Report on

WATER SUPPLY AND TREATMENT ALTERNATIVES

FOR

DARE COUNTY, NORTH CAROLINA

Prepared By:

BLACK & VEATCH, ENGINEERS ARCHITECTS

January 1987

Project No. 02581.113

BLACK & VEATCH, INC.
CONSULTING ENGINEERS
TEL. (919) 672-3600

110 WEST WALKER AVENUE
ASHEBORO, NORTH CAROLINA 27203

MAILING ADDRESS: P.O. BOX NO. 728
ASHEBORO, NORTH CAROLINA 27204-0728

Dare County
Water Supply and Treatment
Study

B&V Project 02581.113
B&V File E
March 31, 1987

Dare County
P. O. Drawer 1000
Manteo, North Carolina 27954

Attention: Mr. Robert V. Owens, Jr.
Chairman of the Board of Commissioners

Gentlemen:

Enclosed is the final report of our study of water supply and treatment alternatives for the Dare County water system. This study was undertaken at the request of County Manager, Jack Cahoon for the purpose of identifying alternatives to the further development of the water supply facilities on Roanoke Island.

The report begins with an introductory section that defines the study purpose and scope. This is followed by a summary of the major findings and recommendations, which can be used as a quick reference section. Several sections then discuss the details of the study and describe needed improvements. The final section describes recommended improvements in greater detail.

It has been a pleasure to complete this study for Dare County. We have received assistance from Jack Cahoon, Phelpie Edmonson, Randy McPhee, and others on their staff, for which we express our deep thanks. We believe that the results of the study, as expressed in this report, will serve Dare County well in meeting water demands in the coming years.

We are available to present this report at a Commission meeting, to discuss it with Dare County's staff, and to answer any questions you may have.

Very truly yours,

BLACK & VEATCH, INC.

David A. Todd

David A. Todd

mlp
Enclosure

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1.0 INTRODUCTION

1.1 SCOPE

This study and report analyzes the water system operated by Dare County, projects future population and water demands, and proposes improvements to meet the projected water demands. The report provides guidance to the Dare County Board of Commissioners and the Dare County Water Department for implementing these improvements, including timing, location, and projected costs.

The study area, shown on Figure 1-1, includes all of Dare County. However, this study and report deals mainly with the existing water system, located on Roanoke Island and the outer banks north of the Oregon Inlet. The study period is from the present (1985) through the year 2005.

1.2 PURPOSE

The purposes of this study were to evaluate the Dare County water system in relationship to area growth and to plan improvements based upon that growth.

Although the present Dare County water system is only five years old, the rapid growth and expansion of the service area have brought the system very close to its 5 mgd nominal capacity. Failure to quickly determine short term growth and construct needed improvements will result in a system that is overloaded in only two years. Failure to determine long term growth patterns and water needs could result in inadequate, improperly located, or unnecessary improvements that will increase costs and be detrimental to operations.

This study addressed these points through five major analyses: population growth trends, projected water demands, water supply sources, water system evaluation, and development of an improvement master plan.

1.3 REPORT CONTENTS

This report describes the study and its results. A summary of findings and recommendations is provided to permit quick assessment of the study results and to serve as a master list of the recommended improvements.

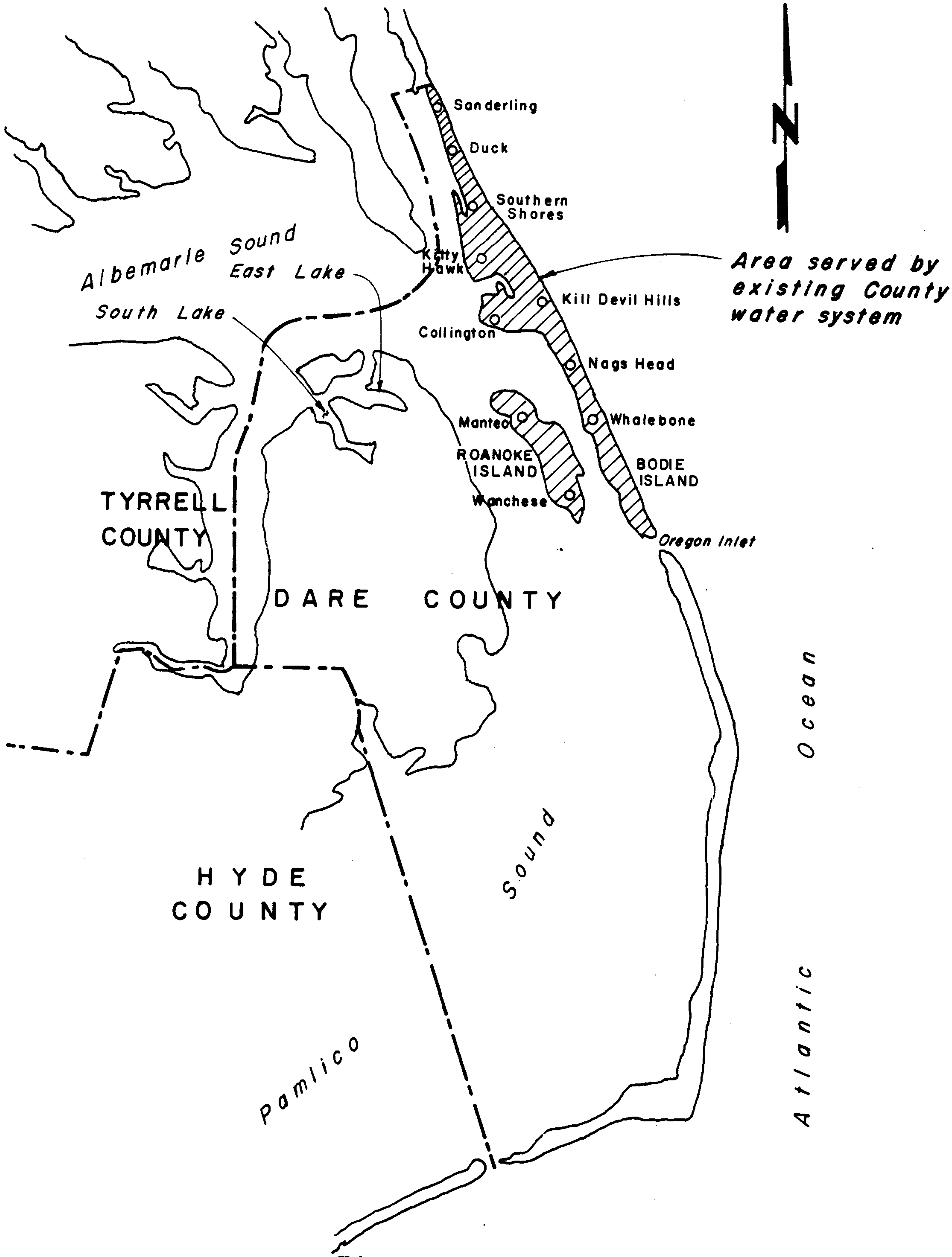

Figure I-I: STUDY AREA

Following the summary are results of the population growth trends and water demand analyses. These form the basis for all recommended improvements. Available water sources are analyzed and evaluated. The existing water system and present operating characteristics are described. Improvements recommended to meet water demands through the year 2005 are presented. Improvements are phased in over the 20 year study period to lessen the capital cost impact. These stages are defined in Section 8.1.

This report is a summary of the analyses performed in the study. Each facet of the study is discussed in sufficient detail to help the reader understand the proposed improvements and their functions.

1.4 REPORT ADDITIONS - 1987

This study and report is built upon the analyses and results of a previous study, Comprehensive Engineering Report on Water System Improvements for County of Dare, March 1984, prepared by Moore, Gardner & Associates, now Black & Veatch, Inc. This new study and report adds the following information.

- Operating data for the years 1984-1986.
- Additional analysis of data from previous years.
- In depth analysis of alternatives to Roanoke Island groundwater. This additional study was conducted as a result of difficulties in the Wanchese community in 1983 and 1984, with private wells being replaced with deep wells at County expense due to the alleged impact of County wells on the private wells. This experience, together with the stated resistance of Roanoke Island residents to the extension of the County distribution system on the island, has required the study of alternative sources of water despite the lower cost of Roanoke Island groundwater development as shown in the previous report.

In addition, certain sections of the report have been rewritten, and the report reorganized, to improve readability and the presentation of information.

1.5 RELATED REPORTS

Reports reviewed and considered under this study are as follows.

- <u>Comprehensive Engineering Report on Water System Improvements for County of Dare</u>, by Moore, Gardner & Associates/Black & Veatch, 1984, hereinafter be referred to as the 1984 MGA Report.

- Although the final report was not available, consideration was given to the work undertaken in the carrying capacity study, by Booz, Allen, and Hamilton, hereinafter be referred to as the 1986 Carrying Capacity Study.

- Letter report, dated November 11, 1984, by Black & Veatch, discussing water use.

- Brown, P.E., et al., <u>Structural and Stragraphic Framework and Spatial Distribution of Permeability of the Atlantic Coastal Plain, North Carolina to New York,</u> USGS Professional Paper 796, 1972.

- Floyd, E.O., Hydrology of Fresh Pond, Moore, Gardner & Associates, Inc., Asheboro, NC, 1979.

- Heath, Ralph, <u>Basic Principles of Groundwater Hydrology with Emphasis on North Carolina</u>, USGS, Raleigh, NC, 1980.

- Peek, Harry, et al., <u>Potential Groundwater Supplies for Roanoke Island and The Dare County Beaches, NC,</u> NRCD, Raleigh, NC, 1972.

1.6 ABBREVIATIONS

AAD	Annual Average Day
AOSD	Average Off-Season Day
APSD	Average Peak Season Day
CHWA	Cape Hatteras Water Association
DEM	Division of Environmental Management
ED	Electrodialysis
gpcd	Gallons Per Capita Per Day
gpm	Gallons Per Minute
IX	Ion Exchange
MG	Million Gallons
MD	Maximum Day
mgd	Million Gallons Per Day
mg/l	Milligrams Per Liter (equivalent to parts per million)
MH	Maximum Hour

NRCD	Department of Natural Resources & Community Development
psi	Pounds Per Square Inch
RO	Reverse Osmosis
TDS	Total Dissolved Solids

2.0 SUMMARY OF FINDINGS AND RECOMMENDATIONS

2.1 FINDINGS

(1) The total permanent population of Dare County is projected to increase from 19,500 in 1985 to 30,000 by 1995, and 48,000 by 2005.

(2) The total permanent plus seasonal population in Dare County is projected to increase from 98,000 in 1985 to 141,000 by 1995, and 201,000 by 2005.

(3) Potential new service areas that could feasibly be served from the County water system are projected to have a population of 2,120 by 1985, 2,700 by 1995, and 3,300 by 2005.

(4) Maximum Day (MD) water demand is projected to be 6.6, 10.3, and 15.2 mgd in the existing service area by 1985, 1995, and 2005, respectively. Inclusion of new service areas on Roanoke Island will increase these demands by 0.2 to 0.3 mgd in 1995 and 2005.

(5) Potential water sales to Currituck County may result in additional MD demands of 0.5, 1.5, and 2.0 mgd in 1987, 1995, and 2005 respectively.

(6) Expansion of the existing water supply on Roanoke Island depends upon additional subsurface investigation and analysis. If the supply can be expanded with a well spacing similar to the existing wells, this system can be economically expanded. This will however, require abandonment of existing private wells and extension of the distribution system.

(7) The least costly alternative to expanding the Roanoke Island water supply is desalination of brackish groundwater on the Outer Banks.

(8) The existing Roanoke Island water treatment plant is over-rated by conventional design standards.

(9) The existing water distribution system north of Kill Devil Hills has inadequate pumping, storage, pressure, and water delivery capabilities to support additional growth beyond 1987.

2.2 RECOMMENDATIONS

(1) Dare County should plan for providing 10.3 mgd and 15.2 mgd maximum day demands by 1995 and 2005, respectively.

(2) Because of the apparent beginning of salt water intrusion, the existing groundwater supply on Roanoke Island should not be expanded unless additional hydrological investigation is made.

(3) The existing water treatment plant on Roanoke Island should continue to operate at maximum capacity until additional treatment capacity is available. At that time the plant should be derated to 3.0 mgd.

(4) The County should construct a water treatment plant on the Outer Banks to treat brackish groundwater. The initial capacity of the plant should be 5.0 mgd, with expansions to 8.0 mgd and 12.0 mgd as demands increase.

(5) A well field and piping system should be constructed to tap brackish groundwater from the Yorktown aquifer. The Baum Tract is a suitable location for the well field, although additional study is required to determine if additional property is required in Stage 2 construction.

(6) A study of brackish surface water should be undertaken to determine if it can be used as an alternative supply to groundwater.

3.0 DEMOGRAPHIC ANALYSIS

3.1 HISTORICAL POPULATION GROWTH

Dare County has experienced significant population growth since 1970. Table 3-1 and Figure 3-1 illustrate this growth, with Table 3-1 broken down by geographic area. This table represents the permanent population only, and does not reflect the recent rapid growth in seasonal population, which is better shown by statistics pertaining to building permits and residence construction. The number of housing units increased from 5,057 in 1970 to 11,006 in 1980. Approximately 60 percent of these are occupied by year-round residents. Thus, based on the number of year-round housing units and 1980 Census population, there are 2.19 persons per housing unit. The geographic areas in the County and the population growth of each are discussed below.

3.1.1 Mainland

Growth on the mainland has been slower than in other parts of the County. Approximately 6 percent of the total permanent population resides in Croaton and East Lake Townships. Growth on the mainland increased by approximately 30 percent between 1970 and 1980. This growth represents only 3 percent of the overall increase in population in the County. The majority of the population on the mainland is concentrated around the Manns Harbor and Stumpy Point areas. The mainland is not served by the County water system.

3.1.2 Roanoke Island

Growth on Roanoke Island has occurred mainly in the vicinity of the Town of Manteo and the Wanchese community. The island is a part of the Nags Head Township. Approximately 30 percent of the County's permanent population inhabits the island. Between 1970 and 1980, the island population increased by 35.6 percent, which represents 16.2 percent of the total growth in the County. At present, only the Town of Manteo and approximately 80 other connections are served by the County water system.

