOR
CEDAR RUN DAM AND RESERVOIR
(October 1981 Prices)

Capital Costs	
Excavation	$ 378,000
Concrete	3,968,000
Gates, Piping, Controls, etc.	1,536,000
Subtotal	$5,882,000
30% Engineering and Contingencies	1,765,000
Total	$7,647,000
Land Costs	
Land	$15,360,000
Improvements	3,840,000
Severance	1,280,000
Subtotal	$20,480,000
Relocation (Residences and Businesses)	2,560,000
Total	$23,040,000

The two principal terrestrial ecosystems at the site of the proposed impoundment are agricultural and upland forest. Areas under cultivation comprise about 1,880 acres. Corn, wheat, barley, mixed hay, oats, and soy beans, and pasture grasses are important open farm habitats supporting quail, deer, Canada geese, rabbit, red fox, woodchucks, dove, and pheasant. There also exists non-game species consisting of blackbirds, meadowlarks, sparrows, finches, and a wide variety of small mammals, reptiles, and amphibians. According to the Virginia Commission of Game and Inland Fisheries, the Cedar Run drainage area contains no endangered mammals, birds, reptiles, or amphibians.

Forestland comprises approximately 2,170 acres. Riparian species, including ash, red birch, flowering dogwood, slippery elm, sycamore, ironwood, oak, pine, black walnut, hickory, and black locust, are the predominant species.

Two wildlife areas to be affected by the proposed project include 741 acres of the Quantico Marine Base and the Merrimac Game Farm. The Quantico area is heavily utilized for hunting and accounts for 75 percent of the woodcock, 15 percent of the turkey, 5 to 7 percent of the deer, and 5 percent of the remaining small game harvest on the 2,000-acre base. At flood stage, this would mean the loss of 375 acres of game farm habitat resulting in a reduction of the harvest.

Cedar Run and its tributaries are typical of the shallow and warm-water streams found in the Virginia Piedmont. The streams support good populations of largemouth bass, bluegill, sunfish, and yellow bullheads, as well as wide varieties of non-game fish. The water quality of the streams appear to be quite good, as indicated by a diverse population of mayflies, stoneflies, and caddieflies. An extensive population of benthic organisms, including a variety of snails, muscles, and crayfish, inhabit the streams.

Under flood pool conditions, substantial portions of eight warm-water streams would be replaced with 2,550 acres of a warm-water lake. The changes that would take place in the conversion from a lotic (running water) ecosystem to a lentic (standing water) ecosystem may be summarized as follows. First, there would be a marked decrease in current velocity. This could affect the species composition of the community. Tolerant species such as smallmouth bass, crappies, and sunfish, would increase, whereas less tolerant stream species such as the darter would decrease. Secondly, the land-water interface of the lake would be much less than that for the stream due to the greater depth and cross-sectional area of the lake. Consequently, the reservoir would be more of a closed ecosystem in comparison to the stream. Lastly, the oxygen would be more variable within the reservoir than in the stream. Oxygen supply is generally not a problem in streams due to their shallowness, constant motion, and contact with the air. Considering these three points, it would be safe to predict that the quality of fishing would change from that presently in the stream.

The fishery downstream of the dam would also be expected to change. The decrease of flow in Cedar Run below the dam would alter the depth, velocity, turbulence, and hydraulic cross sections in the downstream channel. This may consequently affect temperatures, turbidity, and dissolved oxygen. These factors may affect the ability of some aquatic species to survive, especially immediately below the dam. The use of selected draw-off levels and aeration measures could maintain adequate dissolved oxygen concentrations and temperatures.

Encroachment of streambank vegetation resulting from the reduction in average annual flows might benefit wildlife species by providing additional coverage and forage. Degradation of the free-flowing stream and the downstream Occoquan Reservoir should be decreased due to sediment entrapment within the Cedar Run impoundment. An additional benefit might accrue from a continual release of water which could dilute treatment plant effluent concentrations from the Bull Run watershed, a tributary to the Occoquan.

Socio-Economic Concerns

The direct social and economic impacts which might result from the proposed project can be generally associated with present land use practices in the affected area. It is anticipated that the area surrounding the Cedar Run impoundment would retain its rural character for many years; however, the Prince William County Board of Supervisors has recently indicated that some farmland in the vicinity is being converted into residential subdivisions.

Perhaps a more significant impact would be the relocation of land owners, transportation, and utility systems as a result of the project. One hundred and thirty-eight landowners, (99 properties in Prince William County and 39 properties in Fauquier County) would be affected. Seven transportation routes would require relocation including two secondary routes and five unimproved routes. These actions might temporarily disrupt local traffic flow in the area. Since no major roadways pass through the project area, there would be no major disruption of regional transportation flows.

In addition, a preliminary archaeologic appraisal of the region has resulted in the recognition that a high potential for archaeologic and historic resources exists in the Cedar Run area. Fifteen prehistoric sites were revealed through consulting with local artifact collectors. In addition, 13 historical sites and properties were identified.

On the positive side, a new source of water for Prince William County would benefit not only county residents, but the MWA as well, as more Occoquan water could be made available to the rapidly growing areas of the region, particularly Fairfax County. Although no master plan for recreation has been developed, the local authorities have indicated their plans for construction of recreational facilities at the Cedar Run site. It should be noted that even though the Virginia Commission on Outdoor Recreation would be the lead agency in developing the recreational potential of this site, there might be implementation problems in gaining the agreement of all jurisdictions to build the reservoir. Fauquier County, the Federal government, and Prince William County's interest would need to be taken into account as their lands would be impounded. A period of time would be needed for acquisition by the county of those lands belonging to the Marine Corps, assuming that the Marine Corps, as well as the legislative representatives in the area, would agree to this acquisition. A bi-county agreement could be negotiated between Fauquier County and Prince William County to provide them with storage potential that would ameliorate the taking of their land for the reservoir. Recreational benefits could also help to offset the loss to the county.

SITES FROM THE BLACK AND VEATCH REPORT

In 1974, the three Potomac River Water users, WSSC, FCWA, and WAD, retained the consulting firm of Black and Veatch to investigate and develop alternative means of providing additional water supplies adequate for the growing needs of the MWA, including the feasibility of interconnecting the three service areas.

The scope of these investigations was limited to an assessment of the water requirements and potential supply deficiencies of the three service areas. The results of these investigations are presented in the report, Water Supply Storage for Washington Metropolitan Area, Black and Veatch, April 1974.

The purpose of this section is to summarize the data pertaining to the tributary reservoir sites investigated by the above-mentioned report. This will include the location, site description, yield, cost estimate, and environmental impacts of each reservoir site.

The Black and Veatch report indicated that there were substantial number of available reservoir sites within 40 miles of Washington, D.C. that were capable of providing sufficient raw water storage to augment low river flow. For selection of these sites, the report used several planning criteria to include: (a) Does sufficient storage capacity exist to satisfy potential deficits?; (b) Is the proposed reservoir site relatively close to the MWA so that the water can reach water intakes in a reasonable length of time?; and (c) would the proposed reservoir site disrupt any existing transportation, development, or power transmission facilities to an unacceptable degree?

Initially, 50 potential reservoir sites were considered. Using the planning criteria mentioned above, 29 sites were dropped from further analysis. The 21 remaining sites are shown on Figure F-23 and Table F-21 presents the pertinent data on these sites. The costs of these projects have been updated to reflect October 1981 prices.

The reservoir sites for the Black and Veatch report were sized three different ways: 1985 conditions, maximum-sized 1985 conditions, and full utilization of storage. For the purposes of this study, only the full utilization of storage was considered.

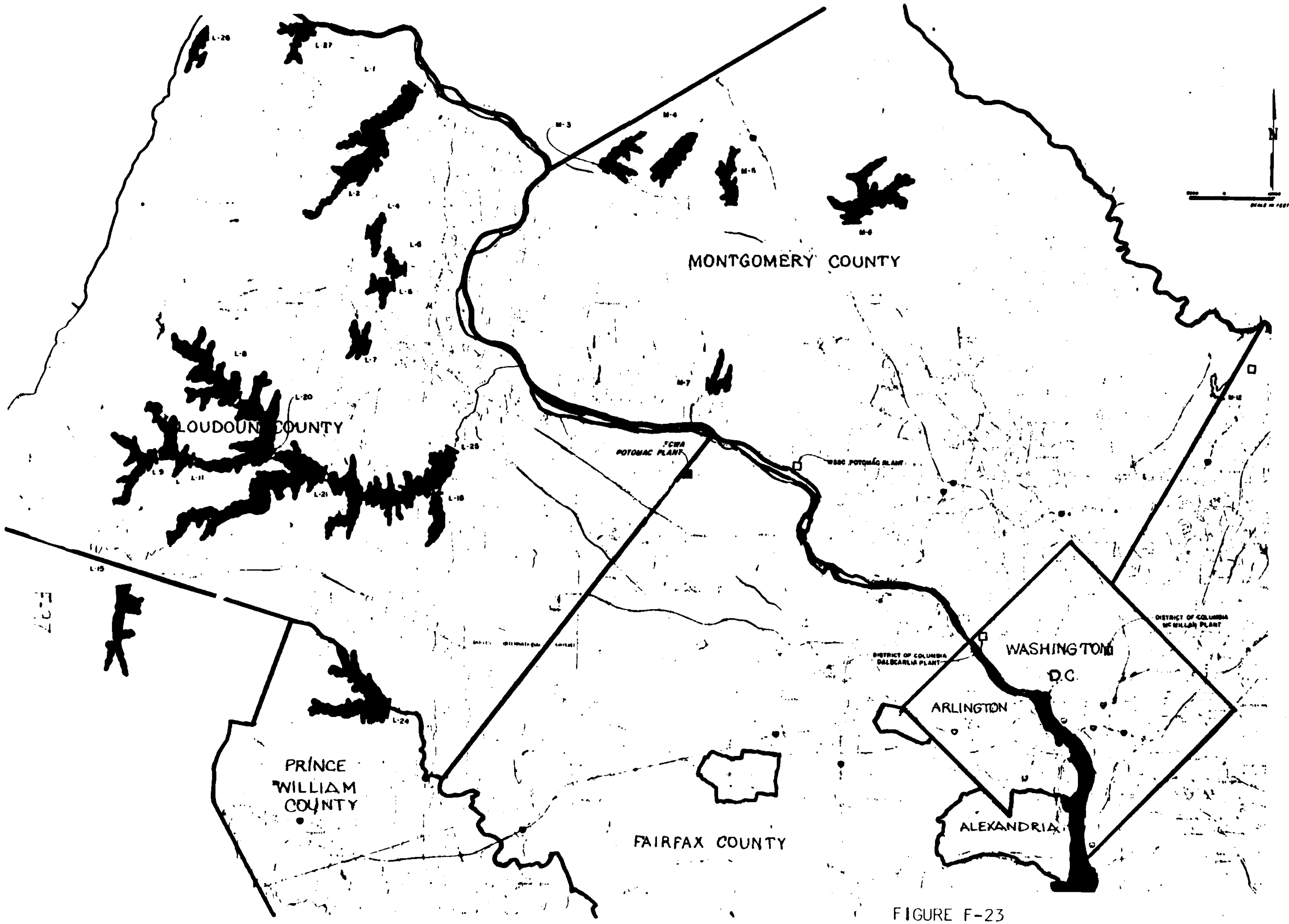


FIGURE F-23

LOCATION OF SITES SELECTED IN THE BLACK AND VEATCH REPORT

TABLE F-21

PERTINENT DATA FOR SITES IN THE BLACK AND VEATCH REPORT

Site	Influent Stream	Drainage Area Sq. Mi.	Top of Dam Elevation Feet, msl	Maximum Pool Elevation Feet, msl	Spillway Crest Elevation Feet, msl	Net Storage MG	Dead Storage MG	Annual Evaporation MG	Annual Gross Storage MG	Dependable Runoff MG	Required Annual Pumping Capability MG	Yield mgd	Cost* $x10^6	Cost per mgd $x10^6
L-1	Catoctin Creek	91.5	364	359	340	12,400	1,375	293	14,068	4,188	9,880	50	61.54	1.23
L-4	Tributary of South Fork Catoctin Creek	1.8	532	527	520	1,210	265	32	1,507	84	1,423	4	25.31	6.33
L-5	Limestone Branch	1.2	502	497	491	1,710	316	26	2,052	53	1,999	4	23.94	5.99
L-6	South Fork, Limestone Branch	2.3	513	508	499	3,100	480	49	3,629	104	3,525	9	26.60	2.96
L-7	Tuscarora Creek	4.5	484	479	470	1,650	320	71	2,041	204	1,937	7	44.68	6.38
L-8	North Fork, Goose Creek	23.6	424	419	405	9,630	1,070	42	10,742	1,080	9,662	31	94.09	3.03
L-9	Beaverdam Creek	22.4	433	428	416	7,790	910	142	8,842	1,025	7,817	24	81.58	3.40
L-11	Beaverdam Creek	38.4	402	397	385	7,140	860	164	8,164	1,757	6,404	21	76.00	3.62
L-15	Cromwell Run	18.1	442	437	425	5,030	670	102	5,802	908	4,894	18	66.82	3.71
L-18	Beaverdam Creek	5.9	313	308	299	1,890	350	76	2,316	268	2,048	7	39.44	5.63
L-20	North Fork Goose Creek	95.0	402	397	379	39,100	4,350	760	44,210	4,347	39,863	125	187.74	1.50
L-21	Goose Creek	268.0	399	394	370	62,100	6,900	1,260	70,260	12,265	58,995	220	295.92	1.35
L-24	Upper Bull Run	22.8	324	319	307	5,390	710	129	6,229	1,098	5,131	17.5	73.55	4.20
L-25	Goose Creek	357.0	301	296	277	5,720	740	323	6,783	16,064	0	20	114.71	5.74
L-26	Piney Run	12.4	510	505	495	2,810	450	50	3,310	590	2,720	10	26.03	2.60
L-27	Dutchman Creek	12.9	390	385	375	3,610	550	50	4,210	600	3,610	12.5	34.60	2.77
M-3	Little Monocacy River	10.8	404	399	387	1,960	360	58	2,378	567	1,811	10	22.95	2.30
M-4	Tributary of Little Monocacy River	3.3	543	538	531	10,650	1,300	221	12,171	171	12,000	32	64.33	2.01
M-5	Tenmile Creek	6.1	504	499	489	9,810	1,090	133	11,033	321	10,712	28	65.29	2.33
M-6	Hookers Branch Seneca Creek	28.1	422	417	403	4,460	620	123	5,203	1,475	3,728	20	68.90	3.45
M-7	Muddy Branch	3.0	302	297	288	1,660	320	35	2,015	143	1,872	6	42.91	7.15

*Based on October 1981 Prices.

Project Descriptions

Most of these 21 sites are located on small tributaries with insufficient flow to maintain full reservoirs during extended drought periods. These reservoirs would require pumps to withdraw water from the Potomac River during high flow periods to refill the reservoir storage. During the low flow periods, water from these reservoirs could be released to make up the flow deficiencies in the Potomac River. Due to high pumping costs, the cost per mgd of these reservoirs are very high. These reservoirs, however, could be used as a supplementary source as their releases could reach the MWA in a short period of time.

Environmental and Cultural Assessment

As mentioned before, there are many sites physically suitable for use as impoundments within the MWA. The Black and Veatch report selected 21 sites. One of the reservoirs, L-18 on Beaverdam Creek, has been constructed since publication of the Black and Veatch report. This reservoir is described in detail in Appendix J, Outlying Service Areas (Fairfax City). The following section briefly describes the environmental and cultural characteristics of the areas where the remaining 20 sites are located. The discussion is not site-specific, but rather it centers around the watershed in which the impoundment sites are located. Fifteen of the selected sites are located in Virginia, while five sites are in Maryland. Figure F-23 shows the locations of these sites. Table F-22 presents the watershed, influent stream, and acres inundated for each site.

Catoctin Creek Basin Sites

Catoctin Creek is a tributary of the Potomac River where two of the proposed sites (L-1 and L-4) are located. The headwaters arise in the Blue Ridge Mountains and form two principal streams, the North Fork and the South Fork. The stream flows northeasterly through the Piedmont Region of Loudoun County and empties into the Potomac River just north of Leesburg, Virginia. The stream has an average flow of 117 cfs (76 mgd) and displays a highly variable flow, particularly during the summer and fall months.

Presently, 22 percent of the watershed is in forest cover (mostly hardwoods), 22 percent in cropland, and 51 percent in pasture. Towns, villages, roads, and other development presently occupy approximately five percent of the watershed. Increases in urban uses are expected to be very significant over the next 10 to 15 years as urbanizing pressures from the Washington, D.C. area exert greater influence in Loudoun County.

In the watershed, a wide variety of small mammals and birds utilize the area. Game animals are common and often visible as they move between the stream and the Catoctin Mountains. Small game are also abundant in the form of raccoon, mink, muskrat, and rabbit. Game birds and waterfowl are found in significant numbers utilizing the stream and its associated habitat.

The water quality is generally good to fair. Fish inhabitants include largemouth and smallmouth bass, chubs, sunfish, catfish, and minnows. There is some recreational fishing, but it is limited by the creek's size and flow characteristics. A 1981 biological survey conducted by the Virginia State Water Control Board showed that Catoctin Creek has a productive and diverse invertebrate community.

TABLE F-22

ENVIRONMENTAL DATA FOR THE
BLACK AND VEATCH RESERVOIR SITES

Site Number	Watershed	Influent Stream	Acres Inundated
L-1	Catoctin Creek	Catoctin Creek	1,540
L-4	Catoctin Creek	Tributary of S. Fork Catoctin Creek	169
L-5	**	Limestone Branch	138
L-6	**	S. Fork Limestone Branch	260
L-7	Goose Creek	Tuscarora Creek	375
L-8	Goose Creek	N. Fork Goose Creek	1,220
L-9	Goose Creek	Beaverdam Creek	970
L-11	Goose Creek	Beaverdam Creek	870
L-15	Goose Creek	Cromwell Run	570
L-20	Goose Creek	N. Fork Goose Creek	1,880
L-21	Goose Creek	Goose Creek	2,600
L-24	Bull Run	Upper Bull Run	1,000
L-25	Goose Creek	Goose Creek	2,920
L-26	**	Piney Run	275
L-27	**	Dutchman Creek	275
M-3	Monocacy River	Little Monocacy River	330
M-4	Monocacy River	Tributary of Little Monocacy River	1,170
M-5	Seneca Creek	Tenmile Creek	690
M-6	Seneca Creek	Seneca Creek	670
M-7	Seneca Creek	Hookers Branch	185

** The influent stream empties directly into Potomac River.

The Commonwealth of Virginia has designated a 16-mile section of Catoctin Creek, from the town of Waterford to its junction with the Potomac River as a "Scenic River." This designation provides for preservation of the stream's scenic and historical attributes and limits development of the stream and its floodplain. Impoundments of any portion of the stream can be carried out only through special permission by the Virginia General Assembly. Impoundment Site L-1 is located on the main stem of Catoctin Creek less than one mile from its confluence with the Potomac River. A dam at this location would inundate most of the main stem which has received scenic river status. Site L-4 is located on an unnamed tributary of the South Fork. This section of river is not included within the area covered by the scenic river status.

The Catoctin Creek Watershed lies east of the historic community of Hillsboro and north of the historic community of Leesburg. Due to the watershed's pristine condition, the watershed contains a high potential for both prehistoric and historic resources. The areas of sensitivities include the floodplains, interfluvial terraces, higher peninsulas formed by bends in the stream where tributaries merge, hollows, and ridge and bluff constrictions within the watershed.

The potential for finding prehistoric sites, starting with Paleo-Indian through the Archaic and Woodland prehistoric cultures is high. The sensitive areas include the small hollows where there exists the possibility of prehistoric quarries, rock shelters, and camp sites as well as early historic colonial mill sites. Similarly, the Catoctin Creek tributary to Impoundment L-4 lies east of the historic community of Waterford. This impoundment consists of two small undeveloped hollows that may also contain prehistoric sites and historic mill resources.

Goose Creek Basin Sites

Nine of the proposed impoundments are located within the Goose Creek watershed. Goose Creek is the largest stream in Loudoun County and has an average discharge of 310 cfs (200 mgd) at Evergreen Mills. The main stem from the Loudoun-Fauquier County line to its confluence with the Potomac River has been designated a State Scenic River. Due to a lack of intensive development in the watershed, and more specifically along the river corridor, there are extensive areas of high scenic and natural value. The associated fish and wildlife resources in the area are also of high quality. Being designated a Scenic River by the State has bestowed protected status upon the stream and thus the development of the area is under close scrutiny. A majority of the land bordering the river is forested, with the remainder involved with agricultural-related uses.

The Goose Creek corridor harbors good wildlife populations. Large mammals include deer, bobcat, gray fox, skunk, raccoon, opossum, rabbit, and gray squirrel. Avian species present include wood duck, quail, woodcock, and turkey. Fish species include sunfish, rock bass, largemouth and smallmouth bass, dace, and darters. Water quality was rated good to fair by the Virginia State Water Control Board in 1980. Additionally, a biological survey revealed a good density and diversity of aquatic invertebrates.

Two of the proposed impoundment sites, L-21 and L-25, are on the portion of the main stem which has received State Scenic River status, and thus could prove difficult for implementation. Also, Site L-20 is located on the lower reach of the North Fork of Goose Creek which appears to have many of the quality characteristics of the main stem. Site L-8 on the North Fork, L-11 and L-9 on Beaverdam Creek, and L-15 on Cromwells Run, all of which are located further up in the Goose Creek drainage area, are basically similar in having reasonably good water quality and being bordered by a mixture of forest, cropland, pasture, and old fields. The environmental impacts of an impoundment on these tributaries would be less than on the main stem of Goose Creek, but could still involve significant habitat losses.

Site L-7 is located on the upper reach of Tuscarora Creek, which is a tributary of Goose Creek. The water quality of Tuscarora Creek was only rated as fair by the State Water Control Board in 1980. It suffers from some nutrient enrichment, and nitrate measurements have been high at some stations. It is impacted by sewage treatment plant discharges, effluent from an asphalt plant and quarry operations, and other sources from the nearby town of Leesburg. It appears that the quality of wildlife habitat along Tuscarora Creek is less than the other potential project sites in the Goose Creek watershed.

In terms of cultural resources, the Goose Creek watershed lies in the Piedmont region and provides a natural path to the Shenandoah Valley where there are several National Register sites. Because the watershed is undeveloped, and is principally forest, cropland, and pasture land, the eight proposed upstream impoundments contain high potential for both prehistoric and historic resources. Immediately, to the east of Goose Creek at Cub Run, 45 prehistoric sites were located during a 1980 survey of a 4-square mile area. The sensitive areas at Goose Creek watersheds include floodplains, interfluvial terraces, terraces at bends in the stream, where tributaries merge, hollows, and where ridge and bluff constrictions occur in the watersheds. Also, standing structures and razed mill sites in the proposed impoundment areas may be of significance.

Bull Run Basin Sites

Bull Run originates in the Bull Run Mountains near the Fauquier County border and flows in a southeasterly direction eventually entering the Occoquan Reservoir. The majority of the watershed area is still agricultural or forested; however, the entire watershed is experiencing urbanization pressure. The topography of the area is gently rolling, with isolated steep areas along streams and in the mountains at the top of the watershed. The high diversity of present land use provides habitat for many of Virginia's wildlife species. Small game species, such as rabbits and quail, are found in good numbers. White-tailed deer, turkeys, and gray and fox squirrel are found in forested and mixed open lands. Migratory and resident game birds such as mourning dove and woodcock, and numerous varieties of song and non-game birds are found in fields and forests. Migratory and resident waterfowl, principally mallards and wood ducks, may be found wintering on area impoundments and streams.

Water quality is generally good in the headwaters and upper reaches of Bull Run, with the lower reaches suffering from the effects of point and non-point pollution. The streams contain good populations of warm-water fish (largemouth bass, smallmouth bass, sunfish and bluegill) but the use of the warm-water streams by fishermen is limited by poor access and low summer flows. Site L-24 is located in the upper reach of Bull Run, which contains good water quality and aquatic resources. Much of the land is forested with agricultural-related activities predominating outside the forested areas.

The Bull Run watershed is undeveloped and is principally forest, cropland, and pasture land. The impoundment (L-24) has a high potential for both prehistoric and historic resources. The sensitive locales in the Bull Run watershed includes the floodplains, interfluvial terraces, high terrraces, points of land on the meandering streams where tributaries merge, hollows at ridges and bluffs where constrictions occur in the floodplain, and in marshes below bluffs (which may have served as kill sites for large animals). Extant millsites and farm structures are the most sensitive historic resources.

Miscellaneous Virginia Sites

This group is composed of sites on small streams which empty directly into the Potomac River. It includes Site L-26 on Piney Run, Site L-27 on Dutchman Creek, and Sites L-5 and L-6 on Limestone Branch. The land bordering the streams is mostly forested, while the land outside is devoted to agricultural uses. Water quality data is limited but due to the lack of development within the drainage areas, water quality is probably good. The terrestrial and aquatic resources are probably good.

The Piney Run and Dutchman Creek (Impoundment Sites L-26 and L-27) lie on the eastern and western slopes of Catoctin Mountain, respectively. These tributaries are hollows and may contain prehistoric sites and historic mill resources. These tributaries of the Potomac River are undeveloped and are principally forested. It should be noted that no on-site investigations were made for this study. For further investigations, however, the hollows should be checked for quarries, road shelters, and campsites, as well as for jasper outcrops at the foot of the Potomac River gap. In Loudoun County, its rural and scenic beauty has preserved many of its early industries, structures, and estates which are now regarded as dominant economic and significant historic resources.

The Limestone and South Fork Limestone Branches (Impoundment Sites L-5 and L-6) lie on the eastern slope of Catoctin Mountain. These tributaries are hollows and which may contain prehistoric sites and historic mill resources. These tributaries are undeveloped, and are principally forest and cropland. The hollows should be checked for quarries, rock shelters, campsites and/or jasper outcrops if these sites are considered further.

Seneca Creek Basin Sites

The Seneca Creek stream system is typical of the Piedmont streams in this area. The major branches are Great Seneca Creek with its headwaters near Damascus, Little Seneca Creek which runs from north to south through the middle of the watershed, and Dry Seneca Creek which rises near Poolesville. The upper tributaries are shallow, rapid-flowing streams with rocky bottoms which combine to form a broad, deep, turbid stream near Seneca's confluence with the Potomac River. The major land types are forest, cropland, pasture, and old field. Typical bottomland hardwood species include ash, willow, pin oak, red maple, and willow oak. The area contains suitable habitat for gray squirrel, rabbit, chipmunk, groundhog, skunk, opossum, mink, otter, beaver, wood duck, quail, and some turkey. There is a noticeable trend for increased urbanization in parts of the watershed.

Water quality in the watershed is generally good. The major problem appears to be high counts of coliform bacteria which frequently exceed the State standards (200 MPN/100ml). The source of this problem appears to be livestock operations. The most common fish species include creek chub, bluntnose minnow, dace, white sucker, common shiner, mottled sculpin, and fantail darter. Smallmouth bass, rock bass, sunfish and rainbow trout also occur. Little Seneca Creek and its tributaries are designated as recreational trout streams and form one of the heaviest stocked trout streams in the State. However, there is no significant trout reproduction as the summer temperatures generally exceed the tolerance level for a sustained population.

Impoundment Site M-5 is located in Tenmile Creek, which is a cool-water tributary of Little Seneca Creek. As mentioned above, this is a recreational trout stream. Dietemann, in a 1974 study of Seneca Creek, Watts Branch, and Muddy Branch for the Maryland National Capital Park and Planning Commission, reported an excellent diversity of fish in Tenmile Creek and recommended that it should be protected by parkland acquisition.

Site M-6 is located on Great Seneca Creek. It is typical of the streams in this area, and an impoundment would affect a mixture of forest, cropland and pasture.

Site M-7 is on Hookers Branch, a small stream which enters the lower reach of Seneca Creek. The land surrounding this relatively small drainage is almost entirely forested. This is reflected in the excellent water clarity which exists there.

The Tenmile Creek and Hookers Run tributaries of the Seneca Creek watershed lie in rural, undeveloped areas which are principally forest, cropland, and pastureland. Hookers Run, is a direct two miles upstream from Lowes Island in the Potomac River Valley. Lowes Island has been studied archeologically and contains prehistoric resources starting with Paleo-Indian era through the Archaic era, and Woodland prehistoric and historic resources, whereas the Seneca and Little Monacacy Rivers have not been systematically surveyed. It should be noted that no on-site investigations were made for this study. For further investigations, it is suggested that the tributaries of these two watersheds, the hollows, river terraces, and transecting historic road corridors should also be checked for prehistoric and historic resources to evaluate their potential resources and signficance.

Little Monocacy River Basin Sites

The Little Monocacy River is a tributary of the Potomac River. It runs through a mixture of forest, cropland, and pasture. There is some industry and other development along the stream, but it has reasonably good water quality. Its fish populations are likely to be composed of species such as dace, minnows, darters, and suckers. Site M-3 is located about midway up the stream above the town of Dickerson. Site M-4 is located further up toward the headwaters.

The Little Monocacy River and tributary upstream impoundments lie east of the Monocacy and Potomac River watersheds which contain numerous recorded prehistoric and historic resources. Both watersheds are primary forest, cropland, and pasture, and the watersheds contain a high potential for both prehistoric and historic resources. The most sensitive areas are the floodplains, interfluvial terraces, terraces at the bends in the streams, where tributaries meet, and hollows. Where historic roads transect the watersheds, the reach should be checked for early historic resources, if the site is considered further.

SITE PROPOSED BY FAIRFAX COUNTY WATER AUTHORITY

In a letter dated 22 February 1980, the Fairfax County Water Authority proposed the investigation of an impoundment site located on an unnamed tributary to the Potomac River in Loudoun County, Virginia (see Figure F-24). The Corps of Engineers investigated this new site using the same criteria used for the Black and Veatch report. Table F-23 presents the data for this new site.

TABLE F-23

PERTINENT DATA FOR SITE PROPOSED BY FCWA

Influent Stream	Unnamed Tributary to the Potomac River
Drainage Area (square miles)	1.85
Top of Dam Elevation (feet above msl)	410
Maximum Pool Elevation (feet above msl)	405
Spillway Crest Elevation (feet above msl)	398
Net Storage (mg)	3310
Dead Storage (mg)	583
Annual Evaporation (mg)	32
Annual Gross Storage (mg)	3925
Dependable Runoff (mg)	85
Annual Pumping Capability (mg)	3840
Yield (mgd)	12
Project Cost ($million)	27.54*
Cost per mgd ($million)	2.30*

*Based on October 1981 Prices.

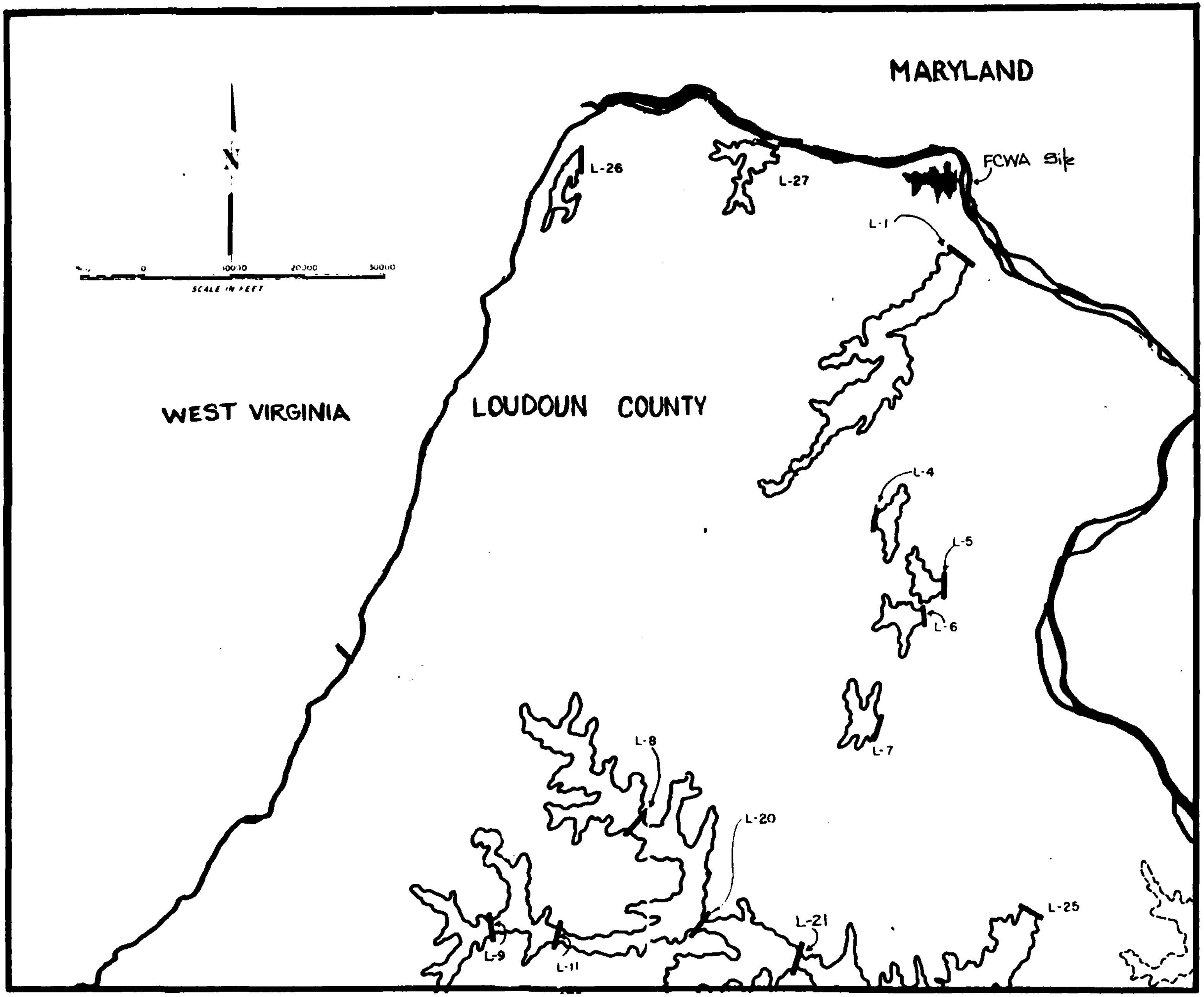


FIGURE F-24

LOCATION OF SITE PROPOSED BY FCWA

The FCWA site has similar characteristics to the sites described earlier as miscellaneous Virginia sites. Like these sites, its influent stream empties directly into the Potomac River. The land bordering the unnamed stream is mostly forested while the land adjacent is devoted to agricultural uses. Water quality data is limited, however, the lack of development within the drainage area would probably be indicative of good water quality. The terrestrial and aquatic resources are probably good.

No on-site investigations were made. From the data and information available, it was clear that the stream valley is undeveloped and principally forested. The area should be further checked for quarries, road shelters, campsites, and jasper outcrops, if the site is considered further for development.

IMPACT EVALUATION OF THE LOCAL RESERVOIR SITES

The proposed 21 sites are located within the same general geographical area and have many characteristics in common. All of the sites have reasonably good water quality. The adjacent lands usually consist of combinations of forest, pasture, old field, and cropland. No extensive wetlands appear to be associated with any of the reservoir sites. No species currently on the Department of the Interior's list of endangered species are inhabiting the project areas except for the occasional transient use by such species as the bald eagle. At the level of analysis for this assessment the similar nature of many of these streams made it difficult to determine an order of preference. However, it was possible to categorize the impoundment alternatives into three groups according to the level of impact as follows:

High Impact:	L-1, L-20, L-21, L-25
Medium Impact:	L-8, L-9, L-11, L-15, L-24, M-5, M-6
Low Impact:	FCWA site, L-4, L-5, L-6, L-7, L-26, L-27 M-3, M-4, M-7

The alternatives in the high impact group are located on designated State scenic rivers or on streams which are closely associated with them. The proposed dams in the high impact group are located on the larger streams in the area and would create the largest impoundments. The amount of loss of free-flowing stream, riparian habitat, and upland habitat, would be greatest with these sites. As these larger stream reaches are relatively uncommon compared to the number of smaller streams in the medium and low impact groups, their loss would be all the more damaging.

The reservoir sites in the medium impact group tend to be located on intermediate-sized stream reaches. The area which would be inundated at maximum storage ranges between 570 and 1,220 acres. The environmental impact of these alternatives would be substantial, but not as great as those in the high impact group.

The sites in the low impact group tend to be located on small streams. Such streams are relatively common in comparison to the larger streams and rivers. The area inundated would be relatively small, ranging from 138 acres to 375 acres (except for M-4). In most cases, the dam sites would be close to the Potomac River, so adverse effects on downstream reaches would be minimized.

This analysis did not take into account the benefits derived from impoundment construction. For example, while the larger impoundments in the high impact group would cause greater losses than the smaller impoundments, they could supply as much water as several of the smaller ones. Although fluvial and upland habitat would be lost, slack water aquatic habitat would be created. The quality of the fishery which would develop in the impoundments would vary depending on the amount and quality of inflow, surrounding land use, physical characteristics, reservoir operating procedures, etc. In some cases opportunities may exist to improve downstream fishery resources by regulating releases.

In addition to the aquatic and terrestrial ecosystem changes and socio-economic impacts common to reservoir construction, there would be significant impacts caused by the operation of this project. In particular, at those times when appreciable withdrawals would be required by the MWA, massive drawdowns would occur which would result in severe adverse impacts local to the reservoir. A rapid reduction in water volume and subsequent changes in water quality could also result in large fish kills within the impoundment. Because of these impacts and the general visual impact of an empty reservoir, the aesthetic character of the impoundment would be severely degraded during the withdrawal period. However, there is the potential for recreation development and use for these projects when they are not being used for water supply purposes. Recreation would have to be restricted during those times when the reservoirs would be drawn down or refilled.

Operation of the reservoir during non-drought periods would be based on keeping the reservoir full. This would require intermittent operation of the pumping station for short periods of time. Minimum releases based on historical flows would be guaranteed.

Impacts resulting from releases of raw water from the Potomac River into the various reservoirs would be in the form of water quality changes stemming from chemical and physical differences between the various bodies of water. The basic premise upon which impacts were assessed implies that the Potomac is generally of a lower water quality than any of the receiving water bodies in the water supply system.

In assessing the impact of altered water quality on the fishery of a given body of water, certain parameters were known to be more significant than others. One of these was Total Dissolved Solids (TDS). Based on monthly test data averages, the Potomac River possesses a TDS level three to five times higher than any of the other water supply streams and reservoirs in the MWA project area. Other test data such as pH, nutrient loads, and turbidity, also indicate, by comparison, a poorer water quality condition in the Potomac. The kind of comparison, however, is misleading as a fast-flowing stream by nature, has quite different chemical and physical characteristics than does a large reservoir. However, one could expect that the potential for reservoir eutrophication would increase with the release of water from the Potomac River into the local reservoirs. The exact interrelationship of the mixing of water from various sources within the MWA can only be determined by an in-depth analysis of existing water quality data as well as predictions of future water conditions based on trends experienced in past years.

In addition to the environmental reservations, the selection of a local reservoir for water supply has economic and other considerations. Loudoun County, Virginia, where most of these reservoirs would be located, has a history of opposing the construction of reservoirs. These reservoirs, even small ones, tend to attract new growth, and Loudoun County generally does not favor more than normal growth in the western part of the County. Further, as mentioned before, because of the substantial pumping needed for filling these reservoirs, their costs per mgd are high.

ANALYSIS OF THE LOCAL RESERVOIR SITES

The 23 potential local reservoir projects (the 20 Black and Veatch sites, Cedar Run Reservoir, raising Occoquan Reservoir and the FCWA site), formed an array of alternatives with a wide range of both economic and environmental impacts. Obviously, a tradeoff between yield, environmental impacts, and cost would be considered in the final selection of one or more of these projects for construction. However, recognizing that this tradeoff would depend heavily on the anticipated need and that this need could vary significantly in the future, a preliminary selection process was undertaken to narrow the list of reservoir alternatives without reducing the range of the tradeoffs significantly.

In the selection process, several factors were examined. First, the array of alternatives were grouped by their respective environmental impacts (Low, Medium, and High). Then, within these groupings, one or more projects were selected for further consideration, based on their yield, cost per mgd, available water supply storage, and any institutional or environmental concerns. These projects then constituted the alternatives considered in the overall MWA Water Supply Study plan formulation discussed in Appendix B, Plan Formulation, Assessment, and Evaluation.

In the group with relatively low environmental impacts, the raising of the Occoquan Reservoir spillway had the lowest cost per mgd ($0.16 million per mgd) as well as the second highest yield within the group. In addition, the Occoquan raising would have minimal environmental impacts since it does not involve construction of a new reservoir. Also, the existing recreation facilities and the spillway structure were designed to accommodate future higher pools, so extensive structural modifications would not be required. Therefore, this project was selected for additional analysis, even though there could be some significant social and institutional effects associated with the acquisition of additional land for flooding easements.

Of the remaining projects in the first impact category, Site M-4 and the FCWA site were also identified for further plan formulation. These two sites had low cost per yield values and high water supply yields for the group (32 and 12 mgd, respectively). Additionally, the FCWA site was recommended by the existing water utility for consideration in the MWA Water Supply Study. Sites M-3, L-26, L-27, and L-6 exhibited very similar characteristics to the FCWA site; however, they had slightly lower yields and somewhat higher cost per yield values. Since only one project of this size was needed for further evaluation, only the FCWA site was carried into the final MWA plan formulation. In future water supply planning efforts (outside of this study), all of these projects would deserve consideration as reservoir sites. The remaining sites in this group, L-5, L-4, L-7, and M-7, were eliminated due to their exceedingly high cost per yield values (greater than $5 million per mgd), and are generally unfavorable sites for MWA water supply development.

The medium impact group consisted of eight sites of which the proposed Cedar Run Reservoir was the least expensive. Although some institutional problems would have to be overcome prior to its construction, these were not considered unresolvable, and therefore, this project was retained for further analysis. The next lowest cost per yield site, Site M-5, was not included in the selected sites because its proposed impoundment is in the area to be inundated by the future Little Seneca Lake. Sites L-8 and L-9 located on Goose Creek tributaries, followed Site M-6 in cost per yield, with values of $3.03 million and $3.40 million per mgd, respectively. These two sites also provide an adequate amount of storage, particularly in comparison to the other sites in the medium impact group. Consequently, these two sites were designated for further evaluation. The remaining sites in this category, Sites M-6, L-11, L-15, and L-24 had no distinct advantages to offer over the previous selections, and were consequently dropped from consideration.

The last impact group, designated as "High Impact," included the reservoir site on Catoctin Creek (L-1) and three sites in the Goose Creek basin (L-20, L-21, and L-25). Since Catoctin Creek and Goose Creek have been designated as State Scenic Rivers in the areas where Sites L-1, L-21, and L-25 are located, these projects would be highly undesirable from an environmental viewpoint, and extremely difficult to finalize due to legal and social constraints. Therefore, they were not considered as alternatives for the final formulation exercise. The remaining site in this group, L-20, had a yield of 125 mgd which was more then three times the greatest yield of the six sites selected up to this point. So, in order to provide a substantial-sized reservoir site in the MWA formulation exercise, Site L-20 was included in the final selected group of reservoir sites. This site also demonstrated the significant tradeoff between high yield and considerable environmental impacts.

Table F-24 provides a summary of the local reservoir selection process undertaken for this study. The list of selected sites includes raising the Occoquan, the proposed Cedar Run Reservoir, the FCWA site, and Sites M-4, L-8, L-9, and L-20 from the Black and Veatch report. None of these projects is necessarily recommended on its own. The environmental and social effects of projects at these sites would have to be investigated in greater detail prior to individual selection. The tradeoff between these impacts and the understood supply needs would be an important consideration.

UPSTREAM RESERVOIRS

This section presents a reevaluation of the upstream reservoirs investigated and studied in previous surveys and reports. Reservoirs have been extensively studied throughout the Potomac River Basin over the years and many have been designed with a water supply purpose. Numerous sites have been identified, but in a large number of instances, years have elapsed since field surveys were undertaken. Because of the inherent utility of reservoirs to provide a means to augment the Potomac River during periods of low flows, a reevaluation of these potential Potomac River reservoir sites was required to determine their viability in terms of current social, economic, and environmental conditions.

RESERVOIRS INVESTIGATED IN THE CORPS' 1963 REPORT

Prepared in response to a 1956 Congressional Resolution, the 1963 Potomac River Basin Report was the first major attempt to assess the basin's potential to alleviate water supply shortages in the MWA, and other communities in the basin. The report recommended the treatment of wastes entering the basin's streams, as well as the construction of 418 headwater reservoirs and 16 major upstream reservoirs. The upstream reservoir list included the Bloomington Lake Project which has recently (July 1981) been operationally completed. During a later review process, two new sites, Sideling Hill and Little Cacapon, were added to the list. Figure F-25 shows the location of these 18 projects. Further details of the history of these projects are given in Appendix A, Background Information and Problem Identification.

TABLE F-24

SELECTION PROCESS FOR THE MWA LOCAL RESERVOIRS

Reservoir Site	Stream	Water Supply Storage (MG)	Additional Yield (mgd)	Environmental Impact	Total Cost ($million)	Cost per mgd ($million)	Selected for Further Plan Formulation	Reason for Selection/ Non-Selection
Occoquan Raising (3-ft) minimal	Occoquan Creek	2,200	12.5	Low	1.98	0.16	Yes	Low cost per mgd; adequate yield; minimal environmental impacts.
M-4	Little Monocacy Tributary	10,650	32	Low	64.33	2.01	Yes	Low cost per mgd; moderate amount of storage; adequate yield.
FCWA Site	Potomac Tributary	3,310	12	Low	27.54	2.30	Yes	Low cost per mgd; recommended by water utility; adequate yield.
M-3	Little Monocacy River	1,960	10	Low	22.95	2.30	No	Greater cost per yield and less storage than other low-impact alternatives.
L-26	Piney Run	2,810	10	Low	26.03	2.60	No	Greater cost per yield and less storage than other low-impact alternatives.
L-27	Dutchman Creek	3,610	12.5	Low	34.60	2.77	No	Greater cost per yield than other low-impact alternatives.
L-6	South Fork Limestone Branch	3,100	9	Low	26.60	2.96	No	Greater cost per yield and less storage than other low-impact alternatives.
L-5	Limestone Branch	1,710	4	Low	23.94	5.99	No	High cost per mgd; low yield.
L-4	Cotoctin Creek Tributary	1,210	4	Low	25.31	6.33	No	High cost per mgd; low yield.
L-7	Tuscarora Creek	1,650	7	Low	44.68	6.38	No	High cost per mgd; low yield.
M-7	Muddy Branch	1,660	6	Low	42.91	7.15	No	High cost per mgd; low yield.
Cedar Run	Cedar Run	8,220	25	Medium	23.04	0.92	Yes	Low cost per mgd; adequate yield.
M-5	Tenmile Creek	9,810	28	Medium	65.29	2.33	No	Part of future Little Seneca Lake impoundment.
L-8	North Fork Goose Creek	9,630	31	Medium	94.09	3.03	Yes	Moderate amount of storage; low cost per mgd.
L-9	Beaverdam Creek	7,790	24	Medium	81.58	3.40	Yes	Moderate amount of storage; low cost per mgd.
M-6	Hookers Branch	4,460	20	Medium	68.90	3.45	No	Greater cost per yield and less storage than other medium-impact alternatives.
L-11	Beaverdam Creek	7,140	21	Medium	76.00	3.62	No	Part of L-9 impoundment; greater cost per yield.
L-15	Cromwell Run	5,030	18	Medium	66.82	3.71	No	Greater cost per yield and less storage than other medium-impact alternatives.

TABLE F-24 (Continued)

SELECTION PROCESS FOR THE MWA LOCAL RESERVOIRS

Reservoir Site	Stream	Water Supply Storage (MG)	Additional Yield (MG)	Environmental Impact	Total Cost ($million)	Cost per mgd ($million)	Selected Further Plan Formulation	Reason for Selection/ Non-Selection
L-24	Upper Bull Run	5,390	17.5	Medium	73.55	4.20	No	Greater cost per yield than other medium-impact alternatives.
L-1	Catoctin Creek	12,400	50	High	61.54	1.23	No	Would inundate State Scenic River.
L-21	Goose Creek	62,100	220	High	295.92	1.35	No	Would inundate State Scenic River.
L-20	North Fork Goose Creek	39,100	125	High	187.74	1.50	Yes	Low cost per mgd; high yield.
L-25	Goose Creek	5,720	20	High	114.71	5.74	No	Would inundate State Scenic River; high cost per mgd.

Recently, the State of Maryland indicated interest in some of these upstream sites and requested reevaluation and review of four of the 18 sites. These sites include: Town Creek Project, Licking Creek Project, Sideling Hill Creek Project, and Tonoloway Creek Project.

The Verona and Sixes Bridge Projects were reformulated and results of the reformulation study were summarized in a May 1973 document titled, Potomac River Basin Water Supply - An Interim Report. Further, these projects were authorized for Phase I Advanced Engineering and Design (AE&D) Study by Section 85(a) of the Water Resources Development Act (Public Law 93-251) of 1974.

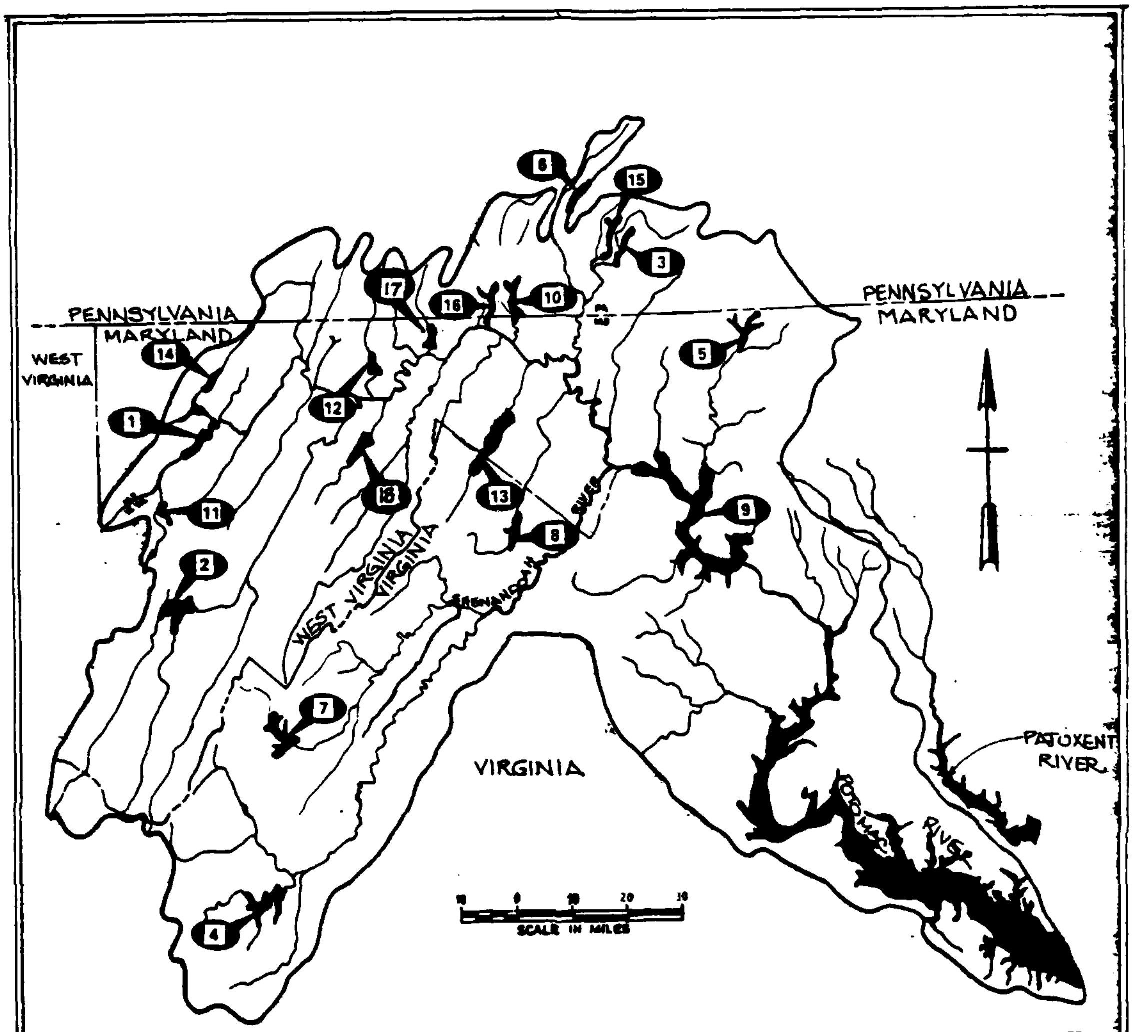
The Phase I AE&D Study for the Verona Project was initiated in August 1975 but was terminated in September 1977 when the non-Federal sponsors including the Commonwealth of Virginia withdrew their support for the project. A summary report recommending no further action was completed in December 1977.

The authorization of a Phase I AE&D Study for the Sixes Bridge Project was conditional, subject to the completion of the MWA Water Supply Study and subsequent NAS-NAE review of both the MWA Water Supply Study Report and the results of the Potomac Estuary Experimental Water Treatment Plant Testing Program (the Experimental Estuary Testing Program is described in detail in an earlier section of this appendix). No funds were appropriated for Phase I AE&D work on the Sixes Bridges Project nor was the project supported by non-Federal interests. Subsequently, the Sixes Bridge Project was deauthorized by the Congress in Section 3(h) of Public Law 97-128, 29 December 1981.

Pertinent updated information on these Potomac River Basin reservoir projects are presented in Table F-25. A discussion of the characteristics and water supply potential of each of these projects follows.

The purpose of presenting these costs (Table F-25) is to show the relative costs per mgd of each project to increase the Potomac River flow. These costs are based on the project design and cost estimates developed for the 1963 report updated to October 1981, except for the Verona and Sixes Bridge Projects whose designs are discussed below. Analysis of the existing project site conditions indicated that there had been no significant changes since 1963; therefore, the original project design and cost estimates were considered valid. Also, the tabulated Bloomington Lake Project costs are the estimated final construction costs. In updating the project costs the ENR Record Construction Cost Index for October 1981 was used.

A reformulation of the Sixes Bridge and Verona Projects in 1972 and subsequent Phase I AE&D work for the Verona Project resulted in project storage reallocation. For the Verona Project, the 104,000 acre-feet of low flow augmentation (LFA) storage was divided into 56,000 acre-feet for water supply and 48,000 acre-feet for recreation. This reallocated storage increased the project yield to 110 mgd. The 65,000 acre-feet LFA storage for Sixes Bridge Project was divided into 39,000 acre-feet for water supply and 24,000 acre-feet for recreation. The remaining 2,000 acre-feet was added to the sediment storage to make a total of 6,000 acre-feet for sediment storage. The reformulated Sixes Bridge Project increased the estimated dependable flow to 85 mgd.



POTOMAC RIVER BASIN SITES

1.	Bloomington	7.	Brocks Gap	13.	North Mountain
2.	Royal Glen	8.	Winchester	14.	Savage II
3.	Chambersburg	9.	Seneca	15.	Back Creek
4.	Verona (Staunton)	10.	Licking Creek	16.	Tonoloway Creek
5.	Sixes Bridge	11.	Mount Storm	17.	Sideling Hill Creek
6.	West Branch	12.	Town Creek	18.	Little Cacapon

FIGURE F-25

LOCATION OF THE POTOMAC RIVER BASIN RESERVOIR SITES STUDIED IN THE 1963 REPORT

TABLE F-25

POTOMAC RIVER BASIN SITES
SUMMARY OF COSTS AND PERTINENT DATA

Project	Bloomington	Royal Glen	Chambersburg	Verona (Staunton)	Sixes Bridge
River	North Branch Potomac River	South Branch Potomac River	Conococheague Creek	Middle River	Monocacy River
Purposes*	LF/FC/R	LF/FC/R	LF/R	LF/R	LF/R
Drainage Area (square miles)	263	640	161.5	325	308
Estimated increase in dependable flow (mgd)	135	312	40	110	85
Dam					
Type	Rolled earth & Rockfill	Concrete Earthwing	Concrete Earthwing	Concrete Earthwing	Concrete Earthwing
Max. Height (ft)	293	218	79	97	73
Length (ft)	2,130	6,300	2,730	1,780	2,240
Spillway	Gated	Gated	Gated	Gated	Gated
Elevation, feet above msl					
Top of dam	1,514	1,200	580	1,222	389
Conservation pool	1,466	1,160	565	1,209	375
Spillway crest	1,468	1,150	550	1,190	350
Storage (acre-feet)					
Sediment	2,700	2,000	1,000	3,500	6,000
Conservation	92,000	228,000	28,000	104,000	63,000
Flood Control	36,200	90,000	—	—	—
Surcharge	10,800	18,000	13,300	35,500	34,000
Total	148,200	338,000	42,300	143,000	103,000
Costs					
Project ($1,000)	174,300	131,202	53,179	152,616	65,836
Allocated Water Supply ($1,000)	57,868	14,563	4,413	77,071	37,790
Cost/mgd ($1,000)	428.7	46.7	110.3	700.6	444.6

Project	West Branch	Brocks Gap	Winchester	Seneca
River	West Branch Conococheague Creek	North Branch Shenandoah River	Opequon Creek	Potomac River
Purposes*	LF/R	LF/R	LF/R	LF/FC/R
Drainage Area (square miles)	77.9	213.5	120.5	11,400
Estimated increase in dependable flow (mgd)	42	95	30	900
Dam				
Type	Earthfill	Earthfill	Earthfill	Concrete
Max. Height (ft)	95	147	80	87
Length (ft)	1,630	860	1,300	2,880
Spillway	Ungated	Ungated	Ungated	Gated
Elevation, feet above msl				
Top of dam	757	1,208	557	255
Conservation pool	737	1,184	526	226
Spillway crest	737	1,184	526	200
Storage (acre-feet)				
Sediment	1,000	1,200	1,000	90,000
Conservation	45,000	120,800	28,300	460,000
Flood Control	—	—	—	460,000
Surcharge	31,500	65,000	47,700	183,000
Total	77,500	187,000	77,000	1,193,000
Costs				
Project ($1,000)	49,895	70,458	47,061	436,322
Allocated Water Supply ($1,000)	1,796	3,382	2,165	68,066
Cost/mgd ($1,000)	42.8	35.6	72.2	75.6

*LF = Low Flow Augmentation; FC = Flood Control; R = Recreation.

TABLE F-25 (Continued)

POTOMAC RIVER BASIN SITES
SUMMARY OF COSTS AND PERTINENT DATA

Project	Licking Creek	Mount Storm	Town Creek	North Mountain	Savage II	Back Creek	Tonoloway Creek	Sideling Hill Creek	Little Cacapon
River	Licking Creek	Stony River	Town Creek	Back Creek WV	Savage River	Back Creek, PA	Tonoloway Creek	Sideling Hill Creek	Little Cacapon River
Purposes*	LF/R	LF/FC/R	LF/R	LF/R	LF/R	LF/R	LF/R	LF/R	LF/R
Drainage Area (square miles)	158	49	144.5	231	48	62.3	112	104	101
Estimated increase in dependable flow (mgd)	89	32	63	89	37	18	54	61	59
Dam									
Type	Earthfill	Earthfill	Earthfill	Earthfill	Earthfill	Concrete	Earthfill	Earthfill	Earthfill
Max. Height (ft)	148	154	127	99	218	68	127	174	151
Length (ft)	1,140	2,080	1,270	1,660	1,300	490	2,350	1,270	1,370
Spillway	Ungated	Ungated	Ungated	Ungated	Ungated	Ungated	Ungated	Ungated	Ungated
Elevation, feet above msl									
Top of dam	581	2,702	739	556	1,709	571	528	620	718
Conservation pool	556	2,681	713	529	1,684	549	503	595	690
Spillway crest	556	2,681	713	529	1,684	549	503	595	690
Storage acre-feet									
Sediment	400	500	500	2,000	500	1,000	500	500	800
Conservation	82,100	27,000	57,500	95,500	39,000	19,700	50,000	54,500	53,000
Flood Control	-	4,500	-	-	-	-	-	-	-
Surcharge	38,000	11,500	38,800	97,500	10,500	26,200	37,500	20,000	28,700
Total	120,500	43,500	96,800	195,000	50,000	46,900	88,000	75,000	82,500
Costs									
Project ($1,000)	48,442	41,845	41,845	70,019	56,308	31,630	59,234	47,778	51,357
Allocated Water Supply ($1,000)	9,785	13,641	6,695	10,852	32,940	2,435	11,136	3,841	5,305
Cost/mgd ($1,000)	109.9	426.3	106.3	121.9	890.3	135.3	206.2	63.0	89.9

* LF = Low Flow Augmentation; FC = Flood Control; R = Recreation.

The project descriptions and environmental assessment information which are presented in the next sections are based on the data presented in the 1963 Potomac River Basin Report. This report included a study of the fish and wildlife resources found in the proposed project areas, which was conducted in 1962. In an attempt to evaluate the proposed reservoir sites, a survey of existing resource values was undertaken. This involved analyzing the 1962 information to see what data were still valid and what needed updating. The results of this survey and reevaluation of the data presented in 1962 report indicated that the types of terrestrial habitats, aquatic ecosystems, and water quality at most of the proposed sites had not been significantly affected since the 1963 Potomac Report was completed. Therefore, assessment of environmental impacts considered in the formulation of the projects for the 1963 report and the conclusions drawn from that assessment are still substantially valid and applicable.

Bloomington Lake Project

The recently completed Bloomington Lake Project was authorized under the Flood Control Act of 1962 for the purposes of water supply, water quality control, flood control, and recreation. The project is located on the North Branch Potomac River, 7.9 miles upstream of its confluence with the Savage River near Bloomington, Maryland. The project construction was initiated in 1971, and the project was operationally completed in July 1981.