TABLE 3-1

POPULATION TRENDS, DARE COUNTY, NC

Township	Population 1970	1980	% Change	% of Total
(3) Atlantic	1,141	3,732	227.5	28
(1) Croaton	540	714	32.2	5
(1) East Lake	88	112	27.3	1
(3) Hatteras	1,333	2,783	108.8	21
(3) Kennekeet	565	1,060	87.6	8
(2) Nags Head	3,328	4,971	49.4	37
Dare County	6,995	13,372	91.2	100%

Township	Housing Units 1970	1980	% Change	% of Total
(3) Atlantic	1,883	4,851	157.6	44
(1) Croaton	214	358	79.9	3
(1) East Lake	42	67	59.5	1
(3) Hatteras	620	1,379	122.4	13
(3) Kennekeet	324	648	100.0	6
(2) Nags Head	1,974	3,676	86.2	33
Dare County	5,057	10,979	117.6	100%

(1) Mainland

(2) Roanoke Island and Part of Outer Banks

(3) Outer Banks

Source: 1980 US Census

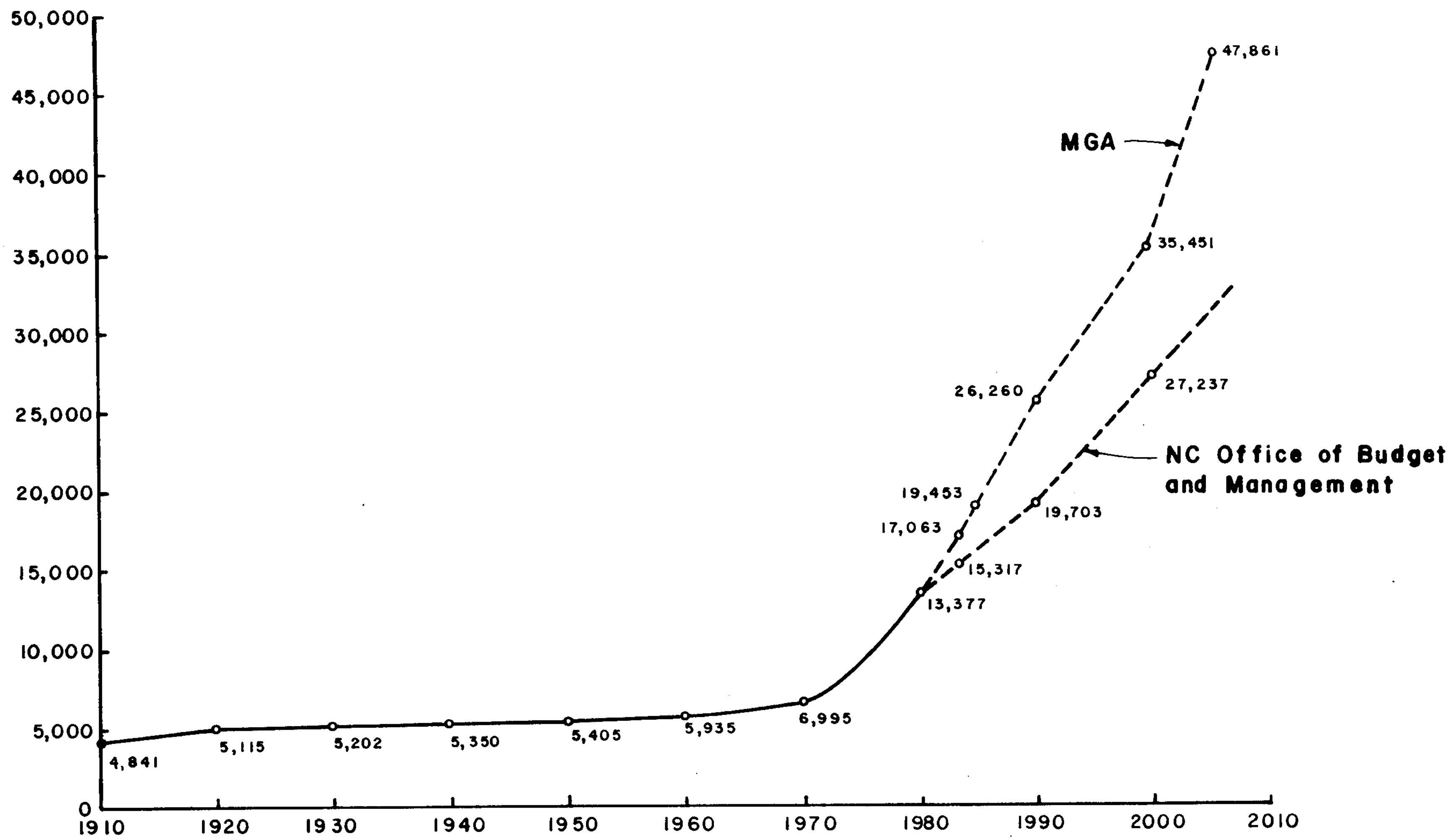

FIGURE 3-1: PERMANENT POPULATION PROJECTION

3.1.3 Outer Banks

The Outer Banks of Dare County are divided into two segments which are separated by the Oregon Inlet. The southern section, Pea/Hatteras Island, comprises the Kennekeet and Hatteras townships. Pea/Hatteras Island has 29 percent of the total County population. Between 1970 and 1980 the area population increased by 102 percent, which is 30 percent of the total County population growth.

The northern segment, Bodie Island, consists of the Atlantic and Nags Head townships. These areas have aproximately 35 percent of the County's population. Growth of Bodie Island population between 1970 and 1980 was 205.6 percent, or 50.2 percent of the total growth in the County. All of this area is within the County water system service area, although the system does not extend to all potential customers at present.

The years 1980 through 1982 saw growth continuing at a rapid pace. Approximately half of the 4,206 building permits were issued for residential construction. Approximately 60 percent of these were for permanent residences, with the remainder being seasonal cottages and timeshare condominiums. Construction of all three types of residences is undergoing growth, with the most rapid growth being in time share condominiums. The permanent population in Dare County in 1983 was estimated to be 17,100. Continued growth has been experienced since 1983.

3.2 LAND USE

An important aspect of projecting future growth and population trends is the amount of land available for development. Dare County consists of 391 square miles of land and 800 square miles of water, for a total of 811,777 acres of land and water. According to the County's Land Use Plan, 15 percent of the total land area is suitable for development. The land use in the County is illustrated in Table 3-2. The actual acreage in each land use class may have changed, since previous reports indicate that large tracts of the mainland could be converted to commercial farming.

It is estimated that 18,900 acres are classified as developed or in a transition stage (subdivided for development). The transitional land is located near or adjacent to developed land, along highways, or in areas where the soils and accessibility indicate a high development potential.

TABLE 3-2

LAND USE CLASSIFICATION

Land Use	Acres
Agricultural	242(1)
Forestry	173,688
Governmental	67,190
Urban (Developed)	6,100
Total Land	247,220
Water	564,557
Total Acreage	811,777

(1) This figure may actually be higher since some cut-over areas are still in transition and could be used for farming.

Source: Dare County Land Use Plan 1976

3.2.1 Mainland

The mainland portion of Dare County consists of approximately 182,500 acres, of which 6,600 acres are classified as developed or transitional. Developed areas are located in the communities of East Lake, Stumpy Point, Mann's Harbor and Mashoes. The population density in these areas is estimated to be approximately 0.15 person per acre.

3.2.2 Roanoke Island

Roanoke Island consists of approximately 12,400 acres, of which 5,116 acres are considered to be developed or suitable for development. The developed areas are centered around the Town of Manteo and the Wanchese community. The majority of the residences are permanent (year-round). Approximately 5,120 persons reside on the island. The population density of the developed area is estimated to be 1.00 person per acre. The island is considered approximately 60 percent developed.

3.2.3 Outer Banks

The Outer Banks consist of approximately 51,633 acres, 26,502 of which are north and 25,131 acres south of Oregon Inlet. Because approximately 16,685 acres of developed or developable land are located to the north of Oregon Inlet, and only 3,116 acres to the south, most of the development is occurring to the north. The municipalities are undergoing growth in the north, as is the unincorporated Duck area north of Southern Shores.

A large concentration of the seasonal population occupies the northern segment of the Outer Banks. The rapid increase in both seasonal and permanent population has brought up the question of what the ultimate saturation population would be.

Assuming that 75 percent of the total developed and developable land on the Outer Banks and Roanoke Island will be used for residential purposes, 280,000 persons could be accommodated during the peak season. This is based on an average density of 15 persons per acre on the Outer Banks and 1.0 person per acre on Roanoke Island.

3.3 POPULATION PROJECTIONS

Calculations have determined that the permanent population is increasing approximately 7 percent and seasonal population by approximately 5 percent per year. The population projections in Table 3-3 are based on current growth trends through the planning period. The accuracy of the projections is based on assumptions that vary with a number of economic, social and climatic conditions. These projections must be updated at intervals of no more than five years to keep the projections in line with actual events.

The carrying capacity study undertaken simultaneously with this study should provide more accurate figures concerning the maximum population and projected rates of development.

The population projection results of this study were compared to the projections of the North Carolina Office of Budget and Management. The State's projections for the years 1990 and 2000 are approximately 25 percent lower than those of this study. This difference emphasizes the need for updating population projections at least every five years. The population projections are shown in Figure 3-1.

3.3.1 Existing Service Area

The populations of the five existing service areas within the County are shown in Table 3-4. Dare County supplies water to all of the areas with the exception of CHWA.

The County service area population was determined by multiplying the number of units by 2.19 persons per unit for permanent units, 6.7 persons per unit for seasonal cottages, and 3.6 persons per unit for multiple unit residences.

3.3.2 Potential New Service Areas

Potential new service areas were identified by analyzing areas with sufficient population density to feasibly support a water system, and areas that are in great need of water service. Each potential new service area is

TABLE 3-3

POPULATION PROJECTIONS (PERMANENT AND SEASONAL)

PERMANENT

Township	% of Total Population	1980	1983	1985	1990	2000	2005
Atlantic	28	3,732	4,778	5,447	7,353	9,926	13,401
Croaton	5	714	853	953	1,313	1,774	2,393
East Lake	1	112	170	195	263	355	479
Hatteras	21	2,783	3,583	4,085	5,515	7,445	10,051
Kennekeet	8	1,060	1,365	1,556	2,100	2,836	3,829
Nags Head	37	4,971	6,314	7,197	9,716	13,117	17,708
Dare County	100	13,372	17,063	19,433	26,260	35,453	47,861

SEASONAL

Accommodations	Units	Occupancy Per Unit	1983	1985	1990	2000	2005
Motels	3,400	3.6	12,240	13,464	16,830	21,037	26,269
Cottage Courts	750	2.4	1,800	1,980	2,475	3,093	3,866
Campgrounds	1,945	3.8	7,391	8,130	10,162	12,702	15,877
Trailers	1,670	3.8	6,346	6,980	8,725	10,906	13,632
Tents	275	3.8	1,045	1,150	1,437	1,796	2,245
Cottages	6,015	6.7	40,300	44,300	55,413	69,265	86,582
Time/Condos	310	6.7	2,077	2,285	2,856	3,570	4,463
Dare County	14,365	N/A	71,199	78,319	97,898	122,369	152,934
Total Permanent & Seasonal			88,262	97,772	124,158	157,822	200,795

Sources: MVERCO (Seasonal Accommodations)

TABLE 3-4

PERMANENT AND SEASONAL SERVICE AREAS POPULATION PROJECTIONS

Existing Service Area	1983	1985	1990	2000	2005
CHWA	7,420	8,500	12,600	20,600	24,400
Nags Head	13,600	17,300	32,200	46,700	61,000
Kill Devil Hills	12,760	16,000	29,000	37,700	46,400
Manteo	980	1,100	1,200	1,300	1,500
Dare County (Kitty Hawk)	16,140	18,400	24,000	32,500	40,900
TOTAL	50,900	61,300	99,000	138,800	174,200
County Water System Service Area	43,480	52,800	86,500	118,200	149,800

Potential New Service Area	1983	1985	1990	2000	2005
Rodanthe-Waves-Salvo	1,274	4,123	5,876	6,610	7,339
Roanoke Island	2,000	2,120	2,440	2,800	3,220
TOTAL	3,274	6,243	8,316	9,410	10,559
Future Service Area for County Water System	43,480	54,920	88,940	121,000	153,020

Notes

1. Rodanthe-Waves-Salvo cannot be economically served from the existing Dare County system.

2. New service areas on Roanoke Island are assumed to be connected in 1985.

discussed in this report. It should be noted that it may not be feasible or desirable to serve all areas from the County system. They are discussed in this report to show the potential for a water system. Should the County decide to extend service to the areas, a detailed study may be required to determine the most feasible water source, be it CHWA, Dare County Water, or a totally new system.

3.3.2.1 Rodanthe-Waves-Salvo. The villages of Rodanthe, Waves and Salvo are well established and relatively isolated communities approximately 14 miles south of Oregon Inlet. Water is provided predominately by individual shallow wells that draw on the fresh water aquifer which is recharged by precipitation. The water in these areas is highly susceptible to salt water intrusion contamination from flooding and by wastewater discharges from septic tanks. As the population increases, the need for a potable water supply will also increase.

Like other areas of the Outer Banks, the villages have experienced a substantial increase in seasonal population. The population projections illustrated in Table 3-4 were taken from the MGA 1982 Engineering Report titled "Water Supply and Treatment Alternatives for Rodanthe-Waves-Salvo." The methodology used in those projections was based on Federal projections.

3.3.2.2 Roanoke Island. The Town of Manteo and the Wanchese community are located on Roanoke Island. The majority of the residents on the Island are permanent inhabitants. The area is approximately 60 percent developed.

The Town of Manteo operates its own water distribution system and purchases treated water from the County. The County also supplies water service to approximately 80 customers in the Manteo area. However, the majority of the residential water supplies on the island are provided by individual shallow wells. The water is susceptible to contamination by flooding and by wastewater discharged from septic tanks. The County uses the groundwater supplies on Roanoke Island as its water source and will continue to depend on this source of groundwater for future supply.

The need for water service on the island will increase as the water demands from the County increase. With increased demand from residents utilizing their own wells and the County obtaining its water from the groundwater source, the stress on the aquifer system will increase.

To eliminate any problems that could result from the competition for water supplies on Roanoke Island, these potential service areas should be brought onto the County system. It is estimated that 2,000 persons are in the service area, which includes Wanchese and areas around Manteo. About 540 possible connections have been identified in Wanchese. Population growth of the island is assumed to be similar to that of Manteo, approximately 3 percent per year. The projected population of the service areas on the island is shown in Table 3-4.

4.0 WATER USE ANALYSIS

4.1 SIGNIFICANT WATER USE DATA

Water use can be measured and quantified in many different ways. For planning and design purposes, the following data have proved to be most useful.

Annual Average Day. Computed by dividing the total annual water production by 365. This value is useful in determining annual operation and maintenance cost and long range water resource requirements.

Maximum Day. The maximum amount of water used in any 24-hour period. This number is used to determine treatment plant capacity. A water system should be capable of supplying the maximum day demand without depleting storage. Also, raw water supply facilities must be capable of delivering the maximum day demand.

Maximum Hour. The average amount of water used in a peak period during a maximum day (or near maximum day). Typically it occurs during late afternoon and early evening for a duration of 3 or 4 hours. Although it occurs for only a few hours, this demand is usually expressed in the same units (mgd) as maximum day and annual average day. This number is used to size distribution mains and pumping and storage, since storage reservoirs are utilized to supply the difference between maximum day and maximum hour rates.

Average Off-Season Day. Computed by dividing the total water production during October through March by 182 days. This value is used for determining water requirements and the approximate system needs during off-peak periods.

Average Peak-Season Day. Computed by dividing the total water produced during June through August by 92 days. This value is used for determining sustained peak usage, which is useful in determining staffing, treatment chemical inventories, and preventive maintenance schedules.

Ratios. The ratios of Maximum Day, Maximum Hour, Average Off-Season Day, and Average Peak-Season Day to Annual Average Day (MD/ADD, MH/AAD, AOSD/AAD, and APSD/AAD are all useful for projecting future water requirements and for recognizing changes in water use patterns. These ratios will fluctuate from year to year, but, in a stable water system, will remain fairly constant with time. A consistent change (up or down) in any of these ratios indicates a change in water use patterns, often caused by changing water system socio-economic status or by changes in industrial or commercial water users. Thus, these ratios can be used to improve the accuracy of projections of future water requirements.

4.2 PAST WATER DEMANDS

Since the County water system has been in operation only since 1980, historical water use records are very limited. Table 4-1 presents past water demand records. This table shows a rapid increase in water usage in all years except 1984 and 1986. This decrease in water use in 1984 was discussed in an MGA letter report dated November 1, 1984. That report identified a number of major leaks in the water system in 1983, which distorted the water use data for that year. Figure 4-1 shows the water production record since the County system began operation in 1980. This figure shows both the erroneous 1983 water production data due to leaks and the large seasonal fluctuation in water use. The increase in metered water use has been continuous since 1980 excepting 1986. Also shown in Table 4-1 are the ratios of MD/AAD, AOSD/AAD, and APSD/AAD computed from historical water use records.

4.3 FUTURE WATER REQUIREMENTS

Figure 4-2 shows a graphical representation of historical and projected water use from 1980 through 2005. The projections for maximum day are based on 105 gallons per capita per day and the population projections of Section 3.0. The annual average day is based upon a MD/AAD factor of 2.1 based on the historical trends. The average off-season day and average peak season day projections are based upon the factors shown in Table 4-1. Also included on Figure 4-2 are the maximum day demands, including potential new service areas on Roanoke Island.