The project controls about 263 square miles of drainage area and provides for the following storage allocation: 2,700 acre-feet for sediment, 92,000 acre-feet for low flow augmentation (LFA) and recreation, 36,200 acre-feet for flood control, 10,800 acre-feet for design surcharge, and 6,500 acre-feet to the top of the dam for a total storage of 148,200 acre-feet. It is estimated that the LFA storage has a safe yield of 135 million gallons per day (mgd). Under the authority of the MWA Water Supply Study, this project was investigated further for its water supply potential. More detailed information regarding this effort is contained in Appendix H, Bloomington Lake Reformulation Study.

The Bloomington Lake watershed is within the Allegheny Plateau physiographic province which is a high, deeply dissected plateau. The area is characterized by the presence of steep, forested slopes. The project inundates approximately 4.4 miles of free-flowing river and 952 acres in the adjacent valley. However, since acid mine drainage has degraded most of the river, there is no fishery, and thus, impacts are minimal. Additionally, in its natural state, the valley offered little or no recreation potential. With the construction of the lake, camping, boating, picnicking, and hiking facilities have been provided. The main impact of Bloomington Lake will be its positive impact on the downstream reach. Even though the acid loading to the North Branch is not expected to significantly change, the presence of Bloomington Dam in conjunction with the Savage River Reservior is expected to produce some instream water quality improvements between Luke, Maryland, and Oldtown, Maryland.

Royal Glen Project

This reservoir site is located on the South Branch of the Potomac River, just downstream of the junction of the North Fork and the South Branch, 3.6 miles due west of Petersburg in Grant County, West Virignia. The project would have a total storage capacity of 338,000 acre-feet including 228,000 acre-feet for LFA, 2,000 acre-feet for sediment, 90,000 acre-feet for flood control, and 18,000 acre-feet for surcharge storage. The 228,000 acre-feet of LFA storage is estimated to have a safe yield of 312 mgd.

The dam would form a large body of water with long arms extending up the South Branch and the North Fork. The South Branch extension would be a meandering arm into the so-called Smoke Hole area, a picturesque natural valley, rich in scenery, and containing a few small farmsteads in the more level bends of the river. The mountains are covered with extensive forests, pierced occasionally by open pastures. Among the mountains are narrow valleys with fast-flowing streams. High cliffs and tall spires of erosion-resistant rock tower above adjacent slopes. The shorelands surrounding this site are steep with continuous slopes from the proposed pool to elevations above 3,000 feet.

The surrounding area is heavily wooded. Most of the forest land has been severely cut over in the past, but some tracts of good woodland, especially in the lower Smoke Hole, still remain. Both the North Fork and South Fork are free-flowing streams with good water quality and high potential for recreation use. The South Branch which runs through the Smoke Hole is a widely known canoe stream and is one of the outstanding natural resources of the Potomac River Basin. Together with the Mononogahela National Forest, within whose boundaries the reservoir would lie, a considerable recreation development in the surrounding area could be expected.

Chambersburg Project

This project would provide water for the immediate and projected future needs of Chambersburg, Pennsylvania, and downstream communities along Conococheague Creek, as well as enhance the recreation potential of the area and provide additional water for the MWA. The reservoir site is located on Conococheague Creek in Hamilton Township, Franklin County, Pennsylvania, with the dam site about two miles due west of Marion, Pennsylvania. The project would provide a total storage capacity of 42,800 acre-feet including 28,000 acre-feet for LFA, 1,000 acre-feet for sediment, and 13,800 acre-feet for surcharge. The project's safe yield is estimated as 40 mgd.

The topography of this site is generally rolling, though some steep inclines do exist along the shores of the proposed reservoir. The majority of the land within the Chambersburg reservoir area is in dairy and livestock agriculture. Tree cover is generally limited to the stream banks and to the steeper slopes.

The water at this site is subject to contamination by municipal sewage, industrial wastes, and agriculture runoff. Restrictions on recreation use of the reservoir for the protection of health might be necessary. High rates of algal growth are possible unless inflowing wastes are diverted around the reservoir. The use of the reservoir area for recreation would be quite limited. The recreation development planned for this site included two camping sites, and a day use area (ball fields, boat launch, etc.). The reservoir would displace about 75 families living on farms and in rural residences.

Verona Project

This project would be located on Middle River, South Fork Shenandoah River, in Augusta County, Virginia, about 4.6 miles due east of Verona and about nine miles east-northeast of Staunton, Virginia. The project as proposed in the 1963 report would provide a total storage of 143,000 acre-feet including 56,000 acre-feet for water supply, 48,000 acre-feet for recreation, 35,500 acre-feet for surcharge, and 3,500 acre-feet for sediment. The project has an estimated safe yield of 110 mgd.

The topography of the Middle River reservoir site is generally rolling with some dominant topographic protrusions above the general level of the Shenandoah Valley. Slope conditions within the reservoir site vary from the vertical precipices to virtually flat-bottom lands. The steeper slopes are tree-covered, and the flatter slopes are used for agricultural purposes. Most of the land area affected by the reservoir is in agricultural use. Its loss might be a significant impact to the local economy. Forests are primarily hardwoods, although there are some extensive stands of Virginia pine on those slopes with very thin soils.

The existing recreation use of the general area is limited. The development planned for the reservoir to provide full public use of the area would consist of several camping areas, day-use areas, swimming and boating. Most of these facilities would be located on the north shore of the reservoir.

Sixes Bridge Project

The project would provide additional water for the MWA in addition to meeting local water supply needs in the Frederick, Maryland area. The project would be located on the Monocacy River, two miles due west of Keysville, Maryland. It would have a full conservation pool at elevation 375 feet above mean sea level which would extend into Adams County, Pennsylvania. The total project storage capacity would be 103,000 acre-feet which includes 39,000 acre-feet for water supply, 24,000 acre-feet for recreation, 34,000 acre-feet for surcharge, and 6,000 acre-feet for sediment. With 65,000 acre-feet of LFA storage, the project has a safe yield of approximately 85 mgd.

The watershed is a gently rolling plain with steep slopes to the west on Catoctin Mountain. The reservoir is located within one of the most productive agricultural areas of the Potomac Basin. Forest cover is extremely sparse on the site and is limited to small patches along the stream valleys and on some of the steeper slopes. Mixed hardwoods predominate the few forested areas although there is some Virginia pine regenerating on the thinner soils. The soils are fertile, though subject to severe erosion. The possibility of high turbidity in a potential reservoir is indicated.

Streams through this site are contaminated by sewage. This would seriously affect water quality in the reservoir. Deep water near the dam might be better than elsewhere in the reservoir. Weed and algal growth might be a problem in the extensive shallow areas. The recreational development planned for this site included three camping areas, two day-use areas, and water-oriented activities.

West Branch Project

The West Branch project would satisfy the water quality control and municipal and industrial water supply needs of the West Branch, contribute to water quality control in lower Conococheague Creek, enhance the recreational opportunities of the area, and furnish additional water for the MWA. The project site is located in Franklin County, Metal Township, Pennsylvania, about 11 miles due north of Mercersburg, Pennsylvania. The Town of Metal, Pennsylvania, lies within the proposed reservoir area. The total storage capacity of the project is estimated to be 77,500 acre-feet which includes 45,000 acre-feet for LFA, 31,500 acre-feet for surcharge, and 1,000 acre-feet for sediment. The project's safe yield is estimated as 42 mgd.

The West Branch is a clear, unpolluted trout stream. Reservoir water quality should be excellent as there are no significant upstream population centers or other sources of pollution. The stream flows through an attractive pastoral valley with backdrops of 1,900-foot high mountains on both sides. The west shoreline is extensively farmed and is essentially unforested, whereas the east shore is steeper, forested terrain. The topography is varied and should lend itself well to recreation use. The proposed reservoir would be a long, narrow impoundment with no major embayments. The reservoir area would cover the community of Metal, Pennsylvania, and thus the project would require relocation of the town.

Brocks Gap Project

The Brocks Gap project would provide for the immediate and projected water needs of the communities along the North Fork of the Shenandoah River and the MWA. The dam site would be located at Brocks Gap where the North Fork of the Shenandoah River has cut through Little North Mountain. Brocks Gap is in Rockingham County, Virginia. Nearly all of the reservoir area would lie within the Proclamation Boundary of the George Washington National Forest. The total storage capacity of the project would be 187,000 acre-feet with 120,800 acre-feet for LFA, 65,000 acre-feet for surcharge, and the remaining 1,200 acre-feet for sediments. The project is capable of adding 95 mgd to low flow.

The North Fork flowing through this valley is a clean, unpolluted stream draining the densely forested mountains. Bottom lands are predominantly cleared for agriculture, while the steeper hillsides are covered with mixed hardwoods and associated conifers including some almost pure stands of white and Virginia pine. Water at this site is of excellent quality and sources of pollution do not now exist upstream from the site. The social impact of this project would be severe. A farm community of about 120 families would have to be relocated. This would require the relocation of approximately 68 farms, 49 residences, 2 churches, and 1 school.

Winchester Project

The Winchester project would be located on Opequon Creek about 600 yards upstream of Virginia State Highway 761 and six miles northeast of Winchester in Frederick and Clarke Counties, Virginia. The project would meet the projected water needs of Winchester, Virginia, and would provide additional flow for the MWA. The total storage capacity of the project would be 77,000 acre-feet with 28,300 acre-feet for LFA, 47,700 acre-feet for surcharge, and 1,000 acre-feet for sediment. The project has a safe yield of 30 mgd.

The area is rolling to hilly, the eastern shore being relatively flat compared to the more hilly western side. The reservoir created by the dam would be a twisting body of water, with one major embayment up Lick Run, and would possess a somewhat ragged shoreline. Although the general topography surrounding the reservoir site is not too severe, the shorelines of the proposed impoundment would be quite steep. Forest cover is somewhat sparse, and is principally confined to the bottom lands, stream banks, and steep slopes. However, sufficient lands with forest cover could be located on which to construct initial recreation developments.

Seneca Project

This project would be located on the Potomac River at Blockhouse Point, 1.6 miles downriver from the mouth of Seneca Creek and would enhance the recreational facilities of the area. It would provide water for the projected water supply needs in the MWA, as well as flood protection for Washington, D.C.

The total storage capacity of 1,193,000 acre-feet provided by the project would be allocated as follows: 460,000 acre-feet for LFA, 460,000 acre-feet for flood control, 183,000 acre-feet for surcharge, and 90,000 acre-feet for sediment. The project has a maximum safe yield of approximately 900 mgd.

The Seneca Project is a large mainstem project. It would have major adverse environmental and social impacts on Washington D.C. and its surrounding area.

Licking Creek Project

This project would be located on Licking Creek at the Pennsylvania-Maryland state line, about 19 miles west-northwest of Hagerstown, Maryland. The project would supplement the available streamflow in the Potomac River at Washington, D.C., and enhance recreational opportunities for the Hagerstown, Maryland area. The project would provide a total storage capacity of 120,500 acre-feet which would include 82,100 acre-feet for LFA, 38,000 acre-feet for surcharge, and 400 acre-feet for sediment. The project has a safe yield of about 89 mgd.

The topography of the site is mountainous, particularly its eastern shore. Some scattered slopes are suitable for recreation developments. The stream bottoms are relatively flat, possess good soil characteristics, and are used extensively for farming. A mixed pine and hardwood forest covers the slopes surrounding the site. Much of the moderately steep land has been cleared for grazing. The streams flowing through this site are clear and apparently uncontaminated. No significant population centers or potential sources of pollution exist upstream of this site.

Mount Storm Project

The Mount Storm project would be located on the Stony River, a tributary of the North Branch Potomac River, about 0.4 miles downstream from the U.S. Route 50 highway bridge, in Grant County, West Virginia. This project would provide water to meet the projected needs of the North Branch area, provide flood protection to the North Branch Valley, and furnish additional water for the MWA. The project would provide a total storage capacity of 43,500 acre-feet which would include 27,000 acre-feet for LFA, 11,500 acre-feet for surcharge, 4,500 acre-feet for flood control, and 500 acre-feet for sediment. The project's safe yield is estimated as 32 mgd.

The Stony River watershed flows in a northerly direction through Mineral County, West Virginia, where it enters the North Branch of the Potomac River at the Mineral County-Grant County, West Virginia boundary. The adjacent lands consist primarily of forested areas with scattered intrusions of farmland.

The flora of the Mount Storm area is typical of the Mid-Appalachian region. In the low-lying valleys, beech, red maple, and yellow poplar predominate. Above this, at elevations of greater than 2,700 feet, the oak/chestnut forest is dominant. Due to the heavy mining operations conducted in the area, negative impacts have occurred to the terrestrial ecosystem. Removal of trees and shrubs has resulted in the loss of wildlife habitat. Vegetation removal contributes to the loss of topsoil due to erosion, the loss of rearing and brood cover, and the loss of food sources and escape cover from predators for many wildlife species. Any one of these losses, or a combination thereof, will result in wildlife population declines.

Wildlife populations supported by areas not impacted by mining, are varied due to the topography of the region. The high ridges offer nesting habitat for resident species of birds while the low valleys offer flyways and cover for seasonal migrants. Mammals that inhabit the area are red squirrel, red fox, opposum, gray fox, etc.

Due to coal mining operations and subsequent acid mine runoff, the water quality of the Stony River in the area of the proposed dam site is poor. The literature indicates (and recent sampling data verifies) that the river in this region is nearly devoid of aquatic life.

Town Creek Project

This project would be located on Town Creek, 5.5 miles northeast of Oldtown, Allegany County, Maryland, and 4.2 miles northwest of Paw Paw, Morgan County, West Virginia. The project would supplement the available dependable flow in the Potomac River at Washington, D.C. Total storage capacity of the project would be 96,800 acre-feet which would include 57,500 acre-feet for LFA, 38,800 acre-feet for surcharge, and 500 acre-feet for sediment. The safe yield of the project is about 63 mgd.

The topography of this watershed is mountainous. The area is characterized by three major land forms: a narrow floodplain with bench-like terraces, the former valley floor, and very steep uplands. The area is dominated by several long ridges - Green Ridge to the east and Warrior Mountain to the west. Forest cover within this site is quite extensive and composed of mixed hardwoods with a considerable amount of Virginia and white pine. Timber quality is low but adequate for recreation site cover.

Streams flowing through this site are clear and apparently free of pollutants. Water quality at this reservoir should be conducive to all forms of water-oriented recreation since there are no significant industrial or population centers upstream from the site.

North Mountain Project

The North Mountainn project would be located on Back Creek in Berkeley County, West Virginia, 2.2 miles due south of Jones Springs and 2.3 miles northeast of Shanghai, West Virginia. The project would supplement the Potomac River flow at Washington, D.C. for municipal and water quality control purposes, and would enhance the recreational opportunities in the Back Creek Valley. The project would provide a total storage capacity of 195,000 acre-feet which would be allocated as follows: 95,500 acre-feet for LFA; 97,500 acre-feet for surcharge, and 2,000 acre-feet for sediment. The safe yield of the project is estimated as 89 mgd.

The topography within the reservoir site is quite varied, ranging from flat-bottom lands to severe slopes. Mountains form a backdrop for both shores of the proposed reservoir. Generally, the land immediately above the conservation pool is rolling to very steep.

The land within the site is approximately 70 percent open and 30 percent woodland. The percentage of woodland increases as the terrain becomes more sloping. Soils within the area are a residual of alluvial soils and shale, and within the confines of the reservoir site itself the soil can be classified as poor to good. Away from the shoreline of the proposed reservoir, soil quality deteriorates and is generally devoted to woodland and grazing. Forest cover is composed of mixed pine and hardwoods. Timber is of poor quality, but it would provide adequate cover for recreation areas. Water flowing through this site is currently clear and lacking in turbidity and pollutants. The reservoir would provide excellent recreation facilities. The large lake would provide many different kinds of recreation such as camping and boating. The project would, however, displace about 60 families presently living in the reservoir area.

Savage II Project

This project would be located on the Savage River, just upstream from the existing Savage River Reservoir and about 10 miles southwest of Frostburg, in Garrett County, Maryland. The project would provide water for the needs of the North Branch Basin, enhance the recreational potential of the area, and furnish additional water to the MWA. The proposed project would provide a total storage capacity of 50,000 acre-feet including 39,000 acre-feet for LFA, 10,500 acre-feet for surcharge, and 500 acre-feet for sediment. The project has a safe yield of about 37 mgd.

The Savage River watershed lies at the headwaters of the North Branch of the Potomac River. It flows in a south-southwesterly direction in Garrett County, Maryland until it reaches the existing Savage River Dam. From this point, the river meanders in a east-southeasterly direction until it reaches its confluence with the North Branch of the Potomac River at Luke, Maryland. The lands adjacent to the Savage River and its tributaries consists of some marginal farmland, but steep forested mountain slopes are predominant.

The Savage River watershed supports a diverse group of plant communities. Those areas that would be impacted from inundation by the reservoir flood pool include forested hillsides, a forest-meadow ecotone, and bottom land/riparian areas. These areas provide excellent diversity in wildlife habitat and a variety of wildlife food sources. Additionally, the Savage River supports one of the finest quality cold-water fisheries that exist in Maryland.

Back Creek Project

The Back Creek project would be located on Back Creek, a tributary of Conococheague Creek, in Franklin County, Pennsylvania, 5.6 miles southwest of Chambersburg and about 3,500 feet downstream from the Turkeyfoot-St. Thomas Road Bridge. The project could provide the necessary supplemental flow to satisfy the projected water quality needs along Conococheague Creek, enhance recreational opportunities in the area, provide limited flood protection for the Conococheague Creek basin, and furnish additional water for the MWA. The project would provide a total storage capacity of 46,900 acre-feet including 19,700 acre-feet for LFA, 26,200 acre-feet for surcharge, and 1,000 acre-feet for sediment. The project's safe yield is approximately 18 mgd.

The reservoir would inundate mostly open farmland although some forest cover exists along the banks of the creek. The topography of the site is generally rolling with the highest bordering hills rising to 700 feet, or some 150 feet above the conservation pool.

Water quality should be relatively good; however, the expected turbidity might prove detrimental to recreation use. This reservoir would be a long, narrow body of water with several half-mile long embayments, though generally its shoreline would be smooth and suitable for development of recreation facilities. Forest cover at this site is extremely limited, and is confined to small patches on the steeper slopes and along stream valleys. The reservoir would displace approximately 40 families and remove approximately 1,500 acres of agricultural land from service.

Tonoloway Creek Project

The reservoir site is located on Tonoloway Creek in Washington County, Maryland, one mile south of the Maryland-Pennsylvania State line, and 1.7 miles east-northeast of Hancock, Maryland. The project would supplement streamflow in the Potomac River at Washington, D.C., for projected water supply and water quality needs, and enhance the recreation opportunities in the area near Hancock, Maryland. The total storage capacity provided by the project would be 88,000 acre-feet including 50,000 acre-feet for LFA, 37,500 acre-feet for surcharge, and 500 acre-feet for sediment. The project has a safe yield of about 54 mgd.

The topography of this watershed is mountainous and dissected by numerous streams. Several prominent ridges dominate the reservoir site, some reaching elevations of 900 feet. The geology of the Tonoloway Creek region is a complex assortment of paleozoic sediments which have been folded and eroded. Mixed forests clad the hilltops and severe slopes. The flatter bottomlands are farmed while many of the more moderately sloping hills have been cleared for pasture.

The streams within the site are clear and unpolluted. Since no significant sources of pollution exist upstream from this site, it is expected that the water quality in the proposed reservoir would be conducive to all forms of water-oriented recreation. The project would displace approximately 20 families living in the reservoir area.

Sideling Hill Project

This project would be located on Sideling Hill Creek in Western Maryland, about six miles west of Berkeley Springs, West Virginia, and 1.6 miles upstream from the junction of Sideling Hill Creek with the Potomac River, north of Lineburg, West Virginia. The project would supplement the available dependable flow in the Potomac River at Washington, D.C., and would enhance the recreation potential of the area between Cumberland and Hancock, Maryland. The project would provide a total storage capacity of 75,000 acre-feet including 54,500 acre-feet for LFA, 20,000 acre-feet for surcharge, and 500 acre-feet for sediment. The project's safe yield is estimated as 61 mgd.

This watershed is essentially mountainous and almost entirely forested. Sideling Hill, a long, straight ridge east of the reservoir site, rises some 1,000 feet above the proposed pool and would dominate its eastern shore. Ridges to the west of the site are not as high, but are topographically similar to Sideling Hill. The land within the proposed reservoir area is estimated to be approximately two-thirds wooded. Once out of the proposed reservoir site, the proportion of woodland increases sharply. The forests are predominantly mixed pine and hardwoods and are somewhat poor in quality. Dense pure stands of Virginia pine exist in many areas, particularly on those sites with poor soil. The water flowing through this site is clear, free of pollutants, and relatively free of silt. No significant population centers or potential pollution sources exist upstream from the site;

thus, water quality at the reservoir should be conducive to all forms of water-contact recreation.

Little Cacapon Project

The Little Cacapon project would be located on the Little Cacapon River in the Gore Magisterial District of Hampshire County, West Virginia, 3.4 miles upstream from the junction of the Little Cacapon River with the Potomac River, and four miles southwest of Paw Paw, West Virginia. The project would improve the supply of dependable flow in the Potomac River at Washington, D.C., and would enhance recreational opportunities in the area. The project would provide a storage capacity of 82,500 acre-feet which would include 53,000 acre-feet for LFA, 28,700 acre-feet for surcharge, and 800 acre-feet for sediment. The maximum safe yield of this project is estimated to be 59 mgd.

This site is steeply rolling and mostly wooded although few small areas of open land exist along the river. The forest cover is predominantly a mixture of hardwoods with several extensive stands of Virginia pine. The streams in the site are apparently clear and pollution-free. No significant population centers or potential sources of pollution exist upstream from this site, and the reservoir water quality would be expected to be conducive to all forms of recreational use.

This area is very sparsely settled and rural in nature. The project would require the relocation of 15 families, mostly living on farms, and remove 600 acres of agricultural land from production. This would not be a significant impact since agriculturally the area is considered poor and a great portion of the land is idle or in brush.

Summary of Environmental Assessment

In summary, in assessing the wildlife resources found at the various project sites, it was apparent that the quality of these resources is consistently high. This was due to several factors, most important of which is the variety of wildlife habitat and the lack of human population pressures in these areas. Game populations are high as indicated by hunting activity and hunter kill data.

Fishery resources in the upper Potomac River basin are somewhat unique and vary in type and quality dramatically from one location to another. The system is unique in that the assemblage of fish species indigenous to the area forms a transition group between northern and southern fish faunas. The fishery also varies tremendously in quality with some of the most productive trout and smallmouth bass streams found on the east coast at the present, as well as areas (North Branch) entirely devoid of aquatic life. Fisherman use of the Potomac and its tributaries is generally quite high with fishing demand increasing yearly.

The most significant change which has taken place in the Potomac River system during the past 20 years is that of water quality. The basin as a whole has probably improved slightly, at least when judged on the basis of physical and chemical parameters. There has been, however, on a localized basis, many instances of a reduction of water quality. This has resulted from point and non-point discharges which are not being regulated in accordance with best management practices. These include industrial discharges such as food processing plants and sand and gravel operations, as well as agriculturally related problems from pesticides, herbicides, and nutrient loading from inorganic fertilizers. Areas impacted most significantly by this type of pollution include portions of the

Shenandoah River and its tributaries, as well as the Conococheague, Opequon, and Monocacy Rivers.

Animal species listed as endangered or threatened in the western Maryland, western Virginia, and eastern West Virginia region includes the Least Weasel, Mustela nivalis allegensis, Eastern Cougar, Felis concolor cougar, and the Indiana Bat, Myotis sodalis. However, the cougar and the weasel have not been sighted in the area for several years. The Black Bear is considered threatened only by the State of Maryland. Endangered or threatened plant species found in this region have only been afforded state protective status and the distribution of these species has not been detailed to any degree.

None of the proposed reservoir sites are located on a designated Federal or State Wild and Scenic River. However, three of the sites (Royal Glen, Chambersburg and Sixes Bridges) are located on the Nationwide Inventory which lists rivers that have been identified as meeting the minimum criteria for further study and/or inclusion into the National Wild and Scenic Rivers System.

In terms of the cultural resources in the upstream impoundments, there appears to be a high sensitivity for archeological and historical resources. Generally, the upstream watersheds which were assessed are undeveloped, rural, and consist of principally forest, crop, or pasture lands, and are located in mountainous terrain, except for the Sixes Bridges site which lies in the rolling Piedmont terrain. These watersheds contain broad to narrow floodplains with gentle to steep slopes. Due to their general pristine conditions of water, soil, ecological, and terrace settings, these watersheds contain a high potential for containing a variety of prehistoric and historical cultural resources which are reflective of Upland Archaic, Multi-component, Quarry, Rock Shelter, Upland Woodland, Flood Plain Woodland, and Historic Mills and Farms.

The areas of cultural sensitivities include the floodplains, interfluves, upper terraces, hollows, marshes, ridges, and bluffs. Recent cultural investigations of the Verona and Sixes Bridges proposed locales showed that these undeveloped watersheds contain various levels of potentially significant prehistoric and historic cultural resources. All of these cultural sensitivities should be addressed and evaluated if any of these upstream impoundments are studied in greater detail in the future. A more detailed assessment would help as a good management practice to avoid further endangering, destroying, or causing increased stress upon the limited cultural resource bases present in these upstream watersheds.

Project Status

Of the 18 projects described, Bloomington Lake Project has already been constructed. This recently completed project provides 92,000 acre-feet of storage capacity for low flow augmentation. Based on the project benefits used for the authorization document, this storage is further allocated into water supply (41,000 acre-feet) and water quality (51,000 acre-feet) of storage. Since the project authorization in 1962, water quality needs have changed due to construction and upgrading of the basin's waste treatment facilities. This has provided a reason to evaluate and reassess the water quality and other project needs. The Bloomington Lake Reformulation Study, authorized under Water Resources Development Act of 1974, evaluated the water supply potential of the recently completed project. Appendix H presents full details of this study.

The Sixes Bridge and Verona Projects were authorized for Phase I AE&D under Public Law 93-251, Water Resources Development Act of 1974. Phase I AE&D work on the Verona Project was initiated in August 1975. In September 1977 the non-Federal sponsors including the Commonwealth of Virginia, withdrew their support and the work on Verona Project was terminated. No further action was recommended.

The Phase I AE&D work on the Sixes Bridge Project was not initiated due to the lack of local support and congressional appropriation. Recently, the Congress has deauthorized the Sixes Bridge Project under Section 3(h) of Public Law 97-128 dated 29 December 1981.

With the exception of these Phase I AE&D work and construction of the Bloomington Lake Project, no further studies or investigations were done on any of the projects in the Potomac River Basin. At present, these projects are not under active consideration and have been placed in inactive status.

RESERVOIR SITES PROPOSED BY THE U.S. FISH AND WILDLIFE SERVICE

As part of the MWA Water Supply Study, the U.S. Fish and Wildlife Service (USFWS) reevaluated and the reservoir sites proposed in the 1963 Potomac River Basin Report. USFWS's reexamination of the potential reservoir sites was based on five parameters: (1) destruction of terrestrial habitat in each proposed flood pool area; (2) aquatic ecosystem alteration in the flood pool areas; (3) aquatic habitat destruction below the dam; (4) existing water quality at each dam site in question; and (5) alternatives to the Corps of Engineers dam site proposals that would also incorporate acid mine drainage abatement, and in turn, habitat enhancement and restoration. Based on these parameters, USFWS determined that with the exception of the North Branch Potomac River, the proposed upstream reservoir sites would result in significant fish and wildlife resource losses. The reduced water quality and impacted fish and wildlife resources of the North Branch resulting from coal mining activities suggest, however, that potential sites for reservoir storage could be implemented in this region without significant resource losses. Further, this preliminary review and examination indicated that not only could potential reservoir storage be obtained in the North Branch Potomac River, but significant aquatic resource restoration and enhancement could be realized. USFWS recommended three sites for further investigation by the Corps of Engineers. These sites were Mount Storm, Laurel Run, and Abram Creek. Of these three, Mount Storm would be located in the same general area where it was proposed in the 1963 Potomac River Basin Study and was described in the previous section. Data for the other two sites, Laurel Run and Abram Creek, are presented in the following paragraphs.

Laurel Run

Laurel Run is the westernmost tributary in Maryland that enters the North Branch Potomac River. The stream flows in a northeasterly direction through the southern tip of Garrett County, Maryland, where it joins the North Branch Potomac River, one mile due east of Red Oak, Maryland. Its main tributaries include Chestnut Ridge Run and Red Oak Run. Lands adjacent to Laurel Run are marginal agricultural lands, flanked by relatively steep, forested mountain slopes.

The proposed site is immediately downstream of the point where Red Oak Run and Chestnut Run join Laurel Run. A dam at this location could provide a storage capacity of 2,680 acre-feet with a top of dam elevation of 2,700 feet above mean sea level. The proposed dam would be 1,100 feet long and more than 100 feet high. The lake behind the dam could provide a surface area of 50 acres.

The terrestrial ecosystem in the Laurel Run watershed is typically a mixed mesic hardwood forest that is indigenous to the mid-Appalachian region. Wildlife species that utilize these forested areas are of the same species that inhabit the Stony River area in which the Mount Storm site is located. Physiographic characteristics are also similar to those found in other segments of the North Branch Potomac River region.

Biological and water quality sampling of the Laurel Run indicates that the stream is of poor quality. This is mainly due to the acid mine runoff.

Laurel Run has a drainage area of less than a square mile (503 acres) including its two main tributaries and would not be able to provide significant flow which could be stored. The proposed dam site would be located in a substantially coal mine area and could experience adverse foundation problems. Due to these reasons, Laurel Run Project was dropped from further consideration.

Abram Creek

The headwaters of Abram Creek originate southeast of Mount Storm, West Virginia. The Creek meanders north through Grant and Mineral Counties, West Virginia, to its confluence with the North Branch Potomac River, near Harrison, West Virginia. The main tributaries to Abram Creek include Johnnycake Run, Wyeroff Run, Glade Run and Emory Creek. The areas that Abram Creek passes through vary from open, moderately sloping stream valleys to steep, forested mountains. Within its drainage area of 42.9 square miles are numerous abandoned and active strip and deep coal mines.

Two dam sites on Abram Creek were investigated. Site A was located about 2.5 miles upstream of the point where Emory Creek jointed Abram Creek. The proposed Site A would control a drainage area of about 30.0 square miles. A dam at this location could provide a storage capacity of 2,200 acre-feet with a top of dam elevation of 2,200 feet msl. The proposed dam at Site A would be 2,100 feet long and about 160 feet high. The lake behind the dam would provide a surface area of 375 acres.

Site B would be located about 0.25 miles upstream from the county line between Mineral and Grant Counties, and about 0.25 miles downstream of the point where Johnnycake Run meets the Abram Creek. The proposed Site B would control a drainage area of 26.4 square miles and provide a storage capacity of 3,900 acre-feet with a top of dam elevation of 2,400 feet msl. The proposed dam at Site B would be 1,700 feet long and about 160 feet high. The lake behind the dam would have a surface area of 650 acres.

Terrestrial habitat for the Abram Creek watershed is similar to that found in other large tributaries of the North Branch Potomac River. However, there has been and still is extensive coal mining in the basin. This has caused severe stress situations in the terrestrial environment due to extensive loss of vegetation and erosion of soils. This has also resulted in significant wildlife losses. To further complicate this, Abram Creek exhibits poor water quality due to acid mine drainage. Due to this acid loading, no aquatic life exists in this stream. Recent sampling data shows that not only is the main stream of Abram Creek severely polluted but also several of its tributaries.

No efforts were made to develop cost data for these sites because of uncertain subsurface conditions. Detailed investigations would be needed to determine foundation design and stability analyses.

ANALYSIS OF THE UPSTREAM RESERVOIR SITES

These 19 reservoir sites in the upstream portion of the Potomac River Basin were examined for the MWA Water Supply Study. The analyses performed during the study indicated that no significant changes had occurred since the original projects were formulated. The current state of the project sites were compared to those evaluated in earlier reports; there were no new site conditions which would cause significant modifications to the design of the reservoir projects. Therefore, the analysis concluded that the previously developed design and costs were still valid and applicable to the MWA Water Supply Study. Subsequently, the costs were updated to October 1981 values.

In addition, reexamination of the original environmental assessment data and field investigations of the project sites revealed that the types of terrestrial, aquatic, and cultural resources had not changed significantly in the 20 years since the original assessment. With perhaps the exception of water quality, there were no significant localized changes; that is, those changes which occurred in one part of the basin were reported at other sites. Changes in water quality, particularly increased non-point pollution problems, were somewhat site-specific; however, these were considered minor in comparison to the overall environmental impacts. The environmental reevaluation concluded that the relative impacts of the alternative sites were still relevant.

Since both the relative costs and environmental impacts for the reservoir sites developed in the earlier reports were still considered valid for the level of detail in the present effort, the earlier formulation analysis was chosen as a means of selecting sites for plan formulation for the MWA Water Supply Study. Of the original reservoir projects from the 1963 Report, the sites selected by the Chief of Engineers in 1969 were Sixes Bridge, Verona, Town Creek, North Mountain, Sideling Hill, and Little Cacapon. These sites were chosen as water supply solutions, as well as to meet flood control, recreation, and water quality needs. Details about the original selection process for these sites are presented in Appendix A of this report. These six sites were selected to be carried into the formulation phase, as presented in Appendix B, Plan Formulation, Assessment, and Evaluation.

In addition, the three sites recommended by the U.S. Fish and Wildlife Service were reviewed for their value as potential alternatives in the final formulation. First, the two headwater sites, Laurel Run and Abram Creek, would have only a limited amount of storage, and consequently could only minimally increase the downstream water supply. Also, their small storage would not provide any measurable improvement in the downstream water quality. For these two reasons, these two sites were eliminated from further consideration.

The other FWS site, Mount Storm, was not very favorable from a cost per yield viewpoint, having a cost of $1.31 million per mgd while five of the six already-selected projects had costs below $1 million per mgd. However, the water quality of Stone River could be substantially improved with a storage project at Mount Storm. This potential for restoration of the aquatic resource warranted further consideration in light of the general environmental concerns about reservoir construction. Therefore, the Mount Storm project was included in the MWA plan formulation process.

A summary of the cost and yield data for the upstream reservoir sites is presented in Table F-26. The sites are listed in order of ascending cost per mgd, for comparison. The seven sites selected for further formulation analysis are noted in Table F-26. Although these sites could provide substantial storage for the MWA, they have received very little support from the affected state and local governments, as well as the local citizenry. In addition, any of the reservoir projects would have considerable impact on the neighboring aquatic and terrestrial systems. Their potential as favorable alternatives would have to be viewed in these contexts.

TABLE F-26

SUMMARY OF THE UPSTREAM RESERVOIR SITES
(October 1981 Price Levels)

Reservoir Site	Stream	Water Supply Storage MG	Additional Yield mgd	Water Supply Storage Cost $million	Water Supply Cost per MG $	Total Cost $million	Cost per mgd $million	Selected for Further Plan Formulation
Royal Glen	South Branch Potomac River	74,300	312	14.56	196	131.20	0.42	No
Seneca	Potomac River	150,000	900	68.07	454	436.32	0.48	No
Licking Creek	Licking Creek	26,800	89	9.78	365	48.44	0.54	No
Town Creek	Town Creek	18,700	63	6.70	358	41.84	0.66	Yes
Bracks Gapp	North Branch Shenandoah River	39,400	95	3.38	86	70.46	0.74	No
Sixes Bridge	Monocacy River	20,500	85	37.79	1,840	65.84	0.77	Yes
Sideling Hill	Sideling Hill Creek	17,800	61	3.84	216	47.78	0.78	Yes
North Mountain	Back Creek, WV	31,100	89	10.85	349	70.02	0.79	Yes
Little Cacapon	Little Cacapon River	17,300	59	5.30	306	51.36	0.87	Yes
Tonoloway Creek	Tonoloway Creek	16,300	54	11.14	683	59.23	1.10	No
West Branch	West Branch Conococheague Creek	14,700	42	1.80	122	49.90	1.19	No
Mount Storm	Stony River	8,800	32	13.64	1,550	41.84	1.31	Yes
Chambersburg	Conococheague Creek	9,100	40	4.41	485	53.18	1.33	No
Verona	Middle River	33,900	110	77.07	2,270	152.62	1.39	Yes
Savage II	Savage River	12,700	37	32.94	2,590	56.31	1.52	No
Winchester	Opoquan Creek	9,200	30	2.16	235	47.06	1.57	No
Back Creek	BAck Creek, PA	6,400	18	244	381	31.63	1.76	No
Laurel Run	Laurel Run	870	Minimal	NA	NA	NA	NA	No
Abram Creek (Site A/B)	Abram Creek	720/1,270	Minimal	NA	NA	NA	NA	No

NA = Not Available

GROUNDWATER

INTRODUCTION

Groundwater has served as a water supply source for many centuries. Groundwater-dependent systems exist throughout the United States and the world. Its universality as well as its generally good water quality make it a very attractive supply source. It is essentially an underground reservoir with a high water supply potential in some localities. However, the ability of the groundwater resource to provide large supplies varies considerably depending on the underlying aquifer characteristics. It was these characteristics which were the focus of the technical groundwater investigations for the MWA Water Supply Study.

Although the major water utilities in the Metropolitan Washington Area (MWA) do not utilize the groundwater source, groundwater has provided a steady supply to several communities in the outlying areas. The potential for even greater groundwater development remains. The costs and impacts for several groundwater development schemes were evaluated for this study. The details of this analysis are presented in the following sections.

NEWS STUDY GROUNDWATER SCHEMES

In the Northeastern United States Water Supply Study (NEWS) two groundwater regions were identified as potential projects for the MWA. The first NEWS project was a wellfield scheme in the Hagerstown area, located about 50 miles west of the MWA. The Hagerstown wellfield scheme consisted of pumping up to 50 mgd from the carbonate rock aquifers underlying the Hagerstown Valley physiographic region. The water would then be discharged directly to a Potomac River tributary upstream of the MWA's raw water intakes at Great Falls, Maryland. The project was planned for operation on an as-needed basis. In order to provide 50 mgd, two wellfield sites of 90 and 80 wells each were designed for the NEWS project. The maximum well depth would be about 300 feet.

The NEWS study also considered groundwater development in the Maryland Coastal Plain region lying east of the MWA. This wellfield scheme was planned to tap the region's deep aquifers, particularly the Magothy and Patapsco formations. To fully penetrate these aquifers and thus maximize their well yields, the wells would have to be drilled to a depth of approximately 1100 feet. For the NEWS study, two specific sites of 25 and 50 wells were investigated. These sites are located in southeastern Anne Arundel County. From the developed wellfields, the water would be piped to McMillan Water Treatment Plant for aeration and sedimentation treatment, and subsequent distribution, according to the NEWS scheme.

The NEWS analysis concluded that both of these schemes would be moderately expensive and energy-intensive water supply sources with power-sensitive operating costs. The availability of water from the Hagerstown Valley wellfield scheme was somewhat questionable due to the highly variable yield patterns in the carbonate rocks. The carbonate aquifer permeability in the Hagerstown Valley is very dependent on the underlying geologic structure and depositional characteristics, i.e., the location of fractures and joints; therefore, the well yield of individual wells would vary considerably within short distances. On the other hand, the Coastal Plain aquifers exhibit relatively homogeneous hydraulic characteristics, and their yield is geographically consistent. Consequently, the Coastal Plain groundwater project would provide a reliable water supply source. Due to the direct connection with the MWA system, the Coastal Plain scheme could also operate on a peak-supply basis as well as for base supply. The Hagerstown Valley project by its distance from the MWA intakes would only provide additional base supply. Both of these projects would have some adverse impacts on the wellfield area environment and the local groundwater users.

For the MWA Water Supply Study, these two NEWS schemes were selected initially for further detailed investigation as groundwater alternatives. However, there was strong citizen opposition to the development of wellfield sites in the Hagerstown Valley area. Additionally, at the July 1977 meeting of the Federal-Interstate-Region Advisory Committee (FISRAC), the State of Maryland recommended that the scheme be dropped from further consideration as an MWA alternative. The State of Maryland also cautioned that use of the Southern Maryland groundwater resource for the MWA would be acceptable only if it were demonstrated that local groundwater users would not be significantly affected by the water transfer.

Given these concerns, the Hagerstown Valley scheme was eliminated, and detailed groundwater modelling efforts were undertaken for the Coastal Plain region. The remainder of the groundwater investigations for the MWA Water Supply Study, then, focused on ascertaining the feasibility of using the Coastal Plain aquifers in southern Maryland to supplement the MWA supply system. .

STUDY AREA DESCRIPTION

GENERAL

The study area for the groundwater investigations was defined as that part of the Coastal Plain of Southern Maryland that falls within a 30-mile radius of Washington, D.C., and is east of the Fall Line, as depicted in Figure F-26. This hemisphere was considered a practical economic limit for developing a groundwater supply for the MWA. The aquifer to the west of the Fall Line, the Piedmont aquifer, was not included in the study because of its small yield.

The study area encompasses large portions of Anne Arundel, Calvert, Charles, and Prince Georges Counties. The area is characterized by gently rolling topography. Elevations in the study area range from sea level to a maximum of 460 feet in northern Prince Georges County. Numerous estuaries and stream valleys dissect the land formations. The Potomac and Patuxent estuaries are the major bodies of water within this study area. Tidal marshes and swamps are commonly adjacent to the estuaries and streams.

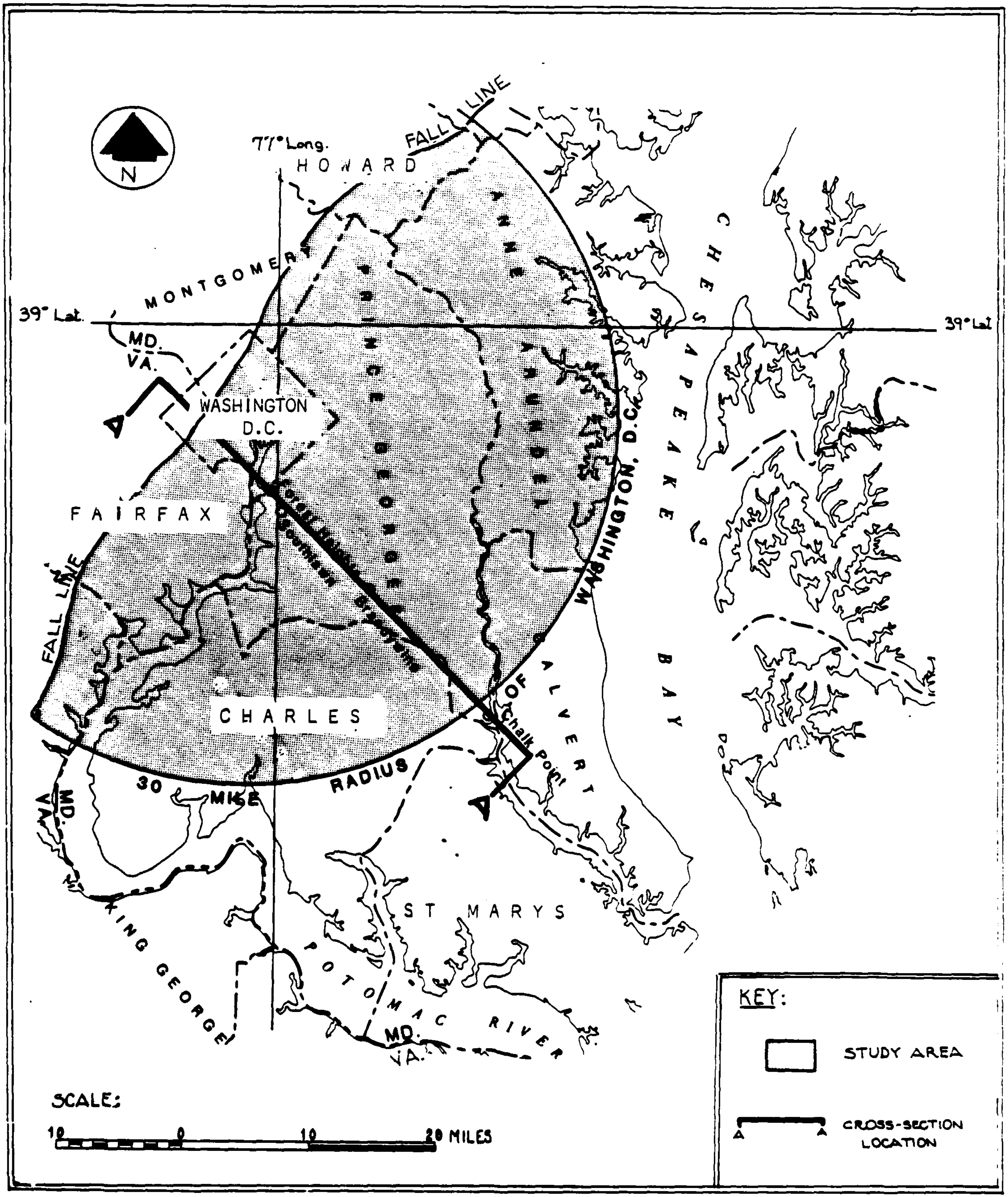
The climate of the southern Maryland area is generally humid and temperate. Rainfall is fairly evenly distributed throughout the year but is highest in summer due to frequent thundershowers. On the average, July and August are the wettest months, while November is the driest. The average annual precipitation for the study area is about 45 inches.

GEOLOGY

The Coastal Plain consists of numerous layers of unconsolidated sediments as indicated in Figure F-27. The wedge of these deposits increases in thickness from a feather edge at the Fall Line to about 2,400 feet in southern Prince Georges County. The basal Coastal

FIGURE F-26

LOCATION OF THE GROUNDWATER STUDY AREA



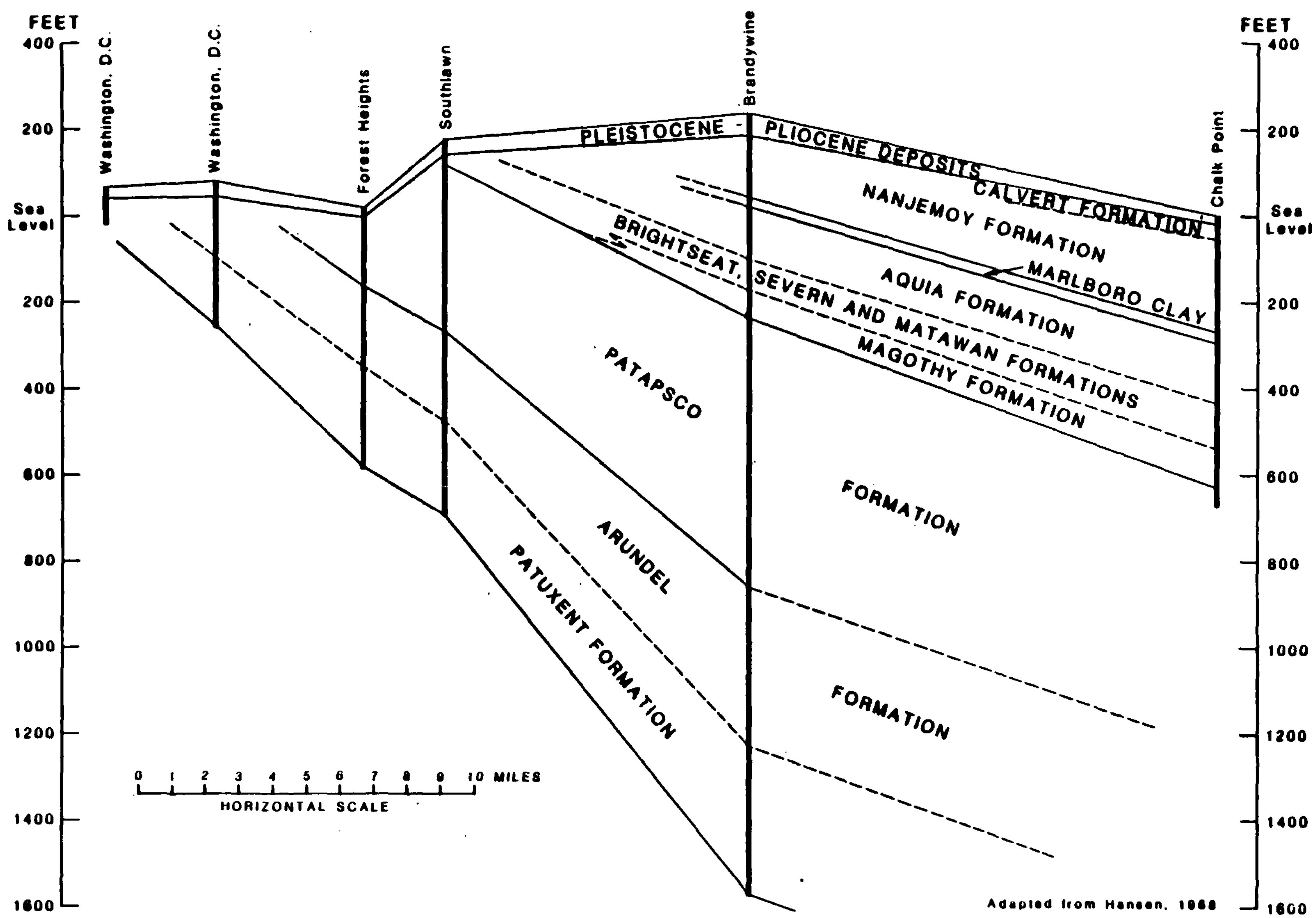


FIGURE F-27

GENERALIZED GEOLOGIC CROSS-SECTION OF THE COASTAL PLAIN SEDIMENTS

Plain unit is the Potomac Group of Early Cretaceous age, the Patapsco, Arundel, and Patuxent Formations, which constitutes most of the bulk of the sediments. This group has an average thickness of 1000 feet compared to the total average wedge thickness of 1470 feet, as noted in Table F-27. Much of the study area surface is covered by a veneer of sand, silt, and clay deposited during the Pleistocene period. A description of the Coastal Plain sediments is presented in Table F-27.

The Coastal Plain sediments in the study area were deposited under various conditions. During Early Cretaceous time, the Potomac Group sediments were deposited in a fluvio-deltaic environment. Deposition of this type consists of complexly related stream and sandbar sand and gravel, intermixed with floodplain and swamp silt and clay. The Patuxent and Patapsco Formations are composed of extensive sand, whereas the Arundel Formation is principally silt and clay. However, because of this constantly changing environment both laterally and vertically, it is difficult today to correlate discrete sand units between wells.

The fluvial environment of Early Cretaceous time was followed by a fluvio-marine phase in the Late Cretaceous represented by the Magothy Formation. These near-shore deposits consist primarily of sand and gravel. Because of the gradual landward encroachment of the sea during the formation of the Magothy, the sand tends to become finer upward. As the sea encroached onto the land, the depositional environment gradually changed to a strictly marine environment.

The marine environment is represented first by the Severn and Matawan Formations, and also by the overlying Brightseat Formation. The sediments of these formations consist almost entirely of silt and clay typical of marine deposition. Above the Brightseat Formation is the Aquia Formation. During Aquia time, the sea began to regress from the land. Finally, during the latter part of the Aquia time, much sand and fine sand were deposited in this shallowing sea. Overlying the Aquia Formation is the fluvial-marine Marlboro Clay, consisting of about 30 feet of very tight clay. Located above the Marlboro Clay, the sediments of the Nanjemoy Formation are an admixture of fine sand, silt, and clay deposited in a marine environment. The sandy Piney Point Formation overlies the Nanjemoy Formation. However, the Piney Point Formation pinches out just east of the study area and therefore was not included in the groundwater investigations. The Calvert Formation directly overlies the Nanjemoy in the study area. The sediments of the Calvert Formation were deposited in a shallow marine sea and consist of clay, silt, and fine sand. Finally, the Pliocene and Pleistocene deposits are irregularly distributed near the surface throughout the area. These are mostly fluvial deposits, and vary from silt and clay to sand and gravel.

These Coastal Plain sediments generally consist of alternating layers of clay, silt, and sand. The sandy layers act as conduits through which groundwater flows. These layers are called aquifers if they yield sufficient flow. The less permeable silt and clay act as confining layers that restrict flow in the vertical directions, thus constraining flow between those aquifers which are separated by confining layers. However, confining layers generally permit passage of small amounts of water between aquifers where pressure differences are sufficient. The whole Coastal Plain wedge is but a single system in which the direction of water movement within the layers is dictated by permeability and pressure differences. Thus, even where the sand layers seem discontinuous, hydraulic continuity with other sand units is commonly good because sediments separating the discontinuous sand layers are not totally impermeable.

TABLE F-27

GENERALIZED LITHOLOGIC DESCRIPTIONS OF THE COASTAL PLAIN SEDIMENTS IN THE STUDY AREA

System	Series	Group	Formation	Approximate Average Thickness	Physical Characteristics	Water-Bearing Properties
Quaternary "	Pleistocene Pliocene	Columbia "	Undivided deposits	30	Irregularly bedded tan to orange sand, gravel, silt and clay.	Yields adequate domestic supplies.
Tertiary	Miocene	Chesapeake	Calvert	110	Brown to greenish-gray sandy clay to fine sand. Generally diatomaceous.	Generally not an aquifer.
"	Eocene	Pamunkey	Nanjemoy	90	Glauconitic fine sand with greenish-gray clayey layers.	Generally not an aquifer.
"	"	"	Marlboro Clay	30	Pink to gray clay.	Not an aquifer.
			Aquia	80	Greenish-gray to brown glauconitic sand with indurated layers.	Principal aquifer in eastern part of study area.
	Paleocene		Brightseat	40	Gray to dark gray micaceous silty to sandy clay.	Not an aquifer.
Cretaceous	Upper Cretaceous		Severn and Matawan	40	Dark-colored sandy clay and sand. Some glauconite. Lighter colored near base.	Not an aquifer.
"	"		Magothy	50	Light-gray to white sand and gravel and interbedded light-colored clay. Contains some lignite, byrite, and glauconite.	Important aquifer in northern half of study area.
"	Lower Cretaceous "	Potomac "	Patapsco	500	Interbedded, variegated sand, clay and sandy clay; in places, upper part consists of about 200 feet of pink clay.	Good aquifer. Used extensively in northwest part of study area.
"	"	"	Arundel	250	Red, brown, and gray clay. Some interbedded sand lenses.	Generally not an aquifer.
"	"	"	Patuxent	250	Mostly gray and yellow sand and interbedded clay. Often contains sand and gravel basal unit.	Good aquifer. Used extensively in northwest part of study area.

Source: Digital Simulation of Groundwater Flow in Part of Southern Maryland, William B. Fleck, U.S. Geological Survey, Towson, Maryland, 1982.

DESCRIPTION OF AQUIFERS

Within the study area, there are four major formations which serve as aquifers. These aquifers include, in ascending order, the Patuxent, the Patapsco, the Magothy, and the Aquia Formations. The locations of major withdrawals from these aquifers are identified in Figure F-28. The aquifers are described in detail in the following sections.

Patuxent Formation

The Patuxent Formation crops out along the Fall Line immediately adjacent to the crystalline Piedmont rocks. It thickens from a feather edge along the Fall Line to about 500 feet on the east side of the study area. The Patuxent aquifer is a multi-aquifer unit consisting of several water-bearing sand layers. Individual sand units range in thickness from several feet to as much as 100 feet. Near Baltimore, these units may comprise as much as 50 percent of the Patuxent Formation. Southward, the percentage of sand diminishes.

Transmissivity, which is a measure of the aquifer's ability to transmit water, is fairly well known in the updip part of the Patuxent Formation, especially between Washington and Baltimore where transmissivity and usage is greatest. Transmissivity data for the rest of the study area are sparse. A map of the estimated transmissivity distribution in the Patuxent aquifer is presented later in this appendix.

A primary source of recharge to the Patuxent aquifer is from precipitation on the outcrop area. Mack (Reference 4) estimates that for Anne Arundel County, about 25 percent of the precipitation recharges the aquifers. This amounts to about 0.5 mgd per square mile for an annual precipitation of 45 inches. In the study area, the recharge area of the Patuxent aquifer is about 300 square miles. Thus, the Patuxent aquifer within the study area receives about 150 mgd recharge. However, most of this recharge is discharged as base flow to streams and rivers or directly to Chesapeake Bay. Mack also estimates that 0.1 mgd per square mile of this recharge moves downgradient into the deeper parts of the aquifers. This amounts to approximately 30 mgd for the Patuxent aquifer.

The Arundel Formation, above the Patuxent aquifer, is a thick confining silt and clay unit that allows only small volumes of water to pass through. Near the outcrop, heads in the overlying Patapsco aquifer are generally higher than heads in the Patuxent, causing downward movement of water through the confining silt and clay into the Patuxent aquifer. Under the Chesapeake Bay, the head gradient is reversed; thus, water is discharged from the Patuxent aquifer upward through the confining Arundel Formation.

Pumpage from the Patuxent aquifer statewide is about 20 mgd. About 80 to 90 percent of this pumpage is from northern Anne Arundel and northern Prince Georges Counties, as noted in Figure F-28. Almost all of the pumpage is from shallow wells in the updip part in or near the outcrop area.

Patapsco Formation

The Patapsco Formation crops out along a belt eastward and parallel to that of the Patuxent Formation. The width of the outcrop ranges from 12 miles south of Baltimore, to 2 miles, on the east side of Washington. The area of the outcrop is about 300 square miles. As in the Patuxent aquifer, recharge on the outcrop area of the Patapsco is estimated as 150 mgd, of which about 30 mgd moves downward to the deep part of the aquifer.

Like the Patuxent aquifer, the Patapsco aquifer contains sandy layers interbedded with silt and silty clay. The sandy layers act as aquifers; however, they tend to be discontinuous. Sand in the Patapsco Formation is generally finer than that in the Patuxent; however, the overall thickness of sand beds is greater. Transmissivity is high in the Baltimore-Annapolis area. Southward, transmissivity is much less. Cumulative thickness of sand beds in the Annapolis area is about 300 feet. Southward, the cumulative thickness decreases, and, in Charles County, it is generally less than 150 feet.

Pumpage from the aquifer is about 20 to 25 mgd and confined to the updip part. Heaviest use is in northern Prince Georges and Anne Arundel Counties and, south of Washington, along the Potomac River in Charles County.

Magothy Formation

The Magothy aquifer crops out in Prince Georges and Anne Arundel Counties. South of central Prince Georges County, it is overlapped by the younger silty clay of the Matawan and Brightseat Formations. The Magothy aquifer ranges in thickness from about 200 feet in the Annapolis area to a feather edge where it pinches out in the southern part of the study area.

The coarser sand and gravel generally occur at the base of the aquifer which overlies the Patapsco clay beds. The sediments of the Magothy become finer upward, grading into the overlying clayey confining beds of the Matawan and Brightseat Formations. Generally, where the aquifer is thicker, transmissivity is higher. An area of high transmissivity is located in the northern part of the study area.

The aquifer is partly recharged at the outcrop area in Prince Georges and Anne Arundel Counties. The area of the outcrop is about 50 square miles. With the assumption that 25 percent of precipitation recharges the aquifer, about 25 mgd is recharged in the study area, and about 5 mgd seeps downward into the deeper parts of the aquifer. Leakage from the overlying Aquia Formation and the underlying Patapsco aquifer provides some additional recharge.

Total pumpage from the Magothy aquifer in Maryland is about 8 to 10 mgd. Figure F-28 shows the location of the principal pumping centers. Although the distribution of pumping centers is relatively even, consumption is heaviest in the Annapolis area with about 4 to 5 mgd of withdrawals. In Prince Georges County, pumpage from the Magothy is slightly less than 1 mgd and, in Charles County, it is about 0.5 mgd.

Aquia Formation

The Aquia aquifer crops out as an irregularly shaped belt from Sandy Point in Anne Arundel County to Indian Head in southwestern Charles County. The maximum width of the outcrop is about 10 miles, and its area is about 170 square miles. Assuming that recharge from precipitation is 0.5 mgd per square mile, total recharge to the Aquia is 85 mgd, 17 mgd of which moves downward into the deeper part of the aquifer. As with the other aquifers, leakage both into and out of the Aquia occurs through the adjacent confining layers. Sandy beds in the Aquia are thickest east of the Chesapeake Bay. In the study area the sand beds are 40 to 70 feet thick.

Total pumpage from the Aquia aquifer is about 4 mgd; however, almost all is outside the study area. Figure F-28 indicates only a few pumping centers within the study area; however, the pumpage is less than 0.1 mgd.

AQUIFER WATER QUALITY

Natural water quality in the four major aquifers is generally good in the study area and suitable for potable supplies with minimal treatment. In the updip areas, the water is commonly safe, low in total dissolved solids (TDS), low in chlorides, with moderately low but acceptable pH values, and somewhat high in iron. The high iron concentrations require some treatment prior to domestic use. As the aquifers go downward, the water tends to become harder, more alkaline (higher pH), with lower concentrations of iron, and higher concentrations of chlorides and TDS. Outside of the study area, the aquifers become quite brackish, to the point of unsuitability for potable use.

According to Otton's report in 1955 (Reference 5), water from the Patuxent aquifer had an average pH of 6.1, and its TDS levels ranged from 18 to 227 mg/l with an average of 91 mg/l. Recent well records from the State of Maryland generally confirm these findings. These analyses indicate an average pH of 6.3 and a TDS range of 14-210 mg/l, with an average TDS concentration of 86 mg/l in the Patuxent wells.

For the Patapsco formation, Maryland groundwater quality analyses indicate an average pH of 6.6 and TDS concentrations ranging from 34 to 265 mg/l. The average dissolved solids concentration was found to be 135 mg/l. The average pH is considerably greater than Otton's value (an average pH of 5.7) for all of southern Maryland.

The Magothy Formation in the study area exhibits an average pH of 7.1, and TDS values between 66 and 224 mg/l. The average TDS concentration is 152 mg/l. Like the Patapsco formation, the observed pH value in the study area is higher than Otton's value of 6.5 for the Magothy aquifer in southern Maryland.