TABLE 4-1
HISTORICAL WATER USE

	1980	1981	1982	1983	1984	1985	1986
Total Water Produced (mg)	160.25	516.07	566.50	738.16	791.83	893.370	856.449
Annual Average Day (mg)	0.98	1.41	1.55	2.02	2.17	2.45	2.346
Maximum Day (mgd)	2.3	3.1	3.4	4.5	4.8	5.0	5.36
Ratio MD/AAD	2.35	2.19	2.19	2.23	2.21	2.04	2.28
Average Off-Season Day (mgd)	NA	0.75	1.06	1.31	1.40	1.56	1.48
Ratio AOSD/AAD	NA	0.53	0.68	0.65	0.65	0.64	0.631
Average Peak-Season Day (mgd)	NA	2.50	2.92	3.66	3.56	3.95	3.77
Ratio APSD/AAD	NA	1.77	1.88	1.81	1.64	1.61	1.61

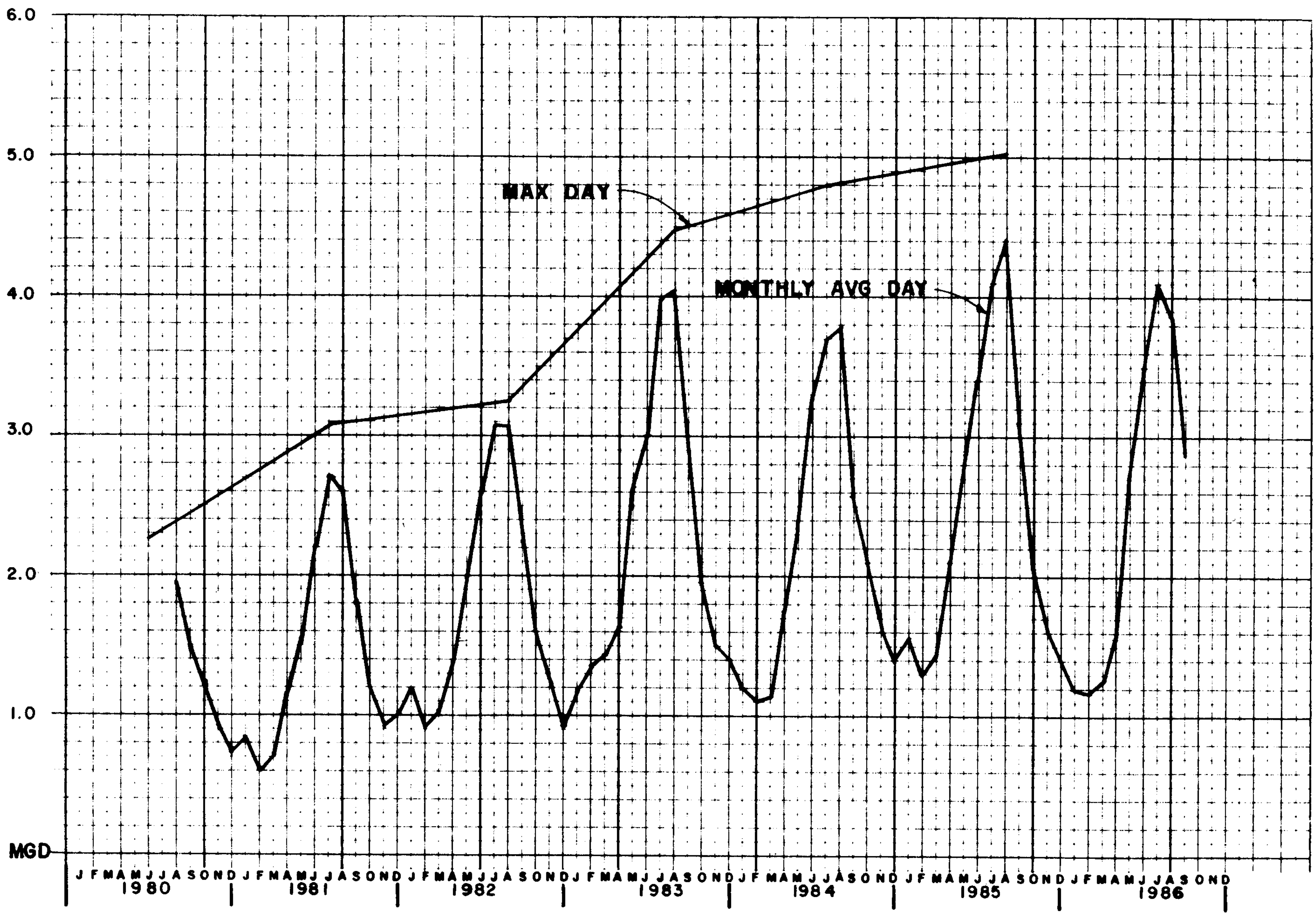

FIGURE 4-1: HISTORICAL WATER PRODUCTION

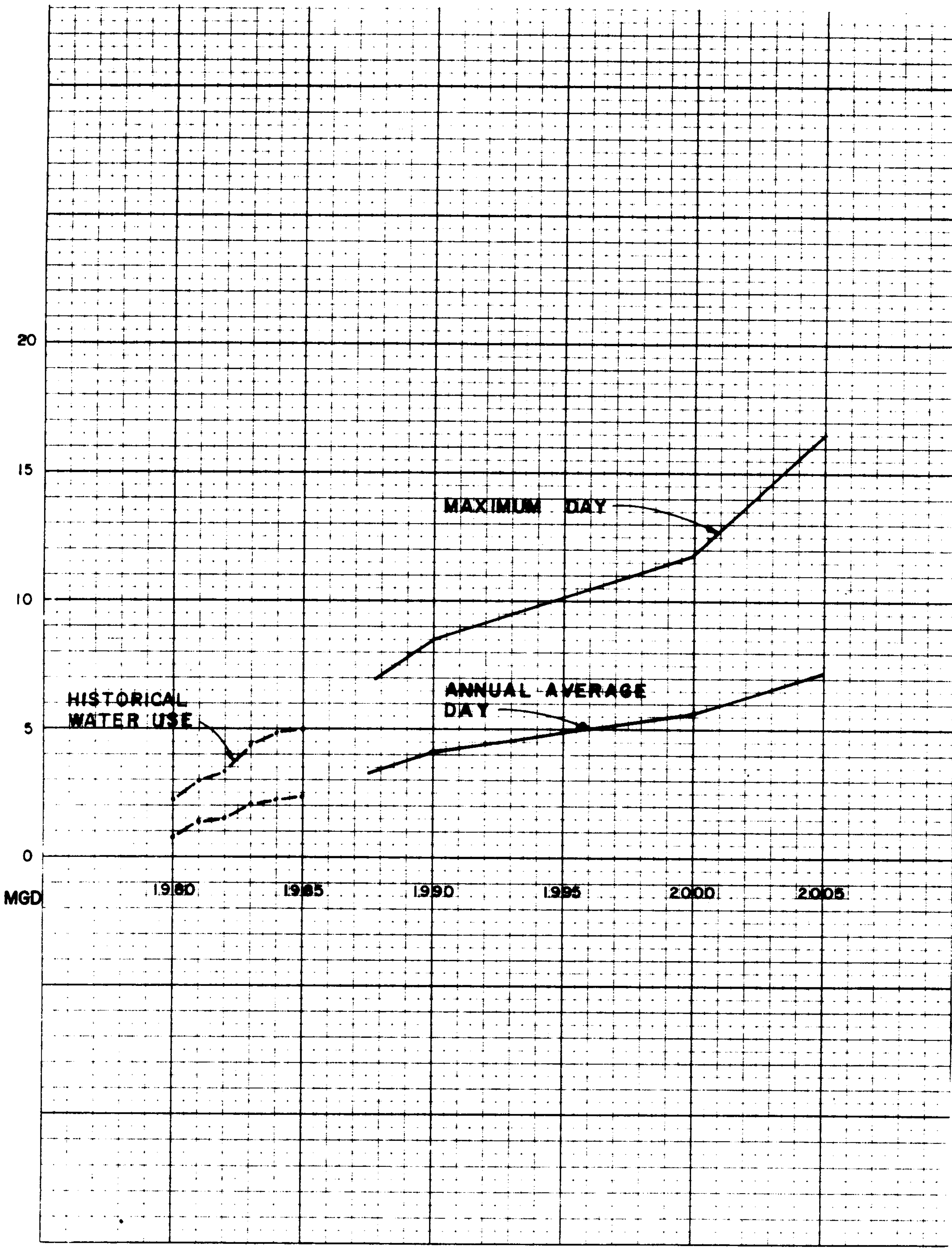

FIGURE 4-2 : PROJECTED WATER USE

As with population projections, water use projections should be updated periodically as new data becomes available. Predictions of future water use are based on many assumptions. Projections by different sources are based on different assumptions, as well as different interpretations of existing data, and can vary greatly. This emphasizes the need to keep projections current.

Tables 4-2 and 4-3 present water use data broken down for the different parts of the service area. This data is presented to facilitate comparison of the projections of this study with any future studies.

TABLE 4-2
PRESENT PEAK SEASONAL WATER DEMAND
DARE COUNTY, NC

Service Area	Peak Deaily Demand (mgd)	Daily Per Capita Demand (gallons)
Nags Head	1.5	111
Kill Devil Hills	1.7	134
Manteo	0.2	102
Dare County	1.1	75
Total	4.5	105*

* Average

TABLE 4-3
WATER DEMAND PROJECTIONS

Service Area	1983	1985	1990	2000	2005
	Existing Service Area (mgd)				
Nags Head (1)	1.5	2.25	3.0	4.5	6.1
Kill Devil Hills (1)	1.7	2.25	3.0	3.8	4.6
Manteo (2)	0.2	0.21	0.24	0.31	0.36
Dare County (2) (Kitty Hawk)	1.1	1.90	2.40	3.2	4.1
Dare County Water System	4.5	6.61	8.64	11.81	15.16
	Potential New Service Area (mgd)				
Roanoke Island		0.22	0.25	.28	0.33
Future Service Dare County Water	---	6.83	8.89	12.09	15.49
*Rodanthe-Waves Salvo		0.40	0.65	.82	1.0

(1) Miller Williams projections to 1990.
(2) MGA projection (105 gpcd).
(3) MGA projection, 1982 Engineering Report.

* Not feasible to serve from Dare County System.

5.0 WATER RESOURCES

5.1 SCOPE

The availability of water resources to supply 15.2 mgd in the study area was examined in this study. A number of alternatives were briefly considered, but not developed in depth. One such alternative is water reuse, which was rejected for both financial reasons and lack of public acceptance. Another alternative is a pipeline to Virginia Beach to receive water from the Lake Gaston pipeline, which was rejected on financial grounds and because of the possibility that this water source may not be available within the time required.

The following water resources were studied in greater depth, and are discussed in the subsections which follow.

Fresh Water

- Outer Banks Surface Water
- Outer Banks Groundwater
- Roanoke Island Groundwater
- Mainland Surface Water
- Mainland Groundwater

Saline Water

- Brackish Surface Water
- Brackish Groundwater

Saline water includes anything from seawater to brackish water requiring treatment by a desalination process. Seawater was not considered a viable water resource due to high capital and O&M costs for treatment.

The analyses have identified the following five alternatives, each of which is discussed in the following subsections.

Alternative	Description
1	Roanoke Island Groundwater
2a	Mainland Groundwater
2b	Mainland Groundwater
3a	Brackish Surface Water
3b	Brackish Groundwater

5.2 OUTER BANKS SURFACE WATER

Fresh Pond is a 27-acre lake located halfway between the Atlantic Ocean and Albemarle Sound and bisected by the boundary between the Town of Nags Head and Kill Devil Hills. This lake served as a water supply source for the two towns until 1980 when the towns entered into a water purchase agreement with the County.

5.2.1 Hydrology

The source of fresh water on the Outer Banks is precipitation. The Nags Head area receives an average annual precipitation of 45 inches. A major portion of the precipitation is lost through evaporation, runoff, and lateral discharge of groundwater to the ocean and the sounds. Only 25 percent of the precipitation is available for percolation to the zone of saturation where it becomes a groundwater supply. The Fresh Pond responds hydraulically in the same manner as a large diameter well. During withdrawal by pumping, a "cone of depression" forms around the lake and behaves as a groundwater divide. The effects of pumping on movement of the divide were investigated by E.O. Floyd of Moore, Gardner & Associates, Inc. in 1979. Floyd found that when water is withdrawn from the pond, the water surface is lowered and groundwater begins to move to the pond from all directions. The gradient difference of the water table surface increases toward the pond with continual withdrawal, and the water table in the vicinity of the pond assumes the shape of an inverted cone with its apex at the pond. Within this cone of depression, groundwater flows toward the pond. The edge of the cone of depression becomes a groundwater divide. The position of the groundwater divide is relatively stationary. This indicates that the water in the zone of influence remains relatively constant throughout the year.

When this water supply was not used for several years, the surface of the lake rose dramatically. This is because evaporation became the only outlet for water in the zone of influence. Pumping during plant operation from the fall of 1985 into summer of 1986 again lowered the water surface, although not as much as in past years when simultaneous withdrawals by both towns lowered the pond level below the minimum necessary for proper operation of the intake. A potential water supply source is still available at Fresh Pond.

5.2.2 Storage

A determination of lake storage at various water surface elevations was made to estimate the volume of water stored by the Fresh Pond. This data was plotted as a stage-storage curve and is shown in Figure 5-1. A tabulation of some data points is shown in Table 5-1. Floyd estimated that during the summer months, pumping imposed 2 to 3 feet of drawdown on the lake in addition to the evaporation losses. The additional drawdown imposed by pumping 1.0 mgd from the lake produced additional flow from the aquifer to the pond. Due to the water storage and transmission properties of the aquifer, sufficent groundwater flow to balance pumping at 1.0 mgd can be obtained from storage within the cone of depression around the pond without causing significant changes in its horizontal dimensions.

5.2.3 Safe Yield

Based on the stage-storage relationship and a monthly water balance, a safe-yield determination was developed for withdrawals from the Fresh Pond. This calculation is for long-term withdrawals and does not consider the water in storage in the pond at the time pumping begins. If the withdrawal is to be of a short duration, the volume of withdrawal could be increased to use the water in storage. Before pumping at the higher yield, the volume of water in storage should be computed. An estimate of pumping duration would be made and the total volume of water required would be determined. By checking the stage-storage curve and the daily safe yield data in Table 5-2, a safe withdrawal rate can be determined. Using this data, 1.0 to 1.5 mgd could be withdrawn from Fresh Pond without adversely affecting the supply, for durations of up to one month, throughout the year. Higher rates could be pumped from January through May and October through December.