In the Aquia aquifer, the average pH was observed to be 7.8. TDS concentrations ranged from 67 to 238 mg/l, with an average of 170 mg/l. These values are similar to Otton's indicating that Aquia water quality within the study area is typical for the Aquia throughout southern Maryland.

As indicated earlier, there is a trending in pH values from the updip (low pH, acidic water) to the downdip (higher pH, more alkaline) areas. This is fairly well outlined in Figure F-29 which shows the regional variation in pH. The data was based on well observations from all four aquifers. The TDS concentrations noted earlier are all within an acceptable range (less than 1000 mg/l) for most domestic and industrial uses.

GROUNDWATER MODELLING BY USGS

As part of the MWA Water Supply Study groundwater investigations, the United States Geological Survey (USGS) was contracted to perform detailed studies of the groundwater availability and the drawdown effects due to large-scale pumping from the Coastal Plain aquifers. This analysis was formally completed in May 1982 with the submission of the

FIGURE F-28

APPROXIMATE LOCATIONS OF THE MAJOR GROUNDWATER PUMPAGE FROM THE COASTAL PLAIN AQUIFERS

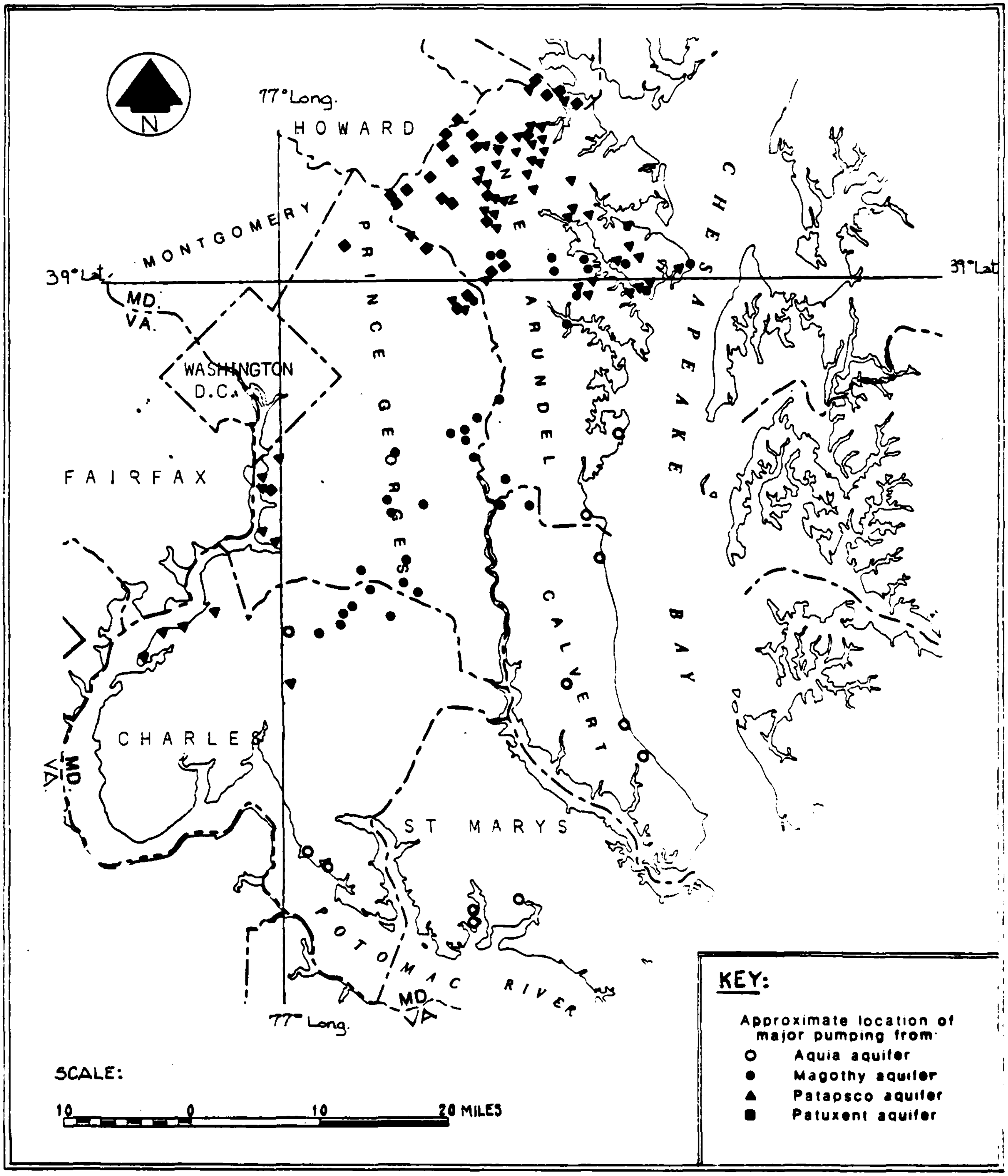
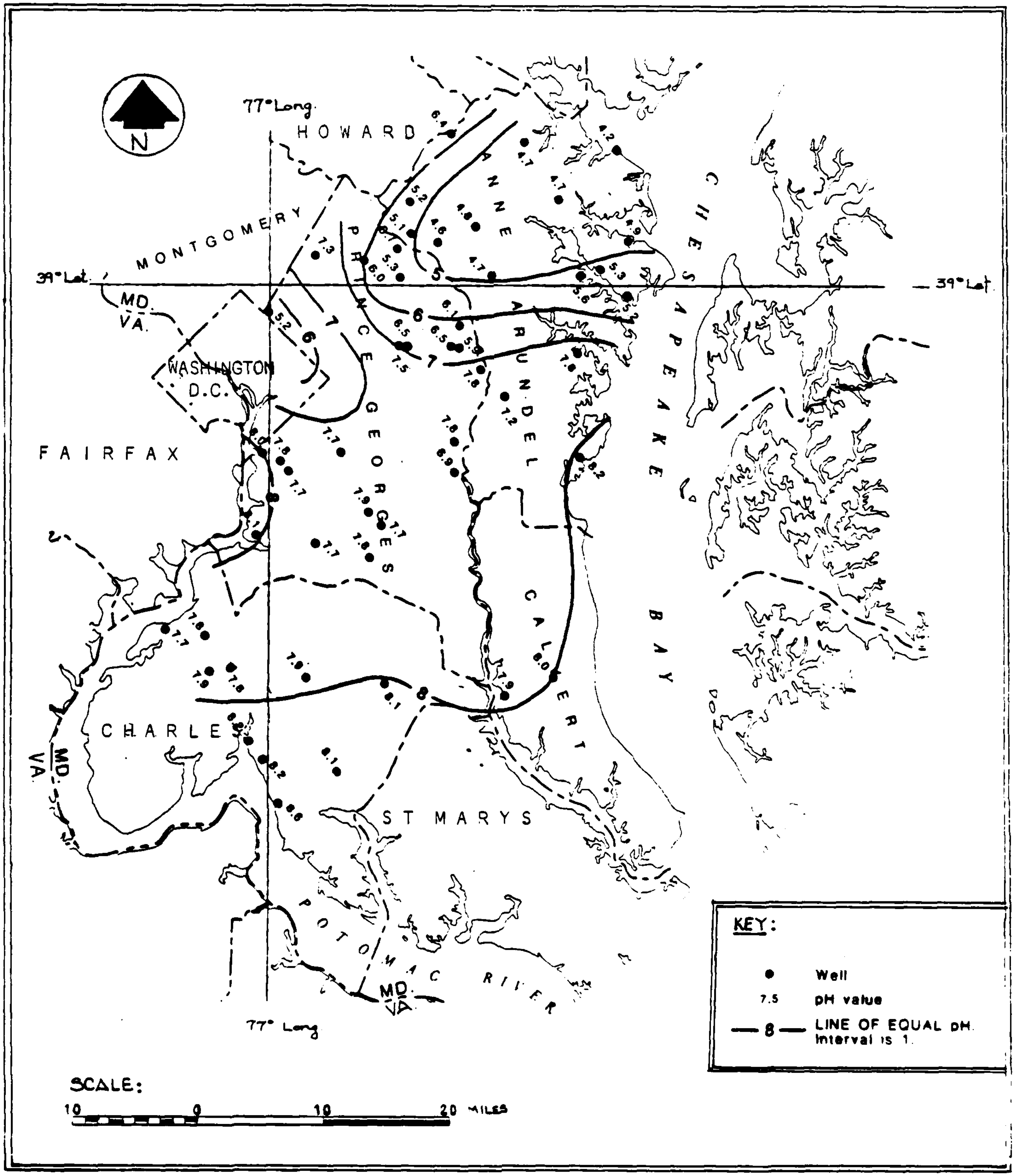


FIGURE F-29

REGIONAL VARIATION IN GROUNDWATER PH



final report. The final report, Digital Simulation of Groundwater Flow in Part of Southern Maryland, is attached to this appendix as Annex F-1. In addition, Maryland Geological Survey's Report of Investigations No. 33, A Quasi Three-Dimensional Finite-Difference Groundwater Flow Model with a Field Application (1979), is also printed in Annex F-1. This report describes the aquifer model used by the USGS in their groundwater simulations for the MWA Water Supply Study.

The first phase of the USGS modelling efforts was the collection and evaluation of existing data for the study area. To this end, the USGS collected hydrologic and geologic data including aquifer depth, thickness, hydraulic conductivity, storage coefficients, and groundwater usage for the Coastal Plain aquifers and confining layers. The original scope of the USGS study included drilling three test wells into the deep downdip parts of the Cretaceous aquifers (i.e. the Patuxent, Patapsco, Arundel, and Magothy Formations). The test wells were desirable in order to obtain a more accurate description of the groundwater system. However, funding limitations for the MWA Water Supply Study precluded the test well drilling. The existing data were considered sufficient for the USGS modelling effort.

The data collection phase was the required input to the second phase of the USGS modelling study, which included the construction, calibration, and testing of a three-dimensional flow simulation model of the Coastal Plain aquifers. This phase is summarized in the proceeding sections. Further details of the simulation modelling are given in Annex F-1.

MODEL DESCRIPTION

The USGS simulation model used a finite-difference numerical algorithm to solve the groundwater flow equations in quasi three dimensions. First, groundwater movement was assumed to be lateral (in two dimensions) within the aquifers, as expressed by a two-dimensional partial differential flow equation. In addition, leakage from overlying and underlying confining formations was included as vertical flow, in the form of a partial differential flow equation. The vertical flow analysis was further simplified by assuming that homogeneous conditions exist and that the head (pressure) distribution in the confining layer is bound by the heads of the adjacent aquifers. From these assumptions, the leakage relationship is reduced to a proportionality such that the confining layer leakage is directly proportional to the head difference in the adjacent aquifers and confining layer permeability, and is inversely proportional to the thickness of the confining aquifer.

The resulting combination of these flow equations is represented by the following equation:

$$T\frac{\partial^2 h}{\partial x^2} + T\frac{\partial^2 h}{\partial y^2} + q_{bottom} - q_{top} = S\frac{\partial h}{\partial t} - W$$

for:

- T = transmissivity (L^2t^{-1})
- h = potential head of water in the aquifer (L)
- x,y = coordinates in two dimensions (L),
- t = time (t)
- q bottom = flux (leakage) out of bottom of aquifer (LT-1)
- q top = flux into top of aquifer (LT-1)
- S = aquifer storage coefficient (dimensionless)
- W = flux in the vertical direction, either recharge or pumpage (LT-1)

This equation was then solved by a computer program using the finite-difference method. Basically, this method involves the substitution of finite-difference approximations for the partial derivates in the flow equation. The finite-difference approximations were based on subdividing the aquifer study area into a grid of rectangular blocks. Within each block, the aquifer properties were assumed to be uniform. The hydraulic head at the center of the block (the node) was assumed to be the average head within that block area. For the Coastal Plain aquifers, the study area was divided into a rectangular grid of 37 rows and 27 columns as depicted in Figure F-30. The major aquifers system was represented by five layers within the model. With the finite-difference method, time is represented by a series of time increments; the model calculations are then made for each discrete time interval.

MODEL ASSUMPTIONS

After establishing the mathematical basis for the physical groundwater system, several system-specific assumptions were formulated to tailor the model hydraulics to the Coastal Plain system. Under ideal circumstances, actual hydrologic data would provide the basis for the model adaptations; however, very little data was available for the Coastal Plain system. Consequently, several of the system assumptions were estimates at best. The basic model assumptions were:

a. The pressure head in the top layer represented water-table conditions, was constant throughout the simulation, and provided recharge to the aquifer through the uppermost confining layer at a rate of 0.6 inches per year.

b. The hydraulic properties of the aquifers were isotropic.

c. All flow within the aquifers was horizontal.

d. With the exception of the recharge areas, all model boundaries were assumed to be no-flow boundaries. In the recharge areas, the boundary was assumed to be constant-head with a flux of 0.1 to 0.3 inches per year during steady-state simulation.

e. Crystalline basement rock, modeled as a no-flow boundary, underlay the Patuxent aquifer.

f. The Patuxent and Patapsco Formations were each treated as single aquifers within the groundwater system. Without further hydrologic data, the numerous sand layers within each aquifer could not be further distinguished.

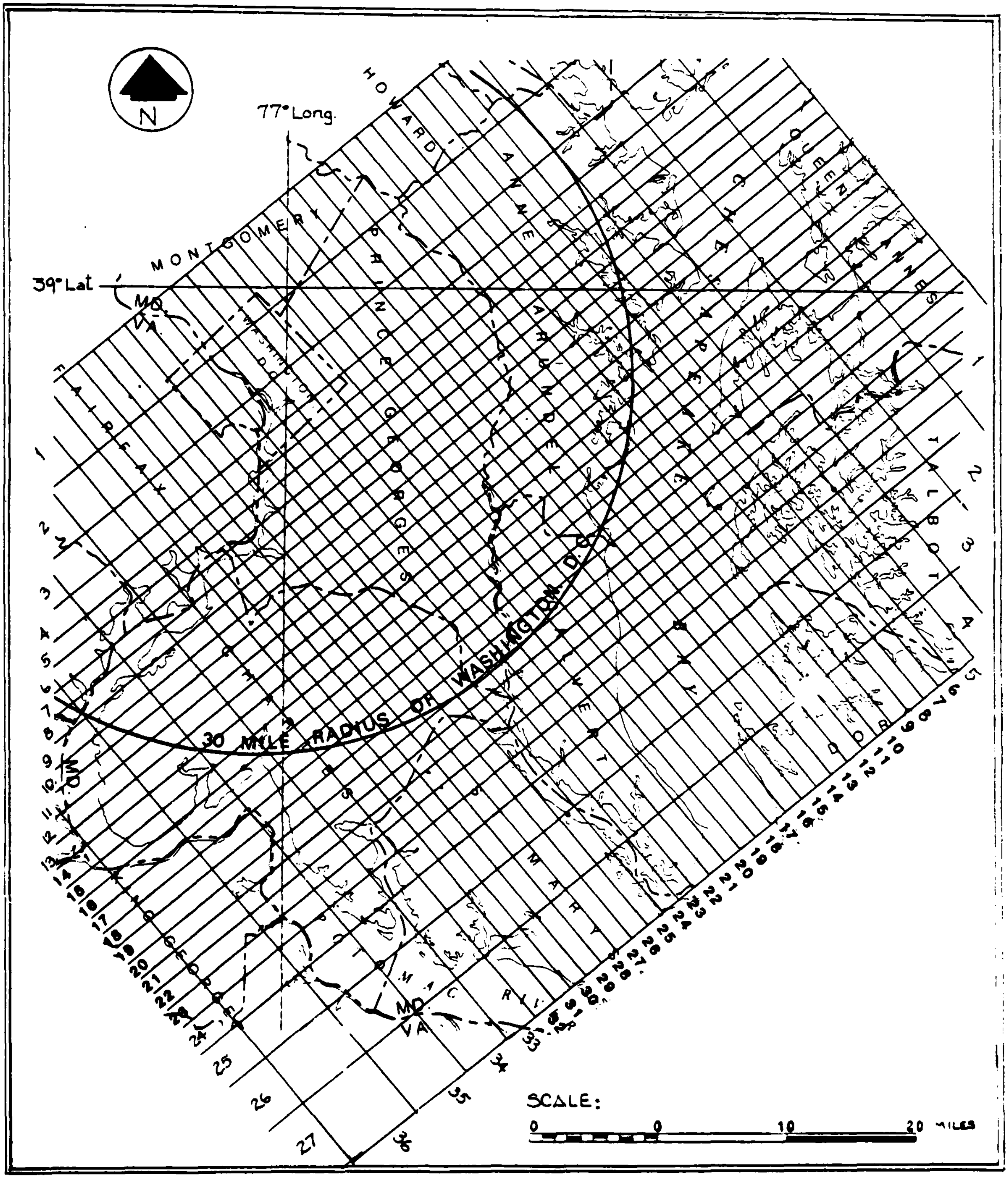
g. Flow through the confining beds was vertical and represented leakage between the aquifers. Under steady-state conditions, this flow was downward for the study area.

MODEL CALIBRATION

The calibration of the Coastal Plain aquifer model was based on steady-state simulations. Normally, groundwater models are calibrated by simulating the known history of pumping, and then comparing the calculated heads to actual field measurements. However, this approach could not be used because of the lack of pumping from the Cretaceous aquifers (Patuxent, Patapsco and Magothy) within the study area. Therefore, steady-state calibration was selected.

FIGURE F-30

MODEL GRID USED IN FINITE DIFFERENCE MODEL



To perform the calibration, pre-pumping head distributions were estimated from the available hydrologic data. Then, steady-state simulations were generated using appropriate values for the conductivity and transmissivity coefficients, and the associated pre-pumping heads were calculated. These computed head distributions were compared to the earlier estimated field distributions. The vertical hydraulic conductivity and transmissivity values were adjusted until the calculation vs. field comparison was favorable, and the calibration completed.

The best-match transmissivity distributions for the steady-state simulation are indicated in Figures F-31 through F-34. These values vary by aquifer and within the aquifer as noted in the diagrams. Generally, the adjusted values varied only slightly from the initial estimates of transmissivity.

In the model calculations, the ratio of the vertical hydraulic conductivity to the confining layer thickness was a key parameter. For the confining layer overlying the Aquia, this value ranged from 1.5×10^{-5} to 1.5×10^{-6}; for the Magothy the range was 1×10^{-4} to 1×10^{-5}; for the Patapsco, it was 1.2×10^{-5} to 1.2×10^{-6}; and for the Patuxent, it was 1×10^{-6} to 1×10^{-7}.

The specific storage for the confining layers was 3×10^{-6} per foot. The storage coefficients for the four confined aquifers were set at 1×10^{-3}. This relatively high value was due to the high percentage of silt and clay, especially in the Patapsco and Patuxent Formations. These calibrated values formed the final link in the development of the aquifer model.

The accuracy of the model calibration depended primarily on the accuracy of the estimated existing pre-pumping head distribution. Since the pre-pumping distribution was based on only a small number of observations, the accuracy of the calibration was not fully substantiated. Nevertheless, the USGS steady-state model was considered to approximately simulate the Coastal Plain aquifer system.

MODEL APPLICATIONS

After the model calibration was completed, the model was used to simulate pumping from the Magothy, Patapsco, and Patuxent aquifers, individually and in combinations. The resultant drawdowns in these three aquifers and in the overlying Aquia aquifer were estimated and used to formulate groundwater supply schemes for the MWA Water Supply Study.

Fourteen tentative sites in the study area were initially selected and tested preliminarily for groundwater development. From this group of 14 locations, four locations were chosen for detailed model and cost analysis. The four sites were selected with respect to the location of present pumping, so that interference effects with those wells would be minimized. In addition, the sites were selected such that the four major counties were each represented. The proposed sites are located in southeastern Prince Georges County, eastern Charles County, southern Anne Arundel County, and northern Calvert County as shown in Figure F-35. Aquifer cross-sections for each site are shown in Figures F-36 through F-39. Although each site was specified to a 2.4 square-mile block, the results of the model simulation would apply to sites within a few miles of the selected site.

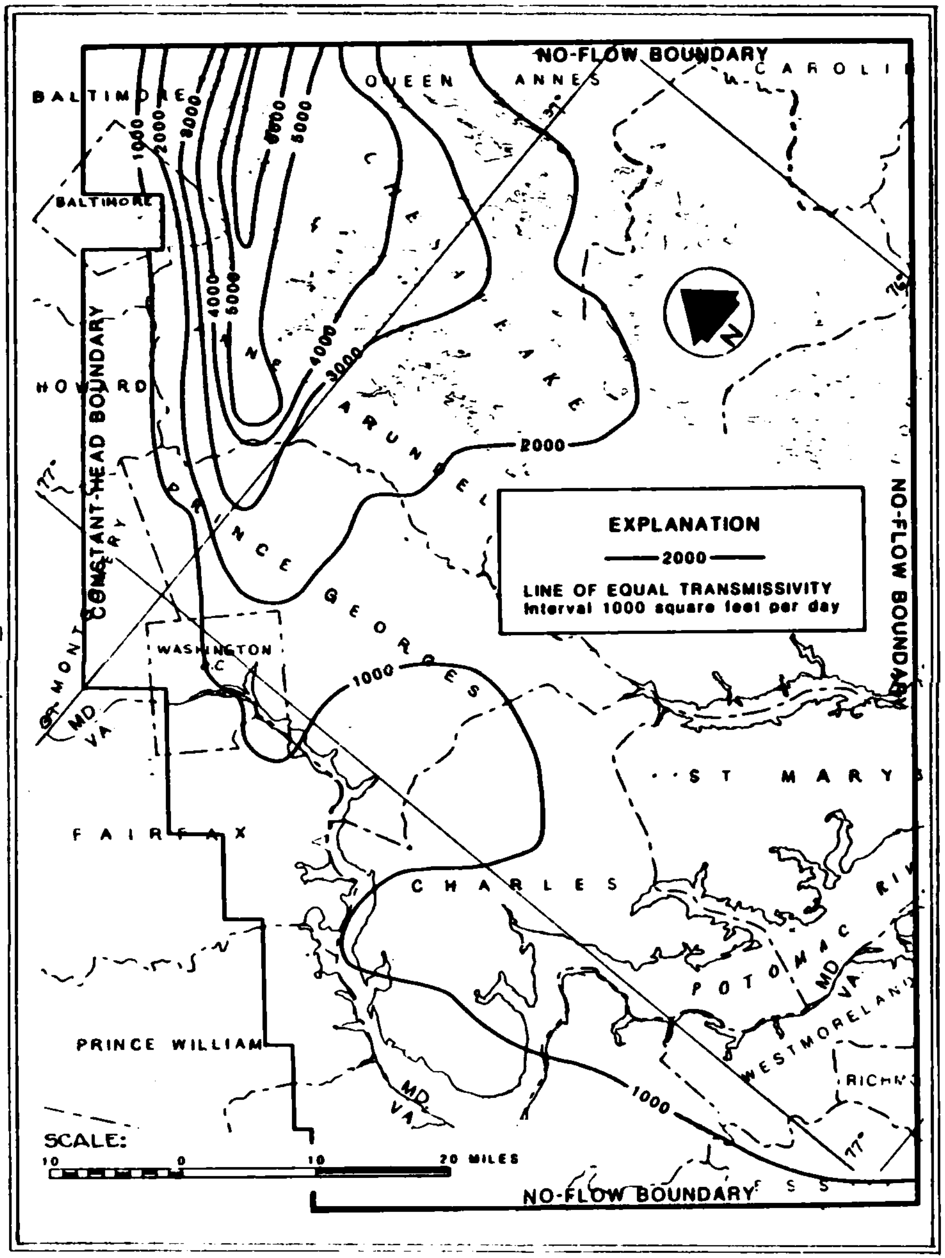


FIGURE F-31

TRANSMISSIVITY DISTRIBUTION IN THE PATUXENT AQUIFER

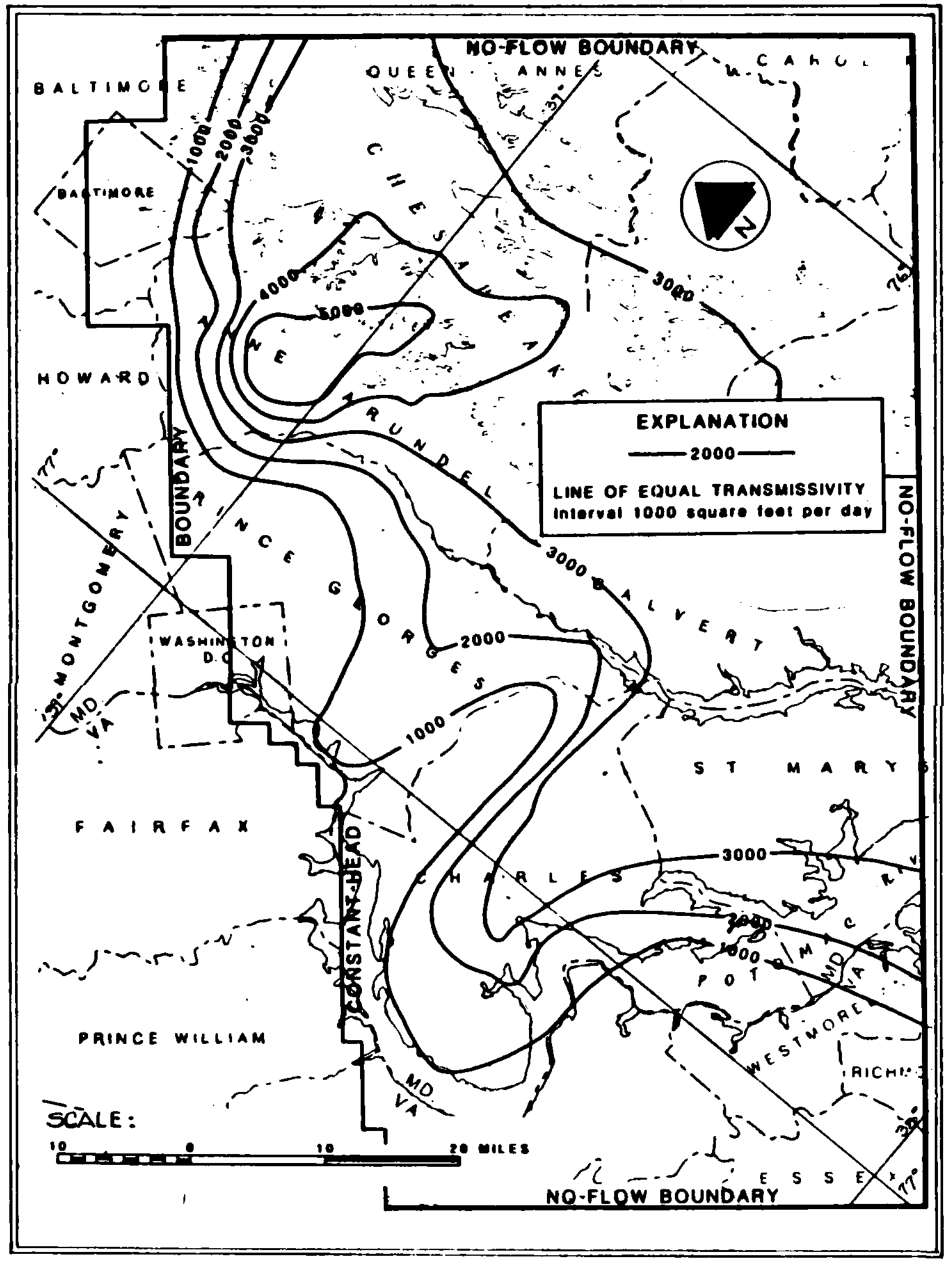


FIGURE F-32

TRANSMISSIVITY DISTRIBUTION IN THE PATAPSCO AQUIFER

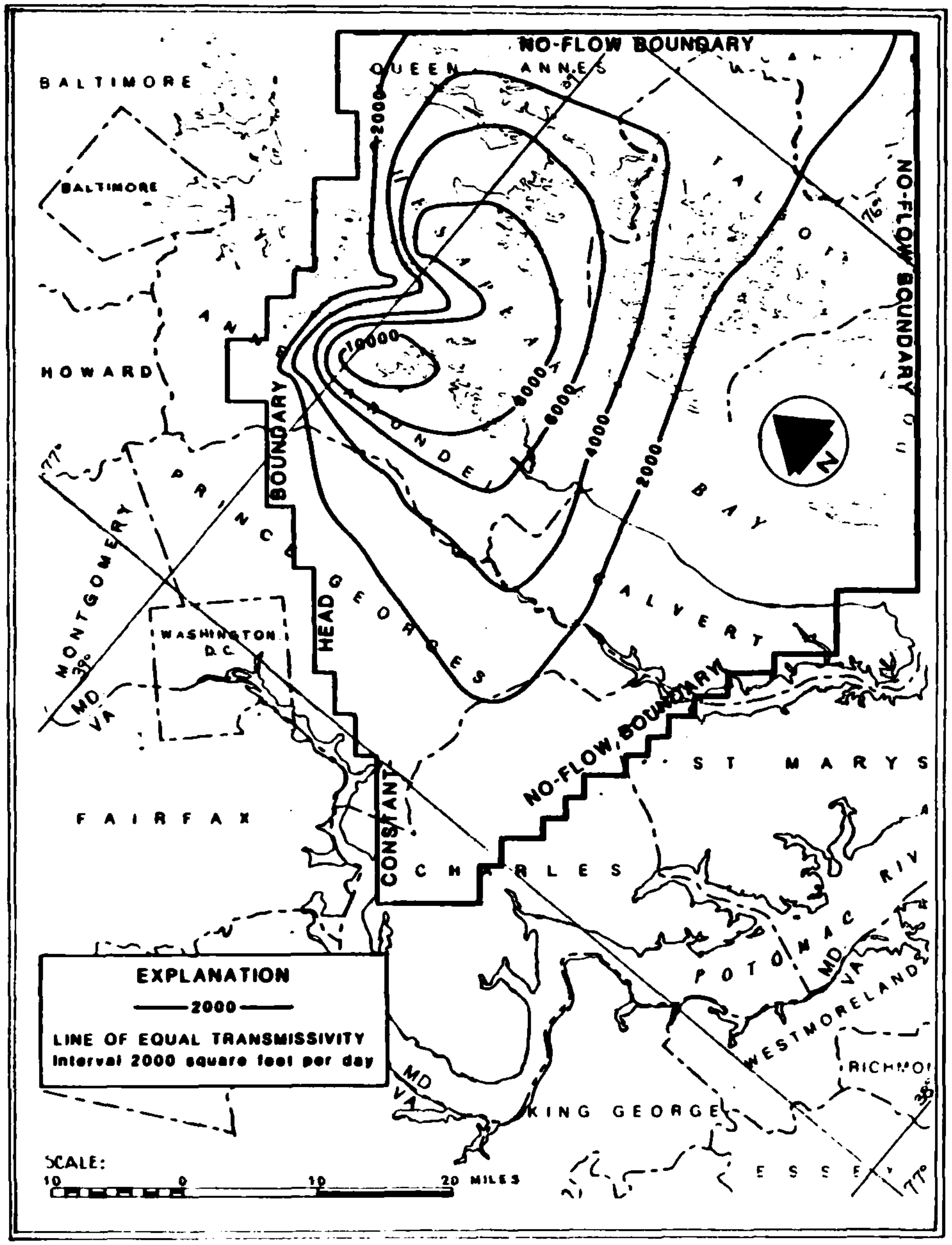


FIGURE F-33

TRANSMISSIVITY DISTRIBUTION IN THE MAGOTHY AQUIFER

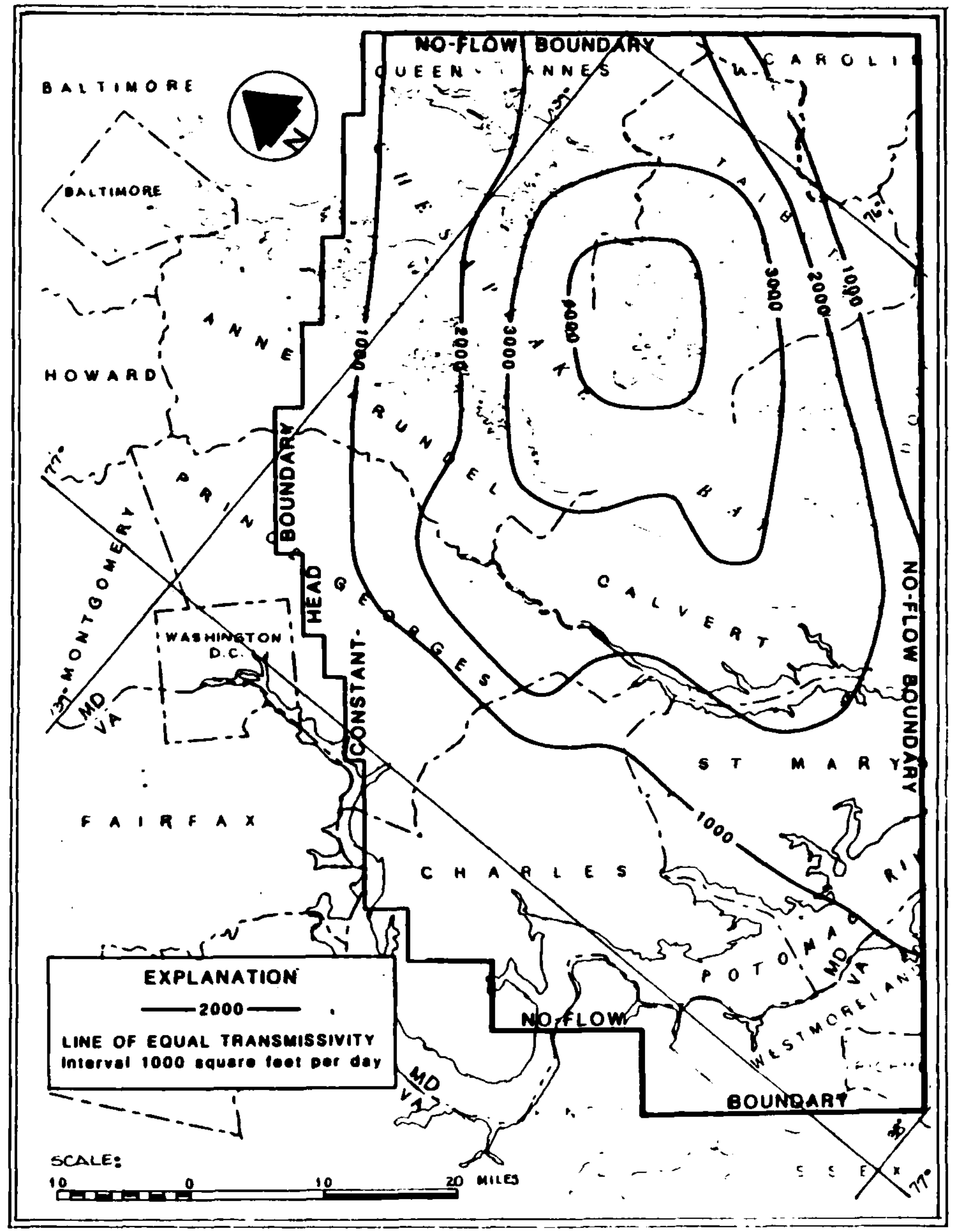
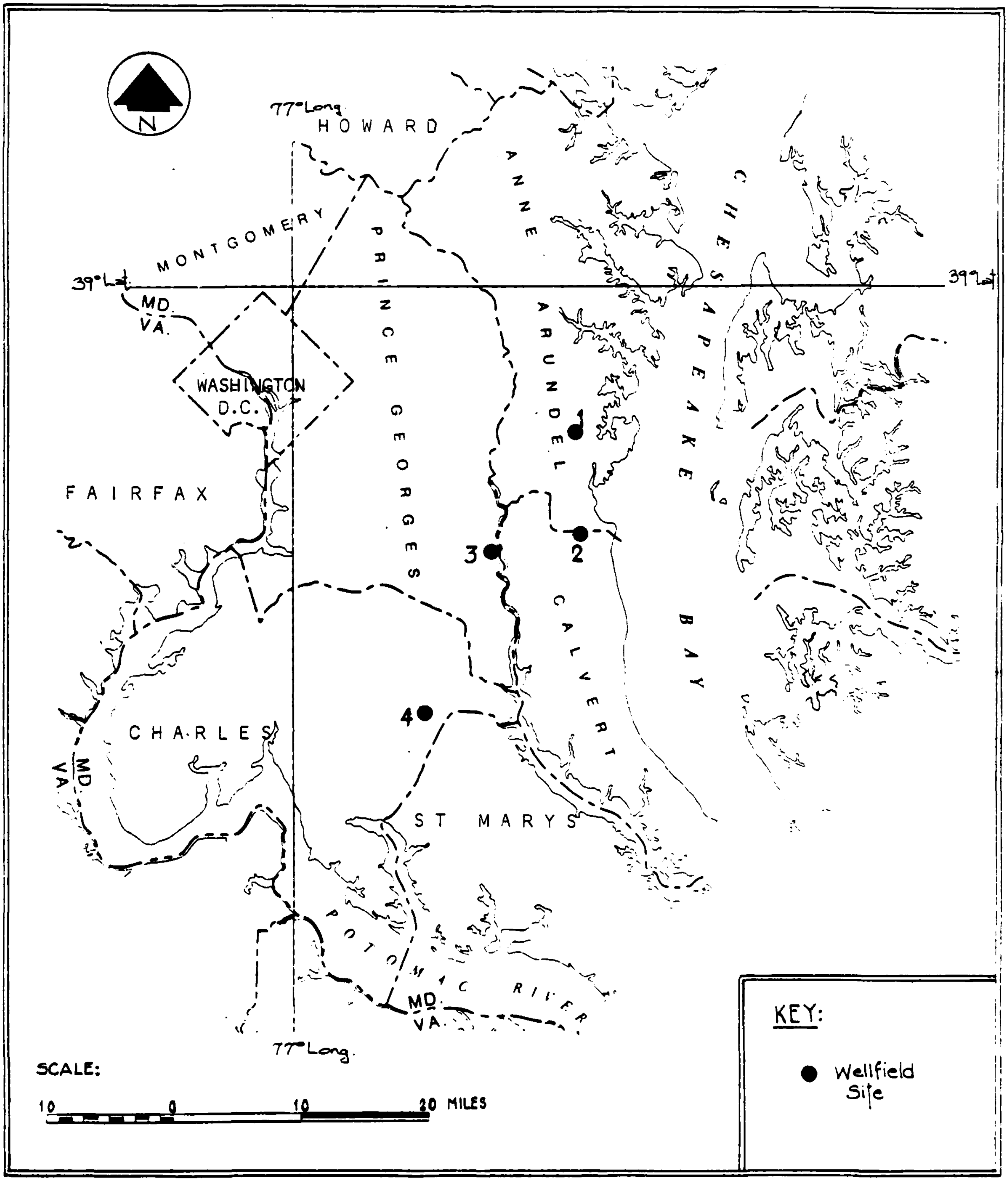


FIGURE F-34

TRANSMISSIVITY DISTRIBUTION IN THE AQUIA AQUIFER

FIGURE F-35

LOCATION OF THE FOUR GROUNDWATER DEVELOPMENT SITES



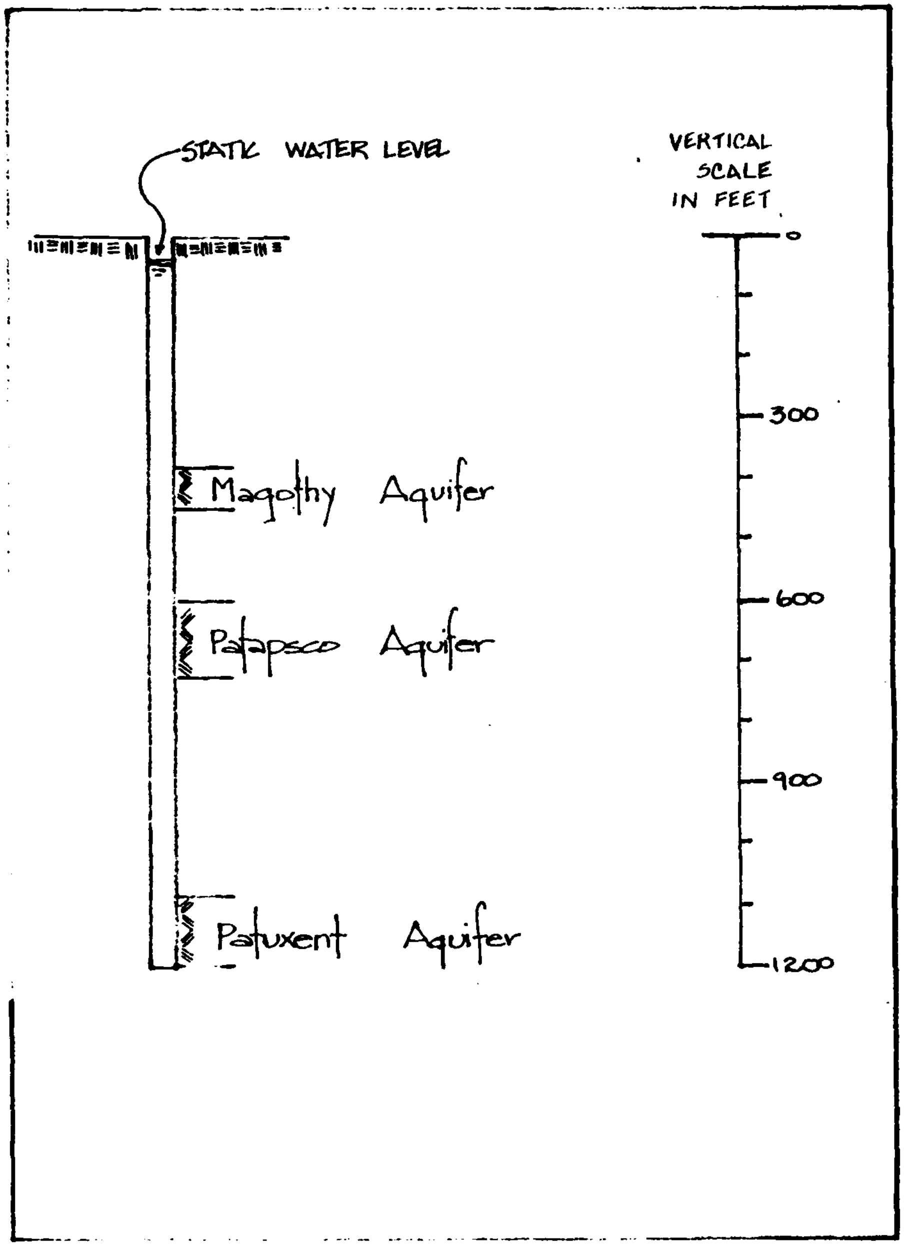


FIGURE F-36

CROSS-SECTION OF SITE I

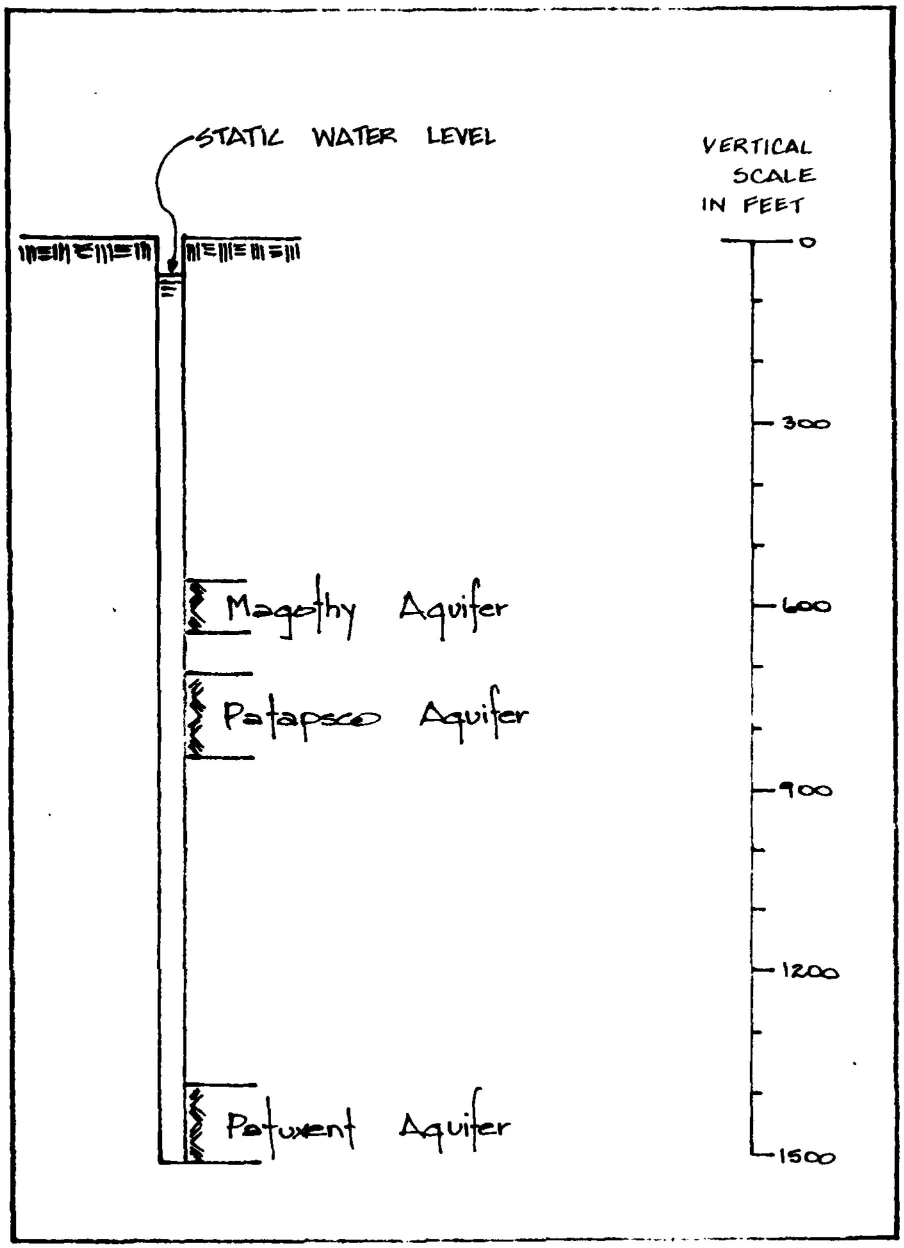


FIGURE F-37

CROSS-SECTION OF SITE 2

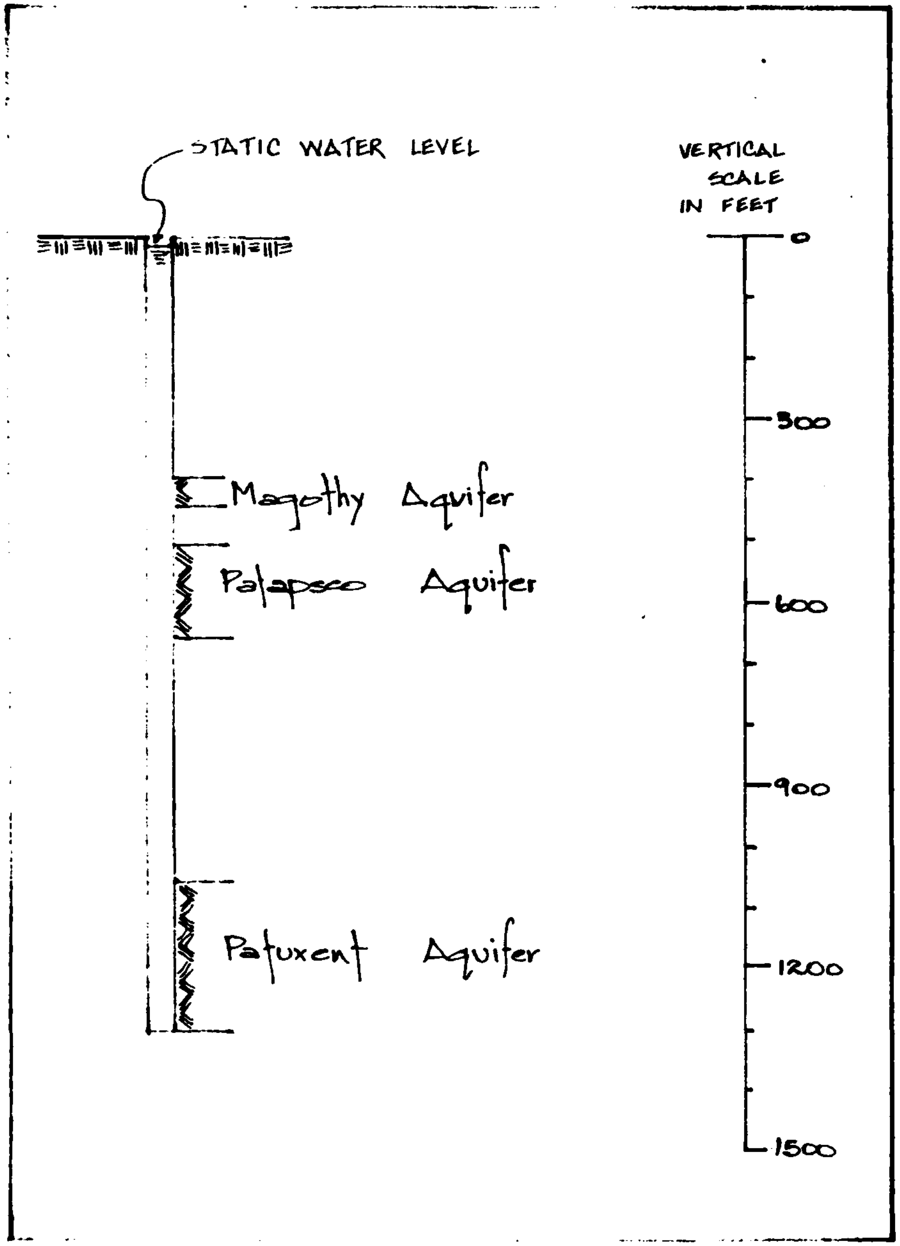


FIGURE F-38

CROSS-SECTION OF SITE 3

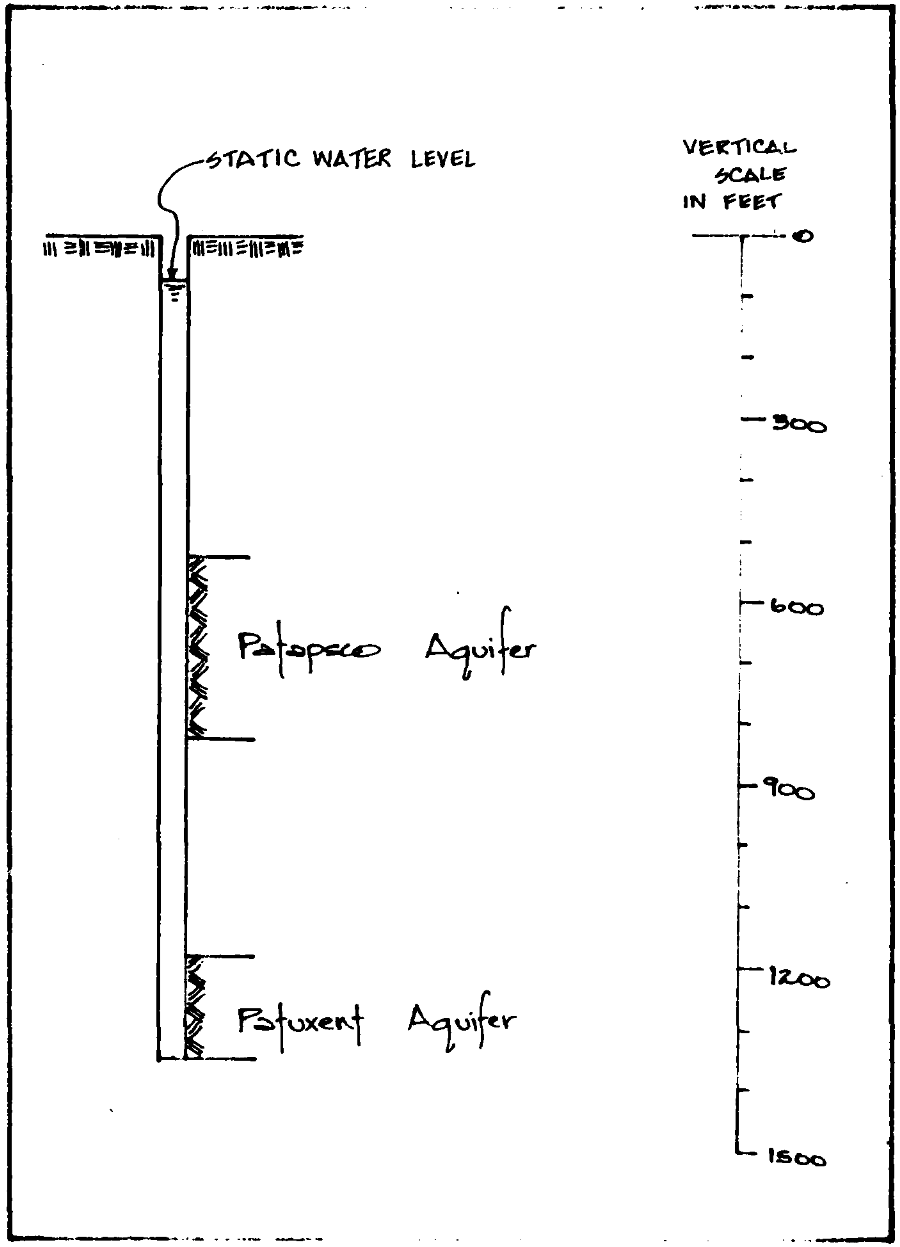


FIGURE F-39

CROSS-SECTION OF SITE 4

For planning purposes, the model simulation tested a two-year drought sequence. During this time, the aquifers were pumped for five months from July through November, then not utilized until the following July, at which time they were pumped for five more months.

Each site was individually simulated (interference effects from other sites were not investigated) for a 10 mgd withdrawal from a single 12-inch diameter well, fully screened for the appropriate aquifer. Then, by using the principle of superposition, the maximum pumping rate for each node was calculated. The maximum pumping rate was defined as that withdrawal which caused the head in the well node to be drawn down to the top of the aquifer. Further drawdown would "dewater" the aquifer and stress the aquifer's recovery capacity. Then using this maximum rate, further simulations were performed to determine average drawdown in the adjacent nodes. The actual head distribution within the node would vary depending on the number and distribution pattern of the wells, the well diameter, the amount of the aquifer screened, and the transmissivity distribution. Within the vicinity of the well, the head gradient would be extremely steep.

A summary of the modelling results is presented in Table F-28. As noted in the table, the Magothy aquifer was not included in the Site 4 analysis since it was not a significant formation at that site.

MODELLING CONCLUSIONS

The groundwater simulations revealed several relationships. Pumping the Magothy aquifer at any of the three available sites produces much smaller drawdowns than the same pumpage from either the Patapsco or the Patuxent aquifers as indicated in Table F-28. However, pumping the Magothy aquifer also results in the greatest drawdown of the overlying Aquia aquifer by a large margin. The model indicated that the recovering time of the Aquia aquifer in the vicinity of the well would be three to six months as shown in Figure F-40 for Site 1. Similar curves would apply for the other three sites.

Of the four sites, Site 1 shows the greatest potential for groundwater development from a supply point of view, followed in order by Site 2, Site 3, and Site 4. Sites 1, 2, and 3 should be able to produce 50 to 60 mgd, individually with proper design and management. However, Site 4 is limited in its potential yield. Groundwater development at that site should be restricted to 20 to 30 mgd total. From the entire Coastal Plain aquifer system, a total yield of 100 mgd is considered a safe limit. Due to the large drawdowns with using only one aquifer, multi-aquifer screening was recommended.

FORMULATION AND DESIGN OF GROUNDWATER ALTERNATIVES

INTRODUCTION

For the groundwater development alternative for the MWA Water Supply Study, a supply scheme was developed to evaluate the Coastal Plain groundwater source's potential for supply to the MWA. This scheme, while somewhat specific in design, was representative of typical potential projects.

TABLE F-28

SUMMARY OF GROUNDWATER MODELLING

Location	Pumped Aquifers	Depth to Bottom of Aquifer (Feet)	Pre-pumping Head Above Top of Aquifer* (Feet)	Unit Drawdown at Well ** (Feet/mgd)	Unit Average Drawdown in Aquia (Feet/mgd)
Site 1	Magothy	450	340	30.9	1.82
	Patapsco	725	570	71.2	0.38
	Patuxent	1200	1000	130.0	0.00
	Magothy, Patapsco	725	340	21.2	1.25
	Magothy, Patapsco, Patuxent	1200	340	18.9	0.83
	Patapsco, Patuxent	1200	570	43.8	0.23
Site 2	Magothy	650	500	50.5	3.00
	Patapsco	850	640	71.7	0.22
	Patuxent	1520	1320	132.0	0.00
	Magothy, Patapsco	850	500	29.7	1.59
	Magothy, Patapsco, Patuxent	1520	500	24.0	1.05
	Patapsco, Patuxent	1520	640	49.6	0.08
Site 3	Magothy	440	390	43.3	1.00
	Patapsco	660	490	98.0	0.20
	Patuxent	1300	1040	173.3	0.00
	Magothy, Patapsco	660	390	39.0	0.60
	Magothy, Patapsco, Patuxent	1300	390	30.0	0.46
	Patapsco, Patuxent	1300	490	61.2	0.12
Site 4	Magothy (not present at this site)				
	Patapsco	825	470	72.9	0.29
	Patuxent	1350	1100	222.0	0.00
	Patapsco, Patuxent	1350	470	52.2	0.11

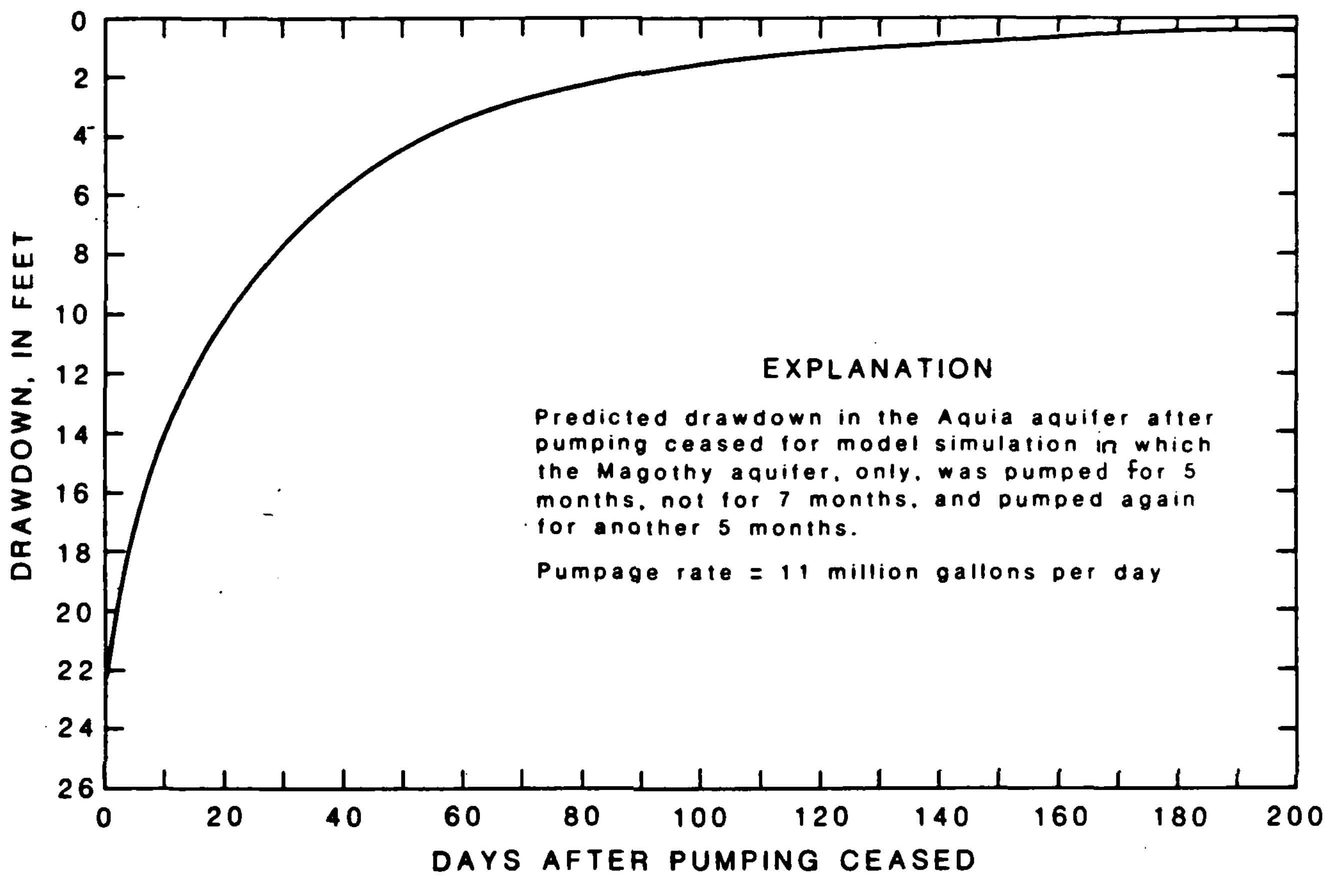
* This value indicates the maximum drawdown at well before "dewatering" of aquifer would occur.

** This value represents the drawdown for a 1 mgd withdrawal, without taking into account any interference effects from other wells.

Source: Digital Simulation of Groundwater Flow in Part of Southern Maryland, USGS, 1982; written communication, Bill Fleck, USGS, 1981.

FIGURE F-40

WATER LEVEL RECOVERY OF THE AQUIA AQUIFER, SITE I



The general groundwater scheme consisted of development of a wellfield at one or more of the four selected sites, pumping the groundwater to a water treatment plant near Upper Marlboro in Prince Georges County, Maryland, and then final transmission of the treated water to the WSSC system via their finished water interconnection near Largo, Maryland. A schematic of the proposed groundwater scheme is drawn in Figure F-41. The groundwater scheme was sized for yields of 25, 50, and 100 mgd total. The assumption was made that this water, up to 100 mgd, would replace other WSSC supplies in the central and southern Prince Georges County areas. An evaluation of the costs and impacts of the various alternatives associated with this scheme follows in the next section.