TABLE 5-1

STAGE STORAGE DATA AT FRESH POND

STAGE	STORAGE
4 ft.	8.7 MG
5 ft.	17.6 MG
6 ft.	26.8 MG
7 ft.	36.4 MG
8 ft.	46.5 MG
Divide	57.0 MG

TABLE 5-2

SAFE YIELD BY MONTH - FRESH POND

Month	Climatological Data			Fresh Pond Climatological Data			Safe Yield = Pws-Rws-Efpa	
	P ft	R ft	E ft	PWS MG	RWS MG	Efpa MG	Month MG	Day MG
Jan. 1980	0.49	0.02	0.18	52.11	2.13	1.64	48.34	1.56
Feb.	0.26	0.01	0.29	27.65	1.06	2.64	23.95	0.83
Mar.	0.54	0.03	0.40	57.42	3.19	3.64	50.59	1.63
Apr.	0.39	0.02	0.63	41.47	2.13	5.74	33.60	1.12
May	0.37	0.02	0.73	39.35	2.13	6.65	30.58	0.99
June	0.28	0.01	0.85	29.77	1.06	7.74	20.97	0.70
July	0.37	0.02	0.87	39.35	2.13	7.92	29.30	0.95
Aug.	0.21	0.01	0.74	22.33	1.06	6.74	14.53	0.47
Sept.	0.42	0.02	0.57	44.66	2.13	5.19	37.34	1.24
Oct.	0.29	0.01	0.38	30.84	1.06	3.46	26.32	0.85
Nov.	0.33	0.02	0.30	35.09	2.13	2.73	30.41	1.01
Dec.	0.34	0.02	0.28	36.15	2.13	2.55	31.47	1.02
Average	0.36	0.02	0.52	38.02	2.13	4.72	31.45	1.03

Watershed (WS) = 326.19 acres

Fresh Pond Area (FPA) = 27.93 acres

P - Precipitation

R - Runoff

E - Evaporation

Pws - Precipitation on Watershed

Rws - Runoff from Watershed

Efpa - Evaporation from Fresh Pond Area

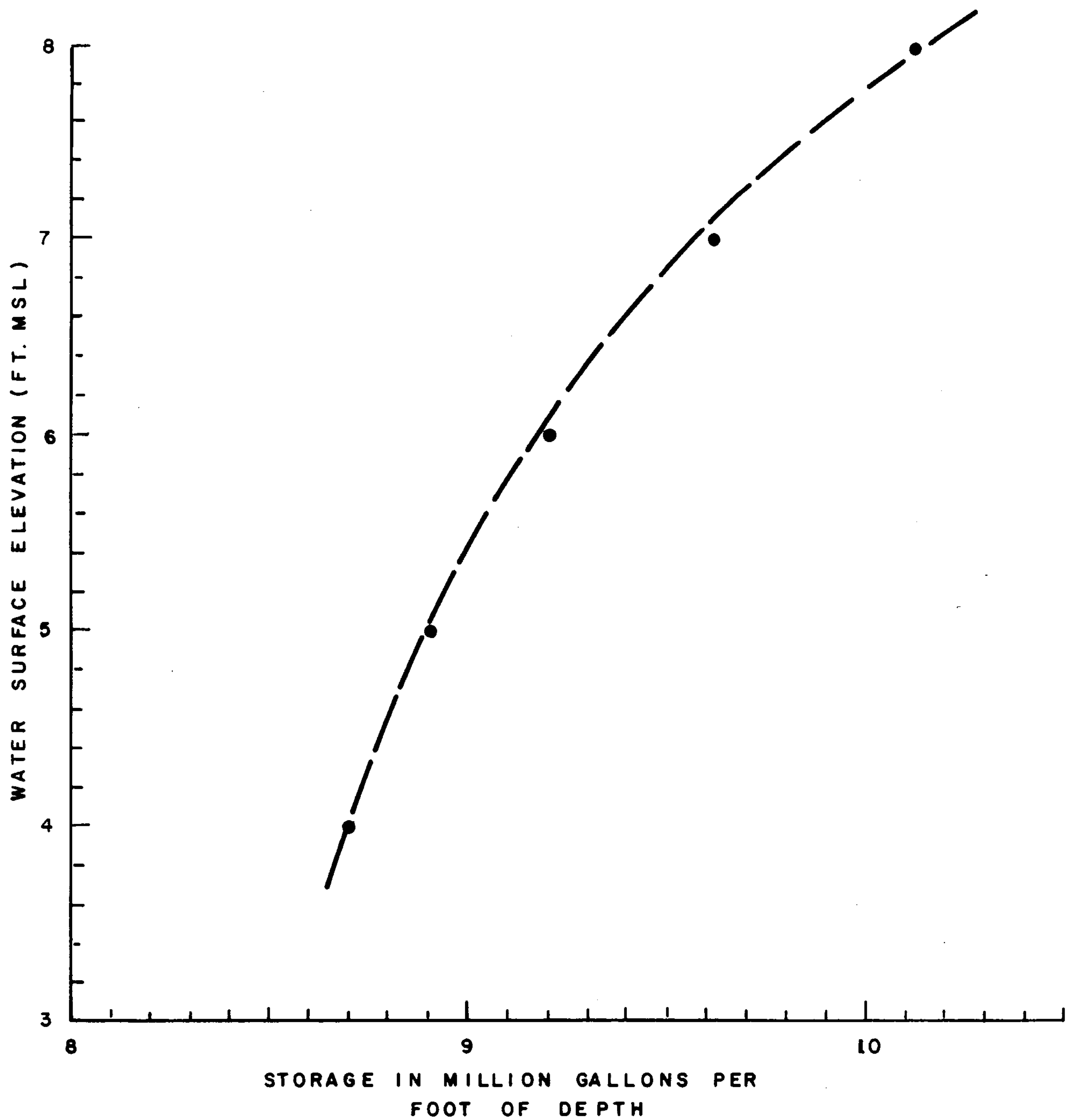

FIGURE 5-1 STAGE – STORAGE RELATIONSHIP AT FRESH POND

5.2.4 Improvements Required For Use

The 1984 MGA Report recommended that the Nags Head Water Treatment Plant at Fresh Pond be upgraded as part of the Phase I improvements, making the plant available for treatment in 1985. The recommended improvements were implemented in 1985, and water production began in November 1985. The improvements were discussed in depth in the 1984 MGA report and are not repeated here. Any further improvements required for this plant are outside the scope of this study.

This plant, hereafter called the Fresh Pond Water Treatment Plant, has a design capacity of 1.5 mgd. The safe yield plus storage of Fresh Pond will not permit operation at this rate for extended periods. On an overall peak season basis, this plant can be run at 0.9 mgd. Plant hydraulics are designed for a maximum rate of 2.25 mgd to meet very short term emergency requirements.

5.3 OUTER BANKS GROUNDWATER

Groundwater in the shallow water table aquifer consists entirely of captured rainfall that is not lost to runoff, evapotranspiration, and discharge to the ocean and sound. As stated elsewhere in this report, only a portion of rainfall, perhaps 25 percent, is actually retained in the aquifer as a potential water supply. Using an average annual rainfall of 45 inches, a well field would have to tap all the water in an area of 5,365 acres to meet the 2005 demand. For actual design, this would have to be increased to account for a 10-year or 20-year low rainfall total. The large quantities of water required rule out the development of Outer Banks' shallow groundwater to meet future water needs.

Deeper Outer Banks groundwater is discussed in Section 5-8.

5.4 ROANOKE ISLAND GROUNDWATER

The groundwater system of Roanoke Island is a complex hydrogeologic feature affected by many physical, meteorologic, and oceanographic factors. The 1984 MGA Report presented considerable detail about the overall characteristics of three aquifers underlying Roanoke Island. The major study work was done by the United States Geological Survey and North Carolina Department of Natural Resources and Community Development. Their work was for the purpose of locating major freshwater supplies and identifying overall aquifer hydraulic characteristics. No area was studied in suitable detail for well field design. Thus, no analysis was made of the cone of depression or the potential for saltwater intrusion in the existing well field or in potential future well fields.

Additional data gathered by the North Carolina Division of Environmental Management has identified the following trends for Roanoke Island groundwater.

(1) Salt water intrusion is occurring in shallow wells in the southwestern end of the Island, possibly as a result of the County's pumping.

(2) Shallow wells in Wanchese have gone dry, possibly as a result of the County's pumping.

A cause and effect relationship between the County's pumping and recent trends in Wanchese can be established only by a study involving analysis of historical data and construction of monitoring wells. DEM is not making available the data upon which the above statements are made, so it is not possible to state whether the data establishes the cause and effect. However, the DEM has indicated that they will probably oppose any attempt to expand the Roanoke Island well field.

A recommendation to expand the development of the Roanoke Island water supply cannot be made unless additional study is conducted. This would involve well drilling and pump testing in new areas, and monitoring area wide drawdown and saltwater intrusion through a monitoring well program. Such a study may indicate that additional water can be safely withdrawn, but may also indicate that the aquifer should not be further developed. Because of the uncertainty, and DEM's probable objections, it is recommended that there be no further development of the existing Roanoke Island water supply. Figure 5-2 shows the required facilities and the opinion of probable cost for expanding the well field and treatment plant as required in Stage 2. This possible water supply is called Alternative 1. Again, expansion of this well field should be done only if further study can verify its feasibility.

Additional details of the hydrogeologic characteristics of groundwater under Roanoke Island are in Appendix A.

5.5 MAINLAND SURFACE WATER

Surface water exists on the Dare County mainland in East Lake, South Lake, and in various canals and natural streams. Water samples taken in several locations in East Lake and South Lake, the results of which are shown in Table 5-3, have revealed that the water is brackish, and that, as a minimum, conventional treatment (coagulation, sedimentation, and filtration) plus desalination would be required. The canals and streams are all slow moving and affected by tides and, in all probability, brackish. The entire area is subject to saltwater contamination by flooding as a result of storms. The high cost of treatment and transmission through a submarine pipeline make this alternative unfeasible and not worthwhile of further study.

5.6 MAINLAND GROUNDWATER

The mainland of Dare County has greater potential than Roanoke Island for groundwater development due to its larger land area. Development of mainland groundwater is an alternative to further expansion of the groundwater supply on Roanoke Island.

5.6.1 Hydrogeologic Framework

The hydrogeology of the mainland of Dare County is very similar to that of Roanoke Island. The stratigraphic sequence along the eastern side of the mainland correlates directly to the sequence on Roanoke Island. The principal water bearing zones present on Roanoke Island are also found near the Manns Harbor area.

TABLE 5-3

MAINLAND SURFACE WATER ANALYSES

SOUTH LAKE:*	Head Near Surface	Middle Surface	2′ Deep
Chloride	1,280	1,380	1,355
Calcium Hardness	64	80	80
Magnesium Hardness	278	258	254
Total Hardness	342	338	334
Nitrate	0.143	0.094	0.111
pH (units)	6.9	7.0	7.0
Total Dissolved Solids	2,732	2,887	2,928
Turbidity (NTU)	2.1	2.0	2.0
Iron	0.395	0.284	0.324
EAST LAKE:			
Chloride	1,490	1,555	1,570
Calcium Hardness	84	92	92
Magnesium Hardness	266	296	246
Total Hardness	350	388	338
Nitrate	0.085	0.126	0.116
pH (units)	6.8	6.9	7.0
Total Dissolved Solids	3,154	3,304	3,300
Turbidity (NTU)	2.0	1.85	2.1
Iron	0.190	0.145	0.166

* All are mg/l unless otherwise noted

DARE COUNTY, NORTH CAROLINA

WATER SUPPLY ALTERNATIVES STUDY - 1985

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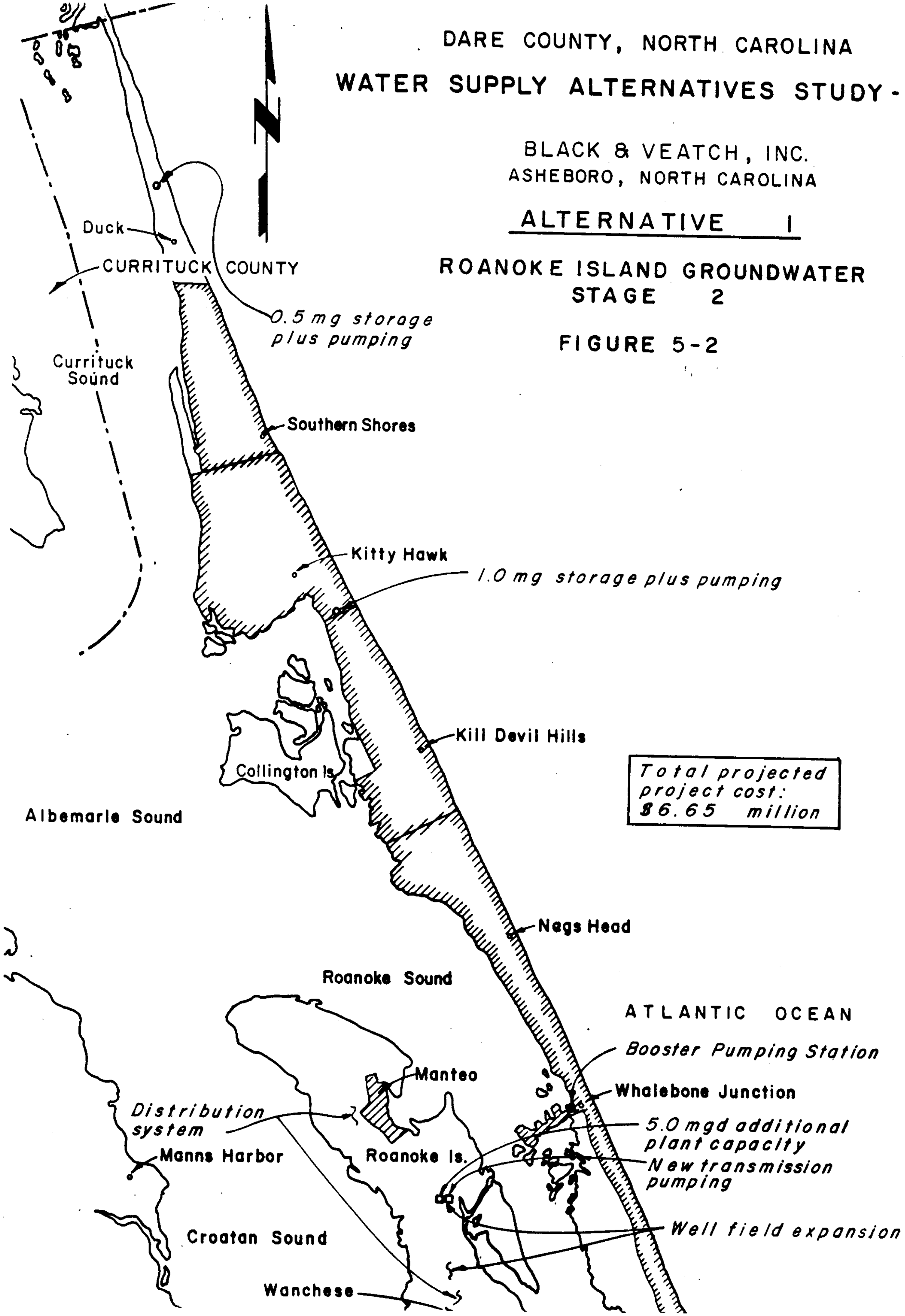
ALTERNATIVE I

ROANOKE ISLAND GROUNDWATER
STAGE 2

FIGURE 5-2



5.6.2 Safe Yield

Test well drilling and pumping tests conducted near Manns Harbor by the North Carolina Division of Environmental Management indicate that ample groundwater supplies exist. The test wells were drilled to depths of about 500 feet and encountered the principal aquifer at depths ranging from 50 to 100 feet. The average thickness of the unit is 75 feet. The transmissivity of the aquifer at this location is 53,000 gal/day/ft with a storage coefficient of 1.03×10^{-2}. The specific capacity of test wells varies from 4.58 gal/min/ft to 7.21 gal/min/ft of drawdown. This data is based on a 4-inch diameter test well screened between 140 and 150 feet. A 10-inch diameter production well located near Manns Harbor, which is screened in the principal aquifer, should produce 500 to 800 gallons per minute. Wells should be spaced at least 2,000 feet apart to avoid pumping interference.

5.6.3 Water Quality

The water quality of the principal aquifer at Manns Harbor is similar to the water which Dare County is recovering on Roanoke Island. Table 5-4 gives the results of water quality testing conducted by the Water Resources Division of the U.S. Geological Survey on the water recovered from the principal aquifer at the test well sites. This analysis indicates that the groundwater is high in hardness, but contains little iron, manganese, or chlorides.