METHODOLOGY OF DISAGGREGATION INTO COMPONENTS

Since there were four wellfield sites and several possible size combinations to choose from, the groundwater scheme was broken down into four major components for detailed analysis. These components were:

a. wellfields for four sites,

b. transmission mains in a conveyance network,

c. pumping stations in the network, and

d. water treatment facilities.

The components were individually designed for a range of sizes with basic common design parameters, and then the appropriately sized facilities were amassed to form each groundwater alternative for further evaluation. Within the array of alternatives for each supply level (i.e. 50 mgd), there would be an alternative for Site 1 development, an alternative for Site 2 development, and for the remaining combinations. The design and cost evaluation for each individual component is detailed in the next several sections. This is followed by a further evaluation of the alternatives as a sum of their facility components.

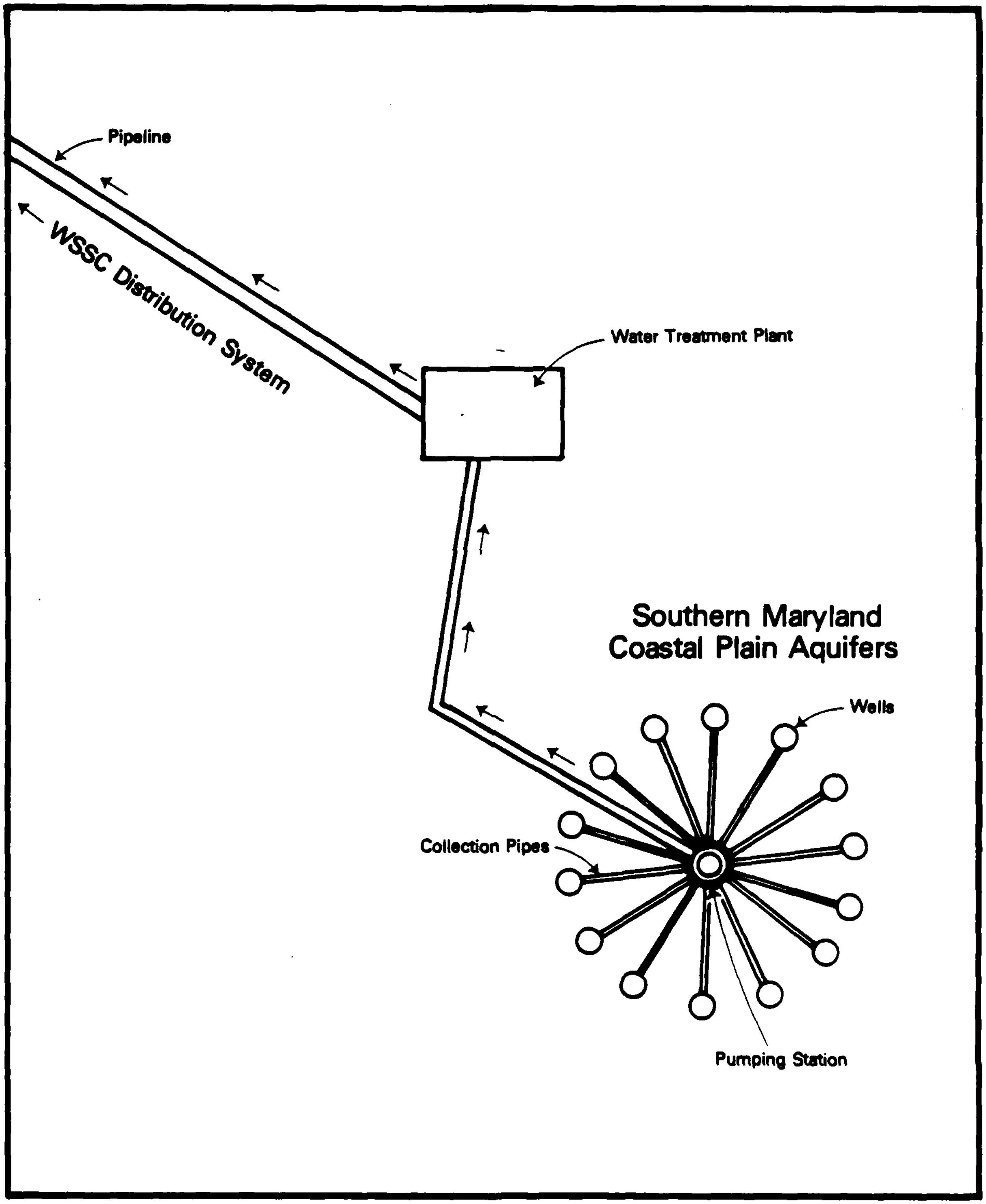
MAPS PROGRAM AND ASSUMPTIONS

The costs for the various components were estimated using the Methodology for Areawide Planning Studies (MAPS) computer program. The MAPS program was developed by the U.S. Army Corps of Engineers' Waterways Experiment Station. It is a generalized planning tool for evaluating water resource alternatives. As such, it provides preliminary design and cost estimates for comparison purposes. These costs should not be utilized as future project estimates, because they do not reflect detailed project planning and site-specific design considerations.

The costs in the MAPS program account for many of the independent variables that normally impact on costs. Consequently, the results are usually more accurate than generalized cost curves available in literature, which are a function of only one or two variables. The MAPS program takes user-specified, engineering design data and applies several cost functions to determine various construction costs and operation and maintenance costs. Itemized construction, total construction, overhead, land, total capital, amortized capital, operation and maintenance, labor, material and supply, power, total operation and maintenance, and average annual costs are provided by the program.

FIGURE F-41

SCHEMATIC OF PROPOSED GROUNDWATER SCHEME



All costs are calculated by the program except for the land cost which is input directly by the user. The costs are based on a set of economic data (user-specified) which for this study reflect October 1981 economic conditions.

The economic data assumed for this study included an Engineering-News Record (ENR) Construction Cost Index of 3672 and a Small City Conventional Treatment (SCCT) Index of 200. The SCCT index reflects municipal wastewater treatment facility costs for 50-mgd plants at various locations in the United States. For the MWA Water Supply Study, indices for Baltimore, Maryland, Philadelphia, Pennsylvania, and the entire United States were considered to yield the value of 200. In addition to these two indices, a power cost of 4.0 cents per kilowatt-hour was assumed. This value reflects the cost of electricity for commercial properties serviced by the Virginia Electric Power Company (VEPCO).

For the amortization calculations, the Federal water supply interest rate of 7.625 percent was assumed. A 50-year payback period was assumed for all amortizations.

All of the above numbers reflect October 1981 values. This economic base was constant throughout the evaluation of components in order to provide a reasonable comparison of the groundwater alternatives to the other MWA water supply alternatives.

WELLFIELD COMPONENT

Each of the four groundwater sites studied earlier were individually evaluated for potential wellfield development for 25 and 50 mgd capacity. These wellfields were eventually combined to form 25, 50, or 100-mgd systems for the MWA. A description of the major design considerations follows.

Since single aquifer screening produced large drawdowns, multi-aquifer screening had been recommended as being yield-efficient; therefore, only multi-aquifer screening was considered in this analysis. For three of the sites, two types of multi-aquifer well screenings were investigated. For Sites 1, 2, and 3, well screening for the Magothy, Patapsco, and Patuxent Formations and for just the Patapsco and the Patuxent Formations were the two options for withdrawals. At Site 4, since the Magothy aquifer was insignificant at this location, only the Patapsco and Patuxent combination was investigated. The selection of which aquifers to screen had an important bearing on the wellfield costs and associated drawdowns in the overlying Aquia aquifer, as will be shown later.

The wellfield was laid out in a radial design with several circles of wells feeding into a central collection point. From there, a large force main would convey the water to the treatment facilities. For the 25-mgd wellfield, 25 wells of 1 mgd capacity were arranged in two concentric circles of 10 and 15 wells with radii of 1600 and 2500 feet, respectively, as diagrammed in Figure F-42. The average radius of the 25-well system was 2100 feet (0.40 miles); this value was used to calculate the costs of connecting pipelines. The wellfield encompassed an area of 0.70 square miles (440 acres). Similarly, for the 50 mgd wellfield, 50 wells of 1 mgd capacity were laid out in three circles with radii of 1600, 2500, and 4000 feet, as diagrammed in Figure F-43. The area and average radius of the wellfield were 1.8 square miles (1150 acres) and 0.57 miles (3050 feet), respectively.

FIGURE F-42

DIAGRAM OF 25-MGD WELLFIELD

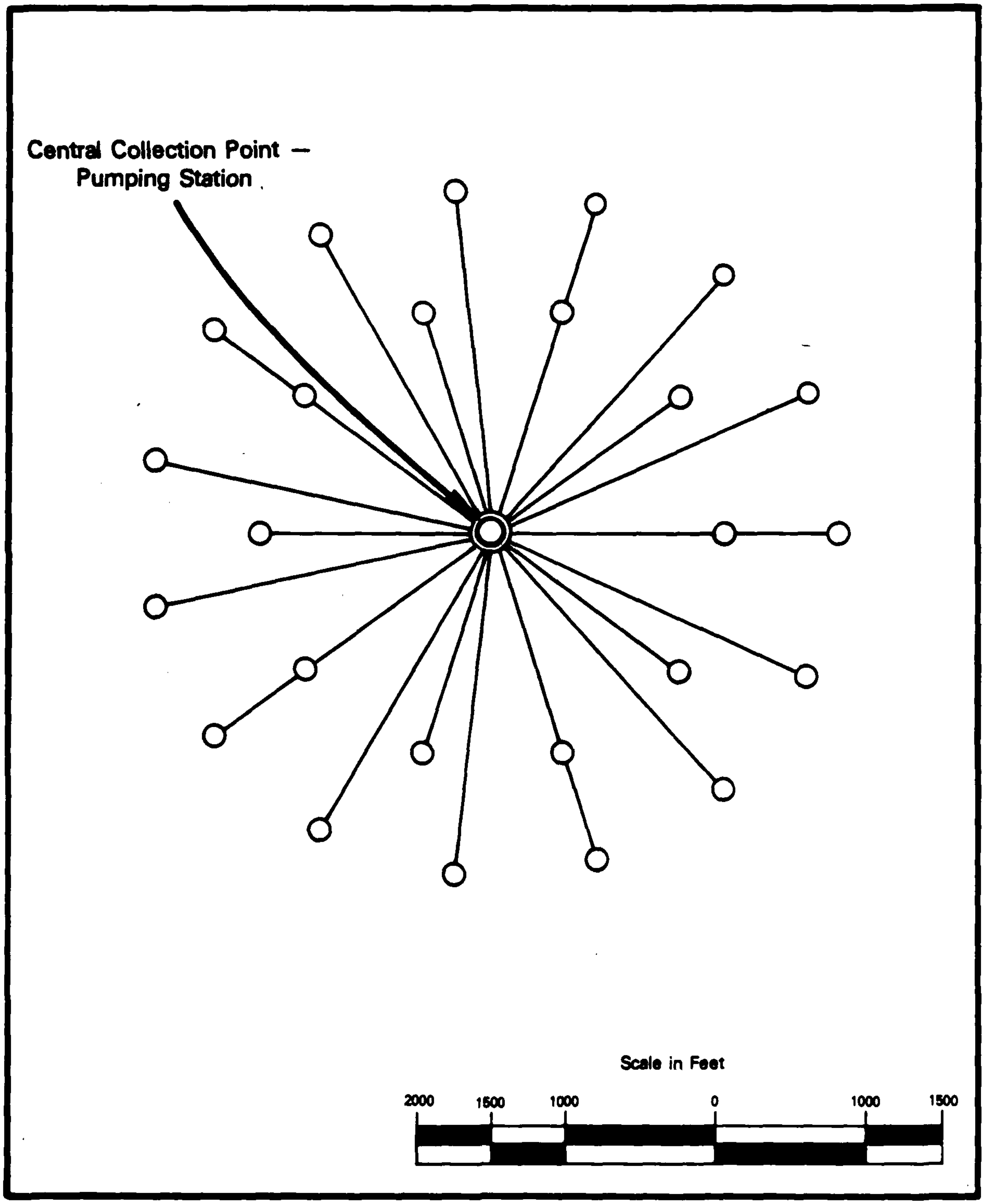
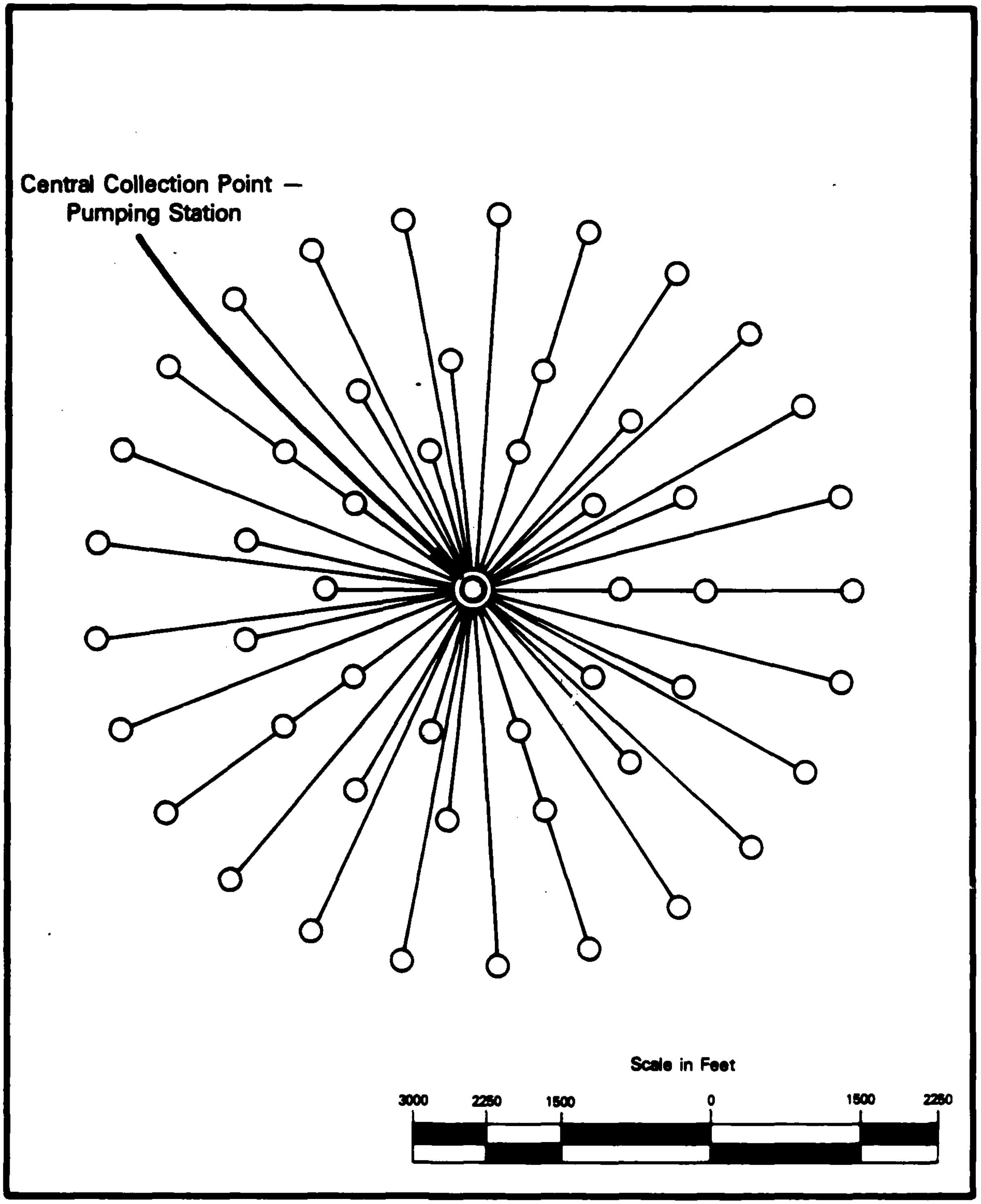


FIGURE F-43

DIAGRAM OF 50-MGD WELLFIELD



For all of the systems, 12-inch diameter wells were specified. Vertical turbine pumps with an efficiency of 80 percent were assumed to provide the requisite pressure head out of the wells. Test wells were included in the cost analysis. Drilling costs of the wells reflected the unconsolidated sediments of the Coastal Plain region. Land needs for the wellfield site included an additional 10 percent for facilities and a protective buffer zone. Land costs were estimated at $4000 per acre for the mainly rural sites.

The wells were drilled to the bottom of the Patuxent aquifer. For Site 1, the well depth was 1200 feet; for Site 2, it was 1520 feet; for Site 3, it was 1300 feet; and for Site 4, the depth was 1350 feet. The groundwater depths from these sites ranged from 20 to 70 feet, as noted earlier in Figures F-31 through F-34. The average well drawdown was also an important design consideration. This value was determined for each site and for the two types of screening. The value was calculated for one well by taking the drawdown at the well for 1 mgd and then adding all of the interference effects for the remaining 24 or 49 wells. Depending on the location of the well, the drawdown varied. Generally, wells in the inner circle experienced the greatest drawdown impacts. The average drawdown for all 25 or all 50 wells was then considered in the design analysis. These values appear in Tables F-29 and F-30. Drawdown calculations for a 50-mgd wellfield at Site 4 confirmed the earlier conclusion that this site could not support more than 30 mgd of withdrawal. The 50-mgd wellfield produced drawdowns exceeding 550 feet; drawdowns of 510 feet or more would dewater the Patapsco aquifer at Site 4.

Using the design data detailed above, the MAPS program calculated capital and annual costs for each of the wellfield sites. The resulting capital and operation and maintenance (O&M) costs are tabulated in Tables F-29 and F-30 for the 25-mgd and the 50-mgd wellfields, respectively.

These costs indicate that there is little difference (4 to 8 percent) between the types of screening in terms of capital costs. However, from an operation point of view, screening for the Magothy aquifer reduced costs significantly. Accompanying this cost savings was a manyfold increase in drawdowns in the top aquifer (the Aquia). This is due to the effective interconnection of groundwater flow between the Magothy and the Aquia aquifers. Drawdowns in the Aquia aquifer at the wellfield site averaged 12 to 26 feet for the 25-mgd scheme, and 23 to 52 feet for the larger 50-mgd scheme. Although the drawdown at the fringe of the wellfield sites, where local wells could be located, would be less than these estimated values, the severity of these drawdowns indicates a significant potential for detrimental impacts on the local water supply. Therefore, in later evaluations, the multi-aquifer screening for the Magothy, Patapsco, and Patuxent Formations was not considered.

TRANSMISSION MAIN COMPONENT

The network of transmission mains was the next component evaluated. The network of mains was divided into pipe segments based on the location of junctions. The various segments and the direction of flow in them are identified in Figure F-44. Pipe routes were chosen so as to follow existing transportation routes as much as possible.

TABLE F-29

25-MGD WELLFIELD SUMMARY*

Aquifer Screening

	Magothy/Patapsco/Patuxent	Patapsco/Patuxent	Cost Diff.
Site 1			
Capital Cost	$4,630,000	$4,840,000	4.5%
O&M Cost	$103,000/year	$173,000/year	6.7%
Drawdown in Aquia at site	21 feet	6 feet	
Average Well Drawdown	95 feet	210 feet	
Site 2			
Capital Cost	$4,840,000	$5,060,000	4.5%
O&M Cost	$136,000/year	$217,000/year	59.9%
Drawdown in Aquia at site	26 feet	2 feet	
Average Well Drawdown	110 feet	250 feet	
Site 3			
Capital Cost	$4,650,000	$4,960,000	6.7%
O&M Cost	$100,000/year	$204,000/year	104%
Drawdown in Aquia at site	12 feet	3 feet	
Average Well Drawdown	100 feet	270 feet	
Site 4			
Capital Cost	Magothy Aquifer	$5,140,000	
O&M Cost	Not at Site	$272,000/year	
Drawdown in Aquia at site		3 feet	
Average Well Drawdown		350 feet	

* Costs are based on October 1981 values, and operation during five months of the year.

TABLE F-30

50-MGD WELLFIELD SUMMARY*

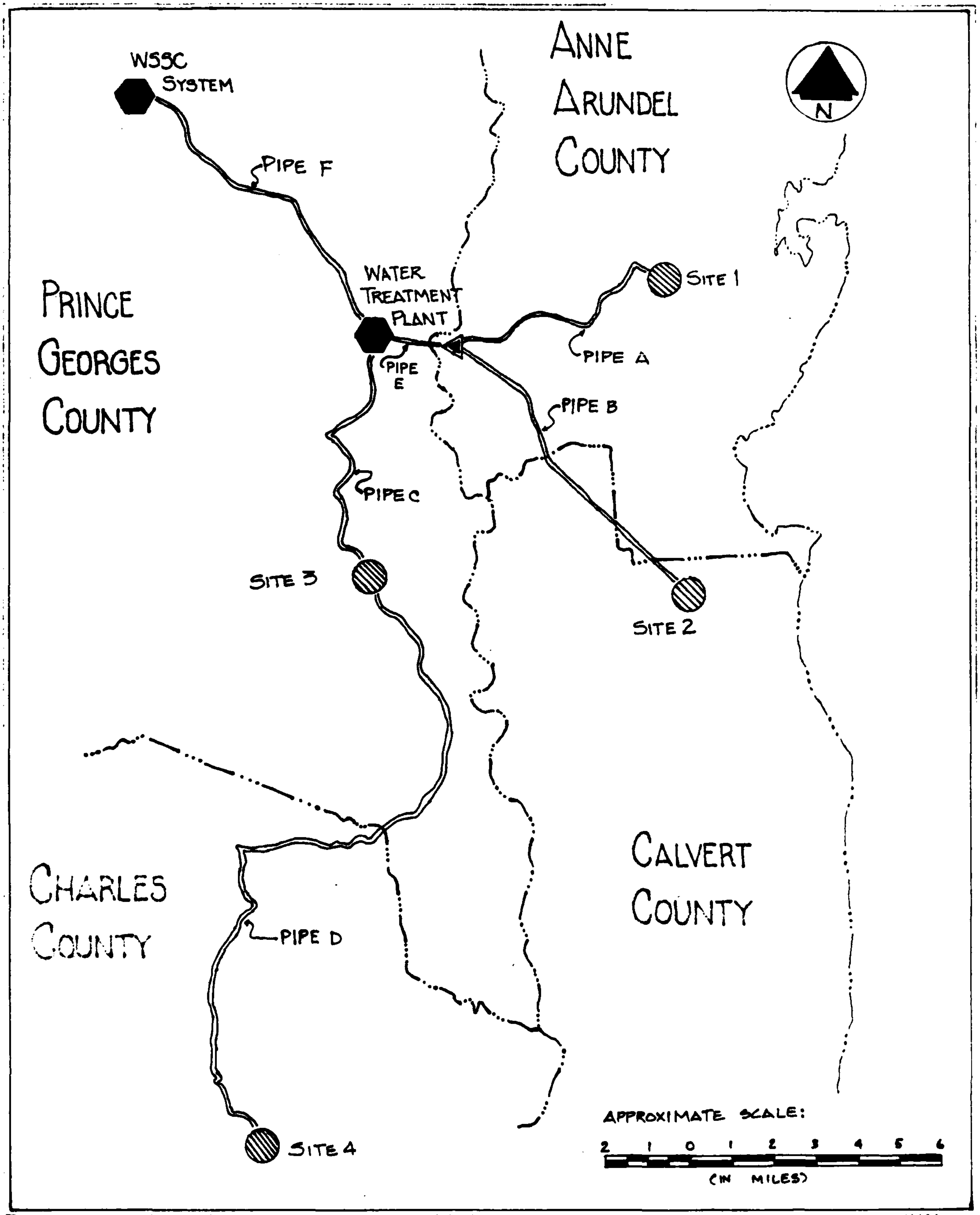
Aquifer Screening

	Magothy/Patapsco/Patuxent	Patapsco/Patuxent	Cost Diff.
Site 1			
Capital Cost	$11,300,000	$11,980,000	6.0%
O&M Cost	$286,000/year	$507,000/year	77.4%
Drawdown in Aquia at site	42 feet	12 feet	
Average Well Drawdown	150 feet	340 feet	
Site 2			
Capital Cost	$11,700,000	$12,300,000	5.1%
O&M Cost	$354,000/year	$595,000/year	68.0%
Drawdown in Aquia at Site	52 feet	4 feet	
Average Well Drawdown	170 feet	380 feet	
Site 3			
Capital Cost	$11,300,000	$12,200,000	8.0%
O&M Cost	$258,000/year	$608,000/year	135%
Drawdown in Aquia at Site	23 feet	6 feet	
Average Well Drawdown	140 feet	430 feet	
Site 4			
Capital Cost	Yield Not Available	Yield Not Available	
O&M Cost	At Site	At Site	
Drawdown in Aquia at Site			
Average Well Drawdown			

* Costs were based on October 1981 values, and operation during five months of the year.

FIGURE F-44

DIAGRAM OF THE CONVEYANCE NETWORK



For the cost analysis, all pipes were considered as force mains, that is, they were designated to flow full. Prestressed concrete cyclinder pipe was selected for the pipe material, this was consistent with raw and finished water interconnections of this size in the MWA. The pipes were assumed to be laid under dry soil conditions with no rock excavation required. The pipe would be laid in a rectangular trench with a depth of 3 feet greater and a width of 1.5 feet greater than the pipe diameter. No concrete cradle was assumed.

There were several characteristics unique to each pipe segment. These identifying data are listed in Table F-31. They include the length of the pipe, the initial elevation, the peak elevation, and the final elevation of the pipe's route. These values were ascertained from the appropriate USGS topographic maps. In addition, the number and type of pipe appurtenances were estimated from the physical layout of the route. The possible appurtenances included gate valves, standard elbows, medium elbows, and long sweep elbows. The percent of terrain-type along the pipe route was also estimated for the cost program. The last identifying item was the cost of right-of-way for each pipe. This value was calculated from the pipe length and a $10 per linear foot estimate of right-of-way costs in the outlying areas of the MWA, where these pipes would generally be located. For Pipe F, the segment which connects into the WSSC system, a final pressure head of 320 feet was assumed. This approximated the system pressure at the point of entry.

The pipe assumptions and identifying data were input to the MAPS program for four levels of flow (25, 50, 75, and 100 mgd average flow; peak flow was two times the average flow) as appropriate. The program determined the capital and operation and maintenance costs, and the velocity and the head requirements at average and peak flows for a range of pipe diameters. The costs included overhead (engineering design, administration, and supervision), materials, excavation, laying the pipe, backfill, valves, and elbows. The pipe velocity associated with varying levels of flow had a direct bearing on the size selection. Since the transmission mains in the network generally carry high quality water, the major velocity constraint is the upper limit where excessive velocity could cause erosion of pipe material or high pressure differentials (i.e. water hammer). The maximum velocity is somewhere between 10 and 20 feet per second (fps) for typical pipe materials, but design velocities are commonly on the order of 4 to 6 fps. For this study, the pipe diameters were selected such that the average velocity was about 3.0 fps and the peak velocity was about 6.0 fps. The selection of pipe diameter also considered the trade-off between larger pipes with lower head requirements versus smaller pipes with greater head requirements, and attempted to determine the most cost-efficient combination of pipes and pumping stations (whose costs are highly head-dependent). The 3.0/6.0 velocity criterion reflects this economic consideration.

The selected pipe and its resultant costs and head requirements are summarized for each pipe segment in Table F-32.

PUMPING STATION COMPONENT

Pumping stations would be required to convey the groundwater supply to the MWA system. For the proposed groundwater scheme, at least two pumping stations would be required, one at a wellfield, and one at the water treatment facility. For schemes involving a combination of wellfields, additional pumping stations would be needed.

TABLE F-31

PIPELINE DESCRIPTION

Variable	Pipe A	Pipe B	Pipe C	Pipe D	Pipe E	Pipe F
Length, feet	35,000	51,000	60,000	105,000	12,000	53,000
Initial Elevation, feet msl	80	90	60	90	40	40
Final Elevation, feet msl	40	40	40	60	40	140
Peak Elevation, feet msl	170	160	240	210	150	200
Peak Station, feet	8,000	39,000	32,000	57,000	7,000	44,000
Final Pressure, feet	0	0	0	0	0	320
Number of Appurtenances						
Gate Valves	1	1	3	6	0	2
Standard Elbows	2	0	4	9	0	2
Long Elbows	5	2	9	12	2	7
Medium Elbows	4	2	8	11	0	5
Cost of Right-of-Way, $	350,000	510,000	600,000	1,050,000	120,000	530,000
Terrain Distribution, Percent						
Commercial	0	0	0	0	0	10
Dense Residential	0	0	10	0	0	30
Sparse Residential	50	50	40	50	50	40
Open Country	50	50	50	50	50	20

TABLE F-32

PIPELINE SUMMARY*

	Average Flow, Mgd			
	25	50	75	100
Pipe A - Site 1 to 1-2 Intersection				
Capital Cost, $	$6,060,000	$10,600,000	-	-
O&M Cost, $/Year	10,900	18,200	-	-
Pipe Size, Inches	48	66	-	-
Head Required, Peak Flow, Feet	112	107	-	-
Head Required, Average Flow, Feet	0	0	-	-
Pipe B - Site 2 to 1-2 Intersection				
Capital Cost, $	8,730,000	15,200,000	-	-
O&M Cost, $/Year	15,600	26,100	-	-
Pipe Size, Inches	48	66	-	-
Head Required, Peak Flow, Feet	175	150	-	-
Head Required, Average Flow, Feet	0	0	-	-
Pipe C - Site 3 to WTP				
Capital Cost, $	10,700,000	18,700,000	29,100,000	-
O&M Cost, $/Year	19,100	32,000	48,600	-
Pipe Size, Inches	48	66	84	-
Head Required, Peak Flow, Feet	270	250	226	-
Head Required, Average Flow, Feet	23	14	2	-
Pipe D - Site 4 to Site 3				
Capital Cost, $	18,300,000	-	-	-
O&M Cost, $/Year	32,700	-	-	-
Pipe Size, Inches	48	-	-	-
Head Required, Peak Flow, Feet	280	-	-	-
Head Required, Average Flow, Feet	45	-	-	-
Pipe E - 1-2 Intersection to WTP				
Capital Cost, $	2,440,000	3,970,000	5,970,000	7,560,000
O&M Cost, $/Year	4,400	6,800	10,000	12,500
Pipe Size, Inches	48	66	84	96
Head Required, Peak Flow, Feet	129	125	119	118
Head Required, Average Flow, Feet	8	6	4	4
Pipe F - WTP to WSSC System				
Capital Cost, $	10,800,000	19,000,000	-	38,200,000
O&M Cost, $/Year	19,400	32,600	-	63,100
Pipe Size, Inches	48	66	-	96
Head Required, Peak Flow, Feet	567	534	-	487
Head Required, Average Flow, Feet	458	449	-	437

* The specific pipe segment was sized for flows as needed for the final evaluation; therefore, all flow levels were not investigated for each segment. Costs represent October 1981 values.

For the component analysis, pumping stations were designed and sized for each wellfield site and for the water treatment plant. For Sites 1, 2, and 3, pumping stations with 25 and 50-mgd capacity were designed. Site 4 was only considered for a 25-mgd station. The pumping station at the water treatment plant (WTP) was sized for 25, 50, and 100-mgd capacity.

For each pumping station, the requisite design heads, peak and average, were obtained from the transmission main analysis. For the Site 1 and Site 2 stations, two pipe segments were involved, so the appropriate pipe heads were combined to estimate the total head requirement. The design head pressures are listed in Table F-33. For each station, the design provided a minimum of two pumps - one to be used as a backup. The efficiency of the pumps was set at 80 percent. The pumping station was considered operational for five months during the year (downtime equal to seven months). Land for the pumping stations was assumed to be available at the facility - the wellfield or the treatment plant. Therefore, no cost for land was included. A wet well was provided at the WTP pumping station.

Using these design considerations, the station costs were evaluated by the MAPS computer program, and are tabulated in Table F-33.

WATER TREATMENT COMPONENT

The groundwater supply in the study area would most likely require some treatment prior to use in the MWA system, although the water quality data indicated that the Coastal Plain aquifers are of high quality. To estimate the costs of a treatment plant for groundwater supply alternatives, a similar neighboring facility was chosen as a design example.

The closest, large plant treating groundwater is located in Indian Head, Maryland, in Charles County, and is fed by wells in the Patapsco aquifer. The basic processes included in this plant are pre-aeration, prechlorination, filtration, iron removal, and disinfection. To cost a similar treatment plant via MAPS analysis, the major treatment processes of chlorination and filtration were highlighted.

For the groundwater scheme, the cost analysis of the chlorination treatment assumed cylinder storage of the chlorine gas and a dosage of 50 lbs of chlorine per 1 million gallons of water. This treatment dosage is equivalent to 6.0 mg/l which should be more than sufficient to destroy all bacteria and leave an adequate residual. For the filtration process, the selected design consisted of a rapid sand filter. The filter was designed as a gravity-type filter with a loading rate of 5 gpm per square foot and a backwash pumping rate of 5,000 gpm. Surface washes and filter backwashes were assumed to occur twice a day. The land cost for the treatment plant was set at $10,000, which allowed for the purchase of 2.5 acres. Using these design specifications, the capital and operation and maintenance costs were generated by MAPS. The estimated costs are provided in Table F-34.

TABLE F-33

PUMPING STATION SUMMARY

	Average Flow, Mgd		
	25	50	100
Site 1 Pumping Station			
Capital Cost, $	2,610,000	4,220,000	-
O&M Cost, $/Year	22,400	38,300	-
Design Head, Peak Flow, Feet	241	232	-
Design Head, Average Flow, Feet	8	6	
Site 2 Pumping Station			
Capital Cost, $	2,840,000	4,510,000	-
O&M Cost, $/Year	22,400	38,300	-
Design Head, Peak Flow, Feet	304	275	-
Design Head, Average Flow, Feet	8	6	-
Site 3 Pumping Station			
Capital Cost, $	2,720,000	4,340,000	-
O&M Cost, $/Year	31,400	47,900	-
Design Head, Peak Flow, Feet	270	250	-
Design Head, Average Flow, Feet	23	14	-
Site 4 Pumping Station			
Capital Cost, $	3,580	-	-
O&M Cost, $/Year	58,400	-	-
Design Head, Peak Flow, Feet	550	-	-
Design Head, Average Flow, Feet	68	-	-
WTP Pumping Station			
Capital Cost, $	3,760,000	6,140,000	10,700,000
O&M Cost, $/Year	292,000	570,000	1,100,000
Design Head, Peak Flow, Feet	568	534	487
Design Head, Average Flow, Feet	458	449	437

TABLE F-34

GROUNDWATER TREATMENT PLANT COSTS*

	Average Flow, Mgd		
	25	50	100
CAPITAL COSTS			
Chlorination	$94,000	128,000	175,000
Filtration	2,348,000	3,858,000	6,409,000
Construction Total	2,442,000	3,986,000	6,584,000
Interface Piping, Sitework (5%)	122,000	199,000	329,000
Subtotal	2,564,000	4,185,000	6,913,000
Engineering (10%)	256,000	419,000	691,000
Profit (10%)	256,000	419,000	691,000
Land	10,000	10,000	10,000
Administrative, Legal	35,000	41,000	52,000
Subtotal	3,121,000	5,074,000	8,357,000
Interest During Construction	205,000	383,000	725,000
TOTAL	$3,326,000	$5,457,000	$9,082,000
ANNUAL COSTS			
Amortized Capital Costs	260,000	427,000	711,000
Operation and Maintenance Costs			
Chlorination	17,000	24,000	36,000
Filtration	174,000	315,000	572,000
TOTAL	191,000	340,000	608,000

* Costs are based on October 1981 price levels, an interest rate of 7.625 percent, and operating during 5 months of the year. Summations may not agree with totals due to rounding.

EVALUATION OF GROUNDWATER ALTERNATIVES

ECONOMIC SUMMARY

As the last part of the groundwater analysis, the various components were grouped to form alternatives for 25, 50, and 100-mgd supplies. These alternatives and their costs are tabulated in Tables F-35 through F-40. For the 50 and 100-mgd evaluation, all possible combinations are not listed since the 25-mgd and 50-mgd analysis indicated that certain sites (notably Site 4) were less advantageous than others.

From an economic perspective, Site 1 appeared to be the most likely candidate for construction for both the 25-mgd and the 50-mgd scheme. However, Sites 2 and 3 were not significantly more expensive. For the 100-mgd scheme, the combination of Site 1 and Site 2 proved to be most cost-effective.

ENVIRONMENTAL IMPACTS

The environmental effects associated with a groundwater system can be divided into two categories: (1) impacts due to construction and (2) impacts due to operation. Impacts during the construction phase deal mainly with the development of the wellfield and the construction of the pipeline and the water treatment plant. The impacts associated with the operational phase are mainly due to the reduction of head in the aquifer.

Construction

In the development of a groundwater system, the most obvious impact would be the alteration of the land use within the wellfield. The extent of this impact would largely depend on the type of land use associated with the wellfield at the time of development. In order to identify and measure the likely impact, an assessment of existing land use was made using 1:24,000 aerial photography and existing reports and publications. In addition the areas were examined for their cultural sensitivity. Table F-41 presents the information that was developed from these sources. Generally, all four wellfields are characterized as being agricultural/forested tracts located in stream valleys in rural parts of Southern Maryland. The major impact associated with the development of the wellfield would be the elimination of agricultural activities within the tract and the disruption and possible elimination of the bottomland hardwood habitat.

The water treatment plant for the groundwater scheme would be located on a small commercial tract of land near the intersection of Routes 301 and 4. Impacts to the environment would be minimal for the WTP development at this site.

TABLE F-35

SUMMARY OF 25-MGD ALTERNATIVES
CAPITAL COSTS*

Component	Site 1	Site 2	Site 3	Site 4
Wellfield at Site 1	$4,840,000	---	---	---
Wellfield at Site 2	---	$5,060,000	---	---
Wellfield at Site 3	---	---	$4,960,000	---
Wellfield at Site 4	---	---	---	$5,140,000
Water Treatment Plant	$3,320,000	$3,320,000	$3,320,000	$3,320,000
Pipe A	$6,060,000	---	---	---
Pipe B	---	$8,730,000	---	---
Pipe C	---	---	$10,700,000	$10,700,000
Pipe D	---	---	---	$18,300,000
Pipe E	$2,440,000	$2,440,000	---	---
Pipe F	$10,800,000	$10,800,000	$10,800,000	$10,800,000
Pumping Station at Site 1	$2,610,000	---	---	---
Pumping Station at Site 2	---	$2,840,000	---	---
Pumping Station at Site 3	---	---	$2,720,000	---
Pumping Station at Site 4	---	---	---	$3,580,000
Pumping Station at WTP	$3,760,000	$3,760,000	$3,760,000	$3,760,000
TOTAL	$33,830,000	$36,950,000	$36,260,000	$55,600,000
Total Cost per Mgd	$1,350,000	$1,480,000	$1,450,000	$2,220,000
Total Cost per MG, 150-Day Supply	$9,000	$9,900	$9,700	$14,800

* Costs are based on October 1981 price levels, and screening of the Patapsco and Patuxent aquifers.

TABLE F-36

SUMMARY OF 25-MGD ALTERNATIVES
OPERATION AND MAINTENANCE COSTS*
$/YEAR

Component	Site 1	Site 2	Site 3	Site 4
Wellfield at Site 1	$173,000	—	—	—
Wellfield at Site 2	—	$217,000	—	—
Wellfield at Site 3	—	—	$204,000	—
Wellfield at Site 4	—	—	—	$272,000
Water Treatment Plant	$191,000	$191,000	$191,000	$191,000
Pipe A	10,900	—	—	—
Pipe B	—	$15,600	—	—
Pipe C	—	—	$19,100	$19,100
Pipe D	—	—	—	$32,700
Pipe E	$4,400	$4,000	—	—
Pipe F	$19,400	$19,400	$19,400	$19,400
Pumping Station at Site 1	$22,400	—	—	—
Pumping Station at Site 2	—	$22,400	—	—
Pumping Station at Site 3	—	—	$31,400	—
Pumping Station at Site 4	—	—	—	$58,400
Pumping Station at WTP	$292,000	$292,000	$292,000	$292,000
TOTAL	$713,000	$762,000	$757,000	$882,000
Total Cost per Mgd	$29,000	$30,000	$30,000	$350
Total Cost per Mgd	$1,350,000	$1,480,000	$1,450,000	$2,220,000
Total Cost per MG, 150-Day Supply	$9,000	$9,900	$9,700	$14,800

* Costs are based on October 1981 price levels, and screening of the Patapsco and Patuxent aquifers.

TABLE F-37

SUMMARY OF 50-MGD SCHEMES
CAPITAL COSTS*

Component	Site 1	Site 2	Site 3	Site 1 (25) and Site 2 (25)	Site 1 (25) and Site 3 (25)
Wellfield at Site 1	$11,980,000	—	—	$4,840,000	$4,840,000
Wellfield at Site 2	—	$12,300,000	—	$5,060,000	—
Wellfield at Site 3	—	—	$12,200,000	—	$4,960,000
Wellfield at Site 4	—	—	—	—	—
Water Treatment Plant	$5,460,000	$5,460,000	$5,460,000	$5,460,000	$5,460,000
Pipe A	$10,600,000	—	—	$6,060,000	$6,060,000
Pipe B	—	$15,200,000	—	$8,730,000	—
Pipe C	—	—	$18,700,000	—	$10,700,000
Pipe D	—	—	—	—	—
Pipe E	$3,970,000	$3,970,000	—	$3,970,000	$2,440,000
Pipe F	$19,000,000	$19,000,000	$19,000,000	$19,000,000	$19,000,000
Pumping Station at Site 1	$4,220,000	—	—	$2,610,000	$2,610,000
Pumping Station at Site 2	—	$4,510,000	—	$2,840,000	—
Pumping Station at Site 3	—	—	$4,340,000	—	$2,720,000
Pumping Station at Site 4	—	—	—	—	—
Pumping Station at WTP	$6,140,000	$6,140,000	$6,140,000	$6,140,000	$6,140,000
TOTAL	$61,370,000	$66,580,000	$65,840,000	$64,710,000	$64,930,000
Total Cost per Mgd	$1,230,000	$1,330,000	$1,320,000	$1,290,000	$1,300,000
Total Cost per MG, 150-Day Supply	$8,200	$8,900	$8,800	$8,600	$8,700

* Costs are based on October 1981 price levels, and screening of the Patapsco and Patuxent aquifers.

TABLE F-38

SUMMARY OF 50-MGD ALTERNATIVES
OPERATION AND MAINTENANCE COSTS*
$/YEAR

Component	Site 1	Site 2	Site 3	Site 1 and Site 2	Site 1 and Site 3
Wellfield at Site 1	$507,000	—	—	$173,000	$173,000
Wellfield at Site 2	—	$595,000	—	$217,000	—
Wellfield at Site 3	—	—	$608,000	—	$204,000
Wellfield at Site 4	—	—	—	—	—
Water Treatment Plant	$340,000	$340,000	$340,000	$340,000	$340,000
Pipe A	$18,200	—	—	$10,900	$10,900
Pipe B	—	$26,100	—	$15,600	—
Pipe C	—	—	$32,000	—	$19,100
Pipe D	—	—	—	—	—
Pipe E	$6,800	$6,800	—	$6,800	$4,400
Pipe F	$32,600	$32,600	$32,600	$32,600	$32,600
Pumping Station at Site 1	$38,300	—	—	$22,400	$22,400
Pumping Station at Site 2	—	$38,300	—	$22,400	—
Pumping Station at Site 3	—	—	$47,900	—	$31,400
Pumping Station at Site 4	—	—	—	—	—
Pumping Station at WTP	$570,000	$570,000	$570,000	$570,000	$570,000
TOTAL	$1,513,000	$1,609,000	$1,630,000	$1,411,000	$1,408,000
Total Cost per Mgd	$30,000	$32,000	$33,000	$28,000	$28,000
Total Cost per MG, 150-Day Supply	$200	$210	$220	$190	$190

* Costs are based on October 1981 price levels, wellfield operation during five months of the year, and screening of the Patapsco and Patuxent aquifers.

TABLE F-39

SUMMARY OF 100-MGD ALTERNATIVES
CAPITAL COSTS*

Component	Site 1 and Site 2	Site 2 and Site 3	Site 1 and Site 3	Site 1 (50) Site 2 (25), and Site 3 (25)
Wellfield at Site 1	$11,980,000	—	$11,980,000	$11,980,000
Wellfield at Site 2	$12,300,000	$12,300,000	—	$5,060,000
Wellfield at Site 3	—	$12,200,000	$12,200,000	$4,960,000
Wellfield at Site 4	—	—	—	—
Water Treatment Plant	$9,080,000	$9,080,000	$9,080,000	$9,080,000
Pipe A	$10,600,000	—	$10,600,000	$10,600,000
Pipe B	$15,200,000	$15,200,000	—	$8,730,000
Pipe C	—	$18,700,000	$18,700,000	$10,700,000
Pipe D	—	—	—	—
Pipe E	$7,560,000	$2,440,000	$2,440,000	$5,970,000
Pipe F	$38,200,000	$38,200,000	$38,200,000	$38,200,000
Pumping Station at Site 1	$4,220,000	—	$4,220,000	$4,220,000
Pumping Station at Site 2	$4,510,000	$4,510,000	—	$2,840,000
Pumping Station at Site 3	—	$4,340,000	$4,340,000	$2,720,000
Pumping Station at Site 4	—	—	—	—
Pumping Station at WTP	$10,700,000	$10,700,000	$10,700,000	$10,700,000
TOTAL	$124,350,000	$127,670,000	$122,460,000	$125,760,000
Total Cost per Mgd	$1,240,000	$1,280,000	$1,220,000	$1,260,000
Total Cost per MG, 150-Day Supply	$8,300	$8,500	$8,200	$8,400

* Costs are based on October 1981 price levels, and screening of the Patapsco and Patuxent aquifers.

TABLE F-40

SUMMARY OF 100-MGD ALTERNATIVES
OPERATION AND MAINTENANCE COSTS *
$/YEAR

Component	Site 1 and Site 2	Site 2 and Site 3	Site 1 and Site 3	Site 1 Site 2 and Site 3
Wellfield at Site 1	$507,000	—	$507,000	$507,000
Wellfield at Site 2	$595,000	$595,000	—	$217,000
Wellfield at Site 3	—	$608,000	$608,000	$204,000
Wellfield at Site 4	—	—	—	—
Water Treatment Plant	$608,000	$608,000	$608,000	$608,000
Pipe A	$18,200	—	$18,200	$18,200
Pipe B	$26,100	$26,100	—	$15,600
Pipe C	—	$32,000	$32,000	$19,100
Pipe D	—	—	—	—
Pipe E	$12,500	$6,800	$6,800	$10,000
Pipe F	$63,100	$63,100	$63,100	$63,100
Pumping Station at Site 1	$38,300	—	$38,300	$38,300
Pumping Station at Site 2	$38,300	$38,300	—	$22,400
Pumping Station at Site 3	—	$47,900	$47,900	$31,400
Pumping Station at Site 4	—	—	—	—
Pumping Station at WTP	$1,100,000	$1,100,000	$1,100,000	$1,100,000
TOTAL	$3,006,000	$3,125,000	$3,029,000	$2,854,000
Total Cost per Mgd	$30,000	$31,000	$30,000	$29,000
Total Cost per MG, 150-Day Supply	$200	$210	$200	$190

* Costs are based on October 1981 price levels, wellfield operation during five months of the year, and screening of the Patapsco and Patuxent aquifers.

TABLE F-41

SUMMARY OF ENVIRONMENTAL DATA
FOR THE GROUNDWATER SITES

Site	Watershed	Acres	Land Use (%) Agricultural	Wooded	Residential	Commercial	Cultural Sensitivity	Notes
1	Tracys Creek	1250	35	65	0	0	Medium	
2	Hall Creek	1250	70	25	5	0	Medium	
3	Spice Creek	1250	60	40	0	0	Medium	Part of wellfield has been designated as a future environmental study area due to the valuable tidal marsh-lands, extensive hardwood swamps and upland forest/agricultural areas.
4	Gilbert Creek	450	60	40	0	0	Medium	
Water Treatment Plant		2	0	0	0	100	Low	

In terms of cultural resources, all four sites are located in small tributary basins. These locations have a good potential for cultural resources since prehistoric peoples usually located along these small meandering streams.

Due to groundwater scheme's adherence to the highway rights-of-way, the pipeline would have a minimal environmental impact since wildlife trails likely would already reflect adjustments made for the highway. However, during the pipeline construction there would be localized short-term traffic disruptions. No cultural resources impacts would be expected since the area is already disturbed.

Operation

The environmental effects due to the operation of the wellfield would largely relate to the resulting drawdown in the aquifers. Possible effects include land subsidence, salt-water intrusion, surface water effects, and effects on existing groundwater users. Based on the pumping rates and recovery time, it does not appear that the drawdowns would be significant enough to cause any problems in the top aquifer (the Aquia) used by the local groundwater withdrawals. Drawdown impacts in the deep aquifers for the three levels of withdrawals are depicted in Figures F-45 through F-47. Since local withdrawals are not from these aquifers, these drawdowns should not impact the local water supply situation significantly. However, further pursuit of this alternative would require more detailed analyses of the drawdown impacts.

SUMMARY

The environmental analysis for this alternative indicated that the four groundwater sites have very similar land use patterns and potential environmental impacts. And although the habitat in the vicinity of Site 3 has been designated as a future environmental study area, the proper design and siting of a future wellfield in this area could minimize impacts to this sensitive environment. Therefore, there would be no preference for any of the groundwater sites from an environmental and cultural perspective.

In economic terms, Site 4 clearly showed a disadvantage. Due to its considerable distance (relative to the other sites) from the MWA, the Site 4 construction costs were significantly higher, at least 50 percent, than the other three sites. In addition, its limited capacity would preclude development beyond 25 or 30 mgd of supply. Of the remaining three sites, the cost analysis for the groundwater schemes for both 25 and 50-mgd production indicated that Site 1 was the most cost-effective. However, the costs for development of Sites 2 and 3 were only 7 to 9 percent higher; further consideration of this alternative should consider these sites, as well. For the 100-mgd alternative, the combination of Sites 1 and 3 proved to be the most economical project at a total cost of $122,460,000.

In general, the groundwater alternative was estimated to cost approximately $1.2 million to $1.3 million per mgd of supply.

FIGURE F-45

DIAGRAM OF DEEP AQUIFER DRAWDOWN WITH 25-MGD ALTERNATIVES

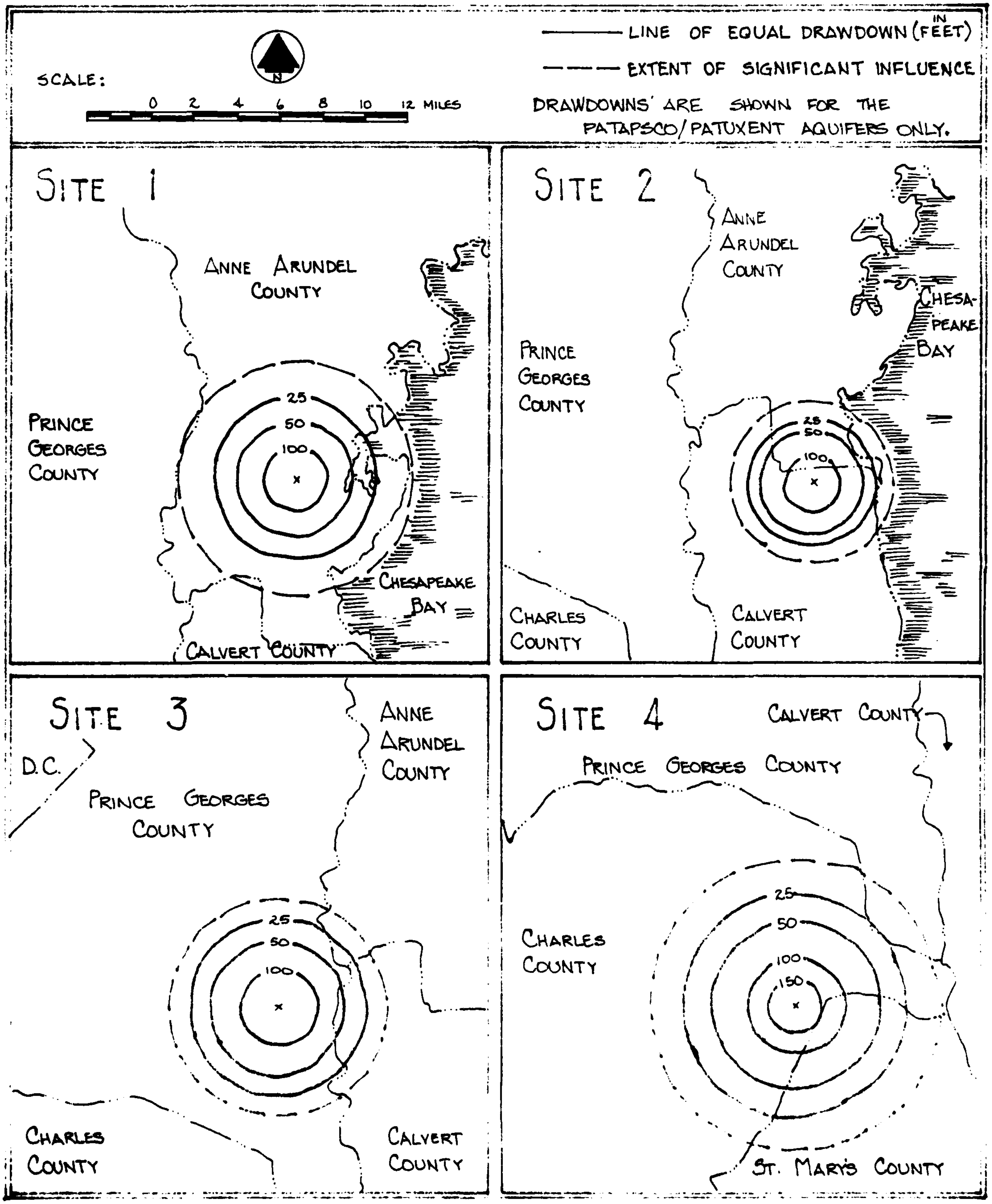
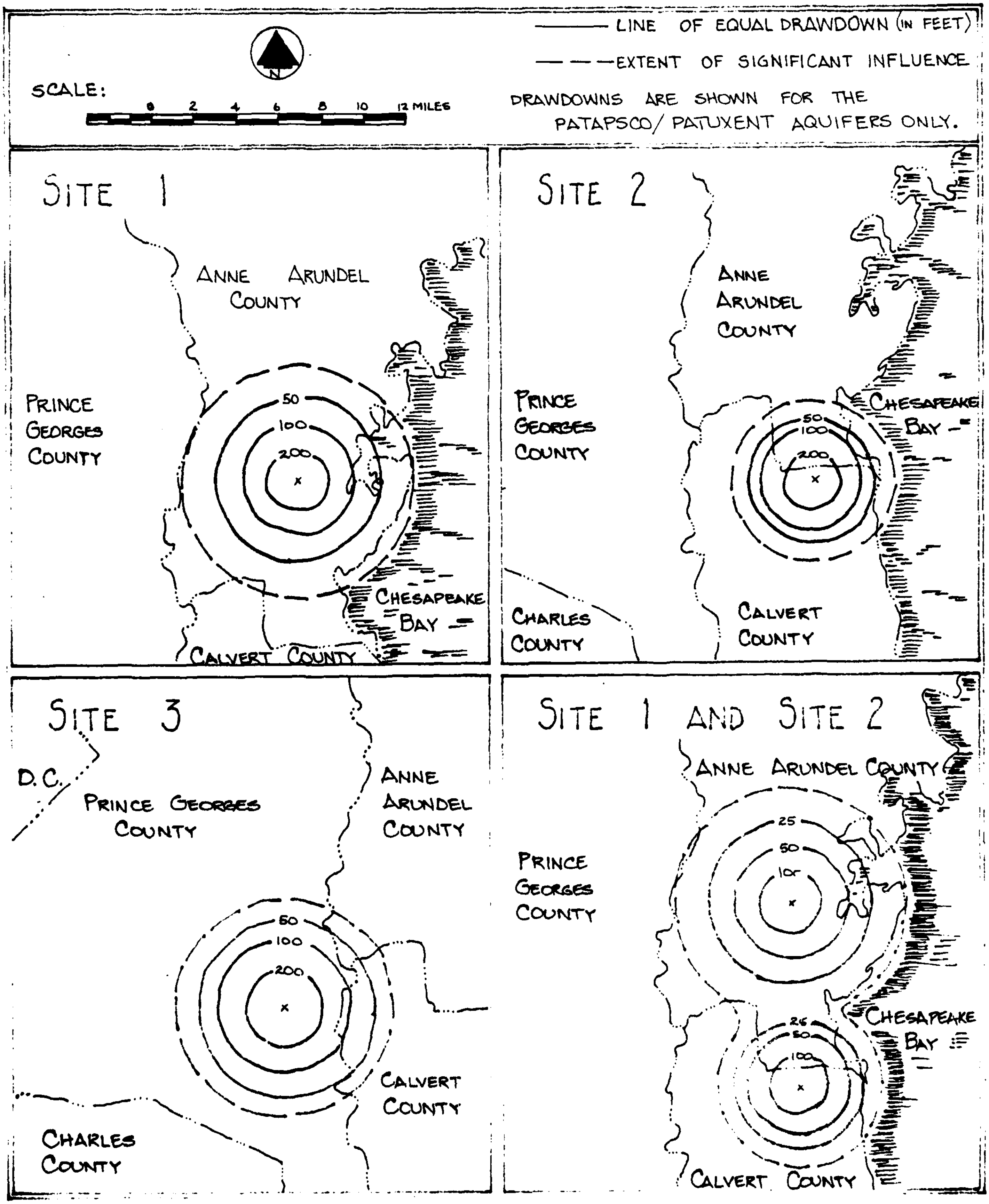


FIGURE F-46

DIAGRAM OF DEEP AQUIFER DRAWDOWN WITH 50-MGD ALTERNATIVES



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APPENDIX F STRUCTURAL ALTERNATIVES(U) CORPS OF
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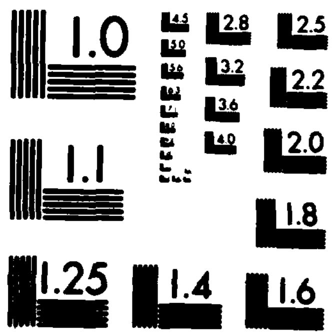

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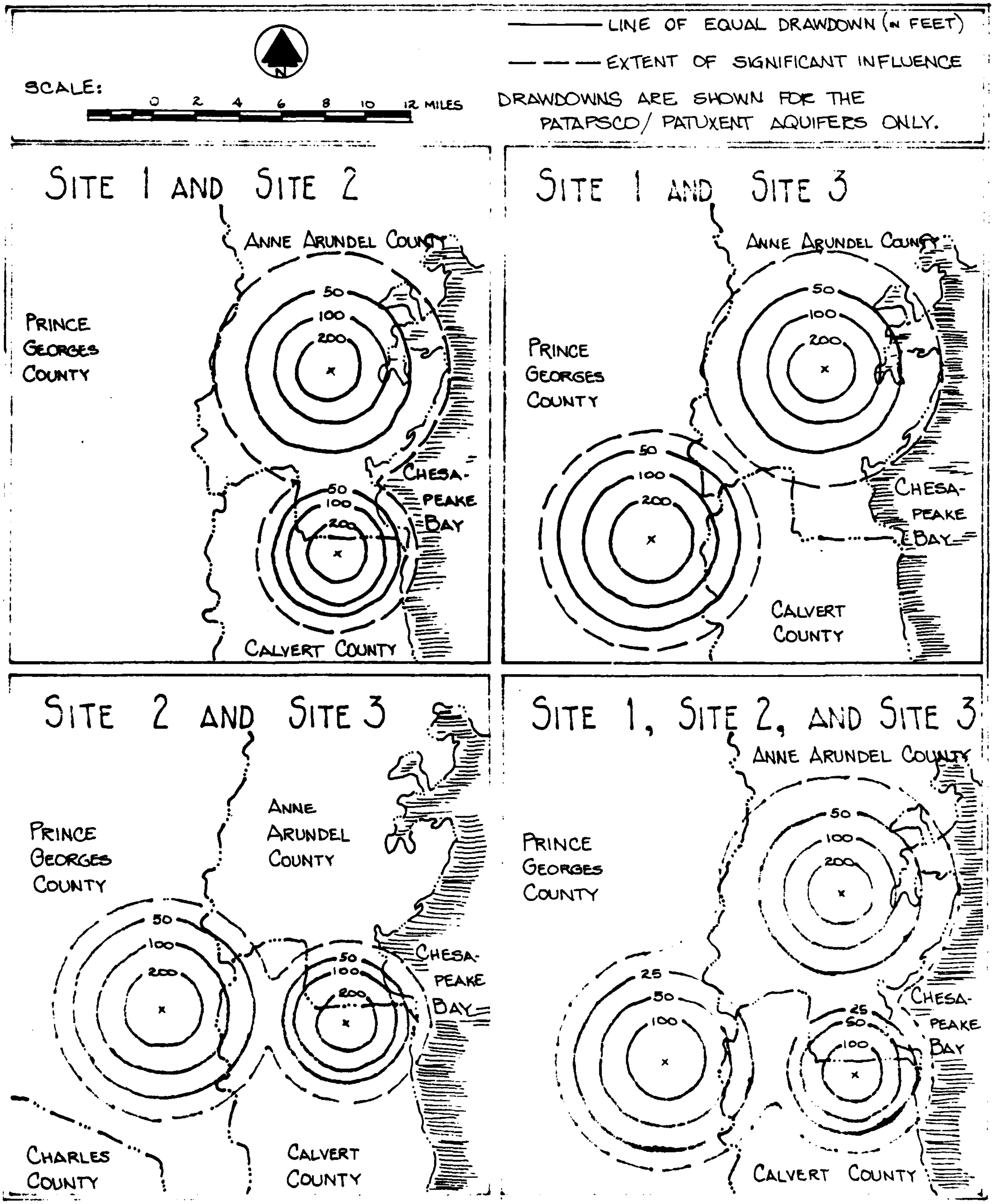
FIGURE F-47

DIAGRAM OF DEEP AQUIFER DRAWDOWN WITH 100-MGD ALTERNATIVES



Generally, as an alternative, the proposed groundwater schemes would have minimal impact to the area habitat. The development of large production wellfields would require fairly extensive purchase of land in southern Maryland. However, the use of these lands could be expanded to multi-purpose properties by combining water supply with recreation or open-space planning. The groundwater alternative, particularly the wellfield, could be somewhat developed as need arises, i.e., in increments of supply, rather than requiring extensive capital development in expectation of need. This would allow for more flexible water supply planning.