5.6.4 Facilities

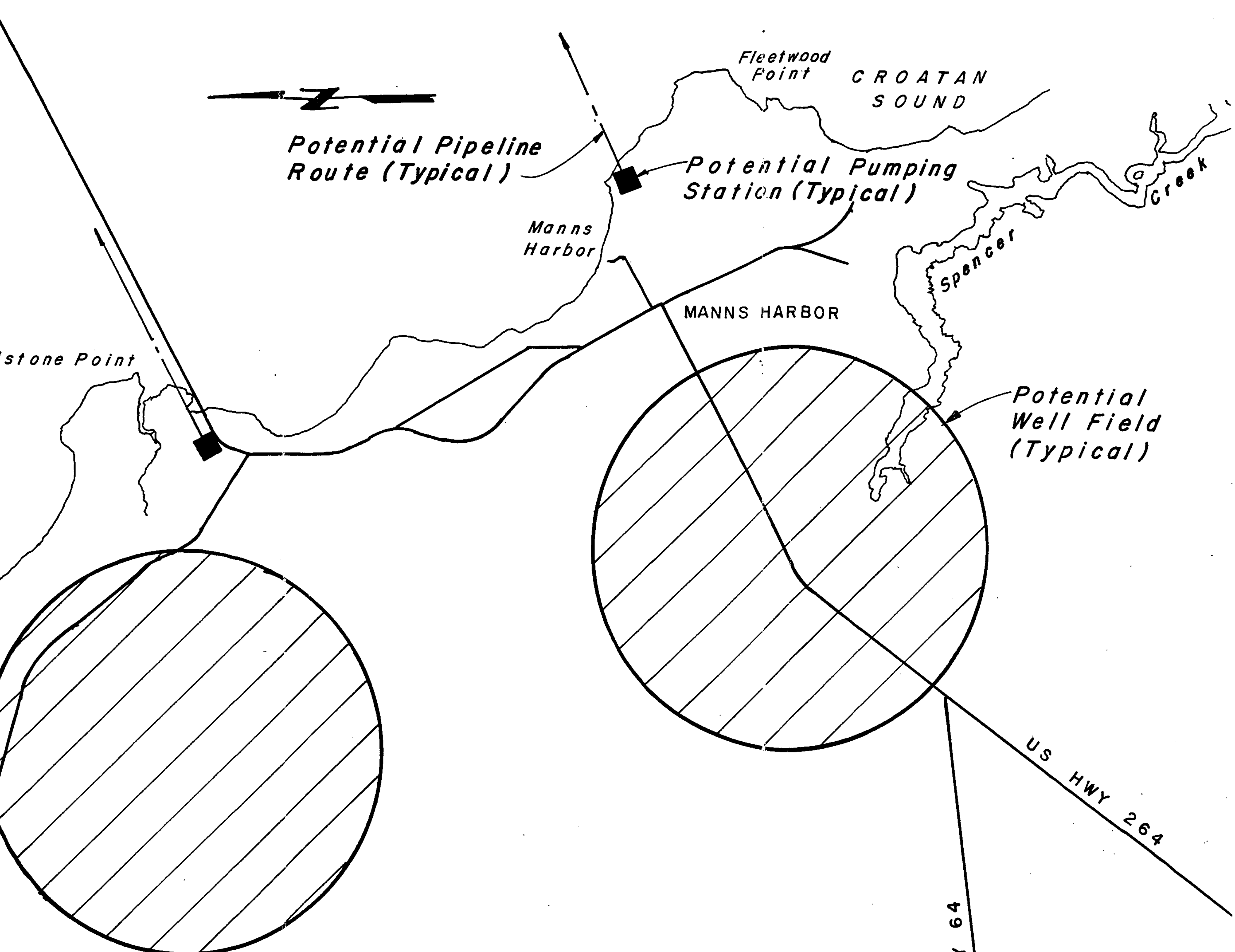
Development of mainland groundwater into a viable supply will require construction of wells, raw water lines, a submarine pipeline, water treatment capacity, and pumping and transmission facilities. Potential well field sites are shown on Figure 5-3. At least seven new wells, producing 500 gpm, each would be required for a 5 mgd system. Water treatment could be provided through expansion of the existing Roanoke Island water treatment plant or by a new water treatment plant constructed at a suitable location. Figures 5-4 and 5-5 show facilities required to implement a mainland groundwater supply for different well field locations, pipeline routes, and treatment plant additions.

TABLE 5-4

MAINLAND GROUNDWATER ANALYSIS

(All values mg/l except as noted)

Silica	24
Aluminum	0.165
Iron	0.096
Manganese	ND
Calcium	68
Magnesium	7.8
Sodium	66
Potassium	8.5
Carbonate	ND
Bicarbonate	398
Sulfate	1.6
Chloride	29
Fluoride	0.2
Nitrate	0.2
Phosphate	ND
Total Dissolved Ss	405
Hardness (as $CaCO_3$)	202
pH (units)	7.7
Color (units)	10
Alkalinity (as $CaCO_3$)	326
Specific Conductivity (umohs)	650


Potential Pipeline Route (Typical)
Potential Pumping Station (Typical)
Fleetwood Point
CROATAN SOUND
Manns Harbor
MANNS HARBOR
Spencer Creek
Potential Well Field (Typical)
US HWY 264
64
stone Point

DARE COUNTY, NORTH CAROLINA

WATER SUPPLY ALTERNATIVES STUDY - 1985

BLACK & VEATCH, INC.
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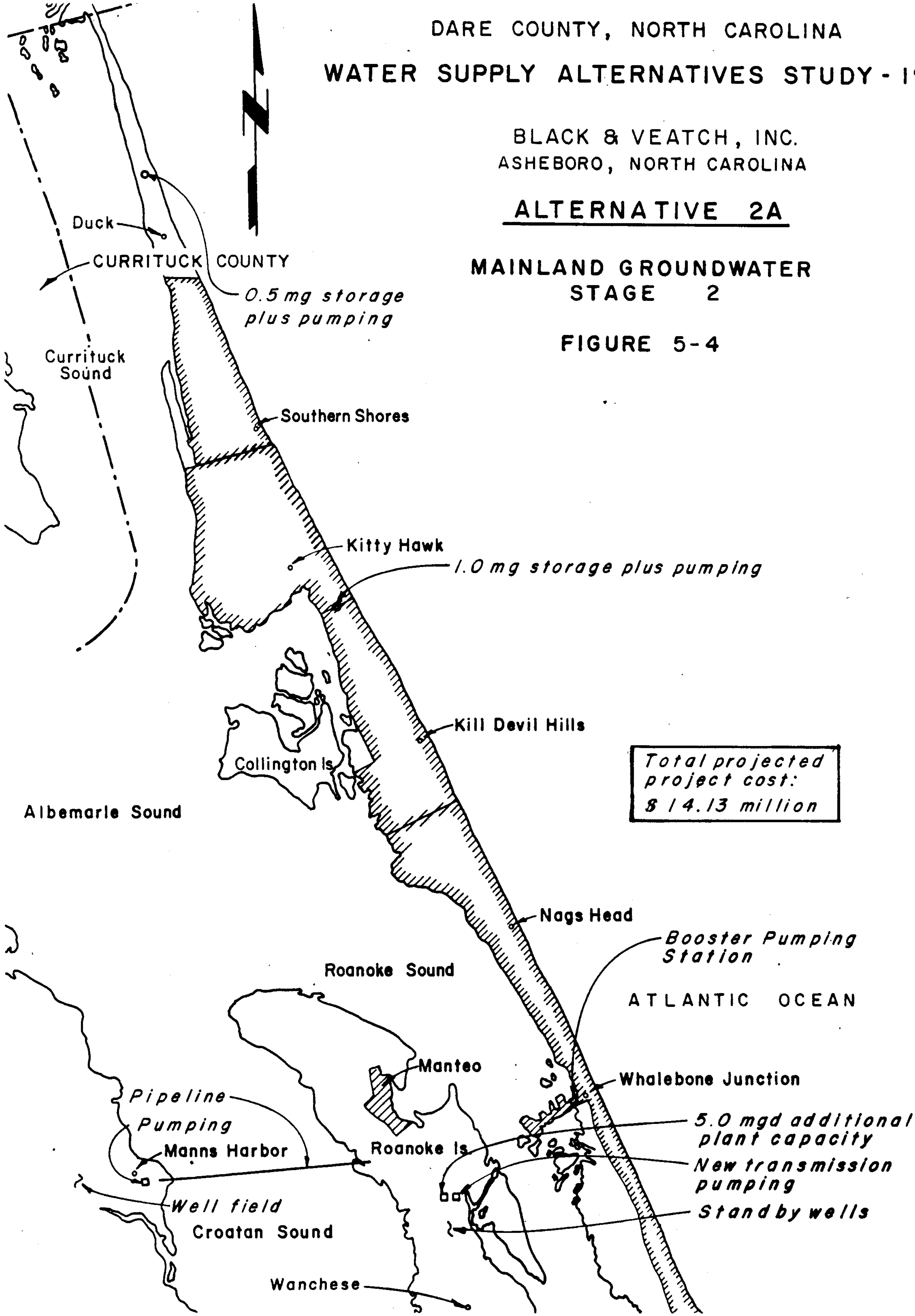
ALTERNATIVE 2A

MAINLAND GROUNDWATER
STAGE 2

FIGURE 5-4



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WATER SUPPLY ALTERNATIVES STUDY - 198

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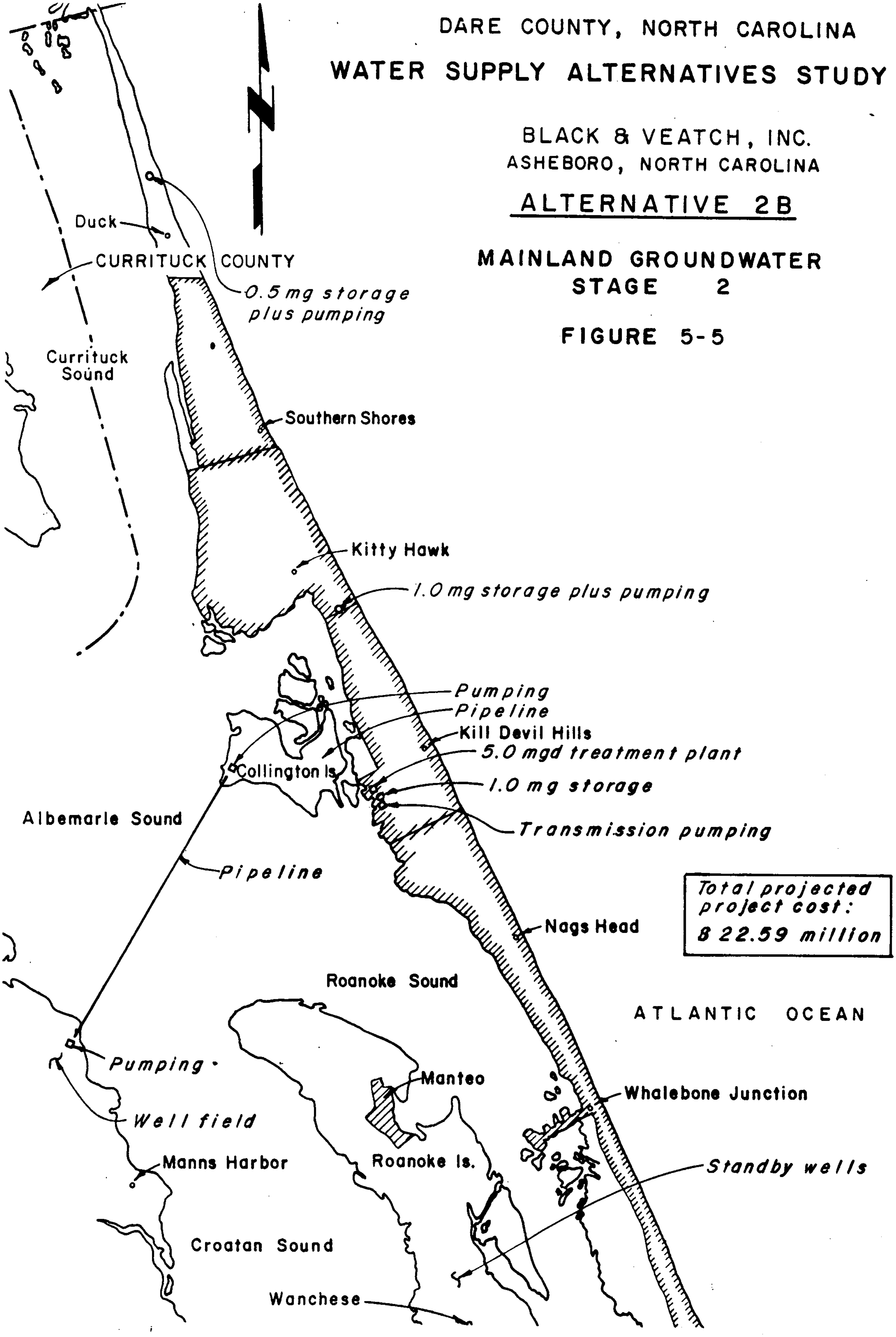
ALTERNATIVE 2B

MAINLAND GROUNDWATER
STAGE 2

FIGURE 5-5



5.7 BRACKISH SURFACE WATER

The sounds and bays between the Outer Banks and the mainland are a potential source of brackish water. The salinity of the surface water varies from 20,000 mg/l TDS in Pamlico Sound near the Hatteras Inlet to as low as 1,000 mg/l TDS in Currituck Sound near the boundary with Currituck County. Salinity at any location in the sounds will vary depending on wind, rainfall, and tides. Figures 5-6a and 5-6b show the potential difference in salinity based on historical data. These values should not be considered the absolute minimum and maximum values possible, but rather as the normal ranges of variation. Samples drawn from the southern end of Currituck Sound and analyzed as part of this study had a TDS of close to 7,000 mg/l, which is outside of the range shown in Figures 5-6a and 5-6b.

The feasibility of using brackish water from the sounds as a water supply depends upon the potential yield or withdrawal rates, the ability to treat and the cost of treatment, and the reliability of the supply. Before water from the sound could be considered potable, some desalination process must be used in conjunction with any required pre- and post-treatment. The high cost of desalination requires that sources be developed near the service area on the Outer Banks, to reduce transmission costs, in order to be potentially feasible. This limits the areas of study to northern Roanoke Sound; Albemarle Sound and its smaller bays near Nags Head, Kill Devil Hills, and Kitty Hawk; and Currituck Sound near Southern Shores and the Duck community.

5.7.1 Quality

No major studies are known to have been performed to definitively determine the quality of water in the sounds. The only available published data pertains to salinity, and is summarized in Figures 5-6a and 5-6b. This data was gathered over many years, but has not been updated recently, and does not concentrate on the areas under study. Before water from any of the sounds mentioned above could be utilized, a water quality study in sufficient detail to develop data for water treatment plant design should be undertaken.