Clearly, the potential for drawdown in nearby local wells would have to be addressed in detail prior to acceptance of the groundwater alternative. However, the studies by the USGS and others have indicated that use of the Patapsco and Patuxent aquifers only would not significantly drawdown adjacent wells tapping the Aquia aquifer. Drawdowns in the deep aquifers would be significant as shown in Figures F-45, F-46, and F-47. However, neither the Patapsco nor the Patuxent are major sources of supply in the vicinity of the three selected sites (Site 4 was excluded for economic reasons as explained above); therefore, these drawdowns should not impact on local water supply withdrawals.

GLOSSARY OF SELECTED TERMS

Anion: A negatively charged ion.

Aquifer: A water-bearing bed that will yield water in a usable quantity.

Cation: A positively charged ion.

Confined aquifer: An aquifer in which groundwater is under pressure significantly greater than atmospheric, and its upper limit is the bottom of a bed of distinctly lower hydraulic conductivity than that of the aquifer.

Constant-head boundary: A boundary at which the head is unchanging with time.

Digital flow model: A mathematical approximation of a flow system.

Downdip: A direction that is downwards and parallel to the dip of a structure or surface.

Finite-difference: A technique in which continuous variables of differential equations are replaced with discrete variables. The relations between these discrete variables are finite-difference equations, and it is these finite-difference equations which are solved numerically on a digital computer.

Fluvio-deltaic: Refers to material formed by the joint action of river and delta.

Fluvio-marine: Refers to material formed by the joint action of river and sea.

Flux: The rate of transfer of fluid across a given surface.

Head: The height above a datum plane of the surface of a column of water.

Hydraulic conductivity: The capacity of a unit cube of rock to transmit water.

Homogeneous: Synonymous with uniformity. A material is homogeneous if its hydrologic properties are identical everywhere.

Isotropy: That condition in which all significant properties are independent of direction.

No-flow boundary: A boundary across which there is no flow.

Outcrop: The part of the formation that appears at the surface of the ground.

Potentiometric surface: A surface which represents the static head. As related to an aquifer, it is defined by the levels to which water will rise in tightly cased wells.

Specific storage: The volume of water released from or taken into storage per unit volume of the porous medium per unit change in head.

Steady-state: A state in which flow is independent of time.

Storage coefficient: The volume of water released from or taken into storage per unit surface area of an aquifer per unit change in head.

Transmissivity: The capacity of an aquifer to transmit water. It equals the hydraulic conductivity times the aquifer thickness.

Trilinear diagram: A diagram in which the composition of the water with respect to cations is indicated by a point plotted in the cation triangle, and the composition with respect to anions by a point plotted in the anion triangle. The coordinates at each point add to 100 percent. The points plotted in the central field are located by extending the points in the lower triangles to points of intersection.

Updip: A direction that is upwards and parallel to the dip of a structure or surface.

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ANNEX F-I

DIGITAL SIMULATION OF GROUNDWATER FLOW

IN PART OF SOUTHERN MARYLAND

by

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U.S. Geological Survey

Prepared in cooperation with the

U.S. Army Corps of Engineers, Baltimore District

Towson, Maryland

1982

DIGITAL SIMULATION OF GROUNDWATER FLOW IN PART OF SOUTHERN MARYLAND

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

DIGITAL SIMULATION OF GROUNDWATER FLOW IN PART OF SOUTHERN MARYLAND

by

William B. Fleck

U.S. Geological Survey

ABSTRACT

A quasi three-dimensional model of the Patuxent, Patapsco, Magothy, and Aquia aquifers in southern Maryland was developed to estimate potential yields from the Cretaceous aquifers and the resultant drawdowns in all four aquifers.

The Patuxent, Patapsco, and Magothy aquifers are of Cretaceous age. The Aquia aquifer is of Paleocene age. These aquifers are composed of fine to very coarse sand and gravel interbedded with some silt and clay layers. These aquifers are part of the wedge of Coastal Plain sediments which vary in thickness in the project area from a feather edge along the Fall Line to about 2,400 feet at the eastern side of the area. Transmissivity distributions of these aquifers are presented as a series of maps for each aquifer.

These aquifers are a major source of water for southern Maryland. Pumpage in 1980 from these four aquifers in the project area is estimated to have been 50 to 60 million gallons per day.

The four aquifers were modeled as confined aquifers with recharge from leakage from an aquifer overlying the entire modeled area. The peripheral nodes were treated as no-flow boundaries, except for nodes in the recharge areas, which were treated as constant-head boundaries. The calibration scheme consisted of simulating a prepumping steady-state head distribution for each aquifer.

Model simulations at four selected sites were of 9 to 11 million gallons per day from the Magothy, 5 to 9 million gallons per day from the Patapsco, and 6 to 10 million gallons per day from the Patuxent. These simulations estimated average drawdowns over a 2.4-square-mile area of 60 to 80 feet, 90 to 140 feet, and 200 to 280 feet, respectively.

INTRODUCTION

The Army Corps of Engineers was authorized by Section 85 of the Water Resources Development Act of 1974 (P.L. 93-251) to make a full and complete investigation and study of the future water resources of the Metropolitan Washington Area.

At the request of the Army Corps of Engineers in September 1977, the U.S. Geological Survey entered into a cooperative agreement to evaluate the water resources of the Cretaceous aquifer system underlying the Coastal Plain in the vicinity of Washington, D.C. The purpose of this evaluation was to determine the feasibility of using groundwater as a water supply during extreme drought in the Metropolitan Washington Area, and to estimate the effects on the aquifer system due to pumping.

The first phase of the study was a collection and analysis of available data. The second phase used these data to build a quasi three-dimensional (3-D) digital-flow model. A previous model study by Papadopulos and others (1974) indicated the total available water from these aquifers within a 30-mile radius of Washington, D.C., to be 170 million gallons per day (mgd).

Papadopulos and others indicated the desirability of drilling test wells into the deep downdip parts of the Cretaceous aquifers to model this system adequately. The original agreement with the Corps provided for drilling about three test wells; however, due to funding limitations, this part of the cooperative agreement was shelved. After some consideration, it was decided that enough new data were available since the report by Papadopulos and others to proceed with this study.

The location of the study area is that part of the Coastal Plain of southern Maryland that falls within an arc with a radius of 30 miles centered on Washington, D.C., on the east, the north, and the south, and the Fall Line on the west. (See Figure 1.) This 30-mile radius was defined by the Corps as a practical economic limit for developing a groundwater supply for the Metropolitan Washington Area. The aquifer to the west of the Fall Line, the Piedmont aquifer, was not included in the study because of its small yield.

The geography of the study area is one of low relief, with elevations from sea level to a maximum of 460 feet in northern Prince Georges County. Numerous estuaries and stream valleys are the most prominent physiographic features. Tidal marshes and swamps are common adjacent to the estuaries and streams.

Southern Maryland is part of the humid temperate climatic belt of Eastern United States. The precipitation distribution is nearly uniform throughout the year. Based on 88 years of record at Annapolis, Md., and 55 years of record at Chettenham, Md., November is the driest month, averaging 3.0 inches, August the wettest, averaging 4.3 inches, and the yearly average is between 44 and 45 inches.

The study area was represented by a flow model that divided the area into a rectangular grid having 37 rows, 27 columns, and 5 layers. The model used a finite-difference numerical algorithm to solve the flow equation in quasi three dimensions. After calibration, the model was used to make estimates of maximum pumping effects on the potentiometric heads in the Patuxent, Patapsco, Magothy, and Aquia aquifer by pumping stresses imposed at selected well sites.

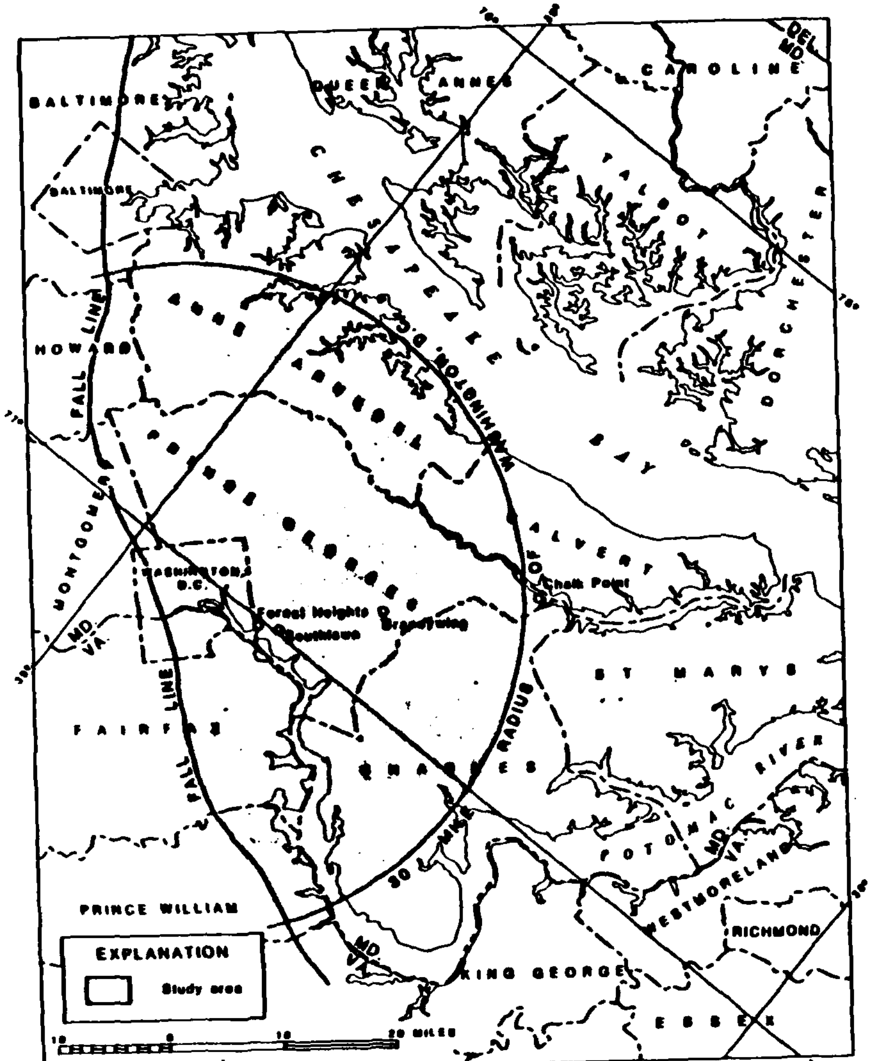

Figure 1. -- Location of study area.

GEOLOGY

The wedge of Coastal Plain deposits within the study area increases in thickness from a feather edge at the Fall Line to about 2,400 feet in southern Prince Georges County (Figure 2). The basal Coastal Plain unit is the Potomac Group of Early Cretaceous age which constitutes most of the bulk of sediments (Table I). Above the Potomac Group lies the Upper Cretaceous formations and the marine Tertiary formations. Much of the surface of the Coastal Plain in this area is covered by a veneer of sand, silt, and clay of Pleistocene age. Refer to Table I for a generalized chart of the geologic units.

A great deal of work has been put into the study of the Coastal Plain sediments of southern Maryland. Since 1949, a series of county reports describing the hydrogeology of each county has been published by the Maryland Geological Survey, including the following four counties within the study area:

County	Reference
Anne Arundel	Mack (1962)
Calvert	Overbeck (1951)
Charles	Slaughter and Otton (1968)
Prince Georges	Mack (1966)

More recently, Harry Hansen (1968, 1972) has studied subsurface correlations of the Coastal Plain formations and aquifers, and Mack and Mandle (1977) have reported on a digital model study of the Magothy aquifer in southern Maryland.

The Coastal Plain sediments in the study area were deposited under various conditions. During Early Cretaceous time, the Potomac Group sediments were deposited in a fluvio-deltaic environment. Deposition of this type consists of complexly related stream and sandbar sand and gravel intermixed with flood-plain and swamp silt and clay. The Patuxent and Patapsco Formations are composed of extensive sand, whereas the Arundel Formation is principally silt and clay. However, because of this constantly changing environment both laterally and vertically, it is difficult today to correlate discrete sand units between wells.

The fluvial environment of Early Cretaceous time was followed by a fluvio-marine phase in the Late Cretaceous, represented by the Magothy Formation. These near-shore deposits consist principally of sand and gravel. Because of the gradual landward encroachment of the sea during Magothy time, the sand tends to become finer upward. As the sea encroached onto the land, the depositional environment gradually changed to a strictly marine environment.

The marine environment is represented first by the Severn and Matawan Formations and by the overlying Brightseat Formation. The sediments of these formations consist almost entirely of silt and clay typical of marine deposition.

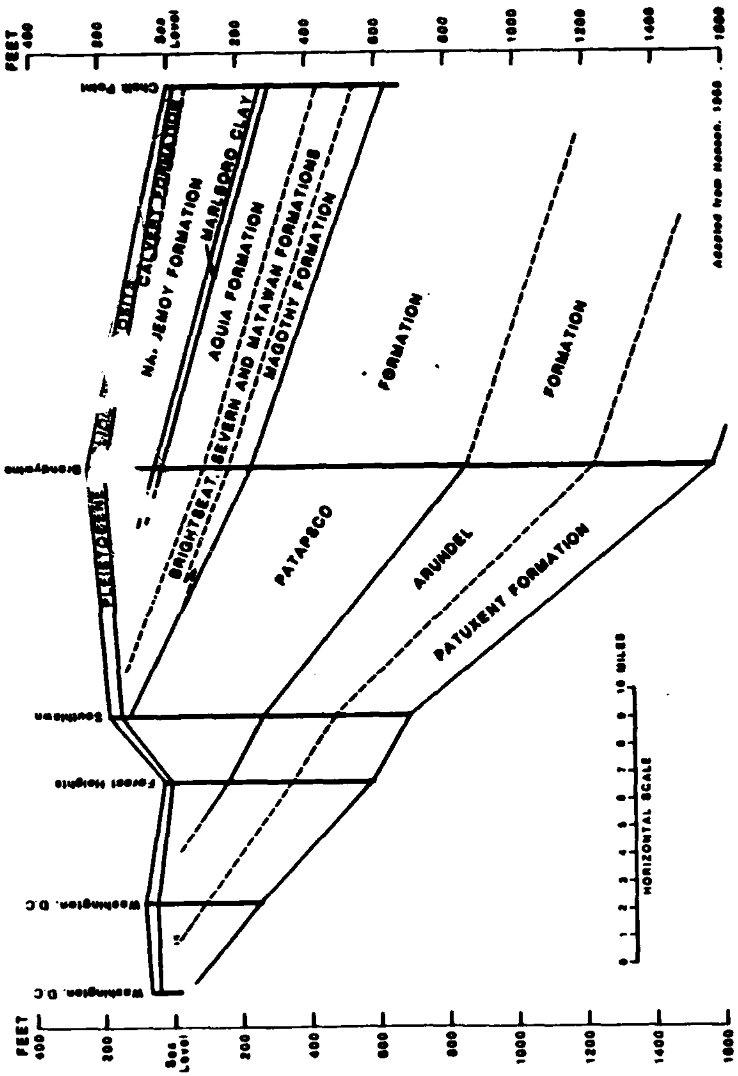

Figure 2. -- Generalized geologic section from Washington, D.C. to Chalk Point, Md.

Table 1.—Generalized Lithologic Descriptions of the Coastal Plain Sediments within the Study Area

System	Series	Group	Formation	Approximate average thickness (ft)	Physical characteristics	Water-bearing properties
Quaternary	Pleistocene	Columbia	Undivided deposits	30	Irregularly bedded tan to orange sand, gravel, silt and clay.	Yields adequate domestic supplies.
Tertiary	Pliocene					
	Miocene	Chesapeake	Calvert	110	Brown to greenish-gray sandy clay to fine sand. Generally diatomaceous.	Generally not an aquifer.
	Eocene	Pamunkey	Nanjemoy	90	Glauconitic fine sand with greenish-gray clayey layers.	Generally not an aquifer.
			Marlboro Clay	30	Pink to gray clay.	Not an aquifer.
	Paleocene		Aquia	80	Greenish-gray to brown glauconitic sand with indurated layers.	Principle aquifer in eastern part of study area.
			Brightseat	40	Gray to dark gray micaceous silty to sandy clay.	Not an aquifer.
Cretaceous	Upper Cretaceous		Severn and Matawan	40	Dark colored sandy clay and sand. Some glauconite. Lighter colored near base.	Not an aquifer.
			Magothy	50	Light-gray to white sand and gravel and interbedded light-colored clay. Contains some lignite, byrite, and glauconite.	Important aquifer in northern half of study area.
	Lower Cretaceous	Potomac	Patapsco	500	Interbedded, variegated sand, clay, and sandy clay; in places, upper part consists of about 200 feet of pink clay.	Good aquifer. Used extensively in northwest part of study area.
			Arundel	250	Red, brown, and gray clay. Some interbedded sand lenses.	Generally not an aquifer.
			Patuxent	250	Mostly gray and yellow sand and interbedded clay. Often contains sand and gravel basal unit.	Good aquifer. Used extensively in northwest part of study area.

Above the Brightseat Formation is the Aquia Formation. During Aquia time, the sea began to regress from the land, and finally, during the latter part of the Aquia time, much sand and fine sand were deposited in this shallowing sea. Overlying the Aquia Formation is the fluvial-marine Marlboro Clay, consisting of about 30 feet of very tight clay. Above the Marlboro Clay the sediments of the Nanjemoy Formation are an admixture of fine sand, silt, and clay deposited in a marine environment. The sandy Piney Point Formation overlies the Nanjemoy Formation. However, the Piney Point Formation pinches out just east of the study area and therefore is not included in Table 1.

The Calvert Formation directly overlies the Nanjemoy. The sediments of the Calvert Formation were deposited in a shallow marine sea and consist of clay, silt, and fine sand.

Finally, the Pliocene and Pleistocene deposits are irregularly distributed throughout the area. These are mostly fluvial deposits and vary from silt and clay to sand and gravel.

HYDROLOGY

The Coastal Plain sediments discussed in the previous section consist of alternating layers of clay, silt, and sand. The sandy layers act as conduits through which groundwater flows. The rate at which the water flows is a function of the permeability of the sediments and the hydraulic gradient. Formations in which water flows at a rate sufficient to supply a well are called aquifers.

The nature of the Coastal Plain sediments is such that the more permeable sandy layers that act as aquifers are often continuous over great distances. In other cases, the sand units are rather discontinuous horizontally and cannot easily be correlated from one location to another. Sands of the Magothy and Aquia Formations are examples of the former case. Contrarily, the sandy layers within the Patuxent and Patapsco Formations are often discontinuous and are not traceable as discrete units throughout the project area.

The less permeable silt and clay act as confining layers that restrict flow in vertical directions, thus constraining flow between those aquifers which are separated by confining layers. Where the aquifer is not confined, the water is under only atmospheric pressure. Confining layers above and below an aquifer tend to confine the flow of water within the aquifer. When a well is drilled into a confined aquifer, water in the well rises in response to pressure transmitted through the aquifer system. Confining layers generally permit passage of small amounts of water between aquifers where head differences are sufficient.

The Coastal Plain sediments consist of layers of aquifers and intervening confining beds. The whole sediment wedge is but a single system in which the direction of water movement both within the sandy aquifers and vertically through the silt and clay confining layers is a function of head differences and permeability. Thus, even where the sand layers seem discontinuous, hydraulic continuity with other sand units is commonly good because sediments separating the discontinuous sand layers are not totally impermeable. This is the case in both the Patuxent and Patapsco Formations.

PATUXENT AQUIFER

The Patuxent Formation crops out along the Fall Line immediately adjacent to the crystalline Piedmont rocks. It thickens from a feather edge along the Fall Line to about 500 feet on the east side of the study area.

The Patuxent aquifer is a multiaquifer unit consisting of several water-bearing sand layers. Individual sand units range in thickness from several feet to as much as 100 feet. Near Baltimore, these units may comprise as much as 50 percent of the Patuxent Formation. Southward, the percentage of sand diminishes.

Transmissivity is a measure of the ability of an aquifer to transmit water. The greater the thickness of sand units and the greater their permeability, the more transmissive the aquifer. Figure 3 indicates the transmissivity distribution of the Patuxent aquifer used in the model. Transmissivities in the updip part of the aquifer, especially between Washington and Baltimore, are fairly well known. Transmissivity data for the rest of the study area are sparse. As indicated on the map, transmissivity is highest between Baltimore and Annapolis. This is also the area of heavy usage. For detailed transmissivity and pumpage data for the aquifers discussed in this report, see Hansen (1972).

A primary source of recharge to the aquifer is from precipitation on the outcrop area. As indicated in the Introduction, average precipitation is about 44-45 inches annually. Mack (1962) estimates that for Anne Arundel County, about 25 percent of the precipitation recharges the aquifers. This amounts to about 0.5 mgd per square mile. In the study area, the recharge area of the Patuxent aquifer is about 300 square miles. Thus, the Patuxent aquifer within the study area receives about 150 mgd recharge. However, most of this recharge is discharged as base flow to streams and rivers or directly to Chesapeake Bay. Mack (1962) estimates that 0.1 mgd per square mile of this recharge moves downgradient into the deeper parts of the aquifers, which is about 30 mgd.

Heavy pumping from the deeper aquifers would result in sizeable cones of depression. These cones, as they extended into the recharge area, would divert groundwater flow, decreasing discharge to base flow and increasing flow to the deeper parts of the Patuxent aquifer.

The Arundel Formation, above the Patuxent aquifer, is a thick confining silt and clay unit that allows only small volumes of water to pass through. Near the outcrop, heads in the overlying Patapsco aquifer are generally higher than heads in the Patuxent, causing downward movement of water through the confining silt and clay into the Patuxent aquifer. Under Chesapeake Bay, the head gradient is reversed; thus, water is discharged from the Patuxent aquifer upward through the confining Arundel Formation.

Pumpage from the Patuxent aquifer statewide is about 20 mgd. About 80 to 90 percent of this pumpage is from northern Anne Arundel and northern Prince Georges Counties. (See Figure 4.) Almost all of the pumpage is from the updip part in or near the outcrop area.

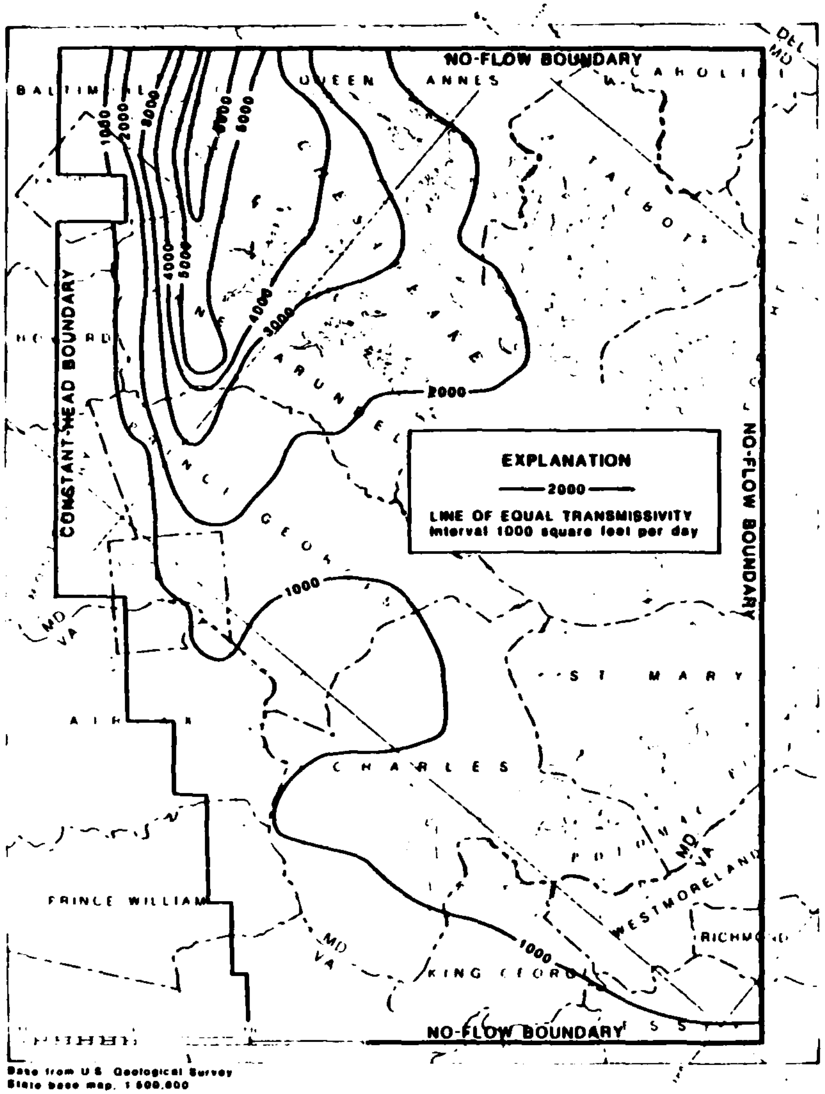

Figure 3 -- Transmissivity distribution used in model, Paluxent aquifer.

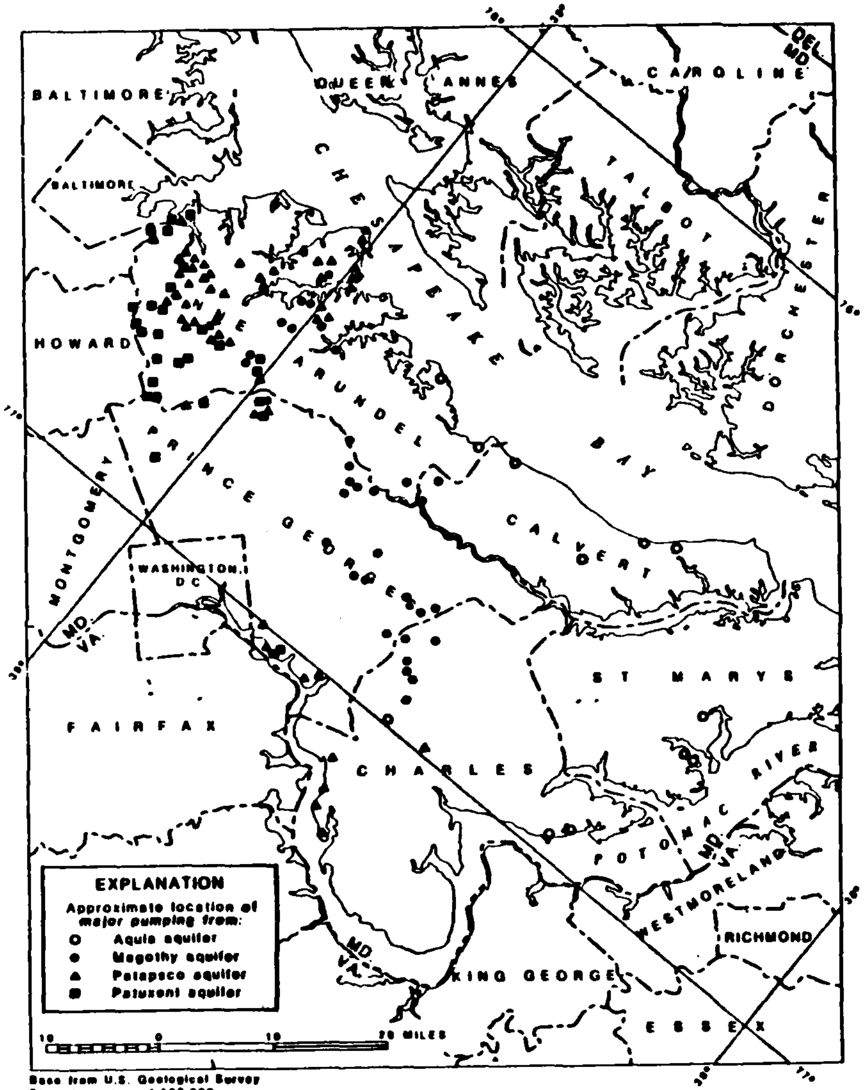

Figure 4. -- Approximate locations of major groundwater pumping.

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PATAPSCO AQUIFER

Like the Patuxent aquifer, the Patapsco aquifer contains sandy layers interbedded with silt and silty clay. The sandy layers act as aquifers; however, they tend to be discontinuous. In this study they are lumped as a single aquifer.

The Patapsco Formation crops out along a belt eastward and parallel to that of the Patuxent Formation. The width of the outcrop ranges from 12 miles south of Baltimore to 2 miles on the east side of Washington. The area of the outcrop is about 300 square miles. As in the Patuxent aquifer, recharge on the outcrop area of the Patapsco is estimated as 150 mgd, of which about 30 mgd moves downward to the deep part of the aquifer.

Sand in the Patapsco Formation is generally finer than that in the Patuxent; however, the overall thickness of sand beds is greater. Transmissivity (Figure 5) is high in the Baltimore-Annapolis area. Southward, transmissivity is much less. Cumulative thickness of sand beds in the Annapolis area is about 300 feet. Southward, the cumulative thickness decreases, and, in Charles County, it is generally less than 150 feet.

Pumpage from the aquifer is about 20 to 25 mgd and confined to the updip part. Heaviest use is in northern Prince Georges and Anne Arundel Counties and, south of Washington, along the Potomac River in Charles County.

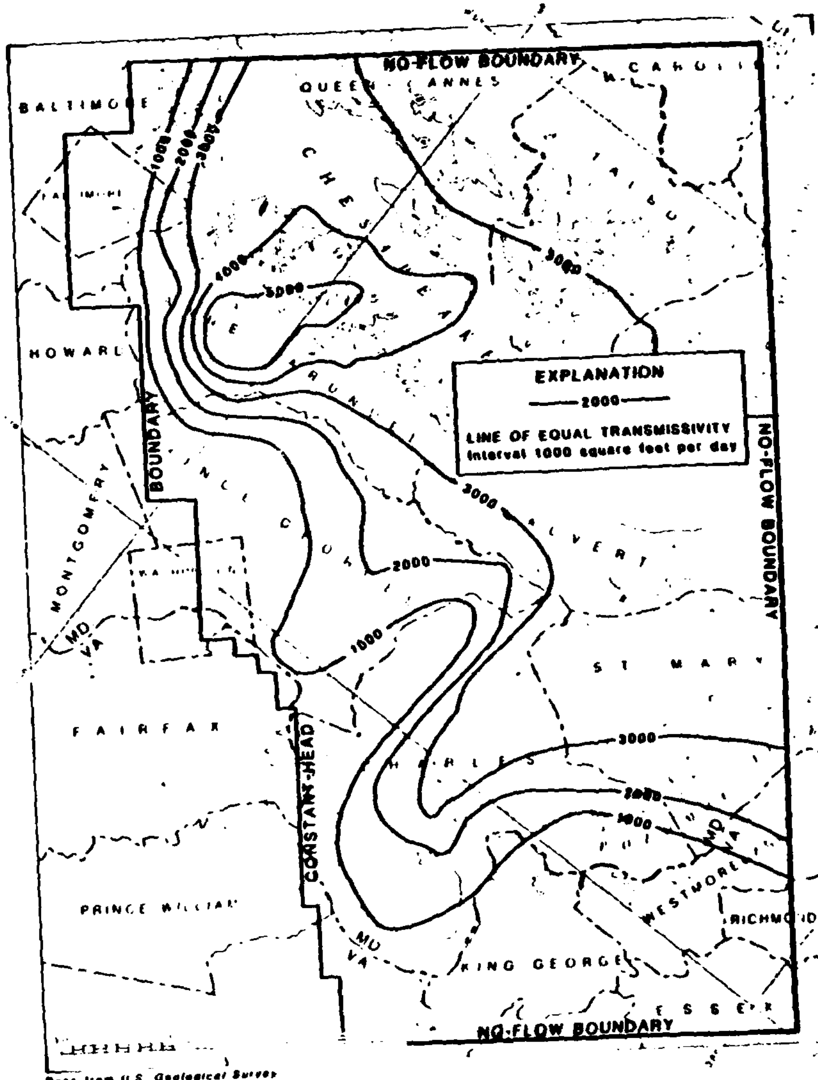
MAGOTHY AQUIFER

The Magothy aquifer crops out in Prince Georges and Anne Arundel Counties. South of central Prince Georges County, it is overlapped by younger silty clay of the Matawan and Brightseat Formations. The Magothy aquifer ranges in thickness from about 200 feet in the Annapolis area to a feather edge where it pinches out in the southern part of the study area.

The coarser sand and gravel generally occur at the base of the aquifer overlying Patapsco clay beds. The sediments of the Magothy become finer upward, grading into the overlying clayey confining beds of the Matawan and Brightseat Formations. Generally, where the aquifer is thicker, transmissivity is higher. Figure 6 indicates an area of high transmissivity in the northern part of the study area.

The aquifer is partly recharged at the outcrop area in Prince Georges and Anne Arundel Counties. The area of the outcrop is about 50 square miles. Again, assuming that 25 percent of precipitation recharges the aquifer, about 25 mgd is recharged in the study area, and about 5 mgd seeps downward into the deeper parts of the aquifer. Leakage from the overlying Aquia Formation and the underlying Patapsco aquifer provides some additional recharge.

Total pumpage from the Magothy aquifer in Maryland is about 8 to 10 mgd. Figure 4 shows the location of principal pumping centers. Although the distribution of pumping centers is relatively even, consumption is heaviest in the Annapolis area, about 4 to 5 mgd. In Prince Georges County, pumpage from the Magothy is slightly less than 1 mgd, and, in Charles County, it is about 0.5 mgd.



Base from U.S. Geological Survey State base map, 1:500,000

Figure 5 -- Transmissivity distribution used in model, Patapsco aquifer.

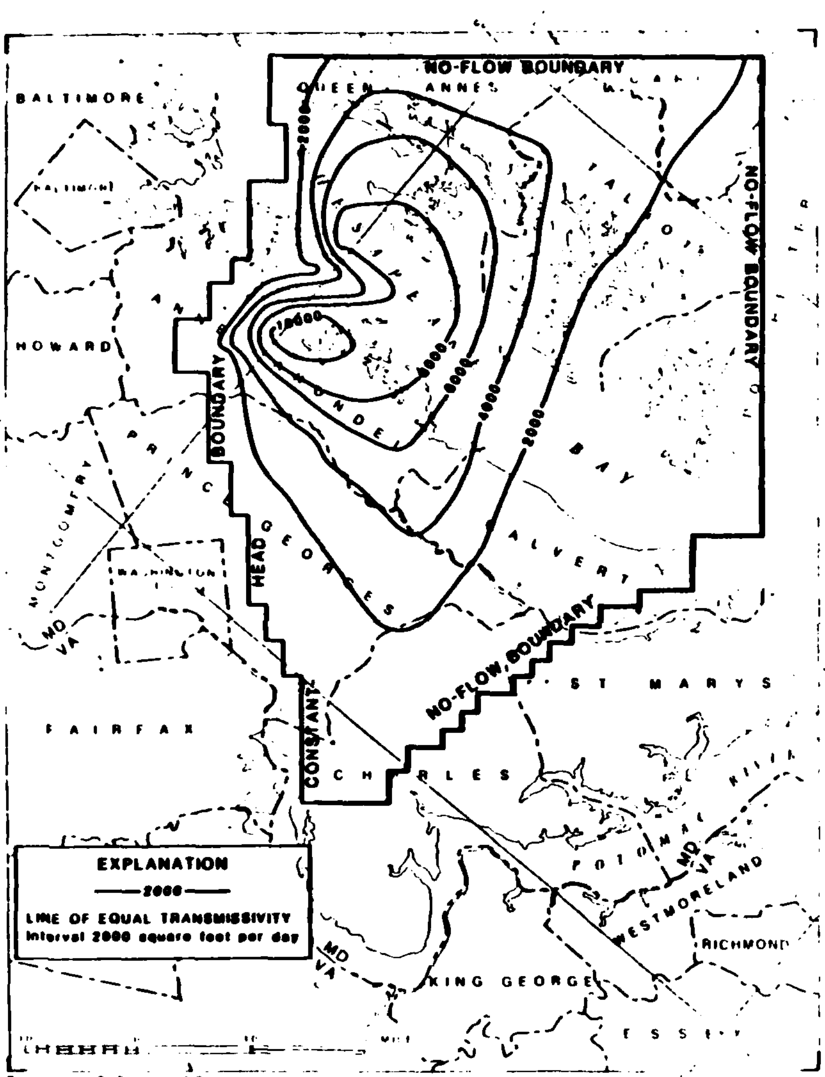


Figure 6 -- Transmissivity distribution used in model, Magothy aquifer.

AQUIA AQUIFER

The Aquia aquifer crops out as an irregularly shaped belt from Sandy Point in Anne Arundel County to Indian Head in southwestern Charles County. The maximum width of the outcrop is about 10 miles, and its area is about 170 square miles. Again, assuming that recharge from precipitation is 0.5 mgd per square mile, total recharge to the Aquia is 85 mgd, 17 mgd of which moves downward into the deeper part of the aquifer. As with the other aquifers, leakage both into and out of the Aquia occurs through the adjacent confining layers. Sandy beds in the Aquia are thickest east of Chesapeake Bay. In the study area the sand beds are 40 to 70 feet thick. The transmissivity map of Figure 7 indicates that sand increases in thickness eastward.

Total pumpage from the Aquia aquifer is about 4 mgd; however, almost all is outside the study area. Figure 4 indicates the few pumping centers within the study area; however, the pumpage is less than 0.1 mgd.

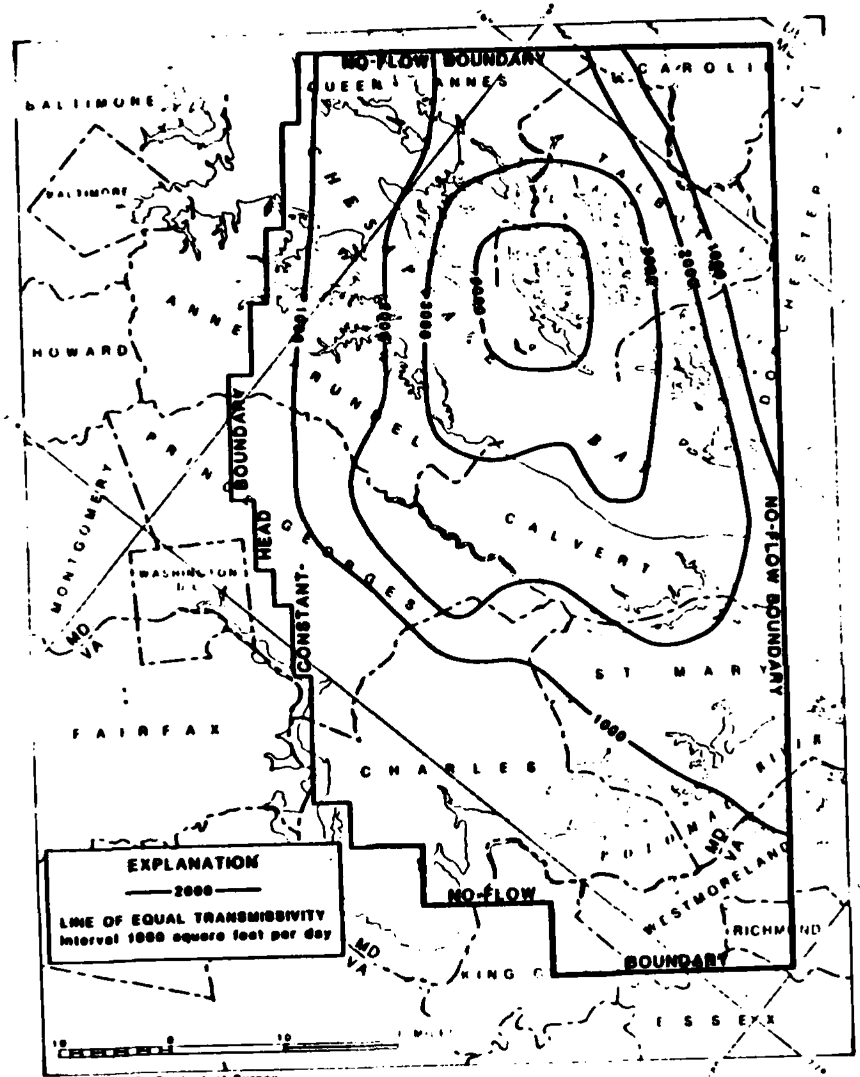


Figure 7. -- Transmissivity distribution used in model, Aquia aquifer.

CHEMICAL CHARACTER OF GROUNDWATER

Representative chemical analyses of water from wells penetrating the four principal aquifers (Patuxent, Patapsco, Magothy, and Aquia) are given in Table 2, which indicates the formation and the chemical results. The major dissolved cations (sodium, calcium, magnesium, and potassium) and the major dissolved anions (bicarbonate, sulfate, and chloride) in Table 2 are plotted in Figure 10.

Analyses of iron, manganese, silica and several minor constituents are also given in the table. The iron, manganese, and some of the minor constituents are reported in micrograms per liter (µg/L), which is equivalent to 1,000 milligrams per liter (mg/L). Temperature and pH are indicated for most of the samples. For a more complete listing of chemical analyses, see Woll (1978).

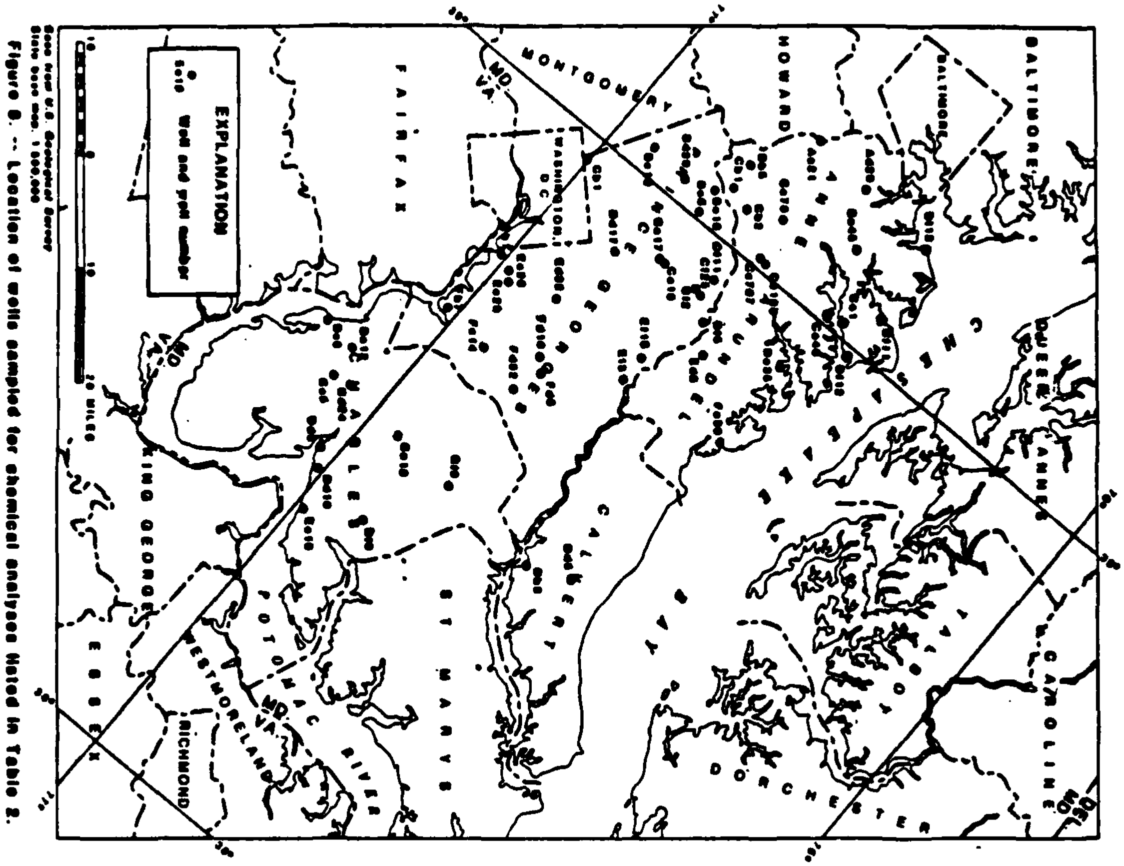
Generally, the groundwater is potable and uncontaminated. However, iron is excessive in places. The U.S. Public Health Service recommends that drinking water contain no more than 0.3 mg/L of iron. An analysis of the iron values in Table 2 indicates that the median ranges from 0.5 to 1.7 mg/L for the four aquifers. As a consequence, the groundwater needs to be treated for high iron concentrations.

Figure 8 indicates the location of wells for which water-quality data are listed in Table 2. The distribution is areally uniform, and most of the wells are within 30 miles of Washington.

Figure 9 indicates the regional variation of pH values listed in Table 2. Groundwater in the northern part of the area tends to be acidic. The pH values for ground water sampled in the rest of the study area are generally greater than 7.0. Although Figure 9 does not distinguish between water of different aquifers, the pattern of pH values, indicated in Figure 9, is similar for each of the aquifers (Mack, 1966, Figure 39). Water from the deep downdip parts of the Cretaceous aquifers is likely to have a pH greater than 7.0.

The chemical composition of groundwater can be conveniently displayed on trilinear diagrams (Back, 1966). On this type of diagram, each analysis is plotted as a percentage of the total milliequivalents per liter of cations (lower left triangle) and anions (lower right triangle). The two points on the lower triangles are then projected into the upper diamond and are plotted as a single point. Thus, the trilinear diagram gives a good representation of the relative cationic and anionic composition of the water sample.

Figure 10 shows the data of Table 2 plotted on trilinear diagrams. Water from the Aquia aquifer is predominantly a calcium and sodium bicarbonate type. Water from the Magothy aquifer is generally a calcium magnesium bicarbonate type with higher percentages of sulfate than water from the Aquia aquifer. Water from the Patapsco and Patuxent aquifers varies considerably in composition, but is predominately a calcium sodium bicarbonate sulfate type. Otton (1955, p. 149-172) describes the chemical character of groundwater in southern Maryland comprehensively. The following summarizes Otton's major conclusions.



Base from U.S. Geological Survey
State base map, 1:500,000

Figure 8. -- Location of wells sampled for chemical analyses listed in Table 2.

Table 2.--Groundwater chemistry

Well No.	Geologic unit[1]	Date of sample	Depth to bottom of sample interval (ft)	Bicarbonate (mg/L as HCO_3)	Nitrogen, nitrate dissolved (mg/L as NO_3)	Elevation of land surface datum (ft above NGVD)	Color (platinum cobalt units)	Specific conductance (micromhos)	pH (units)	Zinc, total recoverable (ug/L as Zn)	Aluminum, total recoverable (ug/L as Al)	Lithium total recoverable (ug/L as Li)	Solids, residue at 180°C dissolved (mg/L)	Solids, sum of constituents, dissolved (mg/L)
						ANNE ARUNDEL COUNTY								
AC 21	PTXN	52-03-31	347	36	0.20	145.00	7	73	6.4	-	3,900	-	48.0	44.0
AD 29	PTXN	48-05-13	300	1	<.05	37.00	3	29	4.7	-	-	-	24.0	24.0
BB 5	PTXN	52-03-31	199	5	.10	115.00	2	39	5.2	-	-	-	30.0	28.0
BE 45	PPSC	46-04-16	67	2	21	80.00	12	114	4.7	-	-	-	75.0	62.0
BF 2	PPSC	43-08-03	378	-	<.05	10.00	-	56	4.2	-	-	-	-	-
BF 2	PPSC	51-05-01	378	< .0	.20	10.00	2	54	4.1	-	<50	-	34.0	28.0
CB 1	PTXN	59-09-15	515	3	.10	126.00	15	22	5.0	210	<50	-	14.0	21.0
CB 2	PTXN	59-12-14	630	.2	.30	170.00	6	44	4.6	100	100	-	37.0	38.0
CC 78	PTXN	62-01-15	872	1	<.05	178.00	2	30	5.2	100	<50	<100	26.0	24.0
CC 78	PTXN	62-01-25	458	< .5	<.05	178.00	0	29	4.5	-	-	-	24.0	-
CC 105	PTXN	73-07-08	1,190	6	.04	130.00	-	46	4.7	-	-	-	-	29.0
CC 107	PTXN	74-07-12	1,105	3	.04	120.00	0	42	-	-	-	-	-	30.0
CE 1	MGTY	46-04-01	213	10	.10	12.00	3	86	5.3	1,300	-	-	53.0	53.0
CE 46	MGTY	46-05-08	96	21	<.05	3.00	2	94	5.6	-	-	-	62.0	62.0
CF 11	MGTY	46-04-16	95	5	<.05	5.00	13	82	4.9	-	-	-	55.0	51.0
DE 35	AQUI	46-08-25	81	220	.20	10.00	3	365	7.8	-	-	-	230	236
DF 12	PPSC	45-03-20	393	9	<.05	10.00	-	-	3.2	-	-	-	66.0	60.0
DF 12	PPSC	60-02-03	393	5	.60	10.00	6	113	3.0	180	<50	-	88.0	65.0
DF 12	PPSC	61-05-26	393	< .5	<.05	10.00	2	113	4.1	200	100	-	68.0	-
ED 8	PPSC	46-08-28	265	190	1.5	125.00	30	342	7.2	-	-	-	209	199
FE 30	AQUI	50-03-27	130	140	.50	5.00	1	280	8.2	-	-	-	164	169
						CALVERT COUNTY								
DB 3	AQUI	50-03-27	419	162	.80	19.00	0	277	7.9	-	-	-	154	-
DB 5	AQUI	47-01-13	552	162	.70	147.00	3	279	8.0	-	-	-	453	163

[1] Geologic unit code:
AQUI = Aquia Formation
MGTY = Magothy Formation
PPSC = Patapsco Formation
PTXN = Patuxent Formation

Table 2.—Groundwater chemistry—continued

Well No.	Geologic unit[1]	Date of sample	Depth to bottom of sample interval (ft)	Bicarbonate (mg/L as HCO_3)	Nitrogen, nitrate dissolved (mg/L as NO_3)	Elevation of land surface datum (ft above NGVD)	Color (platinum cobalt units)	Specific conductance (micromhos)	pH (units)	Zinc, total recoverable (ug/L as Zn)	Aluminum, total recoverable (ug/L as Al)	Lithium total recoverable (ug/L as Li)	Solids, residue at 180°C dissolved (mg/L)	Solids, sum of constituents, dissolved (mg/L)
						CHARLES COUNTY								
BC 6	PTXN	61-03-09	412	135	<0.05	55.00	0	258	7.7	<50	<50	40	185	194
BC 12	PPBC	50-03-28	234	210	.70	155.90	5	357	7.8	-	-	-	224	222
CC 5	PPBC	52-04-02	274	265	.80	170.00	8	411	7.9	-	<50	-	265	264
CD 24	PTXN	60-05-12	970	147	.30	175.00	10	279	7.8	<50	<50	-	200	-
CE 18	PTXN	57-09-16	1,211	-	.20	197.00	-	321	7.9	-	-	-	-	-
CE 18	PTXN	59-06-17	1,211	149	.20	197.00	20	273	7.9	< 5	100	-	210	201
CF 9	MGTY	51-03-21	679	163	1.1	190.00	6	252	8.1	-	100	-	168	170
CF 9	MGTY	52-04-17	679	165	.50	190.00	1	268	7.5	100	<50	-	158	165
DD 5	AQUI	47-01-22	212	184	.30	21.80	1	332	8.5	-	-	-	206	205
DD 10	PPBC	51-03-07	414	216	.40	155.00	12	365	8.2	-	400	-	245	237
DF 9	AQUI	51-03-21	374	143	1.1	152.10	6	252	8.1	-	100	-	168	170
EE 18	AQUI	47-01-27	280	188	.30	10.00	13	327	8.6	-	-	-	204	202
						PRINCE GEORGES COUNTY								
BC 10	PTXN	50-03-17	110	36	6.5	290.00	1	136	7.3	-	-	-	64.0	61.0
BD 29	PTXN	49-03-22	232	4	2.2	120.00	5	27	6.0	-	1,000	-	27.0	20.0
BE 6	PTXN	50-04-13	430	6	< .05	177.00	1	97	5.3	-	-	-	57.0	58.0
BE 16	PTXN	59-09-11	277	17	.70	205.00	0	31	6.7	4,800	<50	-	28.0	26.0
CB 1	PTXN	59-02-08	-	5	7.7	200.00	4	78	5.2	80	<50	-	57.0	73.0
CE 17	MGTY	49-11-04	113	40	1.4	170.00	2	149	7.5	-	1,000	-	108	84.0
CE 18	PPBC	49-11-04	464	73	.10	140.00	1	177	6.5	-	600	-	99.0	101
CF 2	MGTY	49-03-22	171	58	.80	110.00	5	132	6.5	-	3,400	-	128	113
CF 11	AQUI	50-04-17	23	18	3.7	130.00	1	80	6.1	-	-	-	67.0	64.0
CF 25	PPBC	52-04-17	398	17	.10	110.00	0	69	5.9	<50	<50	-	60.0	59.0
BD 17	PTXN	51-04-18	214	39	.50	65.00	2	95	6.6	-	<50	-	58.0	56.0
DF 5	AQUI	50-03-26	90	190	.20	31.00	2	320	7.8	-	-	-	212	212
EB 1	PTXN	49-03-28	588	132	1.6	30.00	5	255	8.0	-	1,000	-	180	177
EC 26	PPBC	52-03-31	324	123	.80	240.00	5	246	7.8	-	<50	-	157	160
EC 29	PTXN	63-06-06	822	81	< .05	75.00	3	346	7.7	<50	<50	-	204	200
ED 32	PTXN	51-03-06	759	113	.20	270.00	3	236	7.7	-	600	-	136	138
EF 3	MGTY	49-06-06	366	178	-	165.00	2	307	6.9	-	-	-	183	177
EF 16	AQUI	63-06-05	188	234	< .05	120.00	0	448	7.8	300	<50	-	309	286
FB 7	PPBC	49-03-31	263	130	1.1	50.00	10	265	8.1	-	300	-	209	191
FC 14	MGTY	50-04-13	150	147	.60	40.00	10	263	7.7	-	-	-	149	154
FD 6	MGTY	49-03-25	404	158	1.8	230.00	5	285	7.7	-	600	-	169	157
FD 10	MGTY	52-04-14	366	153	.30	190.00	1	254	7.9	-	<50	-	149	154
FD 32	MGTY	52-04-17	490	203	.50	230.00	0	320	7.8	<50	<50	-	184	188

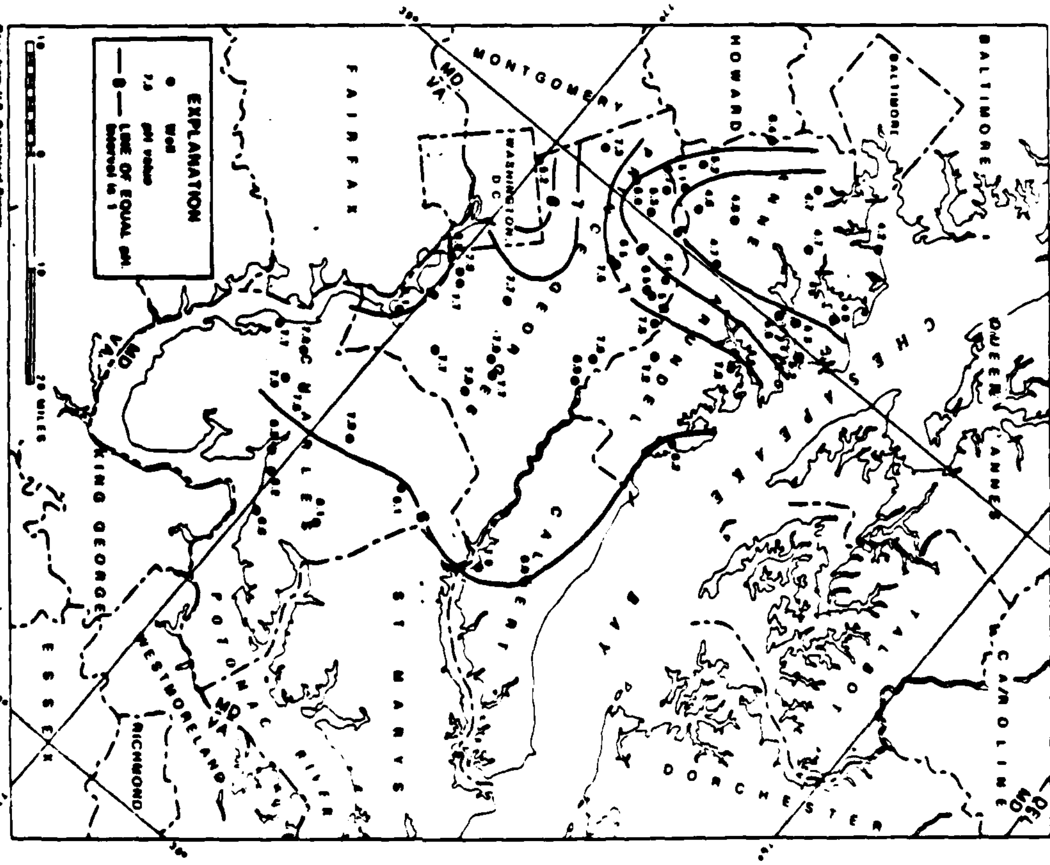
Table 2.—Groundwater chemistry—continued

Well No.	Potassium, dissolved (mg/L as K)	Chloride, dissolved (mg/L as Cl)	Sulfate, dissolved (mg/L as SO_4)	Fluoride, dissolved (mg/L as F)	Silica, dissolved (mg/L as SiO_2)	Copper, total recoverable (ug/L as Cu)	Iron, total recoverable (ug/L as Fe)	Hardness (mg/L as $CaCO_3$)	Hardness, noncarbonate (mg/L as $CaCO_3$)	Calcium, dissolved (mg/L as Ca)	Magnesium, dissolved (mg/L as Mg)	Sodium, dissolved (mg/L as Na)	Manganese, total recoverable (ug/L as Mn)	Temperature (°C)
						ANNE ARUNDEL COUNTY								
AC 21	1.0	3.9	5.5	<0.0	10	-	3,700	9	0	1.8	1.1	2.6	40	-
AD 29	1.0	1.2	6.9	.1	11	-	2,200	4	3	.9	.5	1.5	20	-
BB 5	.3	3.2	7.5	< .0	8.5	-	180	1	0	.3	.1	5.9	10	-
BE 45	1.4	7.0	14	.2	2.8	-	350	25	23	6.2	2.3	6.1	100	13.5
BF 2	-	1.5	2.0	-	-	-	7,500	18	-	-	-	-	-	15.5
BF 2	1.6	2.0	9.0	< .0	9.0	-	2,500	7	7	1.8	.6	1.3	70	15.0
CB 1	-	2.3	3.8	< .0	8.8	30	410	1	0	.3	.1	-	<5	-
CB 2	-	3.0	15	.1	8.6	60	3,400	6	6	1.4	.6	-	200	14.0
CC 78	.7	1.5	8.6	< .0	9.7	<5	1,100	5	4	1.2	.6	1.5	50	16.5
CC 78	-	1.5	-	-	-	-	1,000	5	5	-	-	-	-	14.0
CC 105	1.1	1.8	9.5	.3	8.3	-	300	9	4	2.0	1.0	2.0	100	-
CC 107	1.1	1.4	11	< .1	9.0	-	390	12	10	3.2	1.0	1.6	30	-
CE 1	1.4	1.8	26	.3	6.2	-	30,000	30	22	6.0	3.6	2.2	-	-
CE 46	2.7	2.0	20	.3	13	-	11,000	34	17	7.8	3.5	2.1	-	13.5
CF 11	1.8	1.5	25	.3	9.1	-	21,000	26	22	6.2	2.5	1.8	300	-
DE 35	5.0	2.2	12	.1	33	-	1,400	190	10	67	5.4	2.6	-	13.5
DF 12	1.7	1.2	32	.2	7.6	-	19,000	35	28	7.9	3.8	1.5	600	-
DF 12	1.8	.8	37	.3	8.6	50	19,000	39	35	7.5	5.0	1.2	450	4.5
DF 12	1.9	3.0	34	.3	8.3	70	14,000	31	32	7.0	3.4	1.2	250	-
ED 8	7.2	2.5	15	.1	17	-	1,200	160	6	50	7.7	4.2	-	-
FE 30	5.7	1.2	23	.3	11	-	1,400	130	5	40	8.3	4.4	<5	12.0
						CALVERT COUNTY								
DB 3	15	2.0	10	.2	13	-	510	108	0	22	12	9.8	0	-
DB 5	16	1.1	12	.2	9.8	-	-	104	0	22	12	9.8	-	-

Table 2.—Groundwater chemistry—continued

Well No.	Potassium, dissolved (mg/L as K)	Chloride, dissolved (mg/L as Cl)	Sulfate, dissolved (mg/L as SO_4)	Fluoride, dissolved (mg/L as F)	Silica, dissolved (mg/L as SiO_2)	Copper, total recoverable (ug/L as Cu)	Iron, total recoverable (ug/L as Fe)	Hardness (mg/L as $CaCO_3$)	Hardness, noncarbonate (mg/L as $CaCO_3$)	Calcium, dissolved (mg/L as Ca)	Magnesium, dissolved (mg/L as Mg)	Sodium, dissolved (mg/L as Na)	Manganese, total recoverable (ug/L as Mn)	Temperature (°C)
							CHARLES COUNTY							
BC 6	2.1	2.5	14	1.0	45	<5	70	2	0	.8	<0.0	56	<5	16.5
BC 12	8.6	1.2	13	.5	16	-	400	25	0	5.9	2.4	70	<5	-
CC 5	7.6	.5	7.5	.9	19	-	1,900	29	0	5.9	3.5	88	<5	-
CD 24	-	7.5	13	1.2	40	<5	60	8	0	1.2	1.2	-	10	-
CE 18	-	-	-	-	13	-	2,100	-	-	-	-	-	330	-
CE 16	-	2.0	15	.9	41	<5	1,300	3	0	.4	.6	-	<5	-
CF 9	12	1.6	8.0	.3	32	-	200	68	0	17	6.1	21	<5	14.5
CF 9	11	.9	11	.3	11	<5	730	87	0	21	8.3	20	<5	-
DD 5	6.8	2.5	11	.4	16	-	1,400	14	0	2.9	1.7	69	-	14.0
DD 10	6.0	2.4	16	.7	19	-	980	8	0	1.4	1.2	85	40	-
DF 9	12	1.6	8.0	.3	32	-	200	68	0	17	6.1	21	<5	14.5
EE 10	5.7	2.2	8.5	.3	13	-	960	15	0	3.4	1.6	68	-	-
							PRINCE GEORGES COUNTY							
BC 10	3.8	11	1.8	.2	3.7	-	2,300	32	3	5.6	6.6	6.0	120	8.5
BD 29	.4	3.0	1.8	<.5	7.4	-	470	5	2	1.0	.6	2.0	-	13.5
BE 6	2.3	3.0	31	<.0	3.1	-	58,000	6	1	1.0	.8	14	450	12.5
BE 16	-	.5	.6	<.0	7.9	200	<5	6	0	.3	1.2	-	<5	-
CB 1	-	14	17	<.0	9.9	<5	100	10	5	1.7	1.3	-	110	12.0
CE 17	2.1	12	18	.1	4.4	-	320	61	28	21	2.0	3.4	<5	15.0
CE 18	2.6	2.4	22	.1	7.6	-	1,800	78	18	20	6.8	3.5	200	14.0
CF 2	1.0	2.5	13	.1	45	-	12,730	51	3	17	2.0	3.1	100	14.0
CF 11	3.5	11	4.1	<.0	20	-	16,000	21	6	6.0	1.5	5.1	20	13.0
CF 25	1.8	2.0	14	.2	22	<5	10,000	24	10	7.6	1.2	1.6	120	14.0
DD 17	1.9	3.8	5.9	<.0	10	-	10,000	38	6	8.8	3.9	1.5	210	-
DF 5	6.1	2.1	9.7	.1	33	-	5,200	160	1	59	3.8	2.5	40	-
EB 1	.8	4.5	20	.1	24	-	190	3	0	1.2	.1	60	-	11.5
EC 26	4.8	1.5	24	.1	13	-	950	15	0	3.4	1.6	50	<5	-
EC 29	11	61	9.0	<.0	14	<5	140	10	0	2.0	1.2	62	<5	18.0
ED 32	16	1.6	22	.3	10	-	200	80	0	17	9.1	5.9	170	-
EF 3	1.9	2.1	10	.2	15	-	14,000	150	0	50	5.2	5.0	-	-
EF 16	6.6	1.6	45	.1	28	10	120	230	42	82	7.2	2.0	<5	14.5
FB 7	.6	1.5	28	.3	32	-	390	11	0	2.8	1.0	60	-	13.5
FC 14	9.2	1.2	13	.3	13	-	1,800	120	0	29	11	4.2	<5	12.0
FD 6	1.2	1.5	13	.2	13	-	210	130	0	37	7.9	3.4	-	-
FD 10	4.8	1.8	9.5	.2	16	-	280	120	0	35	8.6	2.7	<5	15.5
FD 32	5.5	1.4	9.2	.1	12	<5	830	160	0	39	15	4.9	<5	-

Adapted from Weil (1978).



Base from U.S. Geological Survey State base map, 1:500,000

Figure 9. -- Regional variation of pH in groundwater observed in all four aquifers.

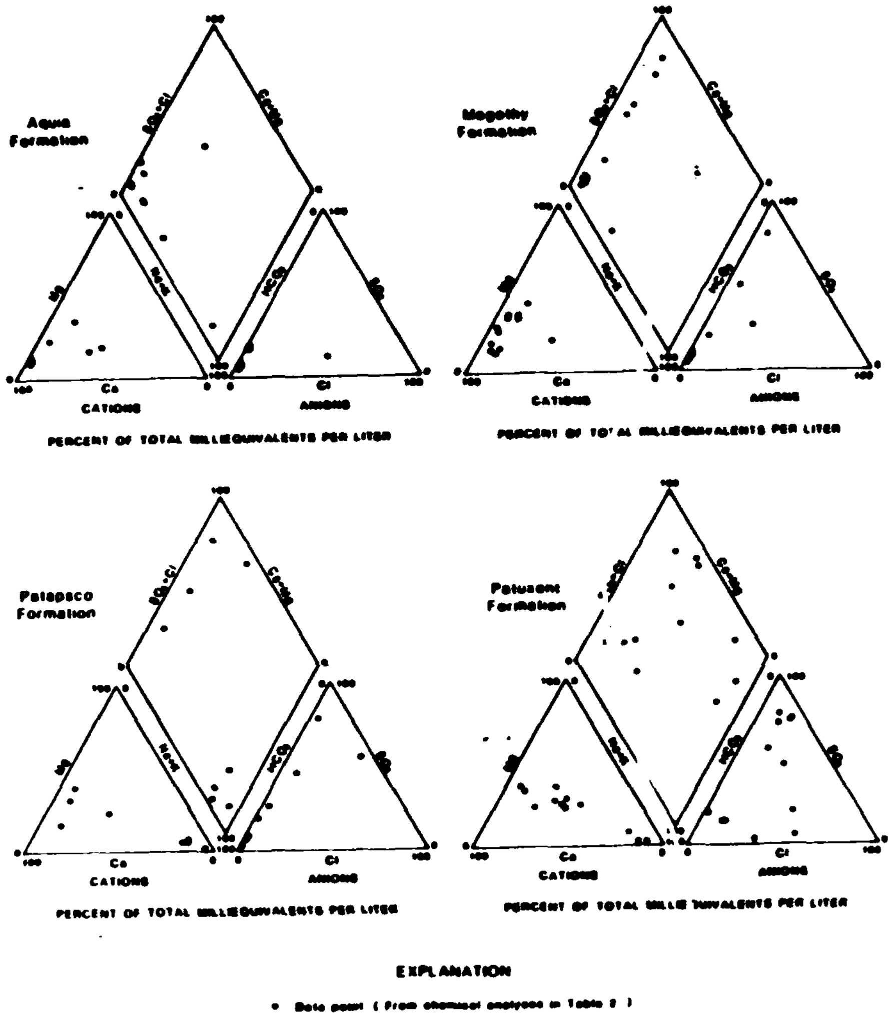


Figure 10. -- Water-analysis diagrams

PATUXENT FORMATION

Water from the Patuxent aquifer has an average pH of 6.1, and its dissolved solids range from 18 to 227 and average 91 mg/L (Otton, 1955). Analyses given in Table 2 indicate an average pH of 6.3, and the range and average of dissolved solids are 14-210 and 86 mg/L, respectively, which is in close agreement with Otton's values for southern Maryland. Water from the Patuxent aquifer varies from calcium bicarbonate and calcium sulfate to sodium bicarbonate and sodium sulfate types, as indicated in Figure 10. The groundwater naturally softens as it moves downgradient, as the calcium cations are exchanged for the sodium cations. Thus, typically, the deeper waters are sodium rich and the shallower waters calcium rich.

PATAPSCO FORMATION

According to Otton (1955), the range of dissolved solids of water from the Patapsco aquifer is from 12 to 345 and the average is 110 mg/L. He also indicates the average pH is 5.7. The average pH in Table 2 is 6.6, and the range and average of dissolved solids are 34-265 and 135 mg/L. Thus, the average pH in the study area is considerably greater, but the dissolved solids are similar to Otton's values for all of southern Maryland. Figure 10 indicates that water in the Patapsco is of the calcium sulfate, calcium bicarbonate, and sodium bicarbonate types.

MAGOTHY FORMATION

Otton (1955) indicates dissolved solids of water from the Magothy Formation range from 67 to 264 and average 131 mg/L. He also mentions that the average pH is 6.5. The average pH, and range and average of dissolved solids in Table 2 are 7.1, 55-224 mg/L, and 152 mg/L, respectively. As with the Patapsco Formation, the average pH in the study area is higher than that indicated by Otton for all of southern Maryland. Figure 10 indicates that the water is of either the calcium bicarbonate or calcium sulfate type. Most of the samples analyzed were taken from relatively shallow depths and therefore represent groundwater that has undergone only negligible sodium-calcium base exchange. The one sample taken at a depth greater than 600 feet (well CP 9, Charles County) is the only sample that has an appreciable percentage of sodium—21 mg/L.

AQUIA FORMATION

Otton (1955) again indicates that dissolved solids of water obtained from the Aquia Formation range from 67 to 300 and average 189 mg/L. The average pH is 7.6. Table 2 indicates the average pH and the range and average of dissolved solids are 7.8, 67-238, and 170 mg/L, respectively. These values and Otton's indicate that the water in the Aquia within the study area is typical for the Aquia throughout southern Maryland. Generally, water from the Aquia ranges from a high calcium to a high sodium bicarbonate type, as indicated in Figure 10.

AQUIFER SIMULATION AND ANALYSIS

THEORY

The purpose of the simulation is to determine the supply potential of groundwater pumped from the Cretaceous aquifers at selected sites and to estimate water-level changes. A quasi three-dimensional digital model by Achmad and Weigle (1979) was used. Effects of leakage from overlying and underlying confining beds are included. Variations of pumping rates with time are represented by a sequence of pumping periods. The model was used to estimate changes in heads (drawdowns) caused by various pumping regimes.

The model solves a two-dimensional flow equation combined with vertical leakage through confining beds.

The two-dimensional flow equation can be expressed:

$$T\frac{\partial^2 h}{\partial x^2} + T\frac{\partial^2 h}{\partial y^2} = S\frac{\partial h}{\partial t} + W \qquad (1)$$

where

T = transmissivity ($L^2 t^{-1}$);
h = potential head of water in the aquifer (L);
x, y = coordinates in two dimensions (L);
W = flux in the vertical direction, such as recharge, or pumpage (Lt^{-1});
t = time (t); and
S = storage coefficient of the aquifer (dimensionless).

Flow through confining beds is assumed to be vertical and is described as:

$$\frac{\partial^2 h'}{\partial z^2} = \frac{S'_s\,\partial h'}{K'\,\partial t} \qquad (2)$$

where

K' = vertical hydraulic conductivity in confining bed (Lt^{-1});
S'_s = specific storage (L^{-1});
h' = head potential in adjoining aquifers (L); and
t = time (T).

Assuming homogeneous conditions and assuming that the head distribution in the confining bed is bounded by the head of the adjacent aquifers, equation 2 can be simplified to show that the leakage into or out of the confining bed is a function of

$$q = \frac{K'}{b'}(h_1 - h_2) \qquad (3)$$

where

q = leakage (LT^{-1});
b' = thickness of confining bed (L); and
h_1, h_2 = heads in adjacent aquifers 1 and 2 (L).

Equation 3 indicates that leakage from confining layers is directly proportional to the head difference in adjacent aquifers and the permeability of the confining bed and is inversely proportional to the thickness of the confining bed.

Combining equation 1 and 3 describes the flow for each aquifer:

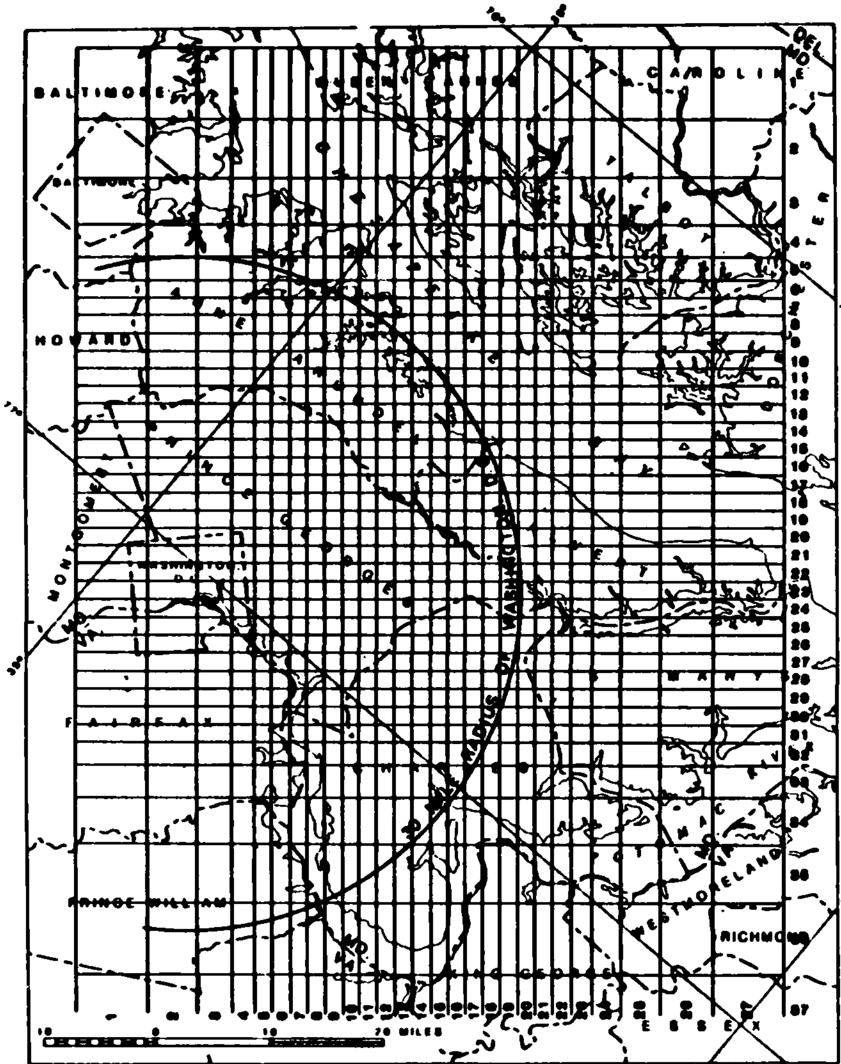
$$T\frac{\partial^2 h}{\partial x^2} + T\frac{\partial^2 h}{\partial y^2} + q_{bottom} - q_{top} = S\frac{\partial Q}{\partial t} + W \qquad (4)$$

where

q_{bottom} = flux (leakage) out of the bottom of the aquifer (LT^{-1});
q_{top} = flux into top of aquifer (LT^{-1}).

The digital model program solves equation 4 by a finite-difference method. Basically, the method involves the substitution of finite-difference approximations for the partial derivates in the flow equation (4).

To apply the finite-difference technique, the aquifer must first be "discretized"; that is, subdivided into a grid of rectangular blocks in which the aquifer properties are assumed to be uniform. (See Figure 11.) The center of the block is called the node. The hydraulic head at a given node is assumed to be an average head over the area of the node. Time dependence of hydraulic head at a given node is handled by dividing time into step increments; the head is calculated at each discrete time interval.



Base from U.S. Geological Survey
State base map, 1:500,000

Figure 11. -- Model grid used in finite-difference model.

THE MODEL

A digital model is a mathematical approximation of a physical system. A conceptual model, a representation of the physical system, must be amenable to mathematical analysis. A conceptual model should be constructed on available hydrologic data. However, since there was little to no data for most of the aquifer system within the study area, many assumptions were made. Also, much of the input data to the model was estimated. Therefore, the model results should be used carefully. However, on the basis of overall hydrologic information, the author believes that the model results probably are adequate to indicate the magnitude of the potential yields from the Coastal Plain aquifers.

The assumptions used for the model are:

1. Heads in the top layer represent water-table conditions and are held constant throughout the simulation. These constant heads provide recharge to the aquifers. Under steady-state conditions, this recharge occurs as leakage through the uppermost confining layer and amounts to about 0.6 inches per year per unit area. During transient conditions, this constant-head boundary may overestimate the amount of water leaking through the confining layers. Present information indicates that the water table has not been affected by pumping; thus, this constant-head boundary seems to be justified.

2. Hydraulic properties of the aquifers are isotropic, and all flow within the aquifers is horizontal. This assumption may not be true; however, there is no detailed information on such properties.

3. In the Patuxent, Patapsco, Magothy, and Aquia aquifers, heads in the boundary nodes in the recharge areas were held constant. Heads in these nodes were determined from contour maps constructed from data obtained from Mack (1962, 1966), Overbeck (1951), Slaughter and Otton (1968), and from information supplied by Chapelle (written commun., 1981), and Mack (written commun., 1980). During steady-state simulation, a flux of 0.1 to 0.3 inches per year per unit area flowed into the aquifers through the constant-head boundary. The rest of the model boundaries were assumed to be no-flow boundaries.

4. Underlying the Patuxent aquifer is the crystalline basement rock, modeled as a no-flow boundary.

5. The Patuxent and the Patapsco are each treated as single aquifers within the system. This simplification resulted from a deficiency of hydrologic data. Thus, the model results approximate average hydraulic heads in the combined sand layers within the Patapsco and within the Patuxent aquifers.

6. Flow through the confining beds is vertical and represents leakage to and from the aquifers. Under steady-state conditions, this leakage is generally downward, except under Chesapeake Bay, where there is an upward component.

The model was used to simulate pumping to determine yields in the Magothy, Patapsco, and Patuxent aquifers and to predict the resultant drawdowns in those three aquifers and in the overlying Aquia aquifer. It requires input of geohydrologic information for each aquifer. These data are defined for each node and are considered representative of the node. The data arrays for each layer are:

(1) a 27 x 37 rectangular grid (Figure 11) with variable nodal spacing (used to give the highest node density in the area of interest);

(2) initial prepumping head distribution;

(3) transmissivity distribution;

(4) storage coefficients;

(5) thickness of overlying confining bed;

(6) vertical hydraulic conductivity of the overlying confining bed;

(7) specific storage of the overlying confining bed; and

(8) location, pumping rate, and well diameter for simulated pumping wells.

CALIBRATION

Models for simulating groundwater flow are usually calibrated by simulating the known history of pumping and comparing heads computed by the model with heads measured in the field. In this study, however, this approach could not be used because of the lack of pumping from the Cretaceous aquifers within the area of interest (Figure 4).

In this study, a steady-state calibration was made. The prepumping head distributions were estimated from the available data. Steady-state model simulations computed *prepumping head* distributions. The computed head distributions were compared with the estimated head distributions. Vertical hydraulic conductivity and transmissivity values were adjusted until the computed head distributions compared favorably with the estimated head distributions.

The accuracy of calibration depends on the accuracy of the estimated prepumping head distribution, which was based on an extremely sparse number of water levels. The steady-state model, therefore, approximately simulates the system. Consequently, the model was not fully calibrated.

Transmissivities used to obtain a best match for steady-state simulation are indicated in Figures 3, 5, 6, and 7. Generally, the adjusted values vary but slightly from the initial estimates of transmissivity. In its calculations, the model uses the ratio of the vertical hydraulic conductivity and the thickness of the confining bed, k'/b', in units of 1/day. For the confining bed overlying the Aquia, k'/b' was 1.5×10^{-5} to 1.5×10^{-6}; for the Magothy, 1×10^{-4} to 1×10^{-5}; for the Patapsco, 1.2×10^{-5} to 1.2×10^{-6}; and for the Patuxent, 1×10^{-6} to 1×10^{-7}. The specific storage for the confining layers was 3×10^{-6} per foot.

Storage coefficients for the Cretaceous aquifers in Maryland range from 1×10^{-3} to 1×10^{-5} (Otton, 1955; Otton, 1981, oral commun.; and Mack and Mandle, 1977). Storage coefficients used for the four confined aquifers in this study were 1×10^{-3}. This relatively high value is due to the high percentage of silt and clay especially in the Potomac Group aquifers. A sensitivity analysis at sites 1 and 3 indicates that decreasing the storage coefficient up to two orders of magnitude results in less than a 4-percent change in drawdowns.

ESTIMATION OF EFFECTS DUE TO PUMPING STRESSES

The purpose of this study was to estimate the effect of pumping from the Cretaceous aquifers. The particular concern of the Corps is the potential for developing the Cretaceous aquifers within 30 miles of Washington, D.C., as an emergency water supply. For planning purposes, the Corps is interested in a pumping scheme in which the pumps are on from July through November, then turned off until the following July, at which time they again would be turned on from July through November. This pumping scheme would represent a 2-year drought.

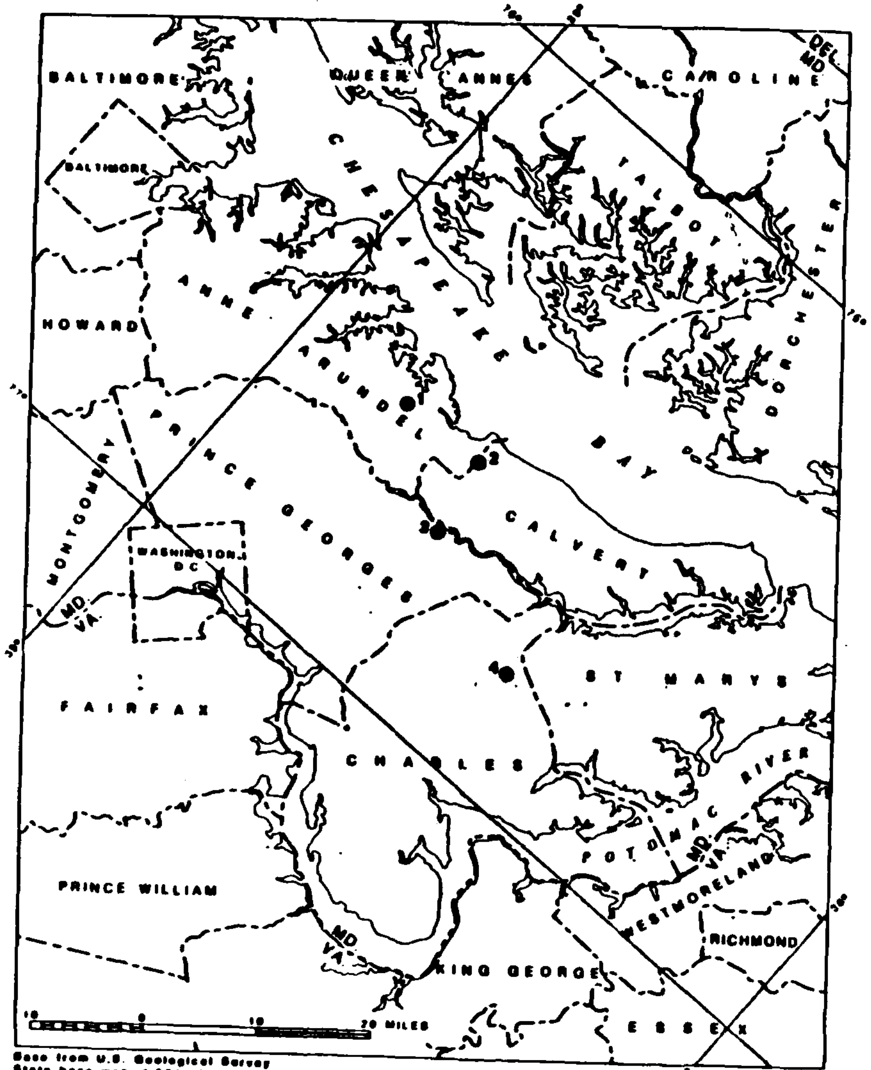
Model estimates were made at each of the four proposed locations. The model simulations assume that, at any given time, only one of the four locations is pumped. The four locations were selected with respect to the location of present pumpage, so that interference effects would be minimized. (See Figure 12.)

The results of the model simulations are presented in Table 3. For each site, a simulation was made for each aquifer at the indicated pumping rate. The pumping rates shown in Table 3 were determined from a test simulation which specified a 10 mgd withdrawal from each site. The principle of superposition (Bennett, 1976) was then utilized to calculate pumping rates at which heads would be drawn down to the top of each aquifer. This analysis assumed wells with 12-inch diameters that operate at 100-percent efficiency.

Each site is represented in the model by a node with an area of 2.4 square miles. The estimated drawdowns in Table 3 are the average drawdowns for that area of 2.4 square miles. Thus, in model simulation no. 1, a pumping stress of 11 mgd was applied to the Magothy aquifer within the nodal area of site 1. The model estimate of the average head drawdown within the nodal area is 60 feet in the Magothy aquifer and 20 feet in the overlying Aquia aquifer. The actual head distribution within the node would vary depending upon a number of factors, *including the* number and distribution of wells, the well diameter, the amount of the aquifer screened, and the transmissivity distribution.

Figures 13 to 16 indicate model estimates of drawdown in the aquifer being pumped at the end of the second 5-month pumping period. Drawdown in the Aquia aquifer is also shown for simulations of pumping from the Magothy aquifer. The estimated average drawdown of the pumped node is given in Table 3. For instance, in Figure 14, simulation 6 shows drawdown greater than 200 feet in the Patuxent aquifer, and, in Table 3, this simulation (no. 6) indicates an average drawdown within the pumped node of 280 feet. This indicates that, within the vicinity of the pumped node, the head gradient is extremely steep. Gradients toward individual wells in a well field would be even steeper and drawdown in wells would be greater than the average drawdowns given in Table 3. At best, these model estimates are rough approximations because input data to the model over most of the study area were sparse.

Also indicated in Table 3 for each simulation is the approximate depth to the bottom of the aquifer and the prepumping head measured in feet above the top of the aquifer. The depth to the bottom of the aquifer indicates approximately how deep wells would need to be in order to take full advantage of the entire aquifer. The head, indicated in column 4, indicates the available drawdown before dewatering of the aquifer occurs.



Base from U.S. Geological Survey
State base map, 1:500,000

Figure 12. - Location of four sites analyzed by model.

Table 3.--Summary of Model Simulations Used to Estimate Effects Due to Pumping Stresses at Sites 1, 2, 3, and 4

Simulation No.	Aquifer pumped	Depth to bottom of aquifer (ft)	Prepumping head above top of aquifer (ft)	Pumping rate in mode (mgd)	Average drawdown in mode (ft)	
					Pumped aquifer	Aquia aquifer
			SITE 1			
1	Magothy	450	340	11	60	20
2	Patapsco	725	570	8	110	3
3	Patuxent	1,200	1,000	8	210	<1
			SITE 2			
4	Magothy	650	500	10	80	30
5	Patapsco	850	640	9	140	2
6	Patuxent	1,520	1,320	10	280	<1
			SITE 3			
7	Magothy	440	390	9	70	9
8	Patapsco	660	450	5	90	1
9	Patuxent	1,300	1,000	6	210	<1
			SITE 4*			
10	Patapsco	825	470	7	120	2
11	Patuxent	1,350	1,100	5	200	<1

* Magothy aquifer not present at site 4.

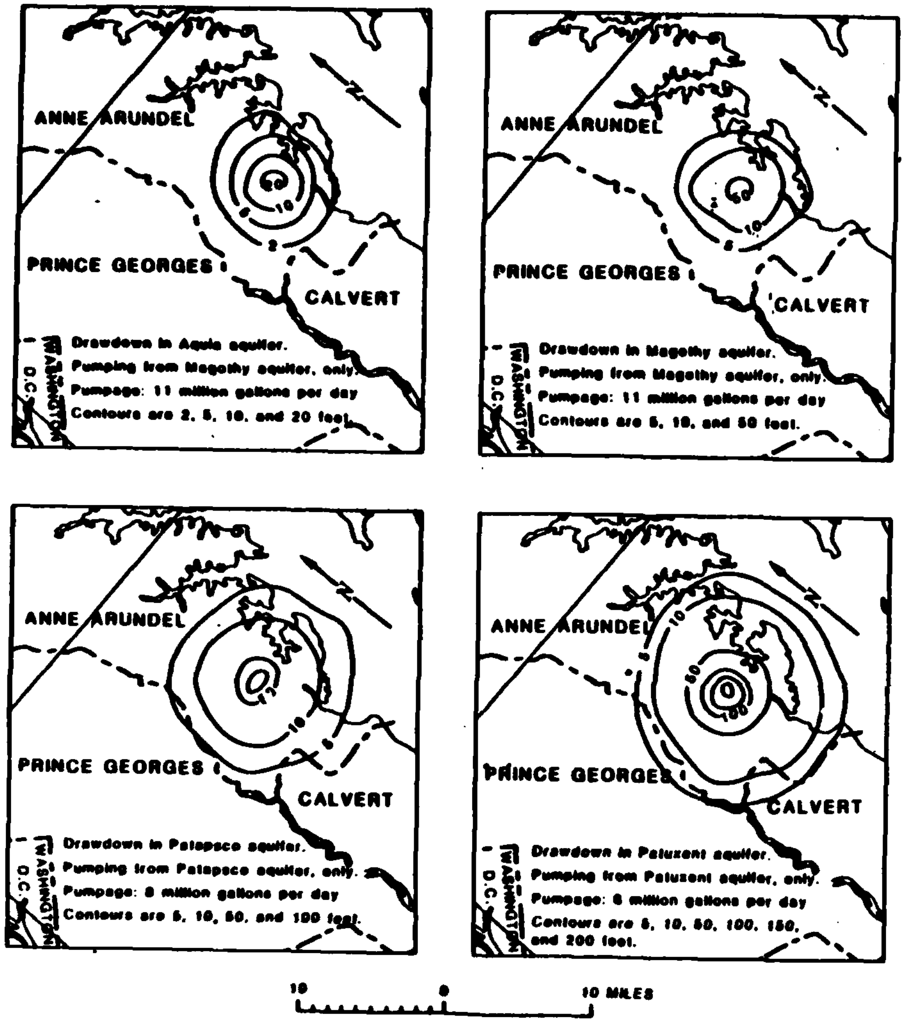


Figure 13. -- Estimated drawdowns for site 1.

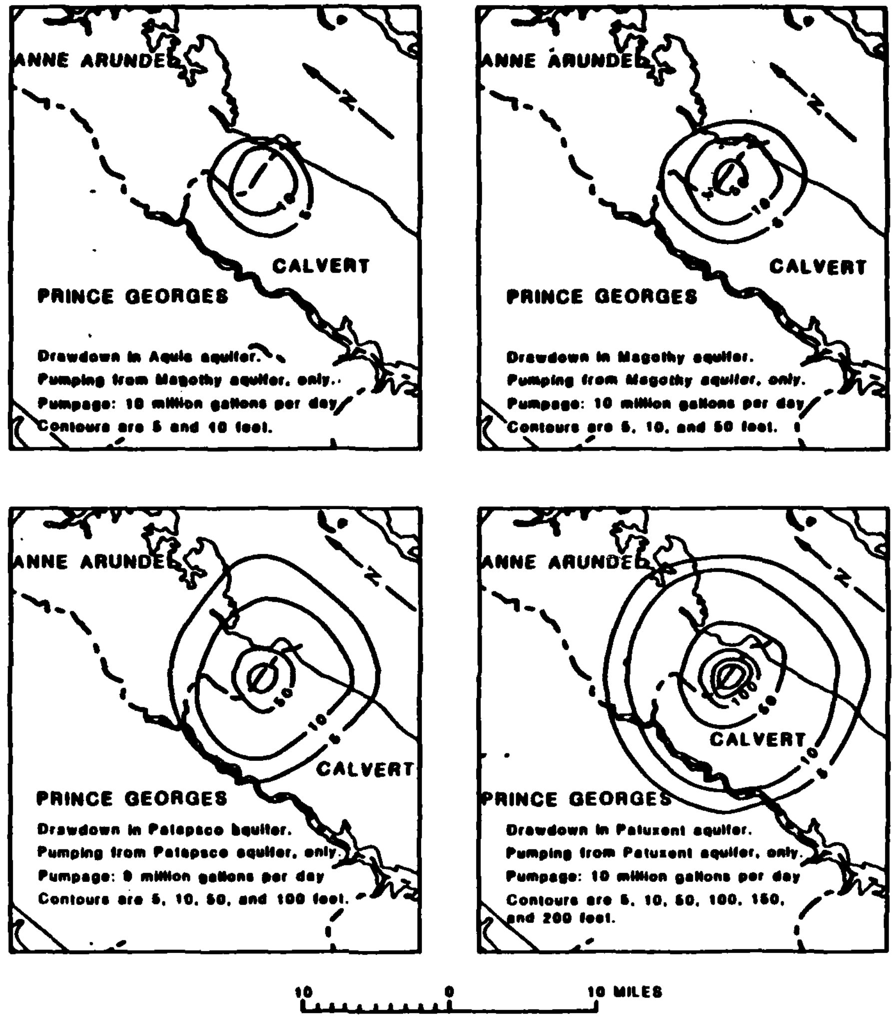
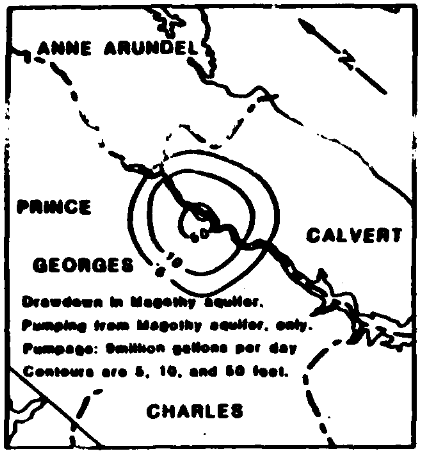
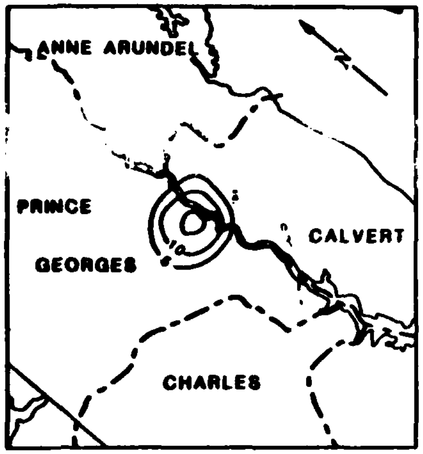


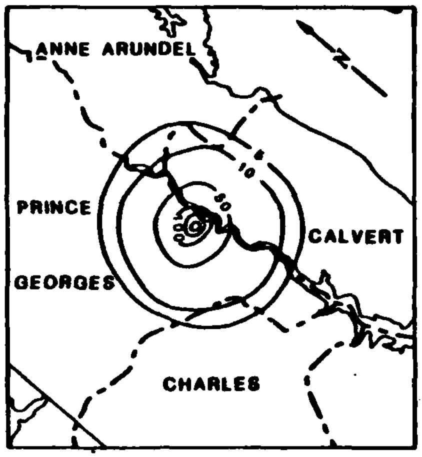
Figure 14. -- Estimated drawdowns for site 2.



Drawdown in Magothy aquifer.
Pumping from Magothy aquifer, only.
Pumpage: 5million gallons per day
Contours are 5, 10, and 50 feet.



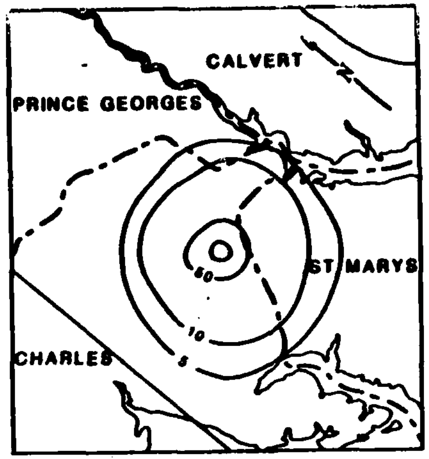
Drawdown in Patapsco aquifer.
Pumping from Patapsco aquifer, only.
Pumpage: 5 million gallons per day
Contours are 5, 10, and 50 feet.

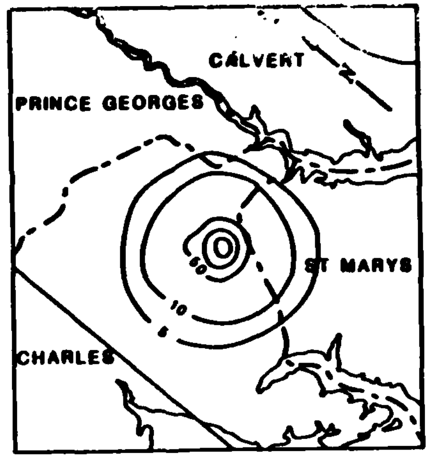


Drawdown in Patuxent aquifer.
Pumping from Patuxent aquifer, only.
Pumpage: 6 million gallons per day
Contours are 5, 10, 50, 100, 150, and 200 feet.

10 0 10 MILES

Figure 15. -- Estimated drawdowns for site 3.





10 0 10 MILES

Drawdown in Patapsco aquifer.
Pumping from Patapsco aquifer, only.
Pumpage: 7 million gallons per day
Contours are 5, 10, 50, and 100 feet.

Drawdown in Patuxent aquifer.
Pumping from Patuxent aquifer, only.
Pumpage: 5 million gallons per day
Contours are 5, 10, 50, 100, and 150 feet.

Figure 16. -- Estimated drawdowns for site 4.

The results indicate that, at the simulated pumping rates, the estimated drawdown is least when pumping is from the Magothy aquifer. For example, in model simulation no. 4, the model simulates a pumping stress of 10 mgd on the nodal area of site 2. At this pumping rate, it is estimated that the drawdown in the Magothy aquifer is 80 feet. The same pumping stress on the Patuxent aquifer (simulation no. 6) would result in an estimated drawdown of 280 feet, and a pumping stress of 9 mgd on the Patapsco aquifer (simulation no. 5) would result in an estimated drawdown of 140 feet. However, pumping the Magothy aquifer also results in the greatest drawdown of the overlying Aquia aquifer. For these same simulations, the estimated drawdown in the Aquia aquifer is 30 feet when pumping from the Magothy aquifer; less than 1 foot when pumping from the Patuxent aquifer; and 2 feet when pumping from the Patapsco aquifer.

For rough approximations, a linear relationship between pumping rates and drawdown may be assumed. Thus, at site 1, a doubling of the pumping rate of 11 mgd from the Magothy aquifer would result in an estimated average drawdown of 120 feet in the pumped aquifer. Similarly, if the total pumping from the Magothy over the 2.4 square miles of site 1 was increased to 55 mgd, the estimated average drawdown would be increased to about 300 feet. Dewatering of the aquifer, in this case, would occur when the drawdown exceeded 340 feet. Thus, the potential of the Magothy aquifer at this site is considerably greater than 11 mgd. However, as the drawdown in the pumped aquifer is increased, the drawdown in the overlying Aquia aquifer would also be increased. Also, drawdown in individual wells would be considerably greater than the average drawdowns indicated in Table 3.

The model indicates that after pumping has stopped, the recovery time of the average water levels (for pumped nodes) in the Aquia to within several feet of the prepumping level is 3 to 6 months. Figure 17 indicates that for simulation no. 1, recovery time of the Aquia aquifer to within 3 feet of the prepumping level after pumping ceased in the Magothy aquifer was 65 days, and, to within 1 foot, it was about 130 days. Similar recovery curves would apply for the other three sites.

The results of this study are model estimates derived from limited input data. Before undertaking the development of the groundwater resources, it would be necessary to have an exploratory testing program to ascertain the accuracy of the model estimates.

The pumping scheme analyzed in this report of two 5-month periods of pumping and the pumping rates indicated in Table 3 would not cause salt-water intrusion; however, it may occur if pumping rates increase or pumping time prolongs. According to Meisler (1980), the limit of the 10,000 mg/L isochlor, which would approximately represent the fresh/salt-water interface, is on the opposite side of Chesapeake Bay from the project area. The limit of the cone of depression that would develop at site 2 when pumping from the Patuxent aquifer at 10 mgd approaches no closer to the salt-water interface than about 10 miles. As the cone of depression expands outward, the chance of salt-water intrusion increases. Pumping rates greater than those indicated in Table 3, or pumping for longer periods, would result in larger cones of depression, thus increasing the chance of salt-water intrusion. Salt-water intrusion into the Aquia aquifer would occur when the cone of depression extended outward far enough to intercept the truncation of the Aquia aquifer by Chesapeake Bay (F. H. Chapelle, oral commun., 1981). Simulations 1 and 4 cause the largest drawdowns in the Aquia aquifer. In neither case would salt-water intrusion from Chesapeake Bay occur. However, pumping at greater rates or for longer period of time might result in salt-water intrusion from Chesapeake Bay.

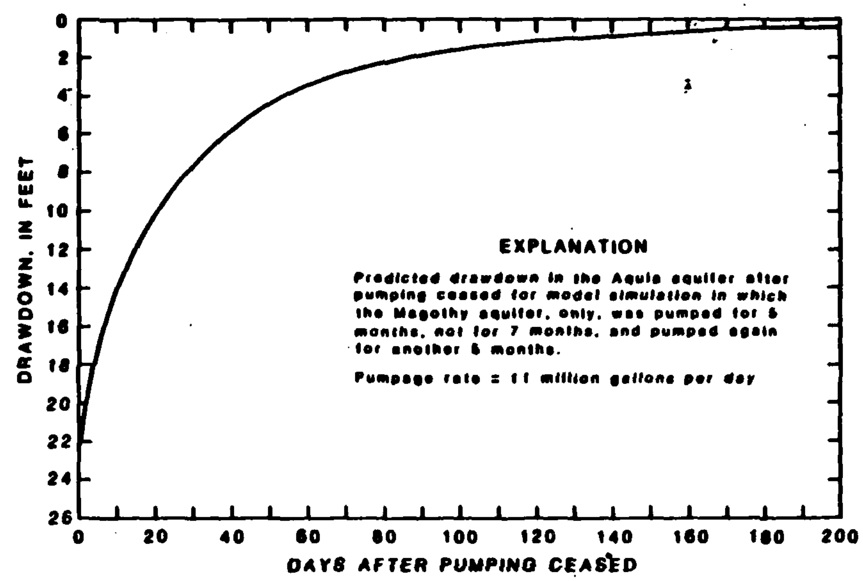


Figure 17. -- Water-level recovery of the Aquia aquifer, site 1.

SUMMARY

The model simulations indicated in Table 3 and in Figures 13-16 assumed that all sand beds for the indicated aquifer were screened and that well efficiencies are 100 percent. Under these assumptions, Table 3 indicates that at site 2, pumping at 10 mgd from wells screened in the Magothy aquifer would result in an average drawdown of 80 feet; at 9 mgd from the Patapsco, the drawdown would be 140 feet; and, at 10 mgd from the Patuxent, the drawdown would be 280 feet. The drawdown in the Aquia aquifer would be greatest when pumping from the Magothy aquifer; for example, at site 2, pumping at 10 mgd from the Magothy aquifer would result in an average drawdown of 30 feet.

Input data to the model for the updip parts of the aquifer were good, but, for the deeper parts where the evaluation was made, the input data were estimates, and therefore the simulated results are subject to greater uncertainty. To improve the reliability of the results, it will be necessary to obtain more information through test drilling.

A previous model by Papadopulos and others (1974) indicated that a short-term supply of 100 mgd would be available from the Magothy and Patapsco aquifers. They assumed a well field about 27 miles long and located about 25 miles east of Washington, D.C. The results of both the present study and that of Papadopulos and others indicate that the Coastal Plain aquifers extending from Washington, D.C., represent a large groundwater potential. However, before any development, it will be necessary to evaluate the extent of the resource and the magnitude of the effects of heavy pumping.

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1/ The name of this agency was changed to the Maryland Geological Survey in June 1964.

Department of Natural Resources
MARYLAND GEOLOGICAL SURVEY
Kenneth N. Weaver, Director

REPORT OF INVESTIGATIONS NO. 33
User's Guide Series

A QUASI THREE-DIMENSIONAL FINITE-DIFFERENCE GROUND-WATER FLOW MODEL WITH A FIELD APPLICATION

by
Grufron Achmad and James M. Weigle

Prepared in cooperation with the Geological Survey
United States Department of the Interior
Mayor and City Council of Ocean City
and
Worcester County Sanitary Commission

1979

CONTENTS

FIGURES

FIGURES, CON'T.

TABLE

A QUASI THREE-DIMENSIONAL FINITE-DIFFERENCE GROUND-WATER FLOW MODEL WITH A FIELD APPLICATION

by

Grufron Achmad and James M. Weigle*

ABSTRACT

This report describes a quasi three-dimensional model which was constructed from a sequence of aquifer areal-flow equations coupled by leakage terms representing flow through the confining beds. In simulating a four-aquifer system in northeastern Worcester County, Md., the model was able to reproduce the hydrographs of several observation wells with differences of less than 1 foot for 60 percent and less than 5 foot for 95 percent of the data points. The computer program of the model is given in the appendices.

* U S Geological Survey

INTRODUCTION

This report describes the quasi three-dimensional ground-water flow model (quasi 3-D model) which was developed by the Maryland Geological Survey in cooperation with the U.S. Geological Survey. Included in this report is a brief discussion of the application of the model in simulating a four-aquifer system in northeastern Worcester County, Md. (fig. 1). The computer program of the model is given in the appendices.

In a multiaquifer system in which the aquifers are separated by semi-permeable layers, the evaluation of ground-water flow requires a simultaneous consideration of the entire system. Jacob (1946) introduced into the nonsteady-state flow interaction between aquifers. Since then many investigators have published various results on the subject. Among these previous works, the analytical solution of a hypothetical two-aquifer system obtained by Neuman and Witherspoon (1969a, 1969b) is of particular interest in the present work, because it was used to verify the solution obtained in this investigation (Achmad, 1979). Briefly stated the Neuman and Witherspoon analytical solution considered a wide range of aquifer properties, solved for drawdowns in the pumped and unpumped aquifers, matched the Hantush solution (1956), and was tested by finite-element modeling methods.

When using an areal model for the simulation of a multiaquifer system, the aquifers must be coupled by terms representing flow through the confining beds. Bredehoeft and Pinder (1970) and Herrera and Rodarte (1973), among others, solved the nonsteady-state vertical-flow equation, taking into account the storage of the confining bed, and used this leakage term to couple the aquifers. The finite-difference multiaquifer model documented in this report adapted a leakage term which is the nonsteady-state solution of the confining-bed vertical flow equation; the solution assumes boundary conditions that are conformable with the head distribution in adjacent aquifers. The model provides two leakage terms representing flow through the lower and upper halves of the confining bed. The alternating-direction implicit procedure (ADIP) used in the model employs a row sweep for all the aquifers, followed by a similar column sweep to complete one iteration. As indicated by Achmad (1979) the model in conjunction with the numerical technique used in this investigation yielded a reasonable match with the analytical solution of Neuman and Witherspoon (1969a, 1969b).

ACKNOWLEDGMENTS

The author gratefully acknowledges the continuous support of H. J. Hansen, of the Maryland Geological Survey, and the valuable assistance of S. Papadopulos, G. D. Bennett, I. H. Kantrowitz, and E. P. Weeks, of the U.S. Geological Survey, in the early stage of this study. Gratitude is expressed to Ren Jen Sun, P. C. Trescott, P. P. Leahy, and L. J. McGreevy of the U.S. Geological Survey, for their helpful review of the manuscript.

QUASI 3-D MODEL

Development of Flow Equation

The quasi three-dimensional model was developed with the assumption that the flow in the aquifers is lateral and mathematically described as:

$$\frac{\partial}{\partial x} T_{xx} \frac{\partial h}{\partial x} + \frac{\partial}{\partial y} T_{yy} \frac{\partial h}{\partial y} = S \frac{\partial h}{\partial t} + W, \qquad (1)$$

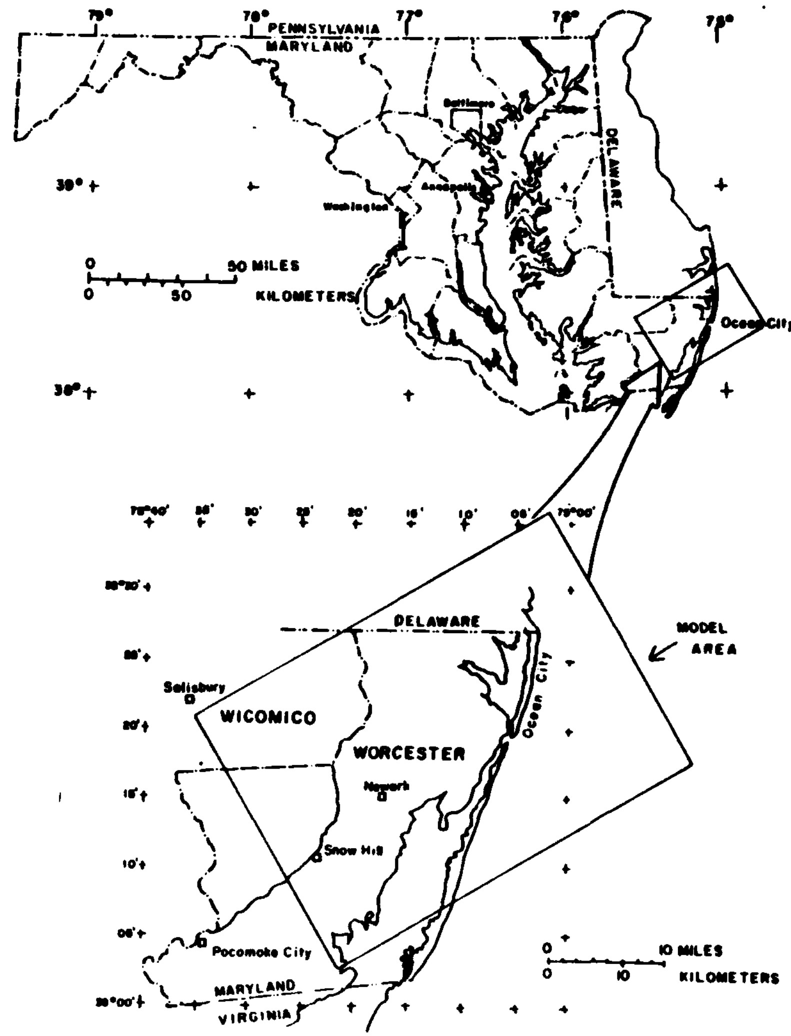

Figure 1. Location of model area

and the flow in the confining beds is vertical and mathematically described as:

$$\frac{\partial}{\partial z}(K'b')_{z z}\frac{\partial h'}{\partial z} = S'\frac{\partial h'}{\partial t}. \qquad (2)$$

Under homogeneous conditions, equation 2 can be written as:

$$\frac{\partial^2 h'}{\partial z^2} = \frac{S_s'}{K'}\frac{\partial h'}{\partial t}. \qquad (3)$$

Furthermore, by assuming that the head distribution in the confining bed is bounded by the head of the adjacent aquifers (h_{a_1} and h_{a_2}) and that initially the head in the confining bed is in equilibrium condition (h'^0), the boundary conditions governing equation 3 can be stated as:

$$h'(z, 0) = h'^0,$$
$$h'(0, t) = h_{a_1}, \qquad (4)$$
$$h'(b, t) = h_{a_2}.$$

Solving the boundary value problem of equations 3 and 4, the head distribution in the confining bed can be described by the following equations:

$$h'(z, t) = h_{a_1} + \left(\frac{h_{a_2} - h_{a_1}}{b'}\right)z + \frac{2}{\pi}(h^0_{a_1} - h_{a_1})\sum_{n=1}^{\infty}\frac{1}{n}\sin\left(\frac{n\pi}{b'}z\right)e^{-\lambda t} - \frac{2}{\pi}(h^0_{a_2} - h_{a_2})\sum_{n=1}^{\infty}\frac{1}{n}\cos(n\pi)\sin\left(\frac{n\pi}{b'}z\right)e^{-\lambda t}, \qquad (5)$$

where

$$\lambda t = n^2\pi^2\frac{K'}{S_s'}\frac{1}{b'^2}t.$$

The amount of flux resulting from the head distribution $h'(z, t)$ in the confining bed is:

$$q(z, t) = -K'\left(\frac{dh'}{dz}\right),$$

or

$$q(z, t) = -\left\{\frac{K'}{b'}(h_{a_2} - h_{a_1}) + \frac{2K'}{b'}(h^0_{a_1} - h_{a_1})\sum_{n=1}^{\infty}\cos\left(\frac{n\pi}{b'}z\right)e^{-\lambda t} - \frac{2K'}{b'}(h^0_{a_2} - h_{a_2})\sum_{n=1}^{\infty}\cos(n\pi)\cos\left(\frac{n\pi}{b'}z\right)e^{-\lambda t}\right\}. \qquad (6)$$

The two leakage terms resulting from equation 6 are q_{top}, evaluated at $z = 0$; and q_{bottom}, evaluated at $z = b$. A more detailed description of the approximate solution of equations 5 and 6 is given by Achmad (1979).

Following the conditions described above, the flow equation for each aquifer becomes:

$$\frac{\partial}{\partial x}T_{xx}\frac{\partial h}{\partial x} + \frac{\partial}{\partial y}T_{yy}\frac{\partial h}{\partial y} + q_{bottom} - q_{top} = S\frac{\partial h}{\partial t} + W. \qquad (7)$$

Finite-Difference Approximations

The finite-difference approximation of the space derivative of the flow equation is obtained by manipulating the Taylor series expansion of function h in the vicinity of a node (x, y, z), assuming that h is continuous, and that higher-order derivatives of h with respect to x exist:

$$h(x + \Delta x, y, z) = h(x, y, z) + \Delta x\frac{\partial h}{\partial x} + \frac{\Delta x^2}{2!}\frac{\partial^2 h}{\partial x^2} + \frac{\Delta x^3}{3!}\frac{\partial^3 h}{\partial x^3} + \frac{\Delta x^4}{4!}\frac{\partial^4 h}{\partial x^4} + \cdots + \frac{\Delta x^n}{n!}\frac{\partial^n h}{\partial x^n},$$

and

$$h(x - \Delta x, y, z) = h(x, y, z) - \Delta x\frac{\partial h}{\partial x} + \frac{\Delta x^2}{2!}\frac{\partial^2 h}{\partial x^2} - \frac{\Delta x^3}{3!}\frac{\partial^3 h}{\partial x^3} + \frac{\Delta x^4}{4!}\frac{\partial^4 h}{\partial x^4} + \cdots + \frac{\Delta x^n}{n!}\frac{\partial^n h}{\partial x^n}.$$

Combining these two equations and neglecting the derivatives higher than the third order (Δx^3), an approximate second order partial derivative of h with respect to x can be expressed as:

$$\frac{\partial^2 h}{\partial x^2} = \frac{h(x - \Delta x, y, z) - 2h(x, y, z) + h(x + \Delta x, y, z)}{\Delta x^2}$$

Similarly, the second order partial derivative of h with respect to y can be expanded as:

$$\frac{\partial^2 h}{\partial y^2} = \frac{h(x, y - \Delta y, z) - 2h(x, y, z) + h(x, y + \Delta y, z)}{\Delta y^2}.$$

The time derivative at a node $(x = i, y = j)$ is approximated as follows:

$$h(i, j, z, t + \Delta t) = h_{(i,j,z,t)} + \frac{\Delta t}{1!}\frac{\partial h}{\partial t} + \frac{\Delta t^2}{2!}\frac{\partial^2 h}{\partial t^2} + \cdots + \frac{\Delta t^n}{n!}\frac{\partial^n h}{\partial t^n}$$

Omitting the derivatives higher than the first order, a partial derivative of h with respect to time can be written as:

$$\frac{\partial h}{\partial t} = \frac{h_{(i,j,z,t+\Delta t)} - h_{(i,j,z,t)}}{\Delta t}.$$

Numerical Method

The finite-difference approximation of the partial differential equation 7 can be stated as follows:

$$\frac{2}{\Delta x_i}\left\{Tx_{i-\frac{1}{2},j,k}\frac{(h_{i-1,j,k} - h_{i,j,k})}{\Delta x_{i-1} + \Delta x_i} - Tx_{i+\frac{1}{2},j,k}\frac{(h_{i,j,k} - h_{i+1,j,k})}{\Delta x_i + \Delta x_{i+1}}\right\} + \frac{2}{\Delta y_j}\left\{Ty_{i,j-\frac{1}{2},k}\frac{(h_{i,j-1,k} - h_{i,j,k})}{\Delta y_{j-1} + \Delta y_j} - Ty_{i,j+\frac{1}{2},k}\frac{(h_{i,j,k} - h_{i,j+1,k})}{\Delta y_j + \Delta y_{j+1}}\right\} + q_{k,bottom} - q_{k+1,top} = \frac{S}{\Delta t}(h^{t+1}_{i,j,k} - h^t_{i,j,k}) + \sum_{i,j,k=1}^{n} Q(x, y, z)\delta(x - x_i)(y - y_j)(z - z_k), \qquad (8)$$

where

$$q_{k,bottom} = -\left\{\frac{K'}{b'}(h_{a_k} - h_{a_{k-1}}) + \frac{2K'}{b'}(h^0_{a_{k-1}} - h_{a_{k-1}})\sum_{n=1}^{\infty}\cos(n\pi)e^{-\lambda t} - \frac{2K'}{b'}(h^0_{a_k} - h_{a_k})\sum_{n=1}^{\infty}e^{-\lambda t}\right\},$$

and

$$q_{k+1,top} = -\left\{\frac{K'}{b'}(h_{a_{k+1}} - h_{a_k}) + \frac{2K'}{b'}(h^0_{a_k} - h_{a_k})\sum_{n=1}^{\infty}e^{-\lambda t} - \frac{2K'}{b'}(h^0_{k+1} - h_{a_{k+1}})\sum_{n=1}^{\infty}\cos(n\pi)e^{-\lambda t}\right\}.$$

Simplifying in a compact algebraic form, equation 8 can be written as:

$$AX_{i,j,k}h_{i-1,j,k} + (-AX_{i,j,k} - CX_{i,j,k})h_{i,j,k} + CX_{i,j,k}h_{i+1,j,k} + AY_{i,j,k}h_{i,j-1,k} + (-AY_{i,j,k} - CY_{i,j,k})h_{i,j,k} + CY_{i,j,k}h_{i,j+1,k}$$
$$= STRM_{i,j,k}(h^{t+1}_{i,j,k} - h^t_{i,j,k}) + QTRM_{i,j,k} + QL1_{i,j,k}(h^0_{i,j,k-1} - h^{t+1}_{i,j,k-1}) - QL2_{i,j,k}(h^0_{i,j,k} - h^{t+1}_{i,j,k})$$
$$- QL2_{i,j,k+1}(h^0_{i,j,k} - h^{t+1}_{i,j,k}) + QL1_{i,j,k+1}(h^0_{i,j,k+1} - h^{t+1,0}_{i,j,k+1}) + R_{i,j,k}(h^{t+1}_{i,j,k} - h^{t+1}_{i,j,k-1}) - R_{i,j,k+1}(h^{t+1,0}_{i,j,k+1} - h^{t+1}_{i,j,k}), \qquad (9)$$

where

$$STRM_{i,j,k} = \frac{S}{\Delta t}$$
$$QTRM_{i,j,k} = \sum_{i,j,k}^{n} Q(x, y, z)\delta(x - x_i)(y - y_j)(z - z_k),$$
$$QL1_{i,j,k} = \frac{K'}{b'}\left(2\sum_{n=1}^{\infty}\cos(n\pi)e^{-n^2\pi^2 t_D}\right),$$
$$QL2_{i,j,k} = \frac{K'}{b'}\left(2\sum_{n=1}^{\infty}e^{-n^2\pi^2 t_D}\right),$$
$$R_{i,j,k} = \frac{K'}{b'},$$
$$t_D = \frac{K'}{S_s' b'^2}t.$$

and

$$AX_{i,j,k} = 2T_{i-\frac{1}{2},j,k}/[(\Delta x_i + \Delta x_{i-1})\Delta x_i],$$
$$CX_{i,j,k} = 2T_{i+\frac{1}{2},j,k}/[(\Delta x_i + \Delta x_{i+1})\Delta x_i].$$

Similarly for $AY_{i,j,k}$, and $CY_{i,j,k}$.