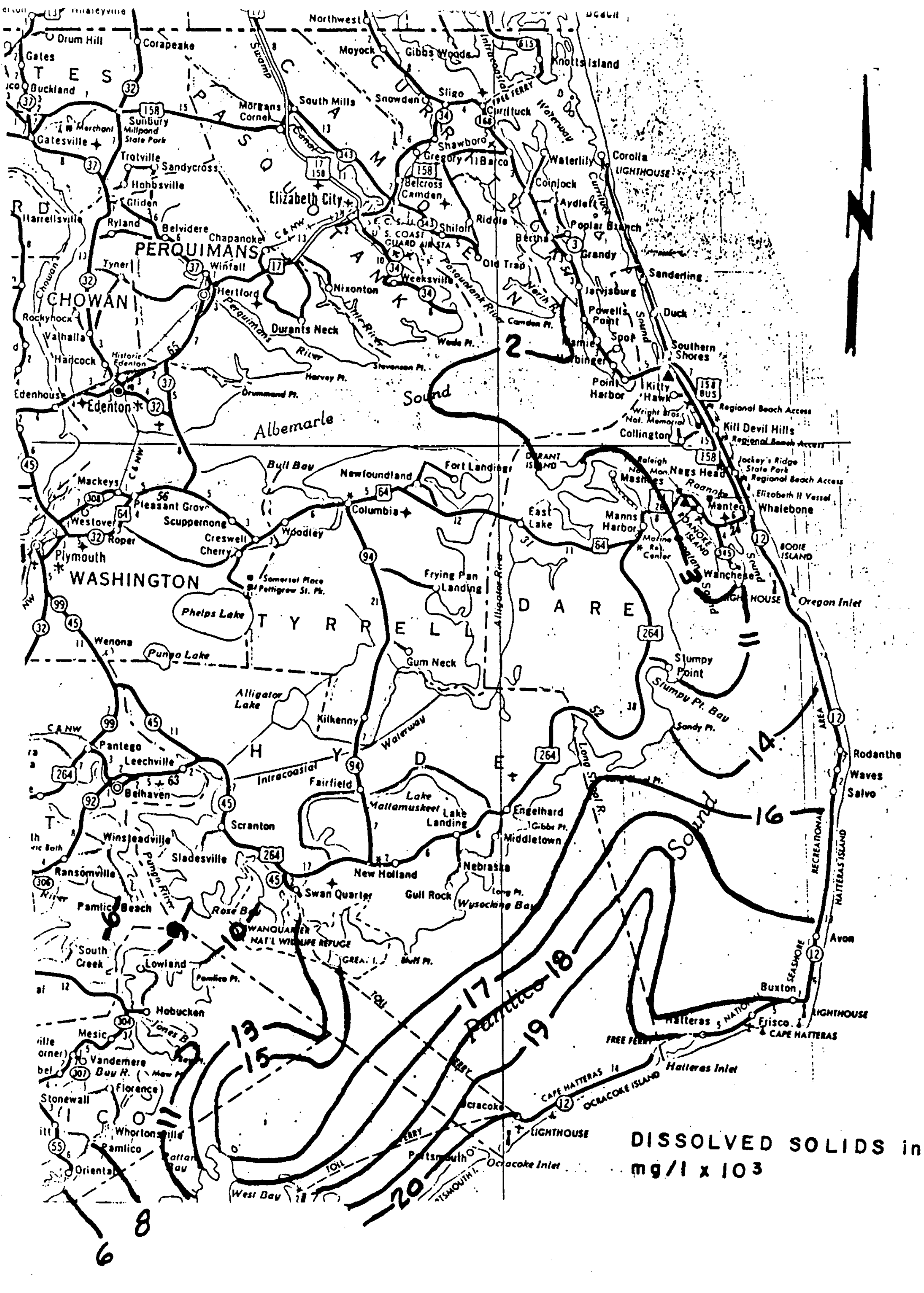


FIGURE 5-6a SALINITY OF SOUNDS
AVERAGE APRIL VALUES

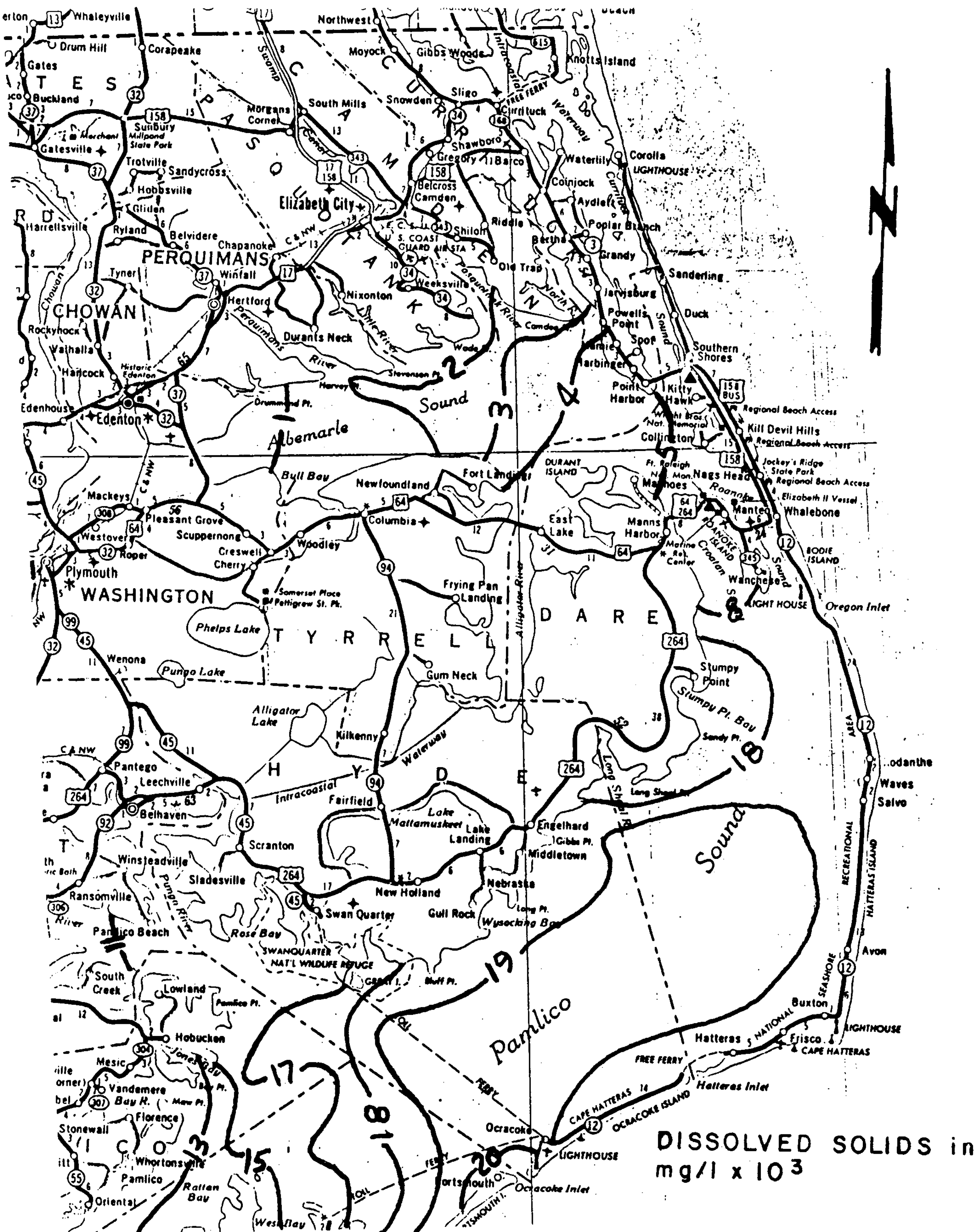


FIGURE 5-6b SALINITY OF SOUNDS
AVERAGE DECEMBER VALUES

In order to obtain a general understanding of the water quality, samples were taken at the southern end of Currituck Sound near the Wright Memorial Bridge. The samples cannot be considered fully representative of water in the area sampled, but will give some indication of the feasibility of treatment. Results of the sample analysis are shown in Table 5-5. As stated above, the salinity is outside of the published range for this location. Since the samples were taken after a prolonged period of little or no rainfall in the contributing watershed, it is likely that saltwater was beginning to move further upstream and that this represents a much higher than average value for TDS. Treating water with this level of TDS is well within the capability of brackish water desalination equipment.

Any surface water will require extensive pretreatment before treatment by reverse osmosis or other desalination processes. Coagulation, flocculation, sedimentation, and filtration will be required for removal of suspended solids and turbidity. Chlorination for disinfection and, potentially, dechlorination for desalination equipment protection will be required. The potential for organics formation should be evaluated, as should the presence of iron and its potential for fouling membrane processes. The filtration process should be conservatively designed to minimize the potential for suspended solids carryover onto the desalination equipment. Therefore, a full surface water treatment plant constructed of materials resistant to saltwater corrosion should precede the desalination process.

The variable quality and temperature of sound water must be considered in plant design. The plant must be sized for the highest predicted TDS level. Plant performance may be substantially above design levels during periods of low salinity. Water temperature greatly affects both RO or ED equipment, with both treatment and capacity being improved at higher temperatures. The low temperatures of the sound water in winter will reduce treatment capacity, although the lower water demands in the winter will offset this decrease. These variations in water quality can be taken into account in plant design, but they make plant operation more difficult. A greater change in sound water quality would occur if major storms opened a new inlet in the Outer Banks which, although improbable, is not impossible. A new inlet could increase the sound water TDS concentration close to that of ocean water.

TABLE 5-5

CURRITUCK SOUND WATER ANALYSIS

Parameters	Raw Untreated Water	Treated Sound Water*
Alkalinity (mg/l)		30.0
Chloride (mg/l)	3547	3380
Fluoride (mg/l)		1.64
Hardness, Calcium (mg/l)		206
Hardness, Total (mg/l)		820
pH (units)		6.6
Total Dissolved Solids (mg/l)	7429	6995
Sulfate (mg/l)		520
Sulfide (mg/l)		<.1
Turbidity (NTU)		.43
Calcium (mg/l)		80.9
Iron (mg/l)		0.037
Potassium (mg/l)		68.1
Magnesium (mg/l)		203
Manganese (mg/l)		0.011
Sodium (mg/l)		1628
Silicon (mg/l)		0.35

* Treated with 0.35 mg/l alum, 6 mg/l Magnifloc, and filtered in 2 ft sand.

A survey of large (at least 1.0 mgd) desalination plants using membrane separation processes in the United States and in the Middle East revealed no plants using surface water as the source of raw water. Avoidance of surface waters is due to the potential for major damage to desalting equipment if any failure occurs in the pretreatment. This historic lack of utilization of surface waters for desalination does not absolutely preclude their use, but it does signal caution and suggests that groundwater is a preferable source, if available.

5.7.2 Quantity

No data is available concerning the amount of water that could be withdrawn from any of the sounds. A brief consideration of the geography and interrelationships of all the sounds suggests that a large quantity of water could be withdrawn with a minimum impact on the sound as a whole. For example, the estimated average volume of Currituck Sound is 563 billion gallons. An average day withdrawal of 6 mgd (one half of a plant supplying a 12 mgd maximum day) would annually remove 2.2 billion gallons, or 0.4 percent of the total volume of Currituck Sound. The volume of the Albemarle/Roanoke Sound system is much greater. It appears that the quantity of water available is more than adequate to meet Dare County's needs within the study period and well beyond.

It is not known what effect withdrawals would have on water quality. This can only be determined by an oceanographic study, which is outside the scope of this study. Due to the expense of undertaking such a study, it is recommended that this be pursued only if it is fairly certain that this water source would be tapped.

5.7.3 Facilities Required

The following facilities would be required to develop one of the sounds as a water supply. These facilities are shown on Figure 5-7 and probable project costs are shown in Table 5-6. Stage 2 includes a 5.0 mgd desalination plant, required by 1989. Stage 3 includes expanding this plant to 12.0 mgd.

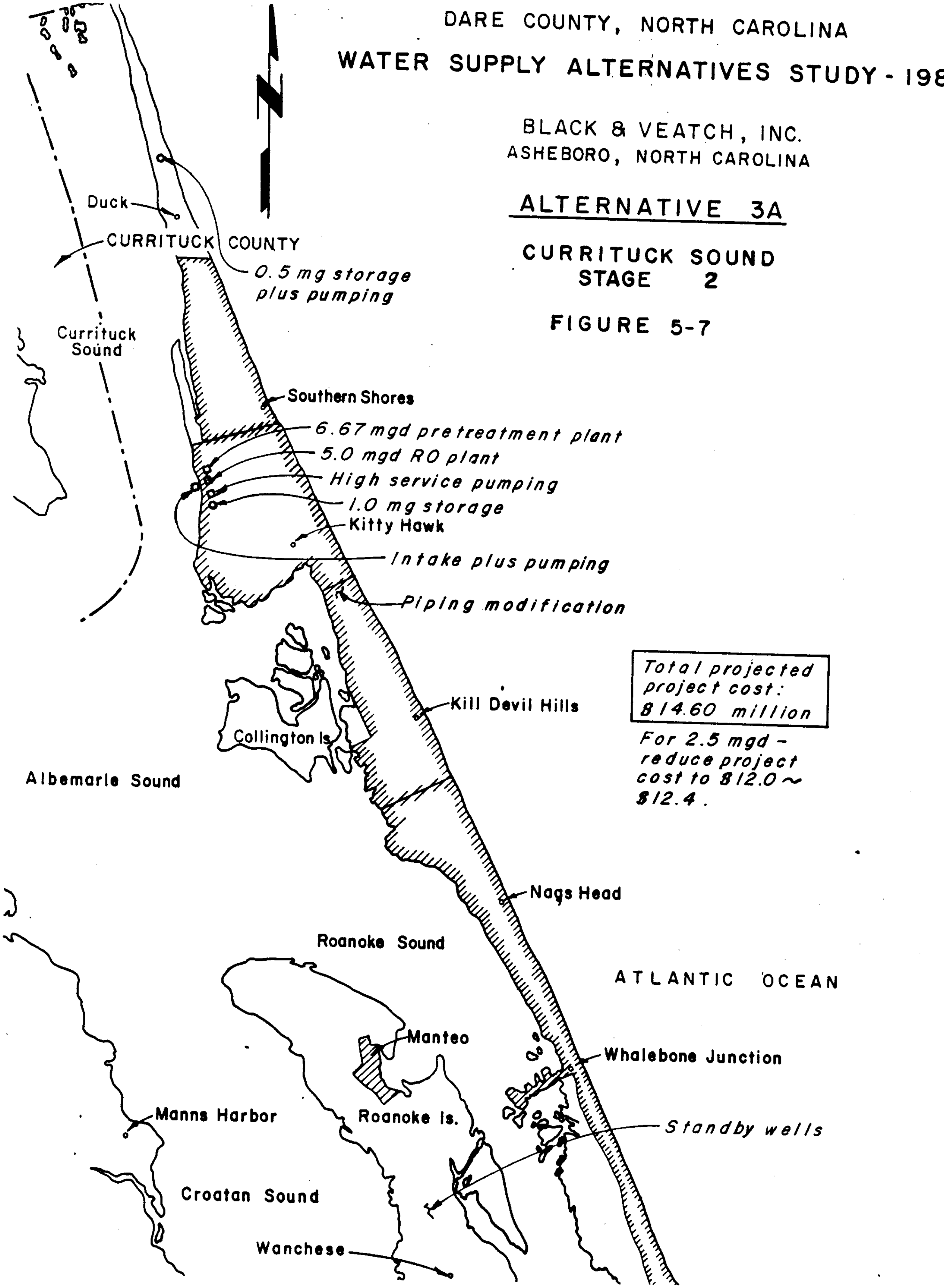
TABLE 5-6

OPINION OF PROBABLE COST

ALTERNATIVE 3A

SURFACE WATER DESALINATION

	Stage 2 ($ x 1,000,000)	Stage 3 ($ x 1,000,000)
Inake Structure and Raw Water Pumping	0.88	0.10
Pretreatment Plant	6.67	5.50
Desalination Plant	5.0	3.86
High Service Pumping	0.25	0.20
5 mg Storage at Treatment Plant	1.30	1.30
Standby Wells on Roanoke Island	0.50	--
	$14.60	$10.96


DARE COUNTY, NORTH CAROLINA
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BLACK & VEATCH, INC.
ASHEBORO, NORTH CAROLINA
ALTERNATIVE 3A
CURRITUCK SOUND
STAGE 2
FIGURE 5-7
Duck
CURRITUCK COUNTY
0.5 mg storage plus pumping
Currituck Sound
Southern Shores
6.67 mgd pretreatment plant
5.0 mgd RO plant
High service pumping
1.0 mg storage
Kitty Hawk
Intake plus pumping
Piping modification
Total projected project cost: $14.60 million
For 2.5 mgd - reduce project cost to $12.0 ~ $12.4.
Kill Devil Hills
Collington Is.
Albemarle Sound
Nags Head
Roanoke Sound
ATLANTIC OCEAN
Manteo
Whalebone Junction
Manns Harbor
Roanoke Is.
Standby wells
Croatan Sound
Wanchese

5.7.3.1 Intake Structure. An intake structure of at least twice the capacity of the planned treatment plant should be located where water depth at low tide is suitable for the intended withdrawal. Relatively low head pumps can be used if the treatment plant is nearby. Inert materials of construction should be used for all wetted parts including pumps, piping, and structure. For a 12 mgd water treatment plant, a 24 mgd intake structure will have a probable cost as shown in Table 5-6.

5.7.3.2 Raw Water Transmission Main. If the potential site in Kill Devil Hills is utilized, a pipeline of approximately one mile long will be required. The pipe should be of inert material, either PVC, fiberglass, or stainless steel, and should be a minimum of 36-inches in diameter. Probable cost is given in Table 5-6.

5.7.3.3 Pretreatment. As discussed in paragraph 5.7.3, a pretreatment plant consisting of coagulation, flocculation, sedimentation, and filtration will be required. The design should be conservative to prevent suspended solids breakthrough. Disinfection should be compatible with the selected desalination equipment. For a 12 mgd desalination process, an 18.5 mgd pretreatment plant is needed to allow a recovery rate as low as 65 percent in the desalination process. An initial stage of 9.25 mgd, constructed of appropriate materials, will have a probable cost as shown in Table 5-6.

5.7.3.4 Desalination. Available desalination processes are covered in greater depth in Section 7.0 of this report. The least costly option for water treatment is reverse osmosis in a spiral wound configuration. Pretreatment, in addition to that described above would likely consist of pH adjustment, chemical addition for prevention of scale formation, and 5-micron cartridge filtration. The TDS of the feed water is high enough that the required operating pressure will be in excess of 500 psi, which eliminates the use of low pressure membranes. Quotations were solicited from six RO system suppliers for the feedwater described in Table 5-3, and the probable cost based on these quotations is given in Table 5-6. This cost includes building space to house the process equipment.