The leakage terms, as calculated in equation 9, are made implicit in terms of head distribution in the corresponding aquifers, and are, therefore, evaluated at the same time step as the head distribution is calculated. For this purpose, at the beginning of each time step the $h^{t+1/2}_{i,j,k+1}$ term in equation 9 is estimated using a straight line head prediction (Achmed, 1973):

$$h^{t+1/2}_{i,j,k+1} = h^{t}_{i,j,k+1} + (h^{t}_{i,j,k+1} - h^{t-1}_{i,j,k+1})\frac{\Delta t_{new}}{\Delta t_{old}}.$$

The alternating direction implicit procedure (ADIP), introduced by Peaceman and Rachford (1955), is used in solving the resulting finite-difference equations. Setting up equation 9 for ADIP solution the linear algebraic equation for each axial direction calculation is the following:

For X direction calculation:

$$AX_{i,j,k}h^{t+1}_{i-1,j,k} + BX_{i,j,k}h^{t+1}_{i,j,k} + CX_{i,j,k}h^{t+1}_{i+1,j,k} = DX_{i,j,k},$$

where

$$BX_{i,j,k} = -AX_{i,j,k} - CX_{i,j,k} - AY_{i,j,k} - CY_{i,j,k} - STRM_{i,j,k}$$
$$- QL2_{i,j,k} - R_{i,j,k} - QL2_{i,j,k+1} - R_{i,j,k+1},$$
$$DX_{i,j,k} = -AY_{i,j,k}h^{t}_{i,j-1,k} - CY_{i,j,k}h^{t}_{i,j+1,k} - STRM_{i,j,k}h^{t}_{i,j,k}$$
$$+ QTRM_{i,j,k} + LTRM_{i,j,k},$$

and

$$LTRM = QL1_{i,j,k}(h^{0}_{i,j,k-1} - h^{t+1/2}_{i,j,k-1}) - QL2_{i,j,k}h^{0}_{i,j,k} + QL1_{i,j,k+1}(h^{0}_{i,j,k+1} - h^{t+1/2}_{i,j,k+1})$$
$$- QL2_{i,j,k+1}h^{0}_{i,j,k} - R_{i,j,k}h^{t+1/2}_{i,j,k-1} - R_{i,j,k+1}h^{t+1/2}_{i,j,k+1}.$$

For Y direction calculation.

$$AY_{i,j,k}h^{t+1}_{i,j-1,k} + BY_{i,j,k}h^{t+1}_{i,j,k} + CY_{i,j,k}h^{t+1}_{i,j+1,k} = DY_{i,j,k},$$

where

$$BY_{i,j,k} = -AY_{i,j,k} - CY_{i,j,k} - AX_{i,j,k} - CX_{i,j,k}$$
$$- STRM_{i,j,k} - QL2_{i,j,k} - R_{i,j,k} - QL2_{i,j,k+1} - R_{i,j,k+1},$$

and

$$DY_{i,j,k} = -AX_{i,j,k}h^{t+1}_{i-1,j,k} - CX_{i,j,k}h^{t+1}_{i+1,j,k} - STRM_{i,j,k}h^{t}_{i,j,k}$$
$$+ QTRM_{i,j,k} + LTRM_{i,j,k}.$$

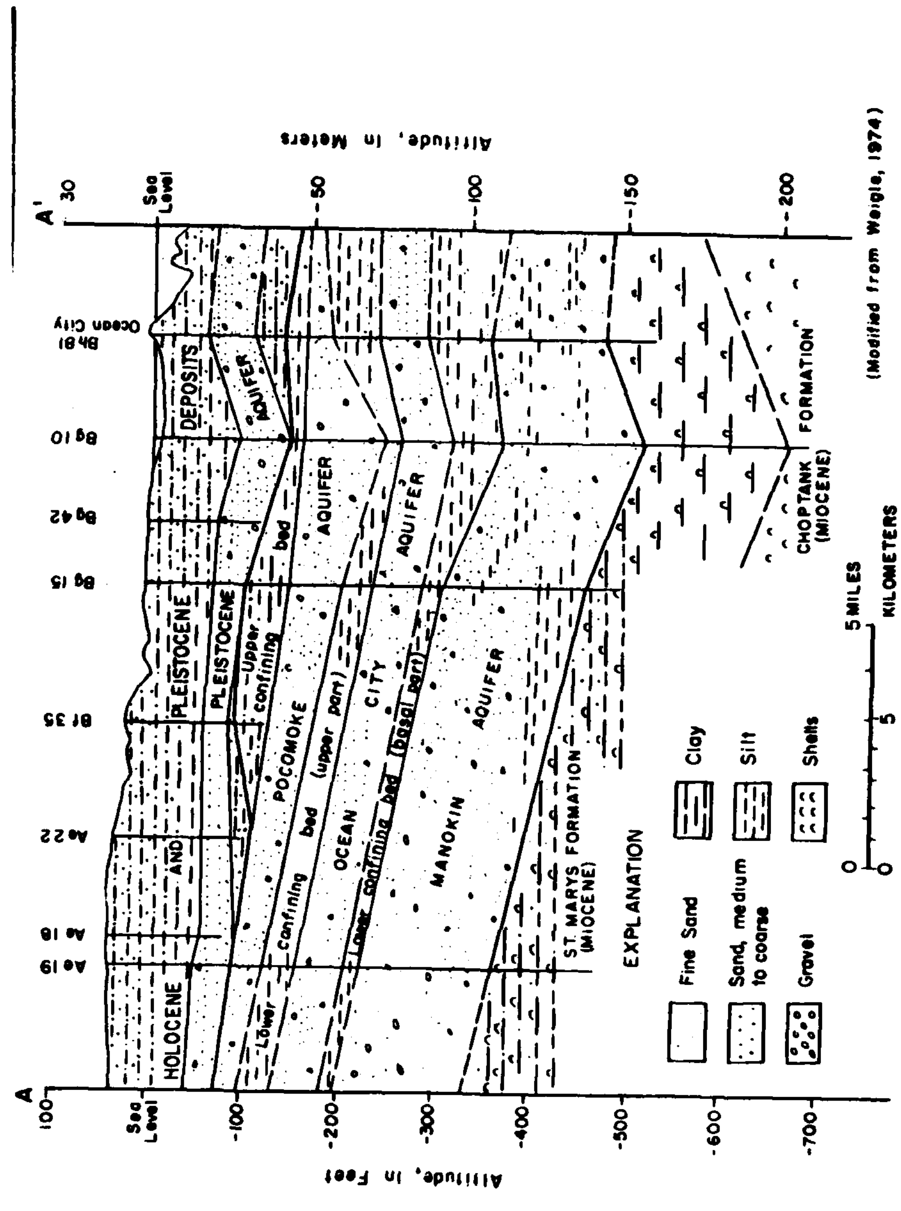

Figure 2. Schematic geologic section A-A'.

In solving the flow equation for a two-dimensional one-layer model the ADIP makes an alternate row and column calculations for all the nodes in that layer. In a multiaquifer system, the flow equation must be solved simultaneously for all the aquifers. In order to simultaneously solve the finite-difference equations the row and column calculations were performed alternately so that the entire system of nodes were covered at each time step. In this manner, the row calculation sweeps the nodes in the first aquifer, proceeds with the ones in the second, and finishes with the ones in the last aquifer; and the column calculation sweeps the nodes in the first, proceeds with the ones in the second, and finishes with the ones in the last aquifer. The procedures are repeated until the closure criterion is met.

FIELD APPLICATION

Description of Aquifer System

The 450-ft (137 m) thick fresh ground-water reservoir in northeastern Worcester County, Maryland contains a series of interconnected aquifers of Miocene, Pliocene (?) and Pleistocene age. Seasonally heavy pumping of 5 Mgal/d (18,925 m^3/d) produces a maximum 30-ft (9 m) areal drawdown in the pumped aquifer and a 5-ft (1.5 m) areal drawdown in the adjacent aquifers. The reduction of pumping during the off-season, combined with recharge from precipitation, results in annual recovery of water levels. The aquifer system is best represented as a multiaquifer system.

The aquifers under study are the four uppermost aquifers, which supply the water needed for farming, domestic use, and public utilities in Worcester County. In that vicinity, the aquifers are known as the Pleistocene, Pocomoke, Ocean City, and Manokin aquifers (fig. 2).

Finite-Difference Grid

The model area, which includes only a portion of the entire aquifer system, covers 875 mi^2 (2266 km^2) and is divided into 35 × 25 elemental blocks of 1-square mile each (fig. 3). Specific conditions are assigned to the boundary nodes of the model to represent the hydrologic setting.

Constant-head conditions are assigned to the boundary nodes at the northwestern corner of the model, which represents a part of the outcrop area of the aquifers. The southeastern boundary nodes are made constant head nodes as they are assumed to represent a part of the aquifer zone of ocean discharge. These assignments establish hydraulic gradient and provide for subsurface flow in the model. The other boundary nodes, which define the aquifer limits or represent flow divides, are assigned no-flow boundary conditions.

The simulation of flow in the aquifers and of leakage between the aquifers creates a recharge-discharge mechanism in the model, and simulates the field conditions.

Source of Data

The data used for the model were derived mostly from recently published reports. Prepumping potentiometric and transmissivity matrices were derived from maps prepared by Weigle (1974). Data were extrapolated to cover the complete model area and an average value was estimated for each element of the finite-difference grid.

Storage coefficients were selected from values reported by Cushing, Kantrowitz, and Taylor (1973). The confining-bed hydraulic characteristics were derived from laboratory tests of Miocene clays cored at Ocean City in 1976 (data filed in U.S. Geological Survey Maryland District Office) and from field and laboratory results obtained at Salisbury (Wolff, 1970).

Model Calibration

Calibration was achieved by adjusting values of the hydraulic characteristics used in the model until the model could reproduce the prepumping potentiometric surface, and match the measured hydrographs of the observation wells.

In general, the amount of change of parameter values permitted was based on: (1) the reliability of the data, (2) the significance of the parameter, and (3) the range of values normally encountered in the field.

Only minor adjustments were allowed in the water levels of the model in the process of duplicating the prepumping potentiometric surfaces. These adjustments consisted of equating the water levels of the Pocomoke aquifer in its subcrop area with the water levels of the Pleistocene aquifer and smoothing the water levels in the Manokin aquifer at the southwest corner of the model. The simulated prepumping water levels are shown in figures 4 to 7.

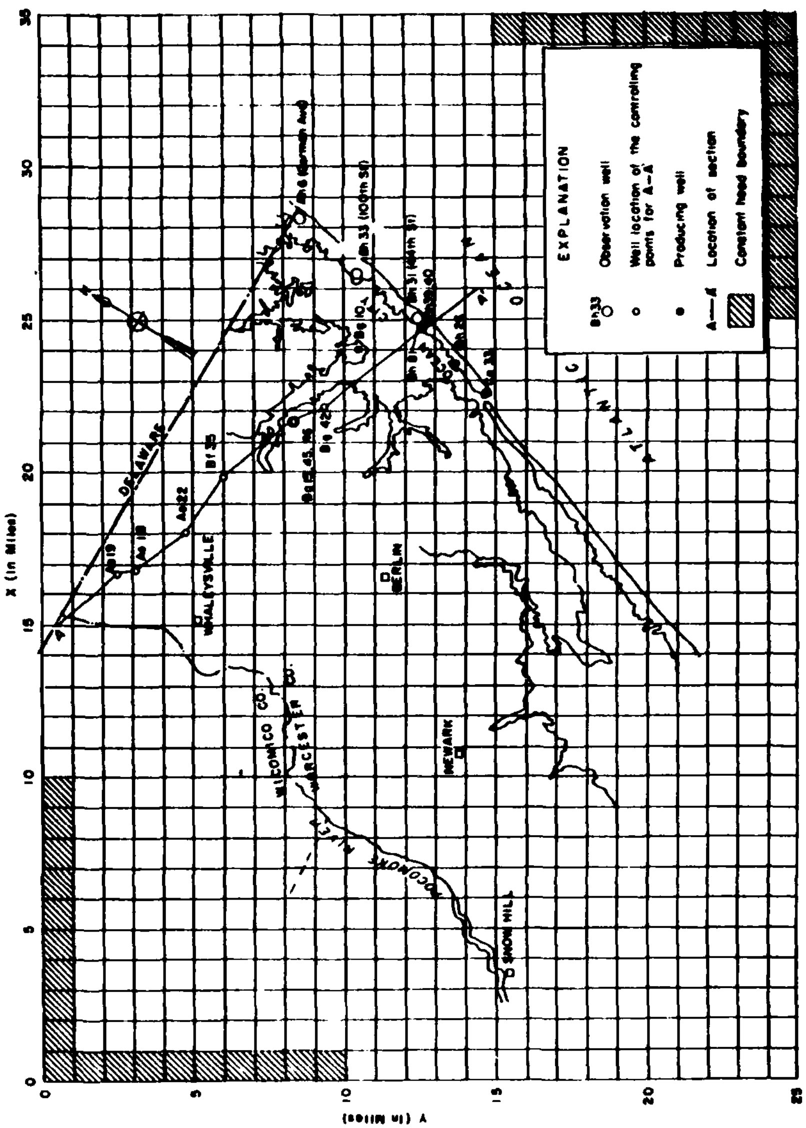

Figure 3. Grid system and location of geologic section A-A'.

During calibration, only minor adjustments were permitted in the transmissivity matrices because the transmissivity and water-level data were considered the most reliable of the model input parameters. The calibrated transmissivity values are shown in figures 8 to 11.

The aquifers were regarded as having a uniform storage coefficient (S) and a single value was assigned to each aquifer. After several trial runs, it was found that the storage coefficient values that gave the best results are as follows: $S_{Manokin} = 0.00012$, $S_{Ocean\ City} = 0.00009$, $S_{Pocomoke} = 0.00012$, and $S_{Pleistocene} = 0.0015$.

The vertical hydraulic conductivity and the specific storage of the confining beds are the least known parameters. Values for vertical conductivity and specific storage of a Miocene clay from Salisbury, Maryland have been reported by Wolff (1970). The values are as follows: hydraulic conductivity ranged from 2.8×10^{-5} to 5.7×10^{-3} ft/day (8.5×10^{-6} to 1.7×10^{-3} m/day); specific storage ranged from 0.3×10^{-4} to 1.8×10^{-4} ft^{-1} (9.8×10^{-5} to 5.9×10^{-4} m^{-1}). Laboratory tests made in 1976 on a Pleistocene clay sample from Ocean City (filed in U.S. Geological Survey Maryland District Office) indicated a vertical conductivity of 9.0×10^{-3} ft/day (3.0×10^{-3} m/day). However, simulation runs indicated that the most satisfactory results were obtained by using the following values:

$S_s' = 0.4 \times 10^{-4}$ ft^{-1} (0.1×10^{-4} m^{-1}), $K'_{vertical}$ for confining bed on top of Pocomoke and Ocean City aquifer $= 3.6 \times 10^{-3}$ ft/d (1.1×10^{-3} m/d), and $K'_{vertical}$ for confining bed on top of Manokin $= 1.9 \times 10^{-3}$ ft/d (5.8×10^{-4} m/d).

Using the hydraulic parameters described above, the production performance of the multiaquifer system at Ocean City was simulated for a 2-year period of 1971 and 1972. The model simulated the effect of pumping from the Ocean City aquifer at four pumping centers in Ocean City (table 1). The pumping was not uniformly distributed throughout the year; it varied seasonally, reflecting the fact that Ocean City is a resort area.

TABLE 1. RATE OF WATER WITHDRAWAL FROM WELLS SCREENED IN THE OCEAN CITY AQUIFER
(In Million Gallons Per Day)

	1971					1972			
	well	1	2	3	4	well 1	2	3	4
Jan		0.386	0.081	0.183	0.610	0.622	0.0	0.242	0.610
Feb		0	.482	.236	.610	.553	.0	.340	.610
Mar		.0	.444	.256	.920	.451	.0	.318	.920
Apr		.0	.736	.441	.920	.678	.0	.483	.920
May		.221	.874	.516	.920	1.041	.146	.627	.920
Jun		.616	1.077	.987	.920	1.118	.578	.916	.920
Jul		1.080	1.280	1.406	.920	1.374	1.370	1.576	.920
Aug		1.061	1.490	1.745	.920	1.510	1.423	1.918	.920
Sep		.582	.814	.879	.920	.801	.786	.966	.920
Oct		.161	.528	.414	.920	.818	.0	.632	.920
Nov		.263	.236	.367	.610	.601	.0	.516	.610
Dec		.447	.0	.251	.610	.618	.0	.518	.610

Explanation
well 1 - Cg 33, South-end
well 2 - Bh 26, 14th and 15th Street
well 3 - Bh 81, 44th Street
well 4 - Bh 39 and 40, Convention Hall
(see figure 3 for well location)

The two-year period of pumping was divided into 25 pumping intervals of one month each. Starting with an initial time step of 24 hours and using a multiplication factor of 1.2, the time step was gradually increased up to 300 hours where it was then kept constant. With this time step, the ADIP converged to a solution within two, or at the most three iterations, which is considered reasonable and effective.

A comparison of simulated hydrographs with records of observation wells is shown in figure 12. The starting head used in the simulation is the prepumping water level, upon which the 1971 and 1972 monthly pumpage was stressed. For places further away from the pumping center like Ah 6, Bg 15, and Bg 16, the prepumping and 1971 water levels are about the same. However, at places nearby a pumping center, as for example Bh 33 and Bh 31,

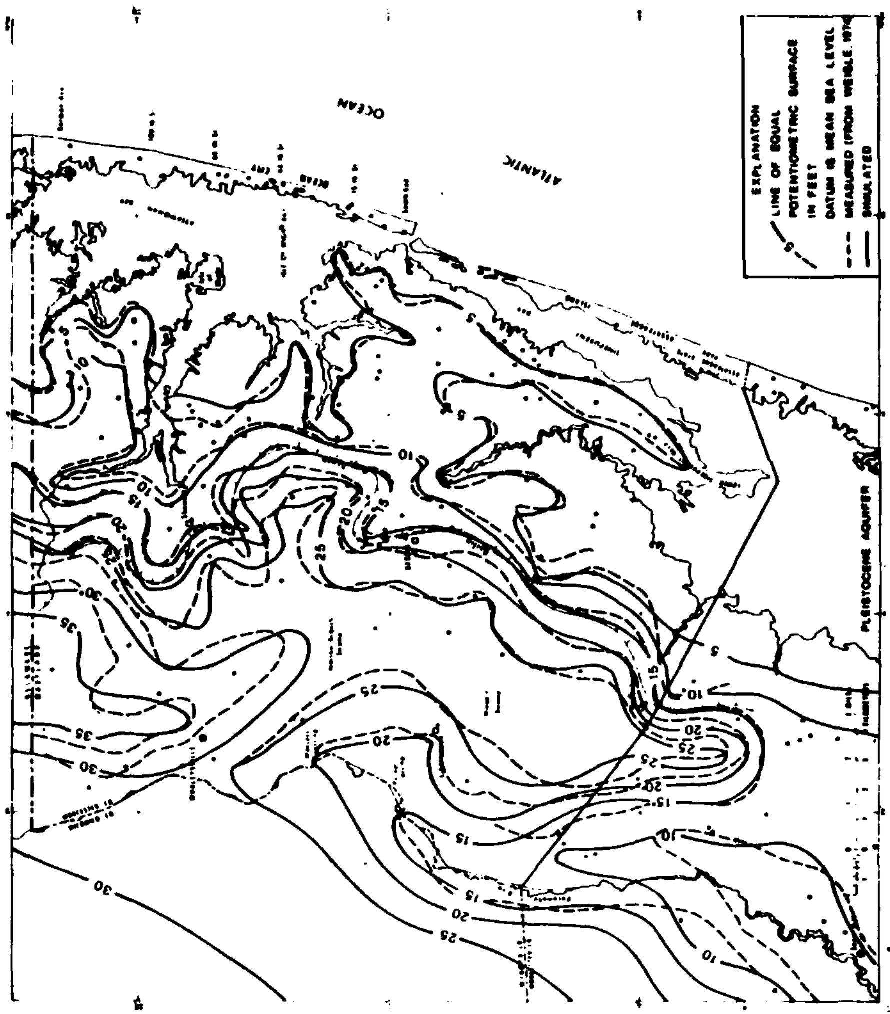

Figure 4. Potentiometric surface of Pleistocene aquifer, prepumping condition

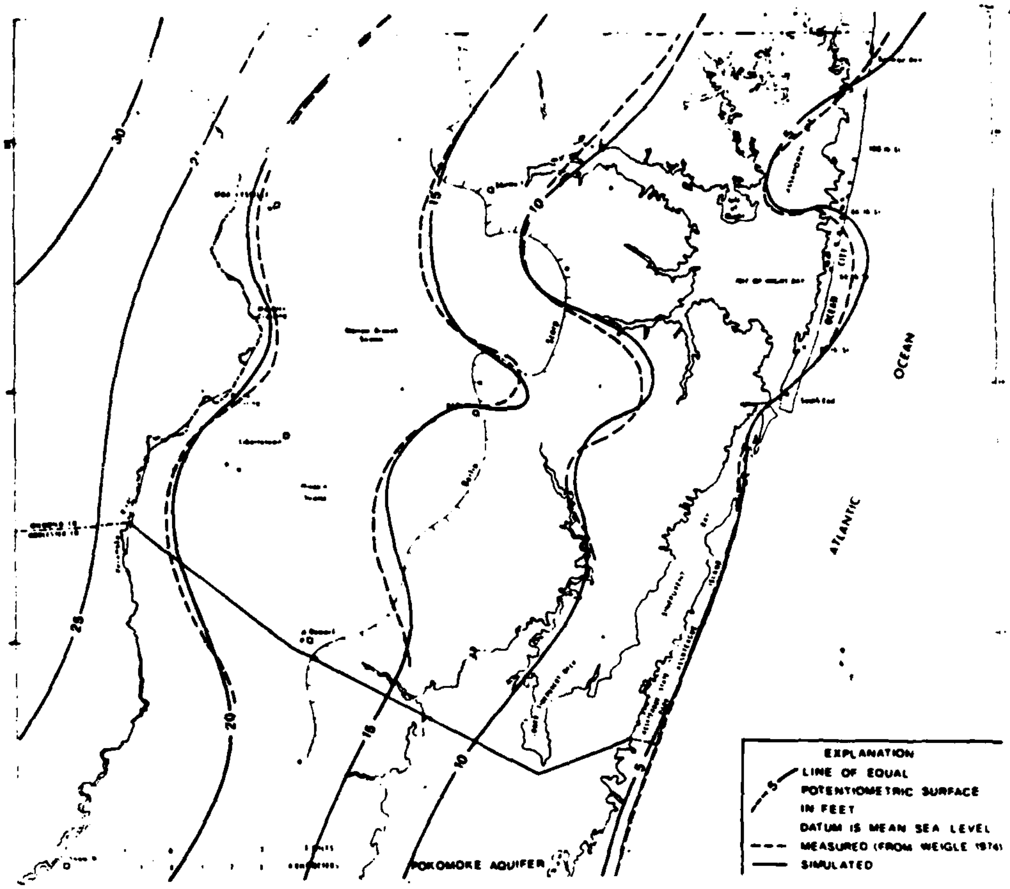

Figure 5. Potentiometric surface of Pocomoke aquifer, prepumping condition

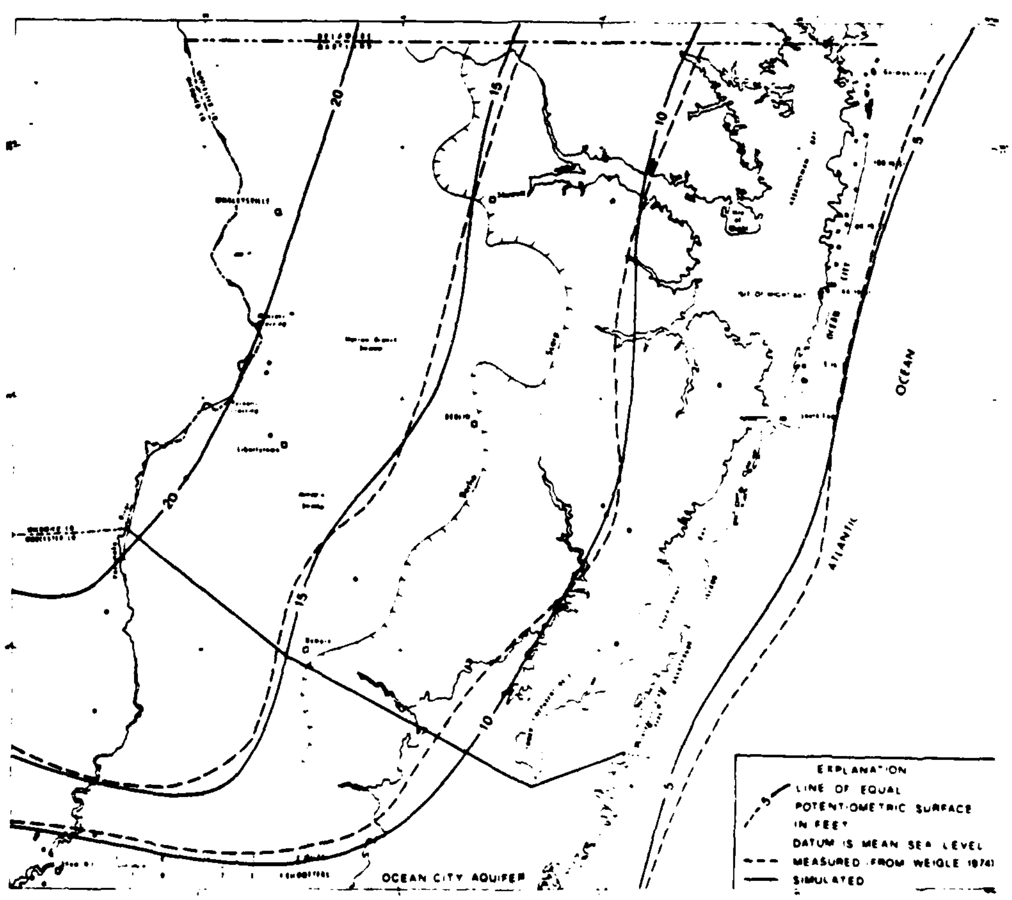

Figure 6. Potentiometric surface of Ocean City aquifer, prepumping condition

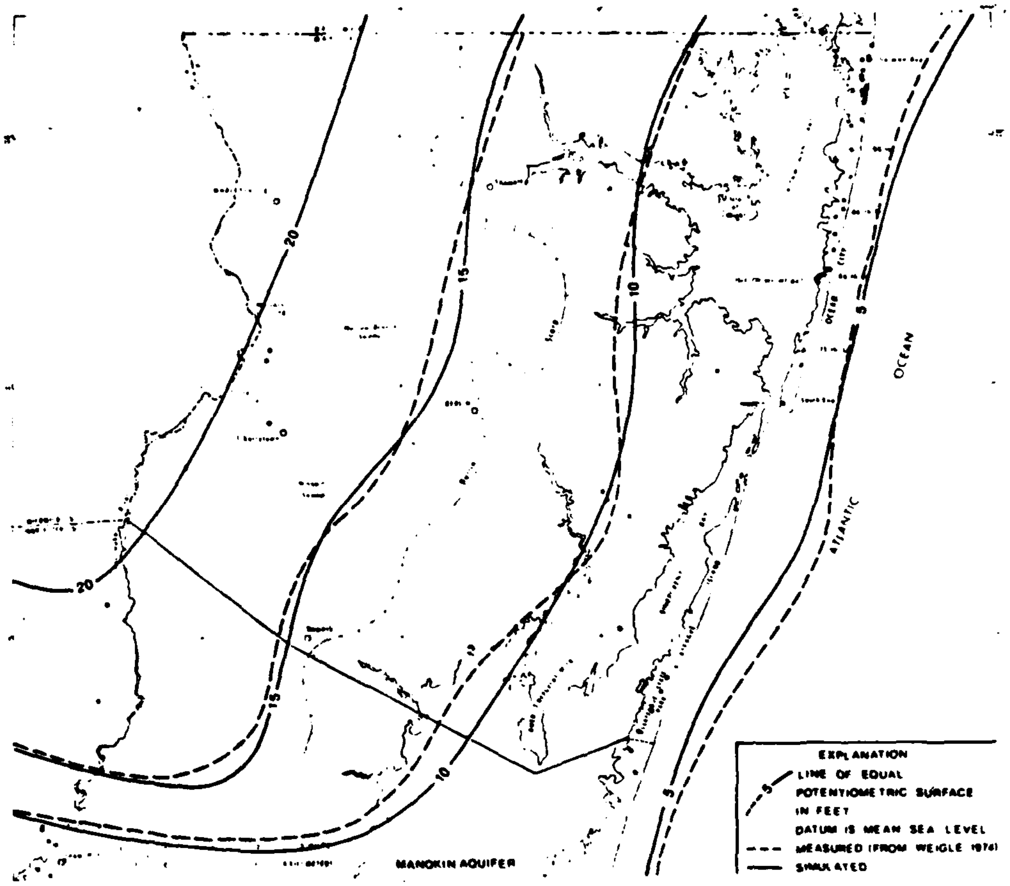

Figure 7. Potentiometric surface of Manokin aquifer, prepumping condition

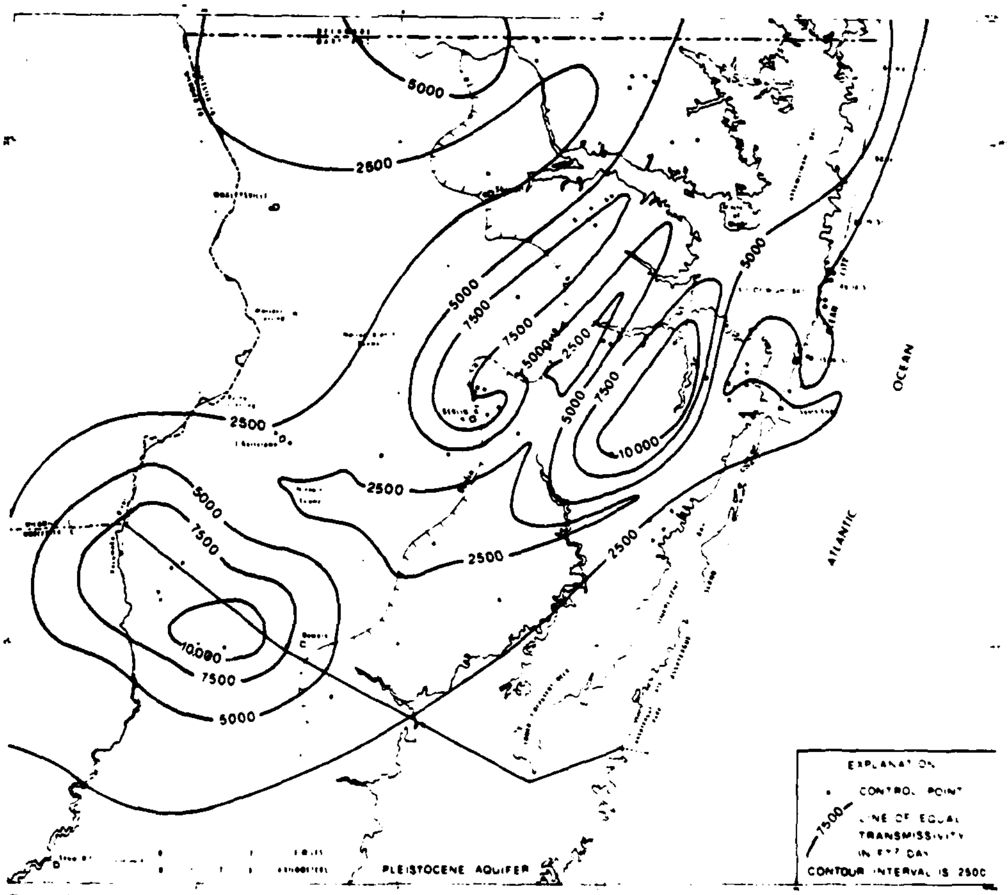

Figure 8. Transmissivity of Pleistocene aquifer used in the model

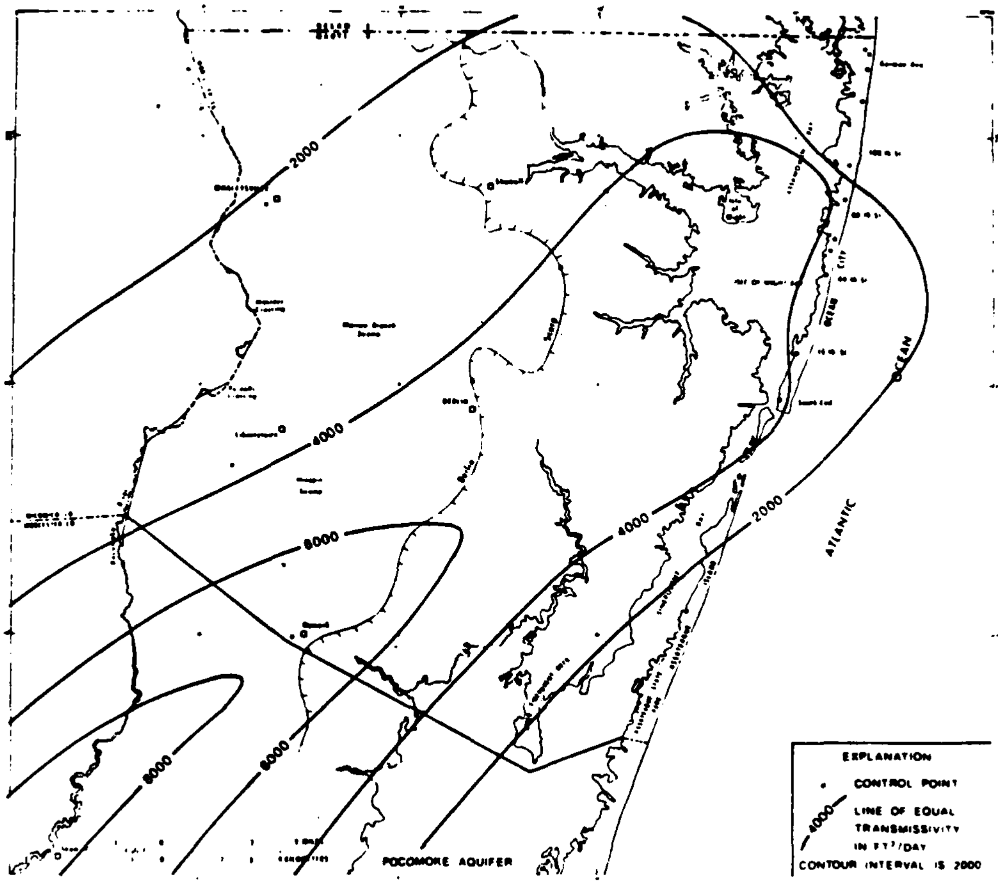

Figure 9. Transmissivity of Pocomoke aquifer used in the model

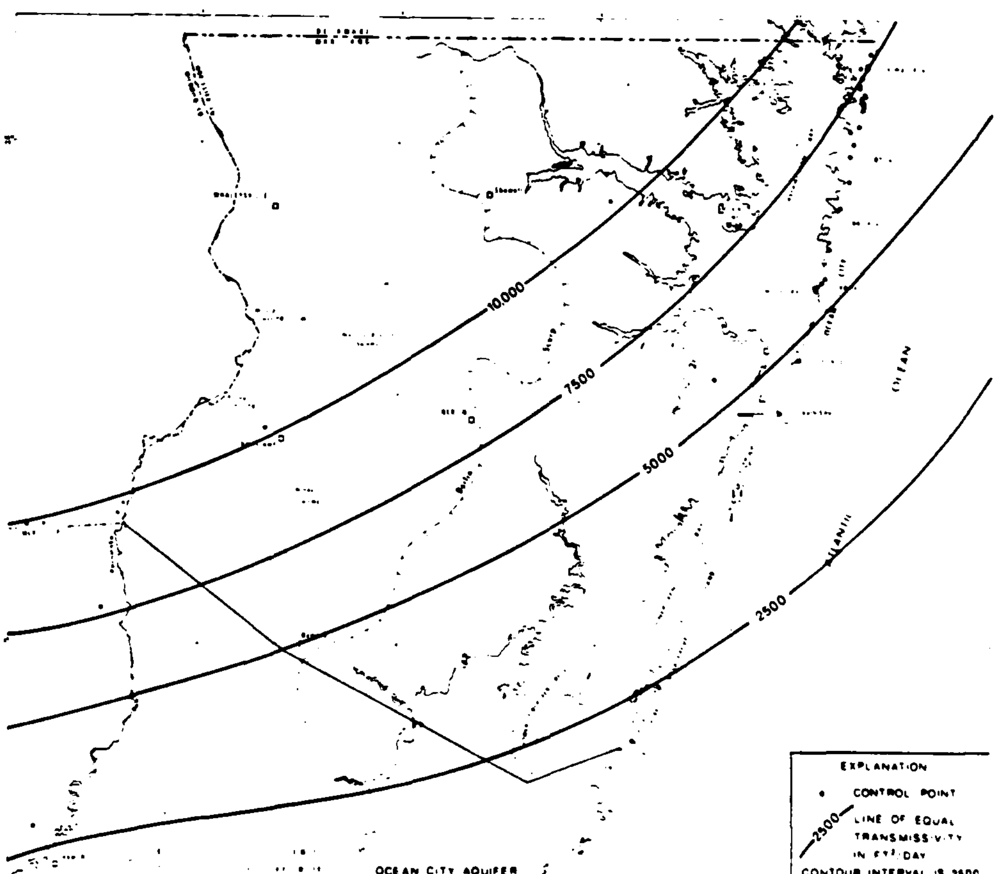

Figure 10. Transmissivity of Ocean City aquifer used in the model

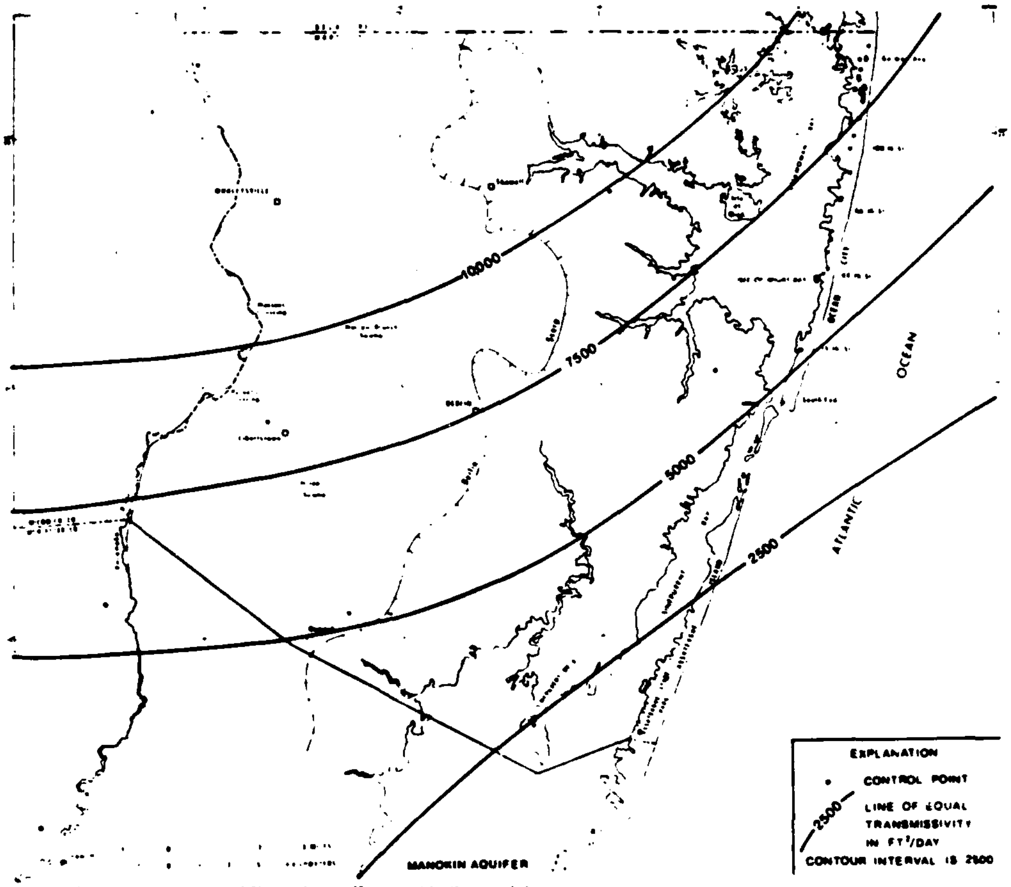

Figure 11. Transmissivity of Manokin aquifer used in the model

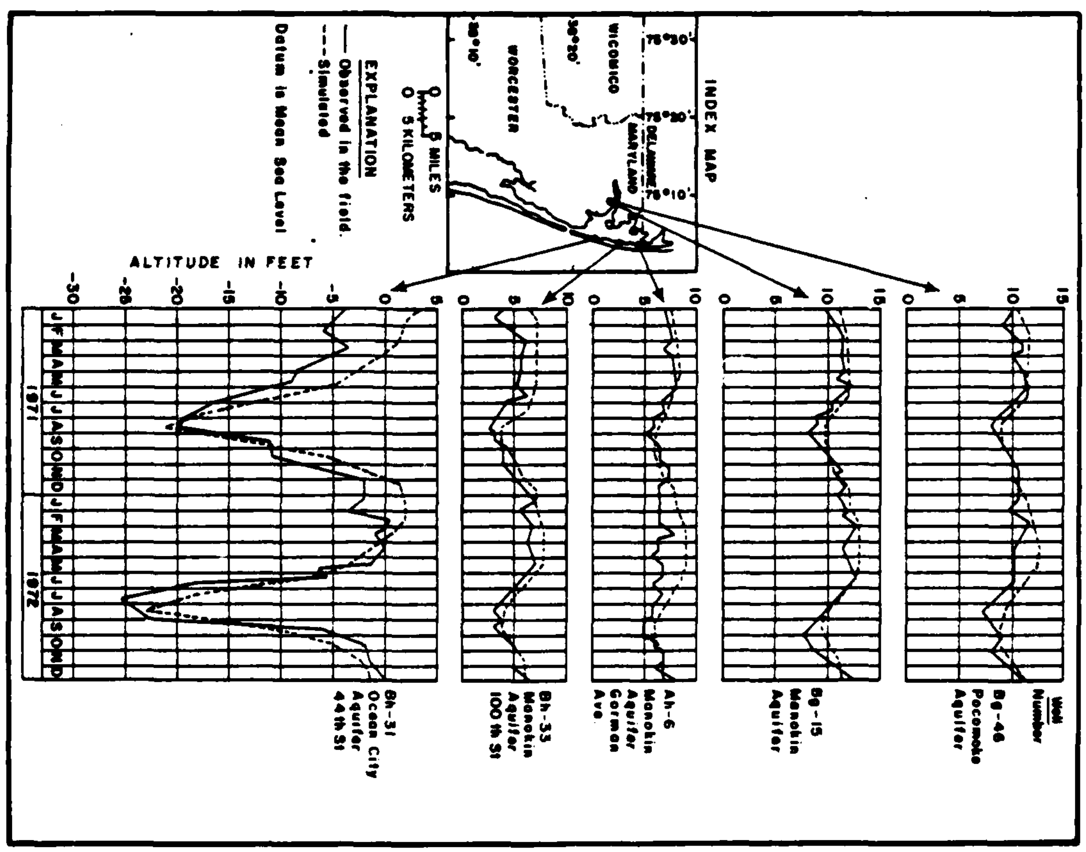

Figure 12. Hydrographs of observation wells

the differences are between 3 to 6 feet each. This accounts for the offset between the observed and simulated hydrographs at the start of the simulation period.

A report detailing the use of the developed quasi 3-D model to predict future ground water trends in the northeastern Worcester County, Md. is currently in preparation by the authors (Weigle and Achmad, 1980).

CONCLUSIONS

A quasi three-dimensional model was developed and used to simulate the four-aquifer system in northeastern Worcester County, Maryland. Over a two-year simulation period, the model was capable of generating a potentiometric head distribution that adequately reproduced hydrographs from several observation wells screened in the pumped aquifer, as well as in other aquifers (Manokin, Pocomoke, Pleistocene). The simulated hydrographs match the ones measured in the field with differences of less than 1 foot for 60 percent and less than 5 feet for 95 percent of the data points.

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APPENDIX I

Notation

A	cross-sectional area of flow (L^2);
b	saturated thickness (L);
D	right-hand side of the flow equation;
$AX, AY, AZ, CX, CY, CZ, BX, BY, DX, DY$	coefficients of the finite-difference equation;
h	altitude of water level (L);
K	hydraulic conductivity (LT^{-1});
q	unit discharge through the entire thickness of aquifer (L^2T^{-1});
S	storage coefficient (dimensionless);
S_s	specific storage (L^{-1});
t	time (T);
T	transmissivity (L^2T^{-1});
W	source term (LT^{-1});
$\Delta x, \Delta y, \Delta z$	space increments, in the X, Y, and Z directions (L);
Δt	time increment (T);
Subscript	
$a1, a2$	aquifer 1, aquifer 2;
i, j, k	position index, in the X, Y, and Z directions;
xx, yy, zz	primary x, y, and z directions.
Superscript	
0	initial time level;
$t-1$	previous time level;
t	current time level;
$t+1$	next time level;
$n-1$	previous iteration level;
n	current iteration level;
$n+1$	next iteration level;
$'$	confining bed index

APPENDIX II

COMPUTER PROGRAM

Main Program

The program was kept simple by writing it as a single main program. In this manner the program can be easily expanded to account for specific problems to be modeled.

For each program, the program has to be dimensioned specifically. The hydraulic parameters are dimensioned according to the number of nodes in the system. The layers are numbered sequentially starting with a smaller number for the top layer and increasing to a higher number for a lower layer.

There are three main loops in the calculation process: (1) time-period loop; (2) time-step loop; and (3) iteration loop. The iteration loop is completed as soon as the calculated water levels of two consecutive iterations are less than the closure criterion. The magnitude of the closure criterion determines the accuracy of the results, and a value of 0.01 is considered sufficient.

Input Data

This part of the main program initializes and defines the hydraulic parameters of the aquifer system. In addition to the values of the hydraulic parameters, the size of the time step, the total period of simulation, the number of time steps, and the calculation options must be read in. The program was written in consistent units with hour as the time unit.

A constant-head boundary node is coded with a negative storage coefficient value, and the program assigns to these nodes a very large storage coefficient (1×10^{20}).

Coefficients Calculations

In order to reduce computational efforts, all coefficients of the finite-difference equations that are independent of time are calculated outside the time-period loop and stored. For this reason, the A and the C coefficients are stored as well as parts of the B and the D coefficients. The leakage coefficients are time dependent, and they are reevaluated for each time step.

Mass Balance

The mass-balance routine computes the amount of flux coming into and going out of the system as pumping stress is applied. The volume released from storage is considered to be a source. The volume pumped is classified as a discharge.

Subroutine Map

This subroutine has been adapted from the Trescott model (1975). This routine will print maps of potentiometric head distribution. By specifying the parameter SCALE, the unit length used in the model is changed to the unit length used in the map. The parameter DINCH is the number of map units per inch. The parameter SPACNG is the multiplication factor for adjusting the value plotted on the map to the value of each node in the model.

In case of the Ocean City model, the parameters: SCALE, DINCH, and SPACNG, and the aquifer names are set by program.