5.7.3.5 Storage. Clearwell, finished water, and pretreated water storage will be required. The costs given in Table 5-6 include a nominal clearwell volume, 5 million gallons of finished water storage, and 5 million gallons of pretreated water storage. This latter is to provide a buffer against inadequate pretreatment.

5.7.3.6 Support Systems. The new treatment plant would become a center of operations for the Dare County Water Department, with offices, meeting rooms, laboratory, and employee facilities. The costs of these facilities is highly variable, depending upon several factors. Table 5-6 includes costs for such facilities in other typical plants.

5.7.3.7 Transmission Pumping and Piping. Locating the plant at the Kill Devil Hills site offers reduced costs for transmission pumping, and eliminates the need for new transmission piping. The probable cost for the Kill Devil Hills site is given in Table 5-6.

5.7.4 Reliability

The chief disadvantage of a brackish surface water supply is the reliability of the system. The variations in raw water quality, the potential for major water quality changes due to changes in Outer Banks geography, and the potential of damage to the RO process equipment contribute to the lack of reliability which must be included in planning and design. The operation of the plant becomes much more critical, and a high degree of operator expertise is the final protection against equipment damage. Increased reliability can be achieved by providing redundancy in the pretreatment plant, buffering storage capacity, and conservative design of the RO process, all of which will increase the system cost.

5.8 BRACKISH GROUNDWATER

Section 5.3 provided a general description of the groundwater that is present on the Outer Banks, and discussed potential development of fresh groundwater resources. This section describes brackish groundwater resources on the Outer Banks and their development.

Very little data has been available describing the brackish water aquifers underlying Dare County. Past drilling has been for the purpose of locating fresh water, and when brackish water was encountered, no attempt was made to identify it as a potential water resource. Previous deep well drilling in the area includes two permanent monitoring wells on the mainland, drilled to depths of approximately 1,000 feet and 1,200 feet and maintained by the State of North Carolina, and a test hole on the outer banks of Currituck County, drilled to a depth of 980 feet and abandoned when no fresh water was found. These borings give a general idea of the geological formations, but they provide no information that could be used in the planning or design of a deep brackish groundwater resource.

To overcome this lack of data, Dare County undertook a drilling program with the following objectives.

(1) Determine the location, thickness, and characteristics of geological strata.

(2) Locate usable aquifers.

(3) Sample and analyze each aquifer for data relative to desalination plant design.

(4) Pump test wells to determine aquifer characteristics.

(5) Conduct extensive pump testing to determine the potential for hydraulic connections between aquifers and for saltwater intrusion.

Two wells were drilled in the locations shown in Figure 5-8. The locations were chosen to make use of available property close to potential treatment plant sites. Well No. 1 in Kill Devil Hills was drilled to a depth of 1,610 feet, and water samples were taken from three aquifers. Well No. 2 in Southern Shores was drilled to a depth of 700 feet without encountering any significant water bearing strata and was abandoned after geophysical logging. Additional description of procedures used and information gained in the test well drilling program is included in Appendix B.

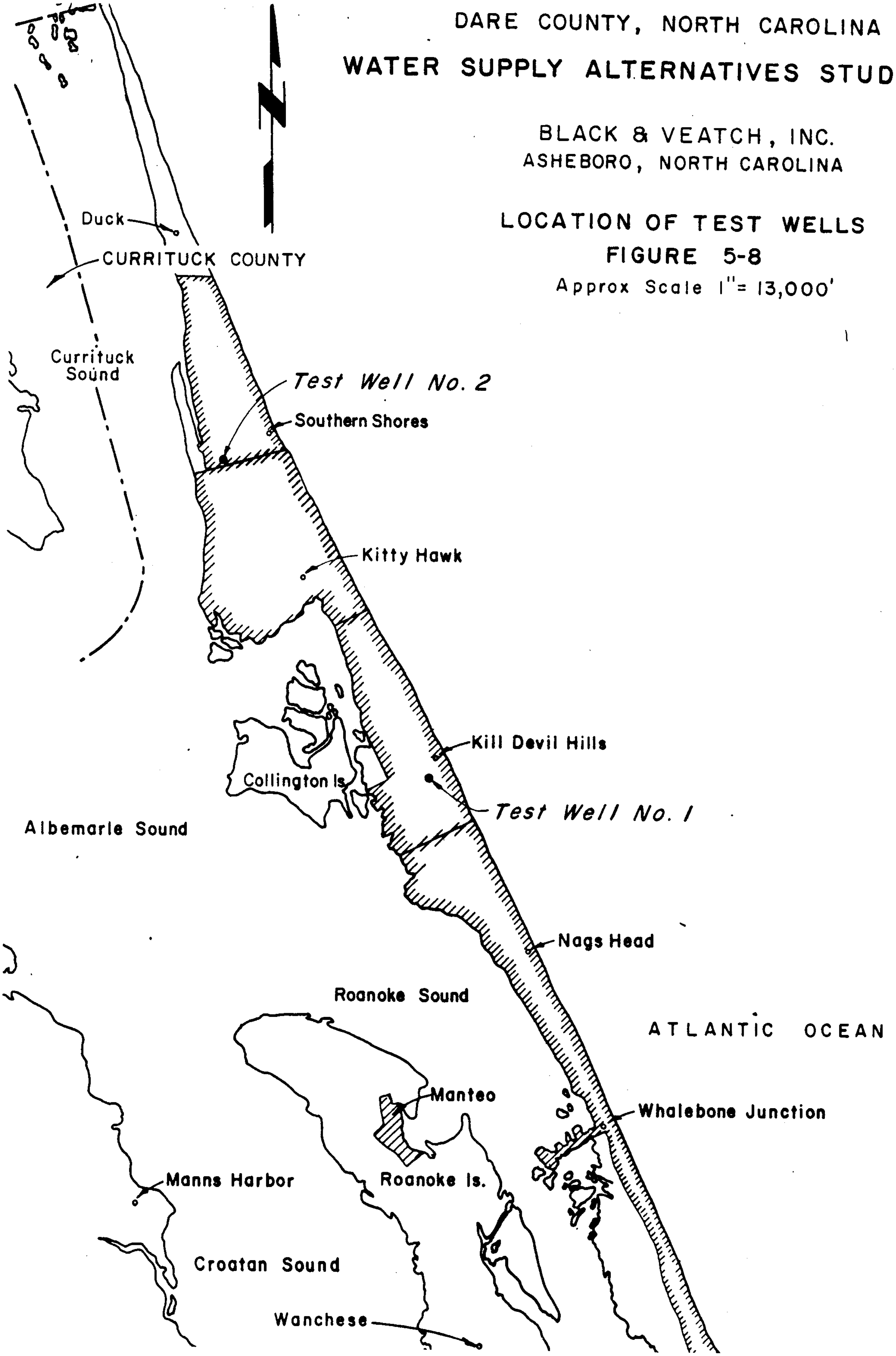
The sections which follow describe significant findings concerning the brackish aquifers.

DARE COUNTY, NORTH CAROLINA

WATER SUPPLY ALTERNATIVES STUDY - 198

BLACK & VEATCH, INC.
ASHEBORO, NORTH CAROLINA

LOCATION OF TEST WELLS
FIGURE 5-8
Approx Scale 1" = 13,000'



5.8.1 Deep Aquifer

An aquifer was located at a depth between 1,160 feet and 1,400 feet. This is the Claiborne formation, commonly called the Castle-Hayne aquifer. It consists of highly fractured, confined limestone and extends over much of eastern North Carolina. It is a fresh water aquifer in some locations. The well drillers estimate a potential well yield in excess of 1,000 gpm. Water samples were taken at 1,200 feet, 1,300 feet and, due to driller's error, at 1,157 feet. Table 5-7 shows the results of these analyses. Although the Castle-Hayne aquifer contains fresh water in some locations in North Carolina, at this test well it is seawater, having TDS and specific ion concentrations similar to ocean water. High yield would reduce the number of wells required and the aquifer confinement will reduce pumping costs; however, development of this aquifer is not feasible at present due to the high cost of converting salt water to potable water. This could be a source of water in the future if brackish water resources are unavailable or if technological advancements reduce the cost of seawater desalination.

5.8.2 Intermediate Aquifer

A confined aquifer consisting mainly of sand was located at a depth between 280 feet and 660 feet. This is the Yorktown aquifer. Water samples were taken at 330 feet and 660 feet. Table 5-8 shows the results of these analyses. The aquifer is stratified, being slightly brackish at the top and somewhat salty at the bottom. This aquifer was further studied as follows. A screen was set from an elevation of 330 feet to 480 feet, and the well pumped at 450 gpm, less than one-half of the driller's estimate of well capacity. The pump tests conducted were step drawdown, 24-hour pumping and recovery, and a long-duration pumping test. This latter test was performed to assess the potential for salt water intrusion and for hydrologic connections between aquifers. The pumping rate and screen setting depth were selected to attempt to draw from the top part of the aquifer only and limit the TDS of the potential supply to 5,000 mg/l or less. The long-duration pump test was conducted for a total of 40 days. Details of the results of this subsurface study are in Appendix B.

TABLE 5-7

CASTLE HAYNE AQUIFER ANALYSIS

PARAMETERS (mg/l unless stated otherwise)	1,300 FEET DEEP B&V	1,300 FEET DEEP BF GOODRICH	1,200 FEET DEEP B&V	1,157 FEET DEEP B&V
Alkalinity	446	442	517	498
Alumimum	<1.0	<DL	<1.0	---
Barium	<0.5	0.14	<0.5	---
BOD(5)	<1.5	---	<1.5	---
Boron	---	7.36	---	---
Calcium	505	455	230	---
*Carbon Dioxide	---	---	---	---
Chloride	21,950	21,034	20,700	15,200
Chlorine	<0.1	---	<0.1	---
Chromuim	0.04	<DL	0.03	---
Color (PCU)	1	---	5	5
Conductivity ($umhos/cm^2$)	35,500	38,200	42,000	---
Copper	0.053	<DL	0.38	---
Flouride	1.34	<DL	1.07	---
Hardness	4,660	4,957	4,240	3,340
Iron	1.48	1.1	0.46	16.7
Lead	<0.05	<DL	0.27	---
Magnesium	944	916	511	---
Manganese	0.04	0.11	0.21	---
Mercury	<0.0005	---	<0.005	---
Nitrate	<0.015	<DL	0.315	---
Nitrite	<0.002	---	<0.002	---
pH (units)	7.1	6.69	6.9	6.9
*Phosphate	---	---	---	---
Phosphorous	0.031	<DL	0.023	---
Potassium	345	345	0.3	---
Selenium	0.010	---	0.004	---

TABLE 5-7 continued

PARAMETERS	1,300 FEET DEEP B&V	1,300 FEET DEEP BF GOODRICH	1,200 FEET DEEP B&V	1,157 FEET DEEP B&V
Silica	3.0	---	29	11.4
*Silicon	---	16.5	---	---
Sodium	12,300	11,203	11,900	9,400
Strontium	34.9	39.9	34.1	---
Sulfate	178	205.7	<2	---
TDS	41,786	38,200	39,322	31,561
TOC	124	---	<1.0	---
*Turbidity (NTU)	14	---	59	125
*Zinc	0.059	---	0.49	---

TABLE 5-8

YORKTOWN AQUIFER WATER ANALYSIS

	DEPTH OF SAMPLE			
	609 ft.		330 ft.	
	B&V	BF GOODRICH	B&V	BF GOODRICH
BOD_5 (mg/l)	<1.5	—	<1.5	—
Alkalinity, as $CaCO_3$ (mg/l)	372	374	404	174
Chloride (mg/l)	9,374	9,239	418	462
Color (PCU)	10	—	30	—
Conductivity @ 25 C (umhos/cm ^{2})	22,000	27,350	1,950	2,055
Fluoride (mg/l)	0.83	<DL	0.12	1.4
Total Hardness (mg/l)	2,630	2,736	44.0	40.9
Nitrate-Nitrogen (mg/l)	0.026	<DL	0.139	<DL
Nitrite-Nitrogen (mg/l)	<0.002	NA	0.003	—
pH (units)	7.4	7.21	8.2	8.42
Total Phosphorus (mg/l)	0.026	<DL	0.359	<DL
Total Solids (mg/l)	18,175	—	1,250	—
Total Volatile Solids (mg/l)	2,965	—	106	—
Total Suspended Solids (mg/l)	21	7.0	74	68.7
Total Dissolved Solids (mg/l)	18,154	18,990	1,176	1,150
Settleable Solids (mg/l)	<0.1	—	<0.1	—
Sulfate (mg/l)	590	675.9	21	29.7
Turbidity (NTU)	29	7.4	23.5	34.0
TOC (mg/l)	108	—	113	—
Carbon Dioxide (mg/l)	—	43.6	—	1.1
Free Chlorine (mg/l)	<0.1	0.0	<0.1	<DL
Silica (mg/l)	34	32.7	11.5	11.4
Silicon (mg/l)	—	15.28	—	5.32
Strontium (mg/l)	16	18.0	0.16	0.13
Silver (mg/l)	<0.01	—	<0.02	—
Aluminum (mg/l)	<1.0	<DL	0.7	<DL
Arsenic (mg/l)	0.111	—	0.080	—
Barium (mg/l)	<1.0	0.08	<1.0	<DL
Boron (mg/l)	—	2.39	—	1.07
Calcium (mg/l)	310	229	5.02	6.2
Cadmium (mg/l)	0.035	—	<0.01	—
Total Chromium (mg/l)	0.04	<DL	<0.02	<DL
Hexavalent Chromium (mg/l)	<0.05	—	<0.02	—
Copper (mg/l)	0.03	<DL	<0.01	<DL
Iron (mg/l)	2.98	<DL	1.44	<DL
Mercury (mg/l)	<0.0005	—	<0.0005	—
Potassium (mg/l)	225	157.5	24.5	20.8
Magnesium (mg/l)	502	520	8.9	6.1
Manganese (mg/l)	0.15	0.05	0.28	<DL
Sodium (mg/l)	5,950	4,626	440	391.9
Lead (mg/l)	0.19	<DL	<0.05	<DL
Selenium (mg/l)	<0.0005	—	<0.001	—
Zinc (mg/l)	0.64	<DL	0.23	<DL

5.8.3 Quality

Table 5-9 and Figures 5-9 and 5-10 show the results of analyses of water samples from the test well during the long-duration pump test. Analysis consisted of tests at the well head for silt density index, temperature, pH, turbidity, and iron concentration. These tests were made every week day, with the exception of iron measurements, which were made at random. Samples were also sent to laboratories at approximately weekly intervals, to be analyzed for a broad range of ions which are factors in desalination process design. The two laboratories utilized were the Black & Veatch laboratory in Asheboro, North Carolina, and the B.F. Goodrich laboratory in Beltsville, Maryland. Two laboratories were used to provide quality control and to guard against analytical errors of critical water quality parameters. The analyses described in paragraphs 5.8.1 and 5.8.2 were made on redundant samples that were sent to each laboratory. The agreement of the results between laboratories was very good, and the redundant sample analysis was not continued during the long-duration pump test.

The water is an average salinity brackish water, with no constituents that would be expected to cause insurmountable problems. Iron in water to be treated by reverse osmosis can cause problems if it is trivalent, or if it is divalent and is present in sufficiently large quantities and could be oxidized to its trivalent state. Iron in concentrations of 0.2 to 0.6 mg/l can be removed by reverse osmosis, but will result in greater membrane maintenance and replacement costs. Pretreatment for iron removal may result in a more economical plant operation. A small pilot plant study is recommended to determine the potential for iron fouling and to provide additional guidance on the need for pretreatment.