APPENDIX III

PROGRAM LISTING

```
C     PROGRAM LISTING                                                  MN001000
C                                                                      MN001010
C                                                                      MN001020
C                                                                      MN001030
C     QUASI THREE-DIMENSIONAL FINITE-DIFFERENCE                        MN001040
C     GROUND-WATER FLOW MODEL                                          MN001050
C     USED TO SIMULATE THE MULTIAQUIFER SYSTEM                         MN001060
C     IN NORTHEASTERN WORCESTER COUNTY, MARYLAND                       MN001070
C                                                                      MN001080
C                                                                      MN001090
C                                                                      MN001100
C     JULY  1978                                                       MN001110
C                                                                      MN001120
C                                                                      MN001130
C     ARA          SURFACE AREA OF AN ELEMENT                          MN001140
C     ATIME        PREVIOUS SIMULATION TIME                            MN001150
C     AX,AY,AZ     A-COEFFICIENTS IN THE X, Y, AND Z DIRECTIONS        MN001160
C     B,G,W        ADI PARAMETERS, THOMAS ALGORITHM                    MN001170
C     BX           B-COEFFICIENT                                       MN001180
C     CDLT         MULTIPLICATION FACTOR FOR THE TIME-STEP SIZE        MN001190
C     CHDT         RECHARGE FROM CONSTANT HEAD                         MN001200
C     CHST         DISCHARGE FROM CONSTANT HEAD                        MN001210
C     CLNT         DISCHARGE FROM LEAKAGE                              MN001220
C     CLST         RECHARGE FROM LEAKAGE                               MN001230
C     CM           CONFINING BED THICKNESS                             MN001240
C     CX,CY,CZ     C-COEFFICIENTS IN THE X, Y, AND Z DIRECTIONS        MN001250
C     DD           TOTAL OF THE RIGHT-HAND SIDE OF THE FINITE DIFFERENCEMN001260
C                   EQUATION                                           MN001270
C     DELTA        SIZE OF TIME STEP                                   MN001280
C     DTN          NEW TIME INCREMENT                                  MN001290
C     DTO          OLD TIME INCREMENT                                  MN001300
C     ERROR        CLOSURE CRITERIA                                    MN001310
C     FC           MULTIPLICATION FOR GRID SPACING                     MN001320
C     FCT          MULTIPLICATION FACTOR FOR TRANSMISSIVITY            MN001330
C     H (HI,HO,    WATER LEVEL (INITIAL-, PREVIOUS-, CURRENT           MN001340
C      HC,HF)       TIME STEP, PREVIOUS ITERATION LEVEL)               MN001350
C     IDDN         PARAMETER CONTROLLING THE PRINTING OF DRAWDOWNS     MN001360
C     IHDN         PARAMETER CONTROLLING THE PRINTING OF HEADS         MN001370
C     IITER        MAXIMUM NUMBER OF ITERATION                         MN001380
C     IMAP         PARAMETER CONTROLLING THE PRINTING OF MAP           MN001390
C     IPCH         CONTROL CODE TO PUNCH HEAD FOR CONTINUATION RUN     MN001400
C     IREP         CONTROL CODE FOR CONTINUATION RUN                   MN001410
C     ISS          CONTROL CODE FOR STEADY-STATE RUN                   MN001420
C     IWCH         CONTROL CODE TO PRINT OUT HEAD FOR CONTINUATION RUN MN001430
C     IWMB         CONTROL CODE TO PRINT OUT MATERIAL BALANCE          MN001440
C     IWRT         CONTROL CODE TO PRINT OUT INPUT DATA MATRICES       MN001450
C     KP           COUNTING NUMBER FOR CURRENT TIME PERIOD             MN001460
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C      KPM1        COUNTING NUMBER FOR THE PREVIOUS TIME STEP
C      NC          NUMBER OF COLUMNS
C      NL          NUMBER OF LAYERS
C      NR          NUMBER OF ROWS
C      NPER        NUMBER OF TIME PERIODS
C      NWEL        TOTAL NUMBER OF WELLS
C      NUMT        TOTAL NUMBER OF TIME PERIOD
C      PUMPT       VOLUME PUMPED
C      W           WELL PRODUCTION RATE
C      QL          COEFFICIENT OF TRANSIENT LEAKAGE
C      R           COEFFICIENT OF CONSTANT LEAKAGE
C      RATE        CONDUCTIVITY OF THE CONFINING BED
C      RADIUS      WELL RADIUS
C      SC          STORAGE COEFFICIENT OF THE AQUIFER
C      SS          SPECIFIC STORAGE OF THE CONFINING BED
C      STORT       VOLUME FROM STORAGE
C      TLNT        RECHARGE FROM TRANSIENT LEAKAGE
C      TLST        DISCHARGE FROM TRANSIENT LEAKAGE
C      TS          TRANSMISSIVITY
C      X,Y,Z       INCREMENTS IN THE X, Y, AND Z DIRECTIONS
C
C
C
C
C
      DIMENSION HI(35,25,4),HC(35,25,4),HF(35,25,4),HO(35,25,4),
     *          DDN(35,25,4),DDNN(35,25,4),
     *          AX(35,25,4),CX(35,25,4),AY(35,25,4),CY(35,25,4),
     *          BX(35,25,4),D(35,25,4),
     *          B(35,4),G(35,4),ARA(35,25),NWR(80,2),NW(4),
     *          TS(35,25,4),R(35,25,4),SS(4),RATE(4),SC(4),CM(4),
     *          SF2(35,25,4),RADIUS(35,25,4),QRE(35,25,4),QL(35,25,4),
     *          CHDT(4),CHST(4),PUMPT(4),CFLUXT(4),TOTL1(4),TOTL2(4),
     *          SUMR(4),DIFF(4),PERCNT(4),TITLE(40),STORT(4),CLST(4),
     *          CLNT(4),TLST(4),TLNT(4),ESM(4),FCT(4)
C
      DIMENSION WL1(35,25,4),QL2(35,25,4),SD(4),QRET(4)
      COMMON    H(35,25,4),SF1(35,25,4),X(35),Y(25),
     *          Q(35,25,4),NC,NR,NL,IKP
C
C
      DATA      PI/3.1415926535898/,TOL/174.673/,BNUM/1E40/,
     *          BNUM1/.5E40/
C
      CALL ERRSET(208,256,-1,1)
C
C
C     DATA INPUT
```



```
C
      READ(5,180) TITLE
      READ(5,210) NC,NR,NL,NPER,DELTA,ERROR,ATIME
     * (K,SC(K),RATE(K),SS(K),CM(K),KK=1,NL)
C
C
      READ(5,340)IITER,IHDN,IDDN,IWRT,ISS,IREP,IPCH,IMAP,IWMB,IWCH
      DO 100 K=1,NL
      READ(5,310) STORT(K),CHDT(K),CHST(K),PUMPT(K)
  100 READ(5,310) CLST(K),CLNT(K),TLST(K),ILNT(K)
      READ(5,320) FC
      READ(5,110)(X(I),I=1,NC)
      READ(5,110)(Y(J),J=1,NR)
  110 FORMAT(20G4.0)
      DO 120 K=1,NL
      READ(5,320) FCT(K)
      DO 120 J=1,NR
  120 READ(5,300) (TS(I,J,K),I=1,NC)
      DO 130 K=1,NL
      DO 130 J=1,NR
  130 READ(5,300) (HI(I,J,K),I=1,NC)
      DO 160 K=1,NL
      DO 140 J=1,NR
      TS(1,J,K)=0.
  140 TS(NC,J,K)=0.
      DO 150 I=1,NC
      TS(I,1,K)=0.
  150 TS(I,NR,K)=0.
  160 CONTINUE
C
      WRITE(6,190) TITLE
      WRITE(6,230) NR,NC,NL,NPER,ICSPR,DELTA,ERROR,ATIME
      WRITE(6,240) FC,(X(I),I=1,NC)
      WRITE(6,250) (Y(J),J=1,NR)
      WRITE(6,260)
      WRITE(6,270)
      DO 170 K=1,NL
  170 WRITE(6,280) (RATE(K),SS(K),CM(K))
      WRITE(6,290)
      WRITE(6, )
      WRITE(6,330) (KK,SC(KK),KK=1,NL)
C
  180 FORMAT(20A4,/,20A4)
  190 FORMAT(1X,30A4,/,10A4,////)
  200 FORMAT(40I2)
  210 FORMAT(4I3,2G5.0,G10.0,/,(I5,4G10.0))
  220 FORMAT(8G10.2)
  230 FORMAT(1X,'# ROW (J)',5X,'= ',I5,/,1X,'# COLUMN (I)',2X,'= ',I5,/,
```

```
     *1X,'# AQUIFER (K)',1X,'= ',I5,/,                                  MN002430
     *1X,'# PMP PERIOD',2X,'= ',I5,/,1X,'STARTING PER',2X,'= '          MN002440
     *,I5,/,1X,'INITIAL DELT',2X,'= '                                   MN002450
     *,F7.2,3X,'HRS',/,1X,'EPSILON E-SUM',' = ',F7.4,3X,'FT',/1X,'INITIAMN002460
     *L TIME',2X,'= ',F7.1,'   HR',/,1X,'STARTING DATE = ', A8,///)     MN002470
 240 FORMAT(////,'      GRID SIZE',//,5X,'MULTIPLICATION FACTOR  =',F10.MN002480
     *1,/,'     DELX :',20F6.0,/,(10X,20F6.0))                          MN002490
 250 FORMAT(/,'     DELY :',20F6.0,/,(10X,20F6.0))                      MN002500
 260 FORMAT(///,1X,'CONFINING BED DATA',//)                             MN002510
 270 FORMAT(/,9X,'CONDUCTIVITY',6X,'SPEC. STORAGE',10X,'THICKNESS',/)   MN002520
 280 FORMAT(/,3E20.7)                                                   MN002530
 290 FORMAT(///,1X,'AQUIFER DATA',//)                                   MN002540
 300 FORMAT(20G4.0)                                                     MN002550
 310 FORMAT(4(1X,E19.6))                                                MN002560
 320 FORMAT(8G10.0)                                                     MN002570
 330 FORMAT(/,I6,E20.7)                                                 MN002580
 340 FORMAT(I2,78I1)                                                    MN002590
     DO 350 I=1,NC                                                      MN002600
     X(I)=X(I)*FC                                                       MN002610
 350 CONTINUE                                                           MN002620
     DO 360 J=1,NH                                                      MN002630
     Y(J)=Y(J)*FC                                                       MN002640
 360 CONTINUE                                                           MN002650
C                                                                       MN002660
     NLM1=NL-1                                                          MN002670
     NRM1=NR-1                                                          MN002680
     NCM1=NC-1                                                          MN002690
     DO 370 K=1,NL                                                      MN002700
 370 NH(K)=0                                                            MN002710
C                                                                       MN002720
     DO 380 I=1,NC                                                      MN002730
     DO 380 J=1,NH                                                      MN002740
 380 ARA(I,J)=X(I)*Y(J)                                                 MN002750
     DO 390 K=1,NL                                                      MN002760
     DO 390 J=1,NH                                                      MN002770
     DO 390 I=1,NC                                                      MN002780
     SF1(I,J,K)=SC(K)*ARA(I,J)                                          MN002790
     R(I,J,K)=RCH*ARA(I,J)                                              MN002800
     H(I,J,K)=HI(I,J,K)                                                 MN002810
     RADIUS(I,J,K)=0.                                                   MN002820
     Q(I,J,K)=0.                                                        MN002830
     QL(I,J,K)=0.                                                       MN002840
     QL1(I,J,K)=0.                                                      MN002850
     QL2(I,J,K)=0.                                                      MN002860
     SF2(I,J,K)=SF1(I,J,K)*2.                                           MN002870
     QRE(I,J,K)=0.                                                      MN002880
 390 CONTINUE                                                           MN002890
C                                                                       MN002900
```



```
C     BOUNDARY CONDITIONS                                               MN002910
C                                                                       MN002920
     DO 400 K=1,NL                                                      MN002930
     DO 400 I=1,NC                                                      MN002940
     DO 400 J=1,NH                                                      MN002950
     IF(SF1(I,J,K).LT.0.) SF1(I,J,K)=1.E40                              MN002960
     SF2(I,J,K)=2.*SF1(I,J,K)                                           MN002970
 400 CONTINUE                                                           MN002980
     IF(IREP.EQ.1) GO TO 420                                            MN002990
     DO 410 K=1,NL                                                      MN003000
     DO 410 J=1,NH                                                      MN003010
     DO 410 I=1,NC                                                      MN003020
 410 HO(I,J,K)=H(I,J,K)                                                 MN003030
     GO TO 450                                                          MN003040
 420 CONTINUE                                                           MN003050
     DO 430 K=1,NL                                                      MN003060
     DO 430 J=1,NH                                                      MN003070
 430 READ(5,470) (HO(I,J,K),I=1,NC)                                     MN003080
     DO 440 K=1,NL                                                      MN003090
     DO 440 J=1,NH                                                      MN003100
 440 READ(5,470) (H(I,J,K),I=1,NC)                                      MN003110
 450 CONTINUE                                                           MN003120
     DO 460 K=1,NL                                                      MN003130
     DO 460 J=1,NH                                                      MN003140
     DO 460 I=1,NC                                                      MN003150
     HF(I,J,K)=H(I,J,K)                                                 MN003160
 460 HC(I,J,K)=H(I,J,K)                                                 MN003170
 470 FORMAT(1X,13F6.1)                                                  MN003180
C                                                                       MN003190
C     A  AND  C  COEFFECIENTS                                           MN003200
C                                                                       MN003210
     DO 490 K=1,NL                                                      MN003220
     DO 490 J=1,NH                                                      MN003230
     DO 490 I=1,NCM1                                                    MN003240
     DENOMI=TS(I,J,K)*X(I+1)+TS(I+1,J,K)*X(I)                           MN003250
     IF(DENOMI.EQ.0.) GO TO 480                                         MN003260
     CX(I,J,K)=2.*TS(I,J,K)*TS(I+1,J,K)/(DENOMI*X(I))*ARA(I,J)*(-1.)    MN003270
     GO TO 490                                                          MN003280
 480 CX(I,J,K)=TS(I+1,J,K)*(-1.)                                        MN003290
 490 CONTINUE                                                           MN003300
     DO 500 K=1,NL                                                      MN003310
     DO 500 J=1,NH                                                      MN003320
 500 CX(NC,J,K)=0.                                                      MN003330
     DO 510 K=1,NL                                                      MN003340
     DO 510 J=1,NH                                                      MN003350
     DO 510 I=2,NC                                                      MN003360
 510 AX(I,J,K)=CX(I-1,J,K)                                              MN003370
     DO 520 K=1,NL                                                      MN003380
```

```
      DO 520 J=1,NW
  520 AX(I,J,K)=0.
      DO 540 K=1,NL
      DO 540 I=1,NC
      DO 540 J=1,NRM1
      DENOMJ=TS(I,J,K)*Y(J+1)+TS(I,J+1,K)*Y(J)
      IF(DENOMJ.EQ.0.) GO TO 530
      CY(I,J,K)=2.*TS(I,J,K)*TS(I,J+1,K)/(DENOMJ*Y(J))*ARA(I,J)*(-1.)
      GO TO 540
  530 CY(I,J,K)=TS(I,J+1,K)*(-1.)
  540 CONTINUE
      DO 550 K=1,NL
      DO 550 I=1,NC
  550 CY(I,NR,K)=0.
      DO 560 K=1,NL
      DO 560 I=1,NC
      DO 560 J=2,NW
  560 AY(I,J,K)=CY(I,J-1,K)
      DO 570 K=1,NL
      DO 570 I=1,NC
  570 AY(I,1,K)=0.
C
      IF(IWRT.EQ.0) GO TO 600
      WRITE(6,610)
      DO 580 K=1,NL
      WRITE(6,640) K
      DO 580 J=1,NW
  580 WRITE(6,630) J,(TS(I,J,K),I=1,NC)
      WRITE(6,620)
      DO 590 K=1,NL
      WRITE(6,640) K
      DO 590 J=1,NW
  590 WRITE(6,630) J,(H(I,J,K),I=1,NC)
  600 CONTINUE
C
  610 FORMAT(///,1X,'TRANSMISSIVITY MATRIX',//)
  620 FORMAT(///,1X,'HEAD MATRIX',///)
  630 FORMAT(1X,I3,1X,20F6.1,/,(5X,20F6.1))
  640 FORMAT(//,1X,'AQUIFER ',I2,//)
C
C     TIME PERIOD LOOP
C
  650 TIME=0.
C
  660 FORMAT(F5.2)
      DO 1580 IPER=1,NPER
      READ(5,320) KP,KPM1,NWEL,TMAX,NUMT,CDLT,DELTA
      DT=DELTA
```



```
      TM=0.
      DO 670 I=1,NUMT
      DT=CDLT*DT
      TM=TM+DT
      IF(TM.GE.TMAX) GO TO 680
  670 CONTINUE
      GO TO 690
  680 DELTA=TMAX/TM*DELTA
      NUMT=I
  690 IKP=KP+ICSPR
      WRITE(6,730) IKP,TMAX,NUMT,DELTA,CDLT
      WRITE(6,740) NWEL
      ICNWEL=0
      IF(NWEL.EQ.0) GO TO 720
      KW=0
C     READ IN PUMPING SCHEDULE
      DO 710 II=1,NWEL
      READ(5,320) I,J,K,Q(I,J,K),RADIUS(I,J,K),NW(K)
      IF(RADIUS(I,J,K).EQ.0.) GO TO 700
      NWR(II,1)=I
      NWR(II,2)=J
      NWR(II,3)=K
      Q(I,J,K)=Q(I,J,K)*5570.4
  700 CONTINUE
  710 CONTINUE
  720 CONTINUE
C
  730 FORMAT('-',58X,'PUMPING PERIOD NO.',I4,':',F10.2,' HRS '/53X,38('
     *-')//53X,'NUMBER OF TIME STEPS=',I6//59X,'DELT IN  HRS  =',F10.3//
     *53X,'MULTIPLIER FOR DELT =',F10.3)
  740 FORMAT('-',63X,I4,' WELLS'/65X,9('-')//)
  750 FORMAT(1X,3I10,2F13.2)
C
C     TIME STEP LOOP
C
      DTO=DELTA
      DO 1520 ISTEP=1,NUMT
      DELTA=DELTA*CDLT
      DTN=DELTA
      ATIME=ATIME+DELTA
      TIME=TIME+DELTA
C
C     LEAKAGE TERMS CALCULATION
C
      PI2=PI*PI
      PRE=0.
C
      DO 840  K=1,NL
```

```
      IF(SS(K).EQ.0.) GO TO 840
      IF(CM(K).EQ.0.) GO TO 840
      TDIM=RATE(K)*ATIME/(SS(K)*CM(K)*CM(K))
      IF(TDIM.GT.10.) GO TO 820
      DO 810  I=1,NC
      DO 810  J=1,NR
      SUM1=0.
      SUM2=0.
      DO 780  N=1,200
      SUM11=SUM1
      SUM21=SUM2
      AN=PI2*N*N*TDIM
      PIN=PI*N
      IF(AN.GT.TOL) AN=TOL
      E=1./(EXP(AN))
      IF(E.LT.0.000001) GO TO 760
      ARG1=(COS(PIN))*E
      ARG2=E
      GO TO 770
  760 ARG1=0.
      ARG2=0.
  770 SUM1=SUM1+ARG1
      SUM2=SUM2+ARG2
      IF(SUM11.EQ.SUM1.AND.SUM21.EQ.SUM2) GO TO 790
  780 CONTINUE
  790 CONTINUE
  800 Q1=RATE(K)/CM(K)
      QL1(I,J,K)=2.*Q1*SUM1*ARA(I,J)
      QL2(I,J,K)=2.*Q1*SUM2*ARA(I,J)
      PRE=RATE(K)/CM(K)
  810 CONTINUE
      GO TO 840
  ... CONTINUE
C 212 WRITE(6,8) TDIM,ATIME
  830 FORMAT(/,1X,'***** TDIM =',E20.7,'AND ELAPSED TIME =',F20.1,'IN HR
     *S')
  840 CONTINUE
  850 CONTINUE
C
C
C     B  AND  D  COEFFICIENTS
C
      DO 860 K=1,NLM1
      DO 860 I=1,NC
      DO 860 J=1,NH
      BX(I,J,K)=SF1(I,J,K)/DELTA-(AX(I,J,K)+CX(I,J,K)+AY(I,J,K)+CY(I,J,K
     *))+R(I,J,K)+R(I,J,K+1)+QL2(I,J,K)+QL2(I,J,K+1)
  860 CONTINUE
```



```
      DO 870 I=1,NC
      DO 870 J=1,NH
      BX(I,J,NL)=SF1(I,J,NL)/DELTA+R(I,J,NL)+QL2(I,J,NL)
     *-(AX(I,J,NL)+CX(I,J,NL)+AY(I,J,NL)+CY(I,J,NL))
  870 CONTINUE
      RTM=DTO/DTN
      DO 890 K=1,NL
      DO 890 J=1,NH
      DO 890 I=1,NC
      HOKM1=HI(I,J,K-1)
      IF(K.EQ.1) HOKM1=0.
      DD=HO(I,J,K)*SF1(I,J,K)/DELTA-Q(I,J,K)+QRE(I,J,K)
     *+QL2(I,J,K)*HI(I,J,K)-QL1(I,J,K)*HOKM1
      IF(K.EQ.NL) GO TO 880
      DD=DD+QL2(I,J,K+1)*HI(I,J,K)-QL1(I,J,K+1)*HI(I,J,K+1)
  880 D(I,J,K)=DD
      DH=0.
      HO(I,J,K)=H(I,J,K)
      HF(I,J,K)=HC(I,J,K)
  890 H(I,J,K)=H(I,J,K)-RTM*DH
      DTO=DELTA
C
C     ITERATION LOOP
C
      ITER=0
  900 E=0.
      EMAX=0.
      ITER=ITER+1
C
C     ALTERNATING DIRECTION PROCEDURES
C
      DO 960 K=1,NL
      DO 960 I=1,NC
      DO 940 J=1,NH
      IF(TS(I,J,K).EQ.0.) GO TO 940
      HKM1=H(I,J,K-1)
      IF(K.EQ.1) HKM1=0.
      DD=D(I,J,K)+(R(I,J,K)+QL1(I,J,K))*HKM1
      IF(K.EQ.NL) GO TO 910
      DD=DD+(R(I,J,K+1)+QL1(I,J,K+1))*H(I,J,K+1)
  910 IF(I.EQ.1) GO TO 920
      HAX=H(I-1,J,K)*AX(I,J,K)
      DD=DD-HAX
  920 IF(I.EQ.NC) GO TO 930
      HCX=H(I+1,J,K)*CX(I,J,K)
      DD=DD-HCX
  930 W=BX(I,J,K)-AY(I,J,K)*B(J-1,K)
      B(J,K)=CY(I,J,K)/W
```

```
      G(J,K)=(DD-AY(I,J,K)*G(J-1,K))/W
  940 CONTINUE
      HA=G(NR,K)
      IF(TS(I,NR,K).EQ.0.) HA=H(I,NR,K)
      H(I,NR,K)=HA
      N=NR-1
  950 HA=G(N,K)-B(N,K)*H(I,N+1,K)
      IF(TS(I,N,K).EQ.0.) HA=H(I,N,K)
      ER=ABS(HA-H(I,N,K))
      EMAX=AMAX1(ER,EMAX)
      E=E+ER
      H(I,N,K)=HA
      N=N-1
      IF(N.GT.0) GO TO 950
  960 CONTINUE
      DO 1020 K=1,NL
      DO 1020 J=1,NR
      DO 1000 I=1,NC
      IF(TS(I,J,K).EQ.0.) GO TO 1000
      HKM1=H(I,J,K-1)
      IF(K.EQ.1) HKM1=0.
      DD=D(I,J,K)+(R(I,J,K)+QL1(I,J,K))*HKM1
      IF(K.EQ.NL) GO TO 970
      DD=DD+(R(I,J,K+1)+QL1(I,J,K+1))*H(I,J,K+1)
  970 IF(J.EQ.1) GO TO 980
      HAY=H(I,J-1,K)*AY(I,J,K)
      DD=DD-HAY
  980 IF(J.EQ.NR) GO TO 990
      HCY=H(I,J+1,K)*CY(I,J,K)
      DD=DD-HCY
  990 W=BX(I,J,K)-AX(I,J,K)*B(I-1,K)
      B(I,K)=CX(I,J,K)/W
      G(I,K)=(DD-AX(I,J,K)*G(I-1,K))/W
 1000 CONTINUE
      HA=G(NC,K)
      IF(TS(NC,J,K).EQ.0.) HA=H(NC,J,K)
      H(NC,J,K)=HA
      N=NC-1
 1010 HA=G(N,K)-B(N,K)*H(N+1,J,K)
      IF(TS(N,J,K).EQ.0.) HA=H(N,J,K)
      ER=ABS(H(N,J,K)-HA)
      EMAX=AMAX1(ER,EMAX)
      E=E+ER
      H(N,J,K)=HA
      N=N-1
      IF(N.GT.0) GO TO 1010
 1020 CONTINUE
      IF(IWIT.EQ.0) GO TO 1030
```



```
      WRITE(6,1040) ATIME,ITER,EMAX,E
 1030 CONTINUE
 1040 FORMAT('0',5X,'TIME =',F13.2,2X,'HRS',5X,'ITERATION #',I3,5X,'EMAX
     * =',E13.6,5X,'SUM EMAX =',E13.6,/)
      IF(ITER.EQ.IITER ) GO TO 1590
      DO 1050 K=1,NL
      DO 1050 I=1,NC
      DO 1050 J=1,NR
      HC(I,J,K)= H(I,J,K)
 1050 CONTINUE
      ICSS=0
      DO 1090  K=1,NL
      ES=0.
      DO 1060  I=1,NC
      DO 1060  J=1,NR
      ESME=ABS(HC(I,J,K)-HF(I,J,K))
      ES  =AMAX1(ES,ESME)
      DDNN(I,J,K)=HF(I,J,K)-HC(I,J,K)
 1060 DDN(I,J,K)=HI(I,J,K)- H(I,J,K)
      IF(ES.LE.ERROR) ICSS=ICSS+1
C     WRITE(6  ,372) ES,K
 1070 FORMAT(5X,'MAX  HF - HC = ',F10.1,5X,'AQUIFER ',I5)
      IF(ES.LE.ERROR) WRITE(6,1080) ES,K
 1080 FORMAT(20X,'***** STEADY STATE FOR HEAD DIFF OF ',E13.6,'******',
     *5X,'LAYER ',I3)
 1090 CONTINUE
 1100 CONTINUE
      IF(ICSS.EQ.NL.AND.ISS.EQ.1) NUMT=ISTEP
C
C     CONVERGENCE TEST
C
      IF(EMAX.LE.ERROR) GO TO 1110
      IF(E.GT.ERROR) GO TO 900
 1110 CONTINUE
      IF(ISTEP.NE.NUMT) GO TO 1130
      WRITE(6,1040) ATIME,ITER,EMAX,E
      WRITE(6,1120) ISTEP,DELTA
 1120 FORMAT(/,5X,'TIME STEP #',I3,5X,'DELTA =',F8.2,'DY')
 1130 CONTINUE
C
C     PRINT OUTPUT
C
 1140 CONTINUE
      DO 1510 K=1,NL
      IF(ISTEP.NE.NUMT) GO TO 1280
 1150 CONTINUE
      WRITE(6,1160) K
 1160 FORMAT('0AQUIFER ',I2,/)
```

```
      IF(IHDN.EQ.0) GO TO 1200                                          MN006270
      WRITE(6,1170)                                                     MN006280
 1170 FORMAT('0',20X,'HEAD MATRIX')                                     MN006290
      DO 1180 J=1,NR                                                    MN006300
 1180 WRITE(6,1190) J,(HC(I,J,K),I=1,NC)                                MN006310
 1190 FORMAT(/,I5,5X,20F6.1)                                            MN006320
 1200 CONTINUE                                                          MN006330
      IF(IDDN.EQ.0) GO TO 1230                                          MN006340
      WRITE(6,1210)                                                     MN006350
 1210 FORMAT('0',20X,'DRAW DOWN')                                       MN006360
      DO 1220 J=1,NR                                                    MN006370
 1220 WRITE(6,1190) J,(DDN(I,J,K),I=1,NC)                               MN006380
 1230 CONTINUE                                                          MN006390
      IF(NW(K).EQ.0) GO TO 1270                                         MN006400
      WRITE(6,1240)                                                     MN006410
 1240 FORMAT('-',50X,'HEAD AND DRAWDOWN IN PUMPING WELLS',/,51X,34('-'),MN006420
     *//,48X,'I     J     K      Q IN MGD      HEAD     DRAWDOWN  AREAL DDN  MN006430
     *     AREAL HEAD')                                                 MN006440
      DO 1260 KW=1, NWEL                                                MN006450
      IF(NWR(KW,3).NE.K) GO TO 1260                                     MN006460
      I=NWR(KW,1)                                                       MN006470
      J=NWR(KW,2)                                                       MN006480
      KC=NWR(KW,3)                                                      MN006490
      IF(RADIUS(I,J,KC).EQ.0) GO TO 1260                                MN006500
      RE=SQRT(ARA(I,J)/PI)                                              MN006510
      HW=HC(I,J,KC)-Q(I,J,KC)*(ALOG(RE/RADIUS(I,J,KC)))/(2.*PI *TS(I,J, MN006520
     *KC))                                                              MN006530
      DRAW=HI(I,J,K)-HW                                                 MN006540
      DRAWA=HI(I,J,KC)-HC(I,J,KC)                                       MN006550
      WHEAD=HC(I,J,KC)                                                  MN006560
      QMGD=Q(I,J,KC)/5570.41                                            MN006570
      WRITE(6,1250) I,J,KC,QMGD,HW,DRAW,DRAWA,WHEAD                     MN006580
 1250 FORMAT('  ',43X,3I5,5F11.2)                                       MN006590
 1260 CONTINUE                                                          MN006600
 1270 CONTINUE                                                          MN006610
 1280 CONTINUE                                                          MN006620
C                                                                       MN006630
C     MASS BALANCE CALCULATION                                          MN006640
C                                                                       MN006650
      PUMP=0.                                                           MN006660
      STOR=0.                                                           MN006670
      FLUXS=0.                                                          MN006680
      CHD1=0.                                                           MN006690
      CHD2=0.                                                           MN006700
      CFLUX=0.                                                          MN006710
      FLXN=0.                                                           MN006720
      FLBXN=0.                                                          MN006730
      FLBXS=0.                                                          MN006740
```



```
      OBTR=0.                                                           MN006750
      PBTR=0.                                                           MN006760
      PUTR=0.                                                           MN006770
      OUTR=0.                                                           MN006780
C                                                                       MN006790
      HU=0.                                                             MN006800
      DO 1360 I=1,NC                                                    MN006810
      DO 1360 J=1,NR                                                    MN006820
      IF(Q(I,J,K)) 1300,1310,1290                                       MN006830
C                                                                       MN006840
 1290 PUMP=PUMP-Q(I,J,K)                                                MN006850
      GO TO 1310                                                        MN006860
C                                                                       MN006870
 1300 CFLUX=CFLUX-Q(I,J,K)                                              MN006880
 1310 CONTINUE                                                          MN006890
      IF(SF1(I,J,K).GE.BNUM1) GO TO 1320                                MN006900
C                                                                       MN006910
      STOR=STOR+SF2(I,J,K)*(HO(I,J,K)-H(I,J,K))                         MN006920
 1320 CONTINUE                                                          MN006930
C                                                                       MN006940
      IF(K.EQ.1) GO TO 1330                                             MN006950
      HUO=HI(I,J,K-1)                                                   MN006960
      HU=HC(I,J,K-1)                                                    MN006970
      GO TO 1340                                                        MN006980
 1330 HU=0.                                                             MN006990
      HUO=0.                                                            MN007000
 1340 CONTINUE                                                          MN007010
      XU=HC(I,J,K)-HU                                                   MN007020
      XUO1=HUO-HU                                                       MN007030
      XUO2=HI(I,J,K)-HC(I,J,K)                                          MN007040
      UTR=XUO1*QL1(I,J,K)-XUO2*QL2(I,J,K)                               MN007050
      UL=XU*R(I,J,K)+UTR                                                MN007060
      IF(UL.GT.0.) FLUXS=FLUXS+UL                                       MN007070
      IF(UL.LT.0.) FLXN=FLXN+UL                                         MN007080
      IF(UTR.GT.0.) PUTR=PUTR+UTR                                       MN007090
      IF(UTR.LT.0.) OUTR=OUTR+UTR                                       MN007100
      HB=HC(I,J,K+1)                                                    MN007110
      HBO=HI(I,J,K+1)                                                   MN007120
      IF(K.EQ.NL) GO TO 1350                                            MN007130
      XB=HC(I,J,K)-HB                                                   MN007140
      XBO1=HBO-HB                                                       MN007150
      XBO2=HI(I,J,K+1)-HC(I,J,K+1)                                      MN007160
      BTR=XBO1*QL1(I,J,K+1)-XBO2*QL2(I,J,K+1)                           MN007170
      BL=XB*R(I,J,K+1)+BTR                                              MN007180
      IF(BL.GT.0.) FLBXS=FLBXS+BL                                       MN007190
      IF(BL.LT.0.) FLBXN=FLBXN+BL                                       MN007200
      IF(BTR.GT.0.) PBTR=PBTR+BTR                                       MN007210
      IF(BTR.LT.0.) OBTR=OBTR+BTR                                       MN007220
```

```
 1350 CONTINUE
      CLS=FLUXS+FLUXS
      CLN=FLXN+FLBXN
      TLS=PUTH+PBTH
      TLN=OUTR+OBTH
 1360 CONTINUE
C
      XIP=0.
      YIP=0.
      XI=0.
      YI=0.
      DO 1480 I=2,NCM1
      DO 1480 J=2,NRM1
      IF(SF1(I,J,K).LT.BNUM1) GO TO 1480
      IF(SF1(I-1,J,K).GE.BNUM1) GO TO 1390
      XI=AX(I,J,K)*(H(I-1,J,K)-H(I,J,K))
      IF(XI) 1370,1390,1380
 1370 CHD1=CHD1+XI
      GO TO 1390
 1380 CHD2=CHD2+XI
 1390 IF(SF1(I+1,J,K).GE.BNUM1) GO TO 1420
      XIP=CX(I,J,K)*(H(I+1,J,K)-H(I,J,K))
      IF(XIP) 1400,1420,1410
 1400 CHD1=CHD1+XIP
      GO TO 1420
 1410 CHD2=CHD2+XIP
 1420 IF(SF1(I,J-1,K).GE.BNUM1) GO TO 1450
      YI=AY(I,J,K)*(H(I,J-1,K)-H(I,J,K))
      IF(YI) 1430,1450,1440
 1430 CHD1=CHD1+YI
      GO TO 1450
 1440 CHD2=CHD2+YI
 1450 IF(SF1(I,J+1,K).GE.BNUM1) GO TO 1480
      YIP=CY(I,J,K)*(H(I,J+1,K)-H(I,J,K))
      IF(YIP) 1460,1480,1470
 1460 CHD1=CHD1+YIP
      GO TO 1480
 1470 CHD2=CHD2+YIP
 1480 CONTINUE
C
      STORT(K)=STORT(K)+STOR
      CHDT(K)=CHDT(K)-CHD1*DELTA
      CHST(K)=CHST(K)+CHD2*DELTA
      PUMPT(K)=PUMPT(K)-PUMP*DELTA
      CLST(K)=CLST(K)+CLS*DELTA
      CLNT(K)=CLNT(K)-CLN*DELTA
      TLST(K)=TLST(K)+TLS*DELTA
      TLNT(K)=TLNT(K)-TLN*DELTA
```



```
      TOTL1(K)=STORT(K)+CHST(K)+CLST(K)
      TOTL2(K)=PUMPT(K)+CHDT(K)+CLNT(K)
      SUMR(K)=STOR/DELTA+CHD2+CHD1+CLS+CLN+PUMP
      DIFF(K)=TOTL1(K)-TOTL2(K)
      PERCNT(K)=DIFF(K)/(TOTL1(K)+TOTL2(K))*200.
      IF(IWMA.NE.1) GO TO 1500
      IF(ISTEP.NE.NUMT) GO TO 1500
C
      WRITE(6,1490) K,STORT(K),STOR,CHST(K)          ,CHD2,CLST(K),CHD1,
     *       TOTL1(K)      ,PUMPT(K),CLS,CHDT(K)              ,CLN,CLNT(K),
     * PUMP,TOTL2(K),SUMR(K),DIFF(K),PERCNT(K)
 1490 FORMAT('0',2X,'AQUIFER',I2,4X,
     *                 'CUMULATIVE VOLUME :',38X,'TIME STEP RATES :',//,
     *1X,'SOURCE :',17X,'STORAGE =',F20.2,26X,'STORAGE =',F20.4,/,20X,
     *'CONSTANT HEAD =',F20.2,19X,'CONSTANT HEAD:  ',/,19X,'
     *    '          ,31X,'IN =',F20.4,/,19X,'TOTAL  LEAKAGE =',F20.2,30X,
     *'OUT =',F20.4,/,20X,'TOTAL   ',F20.2,25X,'LEAKAGE:'/77X, 'IN :
     *    '          ,/,1X,'DISCHARGE :',14X,'PUMPING =',F20.2,27X,'TOTAL  =
     *',F20.4,/,20X,'CONSTANT HEAD =',F20.2,21X,'OUT :              '
     *,/,19X,'                '          ,27X,'TOTAL  =',F20.4,/,19X,
     *'TOTAL  LEAKAGE =',F20.2,29X,'PUMP =',F20.4,/,20X,'TOTAL   ',F20.2,
     *28X,'TOTAL   ',F20.4,//,26X,'BALANCE   ',F20.2,/,
     *23X,'PERCENTAGE   ',F20.2,//)
 1500 CONTINUE
 1510 CONTINUE
      IF(ICSS.EQ.NL.AND.ISS.EQ.1) GO TO 1530
 1520 CONTINUE
 1530 CONTINUE
      TSTORT=0.
      TCHDS=0.
      TCLST=0.
      TPUMPT=0.
      TCHDT=0.
      TCLNT=0.
      DO 1540 K=1,NL
      TSTORT=TSTORT+STORT(K)
      TCHDS=TCHDS+CHST(K)
      TCLST=TCLST+CLST(K)
      TPUMPT=TPUMPT+PUMPT(K)
      TCHDT=TCHDT+CHDT(K)
      TCLNT=TCLNT+CLNT(K)
 1540 CONTINUE
      SUMS=TSTORT+TCHDS+TCLST
      SUMN=TCHDT+TPUMPT+TCLNT
      SPN=SUMS+SUMN
      SMN=SUMS-SUMN
      DIFSN=SMN/SPN*200.
C
```

```
      E(6,1560) TSTORT,TPUMPT,TCHDS,TCHDT,TCLST,TCLNT,SUMS,SUMN,SMN,
     SN
     TINUE
 1560 .. MAT(//,15X,'TOTAL MASS BALANCE :',//,26X,
     *             'STORAGE =',F20.2,26X,'PUMPING =',F20.2,/,18X,'POS C
     *ONST. HEAD =',F20.2,18X,'NEG CONST. HEAD =',F20.2,/,22X,'POS LEAKA
     *GE =',F20.2,22X,'NEG LEAKAGE =',F20.2,//,28X,'TOTAL   ',F20.2,28X,'
     *TOTAL   ',F20.2,//,26X,'BALANCE =',F20.2,/,23X,'PERCENTAGE =',F20.2
     *,//)
 1570 CONTINUE
      IF(ICSS.EQ.NL.AND.ISS.EQ.1) GO TO 1630
 1580 CONTINUE
      IF(IWCM.NE.1) GO TO 1640
      DO 1590 K=1,NL
      WRITE(6,310) STORT(K),CHDT(K),CHST(K),PUMPT(K)
      WRITE(6,310) CLST(K),CLNT(K),TLST(K),TLNT(K)
 1590 CONTINUE
      WRITE(6,1600)
 1600 FORMAT(/,1X,'*** HO(I,J,K) ***',/,4X,'------------',/)
      DO 1610 K=1,NL
      WRITE(6,1160) K
      DO 1610 J=1,NR
      WRITE(6,630) J,(HO(I,J,K),I=1,NC)
 1610 CONTINUE
      WRITE(6,1620)
 1620 FORMAT(/,1X,'*** H(I,J,K) ***',/,4X,'-----------',/)
      DO 1630 K=1,NL
      WRITE(6,1160) K
      DO 1630 J=1,NR
      WRITE(6,630) J,(H(I,J,K),I=1,NC)
 1630 CONTINUE
 1640 CONTINUE
      IF(IPCH.NE.1) GO TO 1690
      DO 1650 K=1,NL
      ....E(7,310) STORT(K),CHDT(K),CHST(K),PUMPT(K)
      WRITE(7,310) CLST(K),CLNT(K),TLST(K),TLNT(K)
 1650 CONTINUE
      DO 1660 K=1,NL
      DO 1660 J=1,NR
 1660 WRITE(7,470) (HO(I,J,K),I=1,NC)
      DO 1670 K=1,NL
      DO 1670 J=1,NR
 1670 WRITE(7,470) (H(I,J,K),I=1,NC)
       IF(MAP.EQ.0) GO TO 1680
      CALL MAP
 1680 CONTINUE
      IF(ITER.EQ.IITER) WRITE(6,1690) ITER
 1690 FORMAT(/,1X,' ***  ',I5,'  ITERATIONS EXCEEDED  ***')

      STOP
      END
```



```
      SUBROUTINE MAP
C
      COMMON     H(35,25,4),SF1(35,25,4),X(35),Y(25),
     *           U(35,25,4),NC,NR,NL,IKP
C
      DIMENSION VF4(11)
      DIMENSION XLABEL(3),TTLE(5),SYM(28),PRNT(122),BLANK(60),DIGIT(122)
     *          ,VF1(6),VF2(6),VF3(7),NA(4),XN(100),YN(13),YLABEL(3)
C
      REAL *4 KPRM
      REAL *8XLABEL,YLABEL,TTLE
      INTEGER OUT
      DATA TTLE/'ALPHAMEN','IC CONTO','URS OF H','EAD'/
      DATA SYM/1H ,1H1,1H2,1H3,1H4,1H5,1H6,1H7,1H8,1H9,1H ,1HA,1HB,1HC,1
     *HD,1HE,1HF,1HG,1HH,1HI,1H ,1H*,1HI,1H-,1H+,1HO,1HR,1HW/
      DATA PRNT/122*' '/,N1,N2,N3,XN1/6,10,133,.083333333D0/,BLANK/60*'
     * '/,NA(4)/1000/
      DATA DIGIT/'1','2','3','4','5','6','7','8','9','10','11','12','13'
     *,'14','15','16','17','18','19','20','21','22','23','24','25','26',
     *'27','28','29','30','31','32','33','34','35','36','37','38','39','
     *40','41','42','43','44','45','46','47','48','49','50','51','52','5
     *3','54','55','56','57','58','59','60','61','62','63','64','65','66
     *','67','68','69','70','71','72','73','74','75','76','77','78',' 79
     *','80','81','82','83','84','85','86','87','88','89','90','91','92'
     *,'93','94','95','96','97','98','99','100','101','102','103',' 104'
     *,'105','106','107','108','109','110','111','112','113','114','115'
     *,'116','117','118','119','120','121','122'/
      DATA VF1/'(1H ','','','    ','A1,F','10.2',')'/
      DATA VF2/'(1H ','','','    ','A1,1','X,A8',')'/
      DATA VF3/'(1H0','','','    ','A1,F','3.1,','12F1','0.2)'/
      DATA VF4/'(1H0','','','    ','X,I2',',2X,','20F6','.1/(','    ','X,2
     *0','F6.1','))'/
      DATA XLABEL /' X DIS- ','TANCE IN','  MILES '/,YLABEL/' Y DISTA','
     *NCE IN ','  MILES '/
      DELX=5280.
      DELY=5280.
      BNUM1=.5E40
      JNO1=NC-1
      INO1=NR-1
      DINCH=2.
      SCALE=5280.
      SPACNG=5.
  100 FORMAT(20G4.0)
  110 FORMAT(//,1X,'LAYER   ',I2,//)
  120 FORMAT(1X,I3,1X,20F6.1,/,5X,20F6.1)
      WIDTH=0.
      YDIM=0.
```

```
      DO 130 I=1,NC
  130 WIDTH=WIDTH+X(I)
      DO 140 J=1,NR
  140 YDIM=YDIM+Y(J)
      IM=MIN0(6*NR+4,124)
      IM=(132-IM)/2
      VF4(3)=DIGIT(IM)
      VF4(8)=DIGIT(IM+5)
  150 XSF=DINCH*SCALE
      NYD=YDIM/XSF
      IF(NYD*XSF.LE.YDIM-Y(NR)/2.) NYD=NYD+1
      IF (NYD.LE.12) GO TO 160
      DINCH=YDIM/(12.*SCALE)
      WRITE(6,170) DINCH
      IF(SCALE.LT.1.0) WRITE(6,180)
      GO TO 150
  160 NXD=WIDTH/XSF
  170 FORMAT(/,' SCALE SHOULD BE ',F10.2)
  180 FORMAT(/,' SCALE SHOULD BE GT 1.')
      IF(NXD*XSF.LE.WIDTH-X(NC)/2.) NXD=NXD+1
      N4=NXD*N1+1
      N5=NXD+1
      N6=NYD+1
      N8=N2*NYD+1
      NA(1)=N4/2-1
      NA(2)=N4/2
      NA(3)=N4/2+3
      NCC=(N3-N8-10)/2
      ND=NCC+N8
      NE=MAX0(N5,N6)
      VF1(3)=DIGIT(ND)
      VF2(3)=DIGIT(ND)
      [illegible]=DIGIT(NCC)
      DO 210 I=1,NE
      NNX=N5-I
      [illegible]-1
      IF(NNY.GE.N6) GO TO 190
      YN(I)=XSF*NNY/SCALE
  190 IF(NNX.LT.0) GO TO 200
      XN(I)=XSF*NNX/SCALE
  200 CONTINUE
  210 CONTINUE
      DO 440 K=1,NL
      DIST=WIDTH-X(1)/2.
      JJ=JN01
      JJ=NC
      LL=1
      Z=NXD*XSF
```



```
      IF(K.EQ.1) WRITE(6,220) IKP
      IF(K.EQ.2) WRITE(6,230) IKP
      IF(K.EQ.3) WRITE(6,240) IKP
      IF(K.EQ.4) WRITE(6,250) IKP
  220 FORMAT('1',5X,'PLEISTOCENE AQUIFER    ',7X,'PERIOD # ',I4)
  230 FORMAT('1',5X,'POCOMOKE AQUIFER       ',7X,'PERIOD # ',I4)
  240 FORMAT('1',5X,'OCEAN CITY AQUIFER     ',7X,'PERIOD # ',I4)
  250 FORMAT('1',5X,'MANOKIN AQUIFER        ',7X,'PERIOD # ',I4)
      WRITE(6  ,410) (TTLE(I),I=1,4)
      DO 400 I=1,N4
      IF (I.EQ.1.OR.I.EQ.N4) GO TO 260
      PRNT(1)=SYM(23)
      PRNT(N8)=SYM(23)
      IF ((I-1)/N1*N1.NE.I-1) GO TO 290
      PRNT(1)=SYM(25)
      PRNT(N8)=SYM(25)
      GO TO 290
  260 DO 280 J=1,N8
      JM1=J-1
      IF(JM1/N2*N2.EQ.JM1) GO TO 270
      PRNT(J)=SYM(24)
      GO TO 280
  270 PRNT(J)=SYM(25)
  280 CONTINUE
      GO TO 350
  290 IF (DIST.LT.0..OR.DIST.LT.Z-XN1*XSF) GO TO 350
      YLEN=Y(1)/2.
      DO 340 L=1,NR
      J=YLEN*N2/XSF+1.5
      KPRM=H(JJ,L,K)/SPACNG
      IF(KPRM) 300,310,300
  300 KPRM=ABS(KPRM)
      IF(KPRM.GT.0..AND.KPRM.LT.1.)  GO TO 320
      KPRM=AMOD(KPRM,20.)
      N=KPRM
      PRNT(J)=SYM(N+1)
      GO TO 330
  310 PRNT(J)=SYM(26)
      GO TO 330
  320 PRNT(J)=SYM(22)
  330 IF(SF1(JJ,L,K).GT.HNUM1) PRNT(J)=SYM(27)
      IF(Q(JJ,L,K).NE.0.) PRNT(J)=SYM(28)
  340 YLEN=YLEN+(Y(L)+Y(L+1))/2.
      DIST=DIST-(X(JJ)+X(JJ-1))/2.
      JJ=JJ-1
  350 CONTINUE
      IF (I-NA(LL).EQ.0) GO TO 370
      IF ((I-1)/N1*N1-(I-1)) 380,360,380
```

```
360 WRITE (6,VF1) (BLANK(J),J=1,NCC),(PRNT(J),J=1,NB),AN(1+(I-1)/6)
    GO TO 390
370 WRITE (6,VF2) (BLANK(J),J=1,NCC),(PRNT(J),J=1,NB),XLABEL(LL)
    LL=LL+1
    GO TO 390
380 WRITE (6,VF2) (BLANK(J),J=1,NCC),(PRNT(J),J=1,NB)
390 Z=Z-2.*ANI*XSF
    DO 400 J=1,NN
400 PRNT(J)=SYM(I)
    WRITE (6,VF3) (BLANK(J),J=1,NCC),(YN(I),I=1,N6)
    WRITE(6,420) (YLABEL(I),I=1,3)
    WRITE(6,430) SPACNG
410 FORMAT(//,46X,4A8,//)
420 FORMAT('0',39X,6A8)
430 FORMAT(//,' CONTOUR INTERVAL  =',F10.2)
440 CONTINUE
    RETURN
    END
```


APPENDIX IV

COMPUTER INPUT DATA-DECK SETUP

Figures 13 to 22 show the arrangement of the input data cards. Values used for the Ocean City model were punched in the cards as an example. The first two cards (fig. 13) are the title cards. A 160-character title can be accommodated. The parameter card determines the dimension of the grid, number of pumping period, size of the first time step, closure criterion and previous simulation time (in hour).

CARD	COLUMN	FORMAT	PARAMETER	EXPLANATION
Title card	1-80	20A4	TITLE	Title of the project
Parameter card	1-3	I3	NC	Number of columns
	4-6	I3	NR	Number of rows
	7-9	I3	NL	Number of aquifers
	10-12	I3	NPER	Number of pumping period
	13-17	G5.0	DELTA	Time step size
	18-22	G5.0	ERROR	Closure criterion
	23-33	G10.0	ATIME	Previous simulation time, 0 when starting a new simulation.

The confining beds are numbered according to the position of the aquifer below it. On top of aquifer 1 is confining bed 1, and so on. The following cards (fig. 14) assign a single default value for a parameter which is assumed to be uniform. In this model the values of aquifer storage coefficient and confining bed hydraulic properties are assumed to be uniform.

Four aquifers were used in the Ocean City model. One card is required for each aquifer and its overlying confining bed.

CARD	COLUMN	FORMAT	PARAMETER	EXPLANATION
Default value card	1-5	I5	K	Counting number for aquifer and its overlying confining bed
	6-15	G10.0	SC(K)	Aquifer storage coefficient
	16-25	G10.0	RATE(K)	Leakance factor K′/b′
	26-35	G10.0	SS(K)	Confining bed specific storage
	36-45	G10.0	CM(K)	Confining bed thickness

The option card (fig. 15) specifies the maximum number of iterations and selects options. Insert 1 if an option is to be performed, and 0 if it is not wanted.

CARD	COLUMN	FORMAT	PARAMETER	EXPLANATION
Option card	1-2	I2	IITER	Maximum number of iteration
	3	I1	IHDN	Print out of computed head
	4	I1	IDDN	Print out of computed drawdowns
	5	I1	IWRT	Print out of transmissivity arrays
	6	I1	ISS	Steady state run
	7	I1	IREP	Continuation run
	8	I1	IPCH	Punch out of head for a continuation run
	9	I1	IMAP	Print out of map
	10	I1	IWMB	Print out of material balance
	11	I1	IWCH	Print out of head for a continuation run

The next cards (fig. 15, 16) assign values to the material balance parameters for a continuation run. For a new simulation, use blank cards.

CARD	COLUMN	FORMAT	PARAMETER	EXPLANATION
Material-balance parameter	2-20	E19.6	STORT(K)	Volume from storage
	22-40	E19.6	CHDT(K)	Recharge from constant head nodes
	42-60	E19.6	CHST(K)	Discharge from constant head nodes
	62-80	E19.6	PUMT(K)	Volume pumped
	2-20	E19.6	CLST(K)	Recharge from leakage
	22-40	E19.6	CLNT(K)	Discharge from leakage
	42-60	E19.6	TLNT(K)	Recharge from transient leakage
	62-80	E19.6	TLST(K)	Discharge from transient leakage

The grid spacing is specified by the next set of cards (fig. 17).

CARD	COLUMN	FORMAT	PARAMETER	EXPLANATION
Grid spacing	1-10	G10.0	FC	Multiplication factor for grid spacing
	1-80	G4.0	X(I)	Grid spacing in the X-direction, 20 values in each card.
	1-80	G4.0	Y(J)	Grid spacing in the Y-direction, 20 values in each card.

The next group of cards specify the transmissivity arrays (fig. 18, 19). One factor card followed by one set of value cards are needed for each aquifer. Each number on the transmissivity-value cards is multiplied by the number on the factor card as the matrix is read.

CARD	COLUMN	FORMAT	PARAMETER	EXPLANATION
Factor card	1-10	G10.0	FCT(K)	Multiplication factor. One card for each aquifer.
Transmissivity value card	1-80	G4.0	TS(I, J, K)	Transmissivity, 20 values for each card. Each new row started with a new card.

Head arrays for each aquifer follow the transmissivity arrays (fig. 20, 21). If initial conditions are related to drawdown rather than head, drawdown values are entered in the head arrays.

CARD	COLUMN	FORMAT	PARAMETER	EXPLANATION
Head	1-80	G4.0	HI(I, J, K)	Head or drawdown distribution, 20 values on each card. Each new row started with a new card.

Pumping period and well cards follow the head cards (fig. 22). One pumping period card plus one well card for each well pumped during the period are required for each pumping period. In the example shown on figure 22, no wells are pumped during the first period (KP = 1, NWEL = 0); thus no well cards are required. Four wells are pumped during pumping period number two (KP = 2, NWEL = 4); thus four well cards are required (only the first and last well cards are shown in the example).

CARD	COLUMN	FORMAT	PARAMETER	EXPLANATION
Pumping period	1-10	G10.0	KP	Current pumping period counting number
	11-20	G10.0	KPM1	Previous pumping period counting number
	21-30	G10.0	NWEL	Number of well
	31-40	G10.0	TMAX	Total period of pumping
	41-50	G10.0	NUMT	Estimated number of time steps
	51-60	G10.0	CDLT	Multiplication factor for the size of time step

CARD	COLUMN	FORMAT	PARAMETER	EXPLANATION
Well card	1-10	G10.0	I	X position of the well
	11-20	G10.0	J	Y position of the well
	21-30	G10.0	K	Z position of the well
	31-40	G10.0	Q(I, J, K)	Rate of withdrawal
	41-50	G10.0	RADIUS (I, J, K)	Radius of the well
	51-60	G10.0	NW(K)	Total number of wells in aquifer K

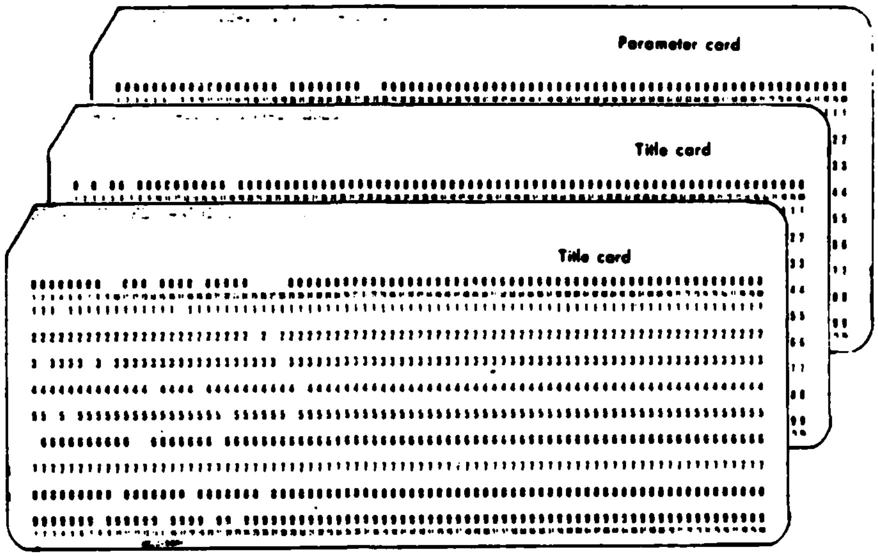

Figure 13. Data-deck setup: Title and parameter cards.

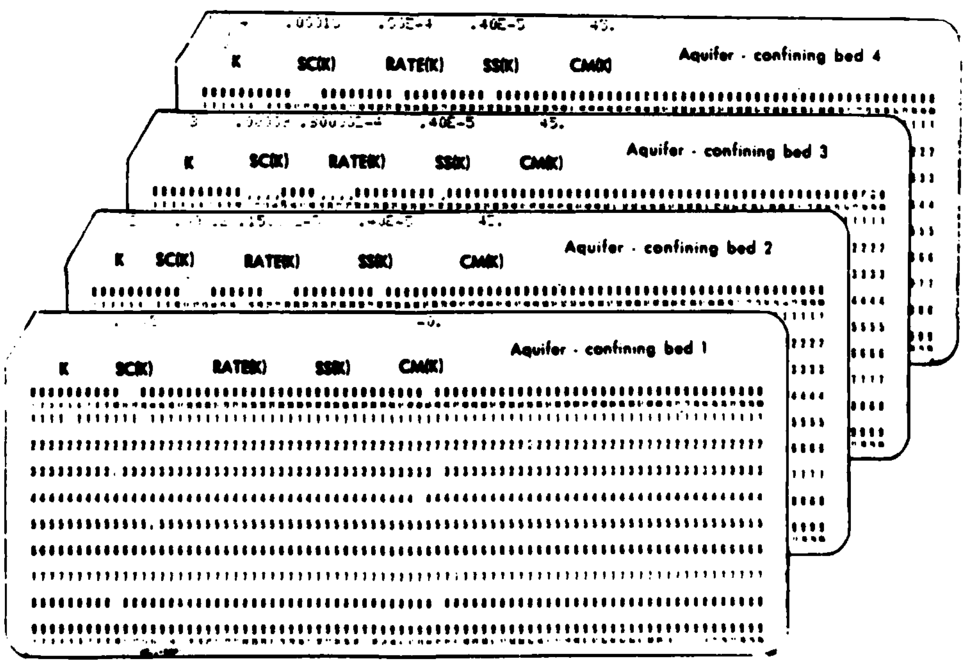

Figure 14. Data-deck setup: Aquifer and confining bed parameter default value cards.

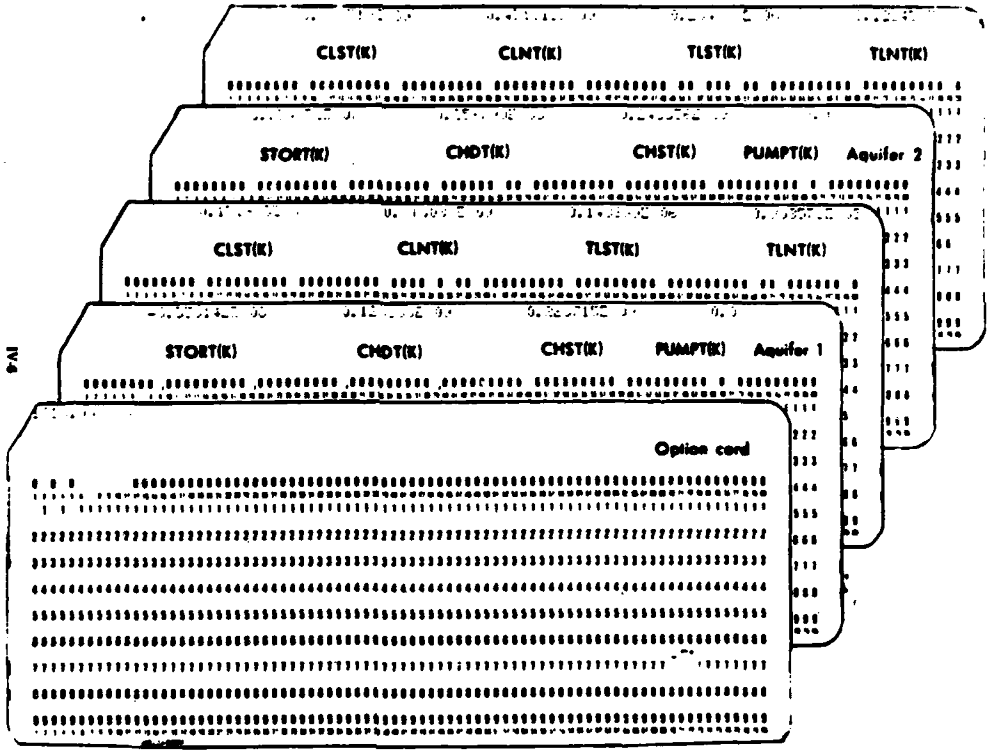

Figure 15. Data-deck setup: Options and material-balance values.

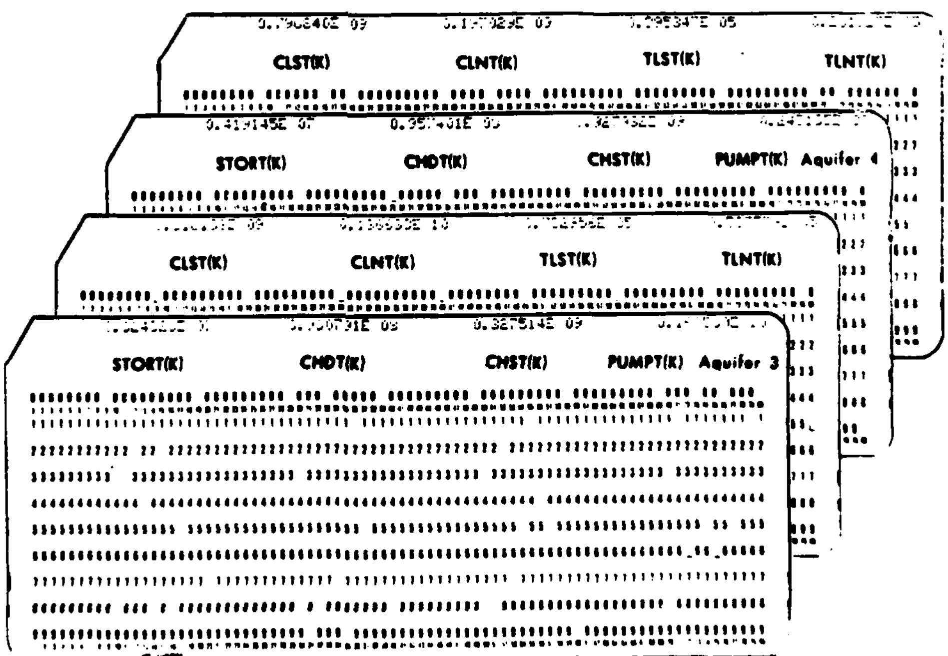

Figure 16. Data-deck setup: Material-balance values (continued).

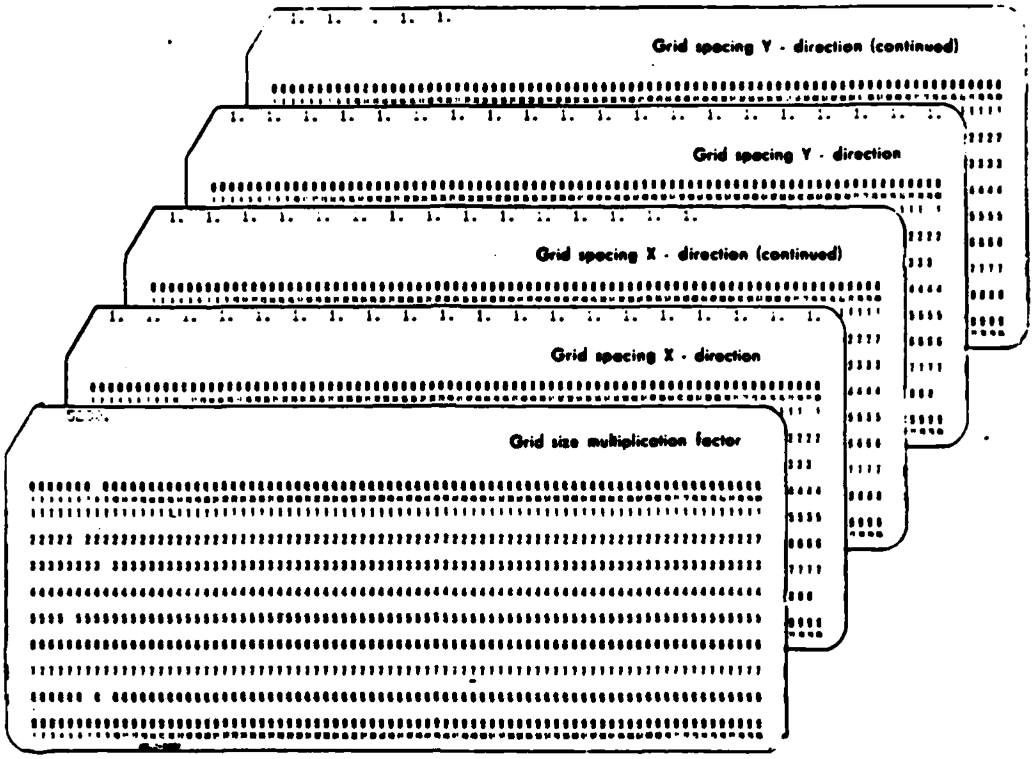

Figure 17. Data-deck setup: Grid spacing.

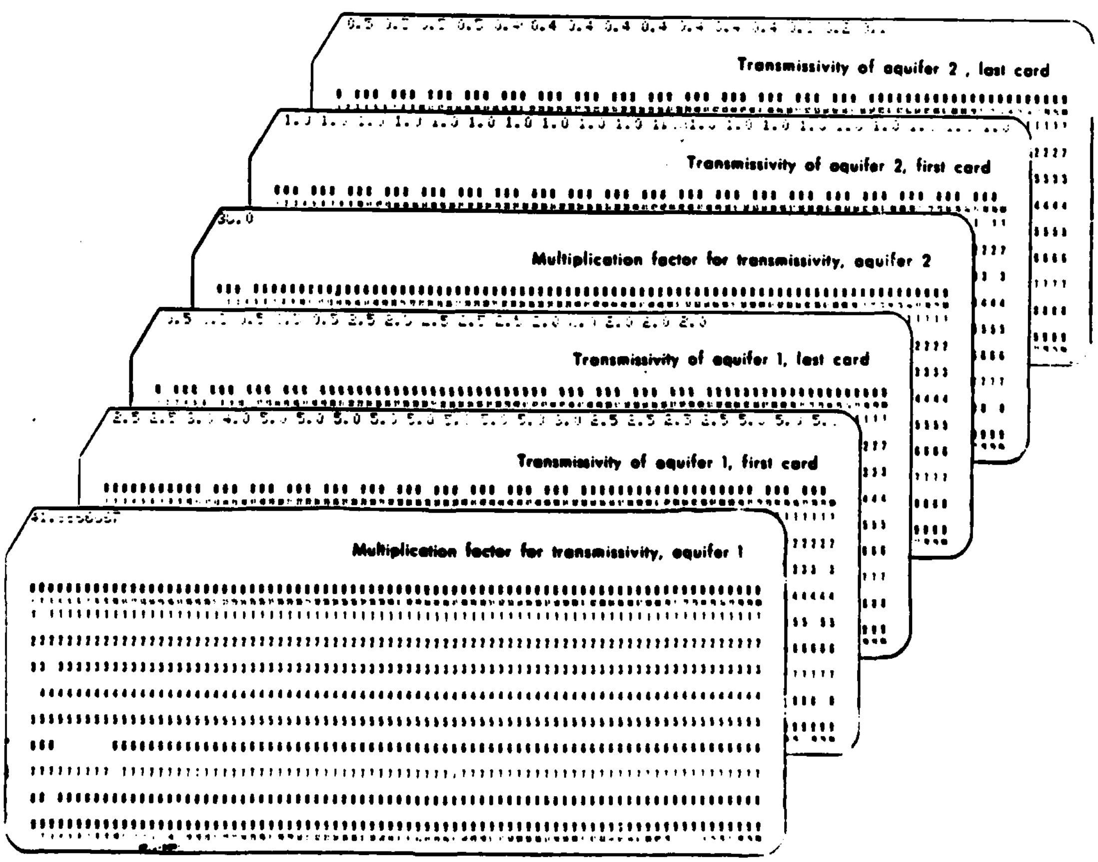

Figure 18. Data-deck setup: Transmissivity arrays.

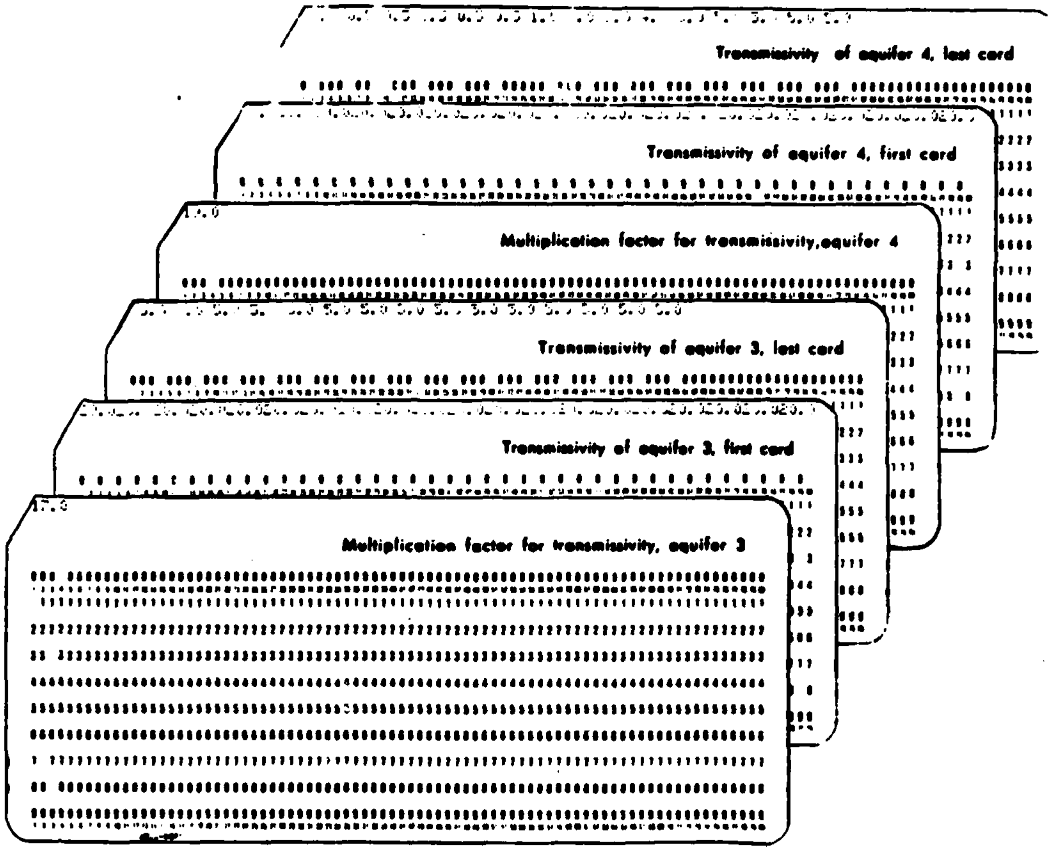

Figure 19. Data-deck setup: Transmissivity arrays (continued).

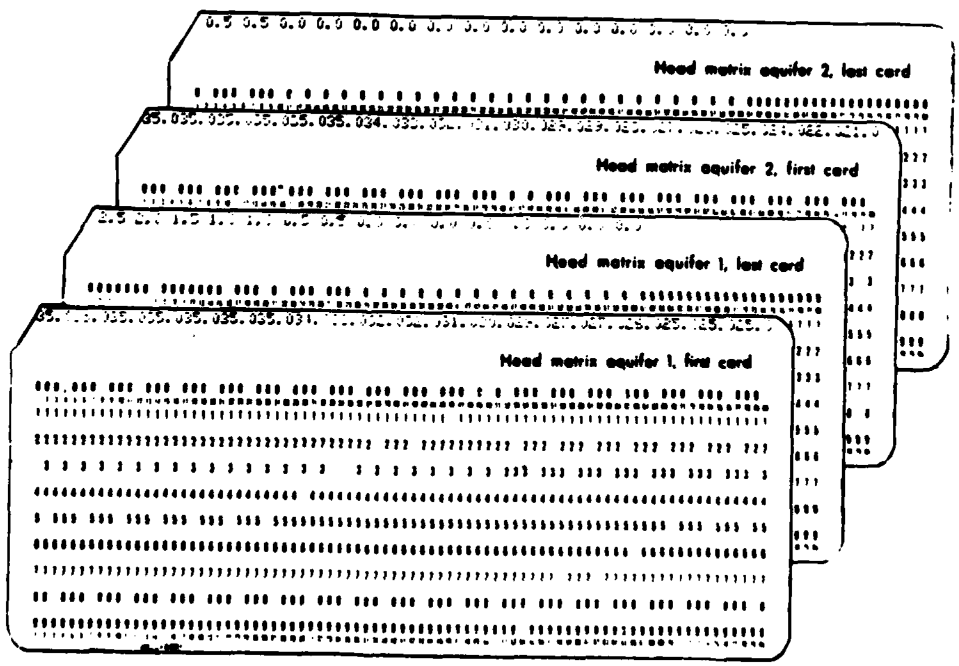

Figure 20. Data-deck setup: Head matrix.

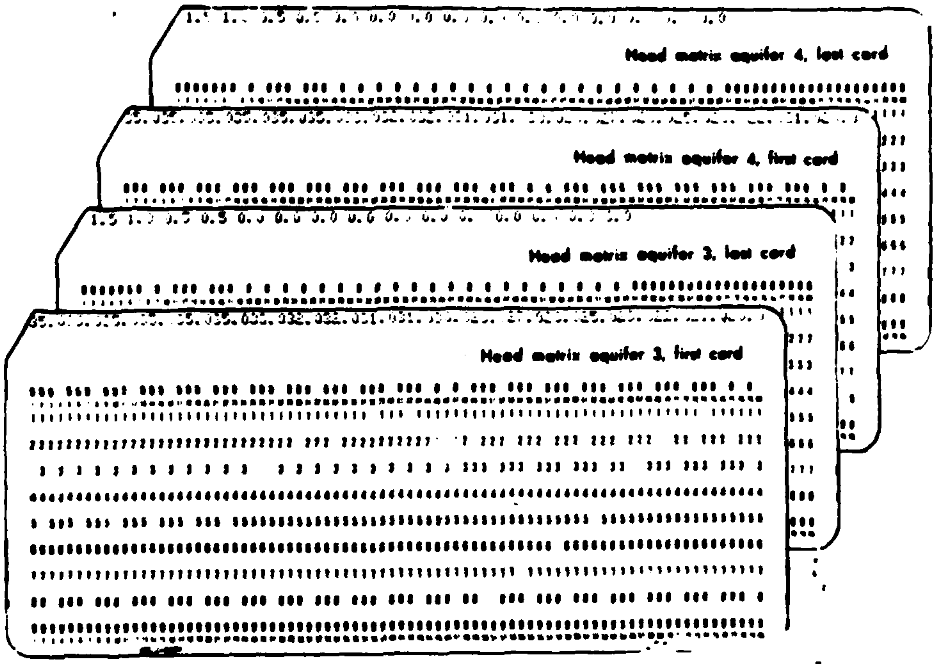

Figure 21. Data-deck setup: Head matrix (continued).

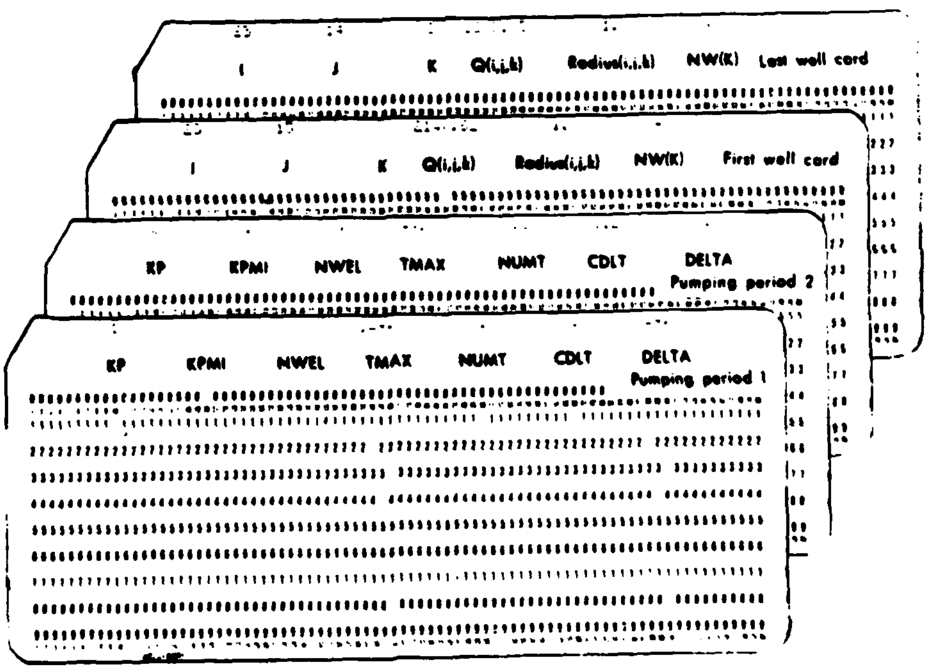

Figure 22. Data-deck setup: Pumping period and well cards.

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