5.8.4 Quantity

The quantity of water which can be withdrawn from the Yorktown aquifer for treatment, from either a single well or from a well field, is dependent upon the geological characteristics of the formation, the amount of water in

TABLE 5-9

YORKTOWN AQUIFER BLENDED WATER ANALYSIS

	Day of Sample				
	2	5	13	19	41	41
	B&V	BF Goodrich	B&V	B&V	B&V	BF Goodrich
Alkalinity, as $CaCO_3$ (mg/l)	292	294	294	296	297	290
Aluminum (mg/l)	—	<DL	—	—	—	<DL
Barium (mg/l)	<0.5	<DL	<0.2	<0.2	<0.1	0.006
Boron (mg/)	<0.5	1.24	<10	<5	<10	1.23
Calcium (mg/l)	71.6	49.03	58.9	50.9	50.8	54.97
Carbon Dioxide (mg/l)	—	12.9	—	—	—	12.4
Chloride (mg/l)	1880	1999	2122	2074	2100	2183
Chlorine (mg/l)	—	<DL	—	—	—	<DL
Total Chromium (mg/l)	—	<DL	—	—	—	<DL
Color (PCU)	10	—	10	30	5	—
Conductivity @ 25C (umhos/cm^2)	—	6200	—	—	—	7080
Copper (mg/l)	—	<DL	—	—	—	<DL
Fluoride (mg/l)	0.25	0.3	0.3	0.35	0.2	0.3
Hardness (mg/l)	412	480.7	452	424	378	606.7
Iron (mg/l)	0.98	0.417	0.38	0.21	0.02	0.588
Lead (mg/l)	—	<DL	—	—	—	<DL
Magnesium (mg/l)	72.5	86.2	91.5	92.1	10.0	113.0
Manganese (mg/l)	<0.01	<DL	<0.01	<0.01	<0.01	<DL
Mercury (mg/l)	<0.005	—	—	—	—	—
Nitrate (mg/l)	—	<DL	—	—	—	<DL
pH (units)	7.9	7.2	7.9	7.9	7.9	
Phosphate (mg/l)	—	<DL	—	—	—	<DL
Phosphorous (mg/l)	0.169	<DL	0.177	0.156	0.18	<DL
Potassium (mg/l)	43.1	51.8	62	68	74	71.6
Selenium (mg/l)	0.0054	—	0.0013	0.0035	<0.0005	—
Silica (mg/l)	—	16.5	15.6	14.0	15.2	16.5
Silicon (mg/l)	5.7	7.73	—	—	—	7.74
Sodium (mg/l)	999	1146	1080	1080	1120	1206
Strontium (mg/l)	2.3	2.55	2.5	2.4	—	3.31
Sulfate (mg/l)	125	122.7	140	130	125	141.5
TDS (mg/l)	3760	3800	3890	3981	4130	4128.1
TOC (mg/l)	78	—	5.0	45	12.5	—
Turbidity (NTU)	2.6	2.95	4.2	3.5	3.6	3.65
Zinc (mg/l)	0.07	<DL	0.03	0.05	0.01	<DL

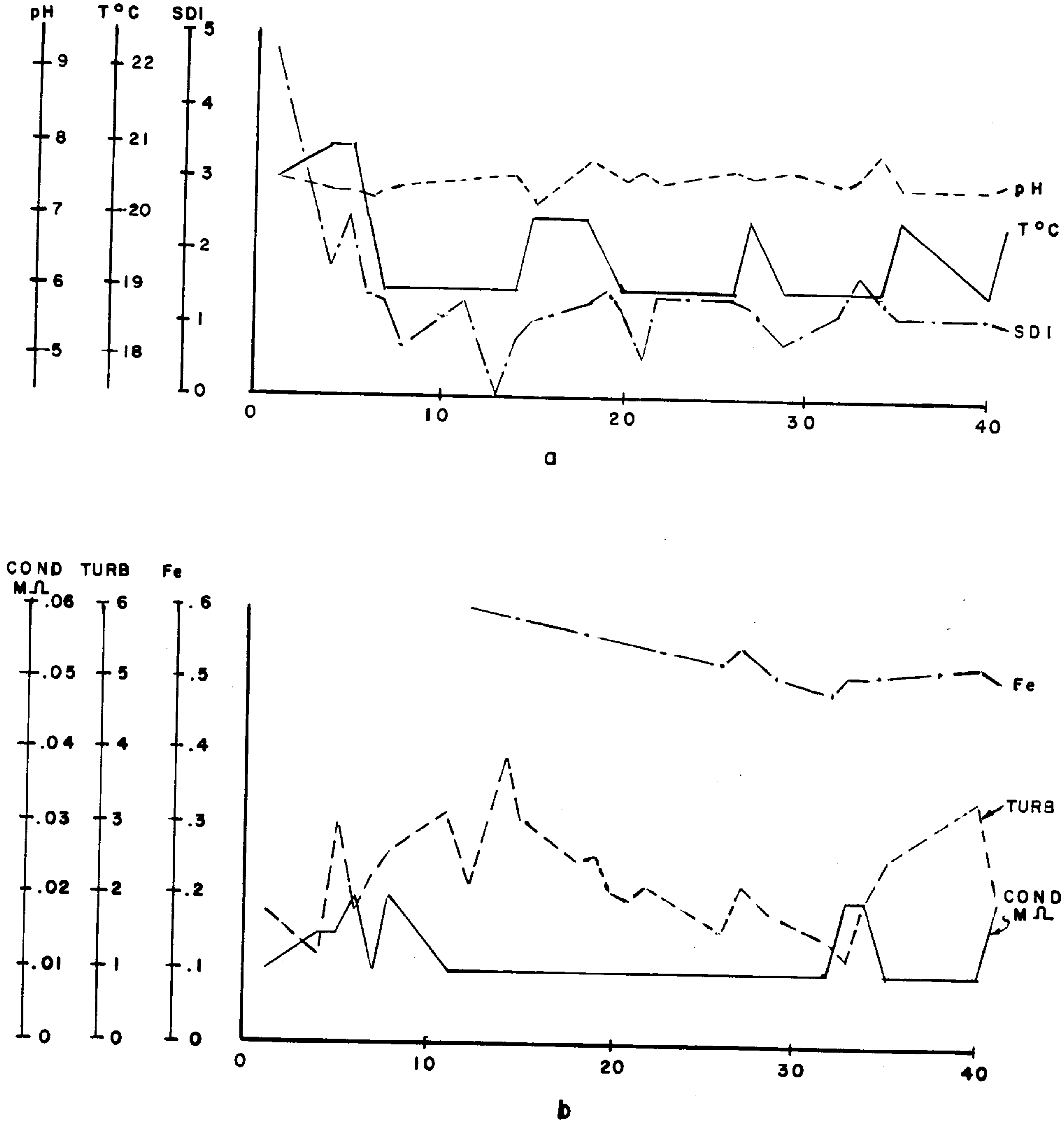


FIGURE 5-9 WELL HEAD ANALYSIS
TEST WELL NO. 1

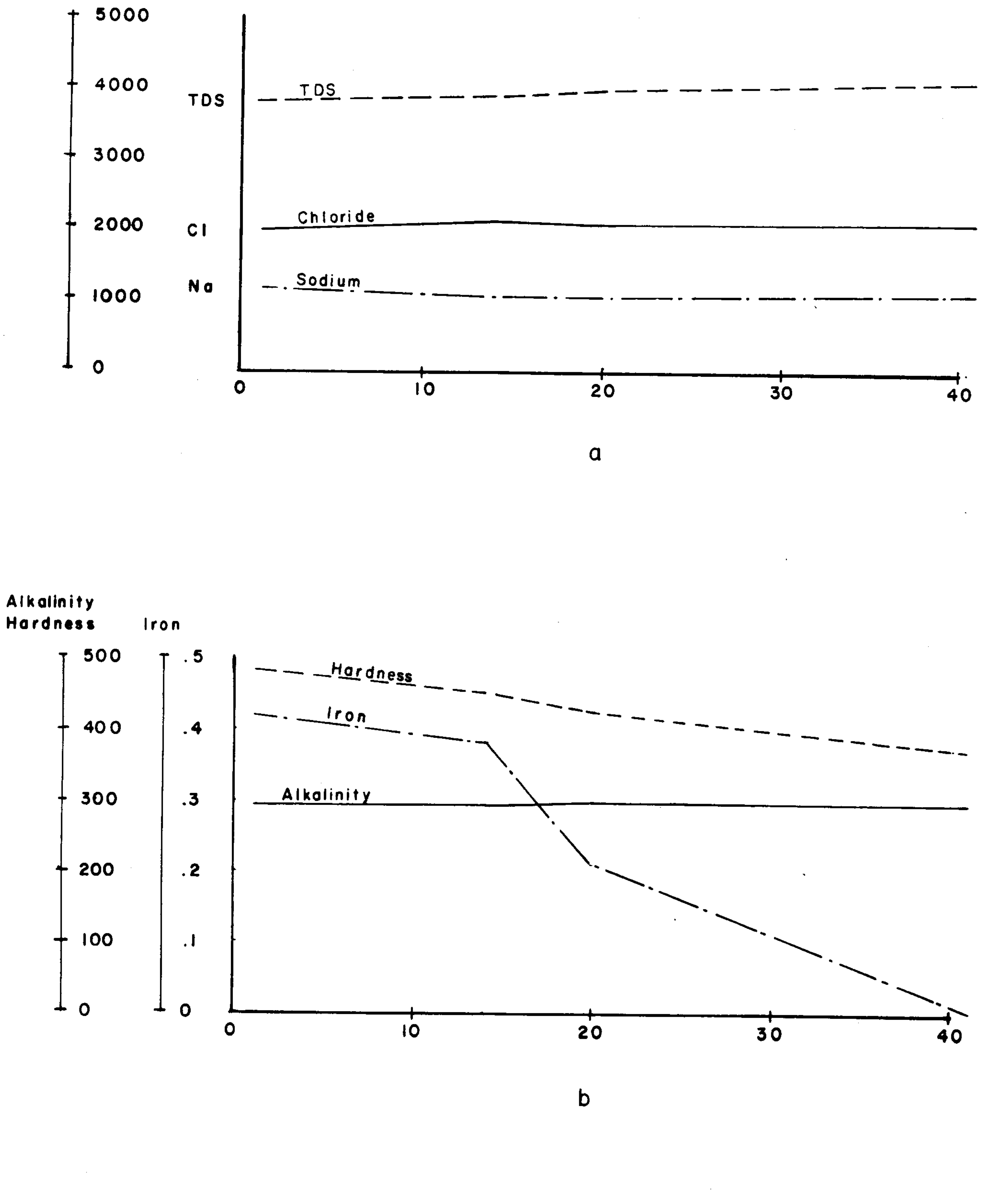


FIGURE 5-10 LABORATORY ANALYSIS
TEST WELL NO. 1

storage in the aquifer, and preservation of water quality. As mentioned previously, the characteristics of the formation will allow high yielding wells, in excess of 1,000 gpm. This is confirmed by drawdown measurements during pump testing. The apparent specific capacity of the well is 15 gpm/foot. At a well yield of 1,000 gpm, the drawdown is approximately 67 feet. This is within usual well design parameters.

Well field yield is related to aquifer storativity, leakance from adjacent aquifers, well spacing, and formation transmissivity. These factors were evaluated during this study, and a detailed report on this analysis is included in Appendix B. The conclusion from the hydrogist's report can be summarized as follows.

- The aquifer can safely yield 6.67 mgd, which is the amount of raw water needed for the 5 mgd plant planned for Stage 2 improvements.
- Well spacing should be approximately 1,500 feet, although this could be increased or decreased somewhat as the wells are installed and additional data is obtained.
- There is sufficient space on the Baum Tract for installing the 6.67 mgd well field.

The quality of the water withdrawn by the wells is another factor affecting well yield. As stated in Section 5.8.3, water quality varies with depth, as the TDS concentration increases towards the bottom of the aquifer. As pumping begins, the higher salinity waters could migrate towards the well both vertically and horizontally. Figure 5-11 is a schematic presentation of what can happen if the aquifer is over stressed. The figure shows a theoretical salinity interface between water acceptable for treatment and unacceptable water, which migrates toward the well field. The potential for saltwater intrusion can be minimized by limiting well yield and well field pumping, by adequately spacing wells, and by properly locating wells within the aquifer.

Well field study is continuing while this report is being written to gain additional information on the aquifer characteristics and behavior. The design of the well field should not be finalized until an additional well is constructed and tested.

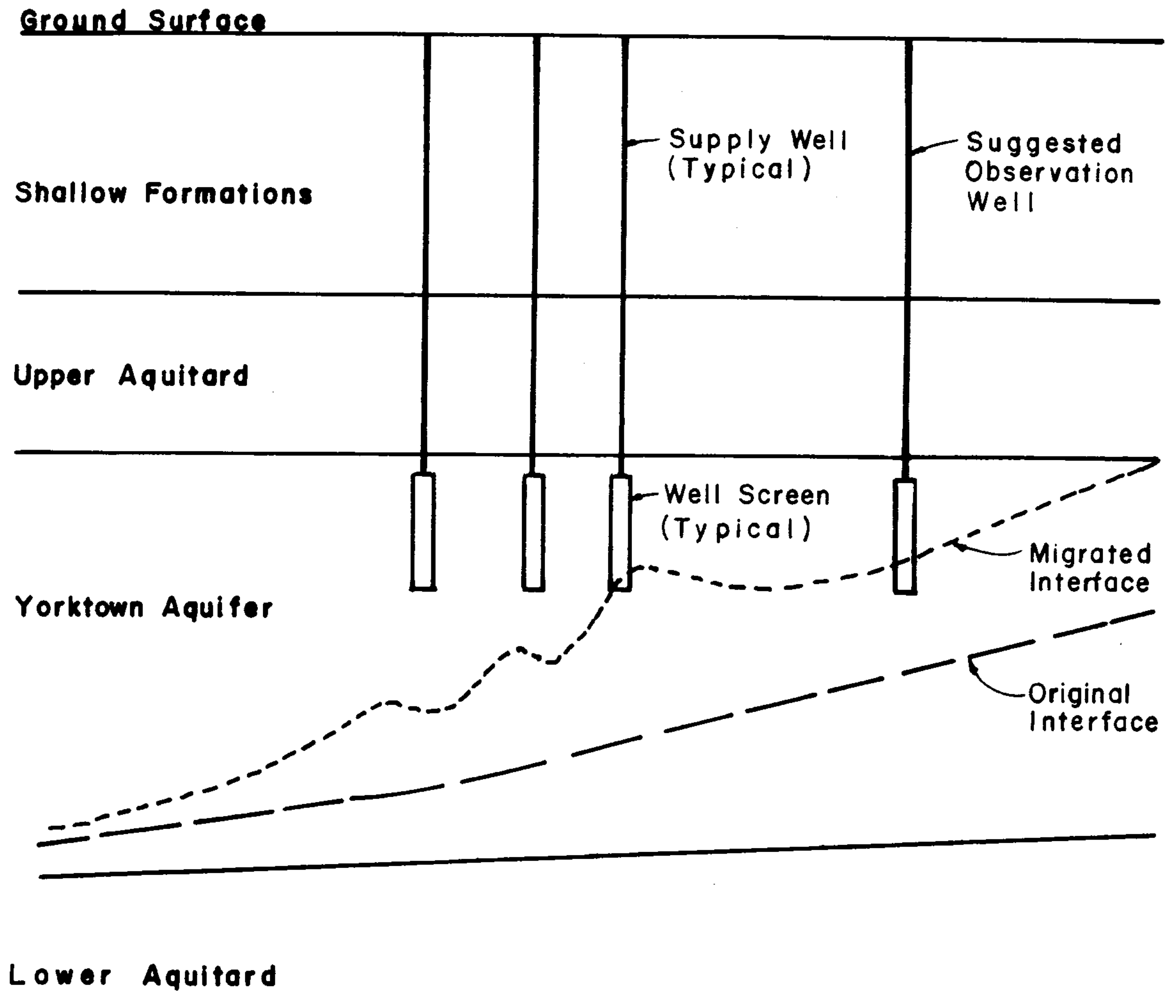


FIGURE 5-11: POTENTIAL EFFECT OF
PUMPING YORKTOWN AQUIFER

5.8.5 Facilities Required

The following facilities would be required to develop the Yorktown aquifer as a water supply. The facilities are also shown in Figure 5-12, and Table 5-10 shows the opinion of probable project cost.

5.8.5.1 Water Supply Wells. Water supply wells should be developed with capacities of 450 to 500 gpm. The well spacing should be 1,500 feet, screen depth from 300 to 500 feet, and the anticipated pumping level 40 to 60 feet. Ten wells will be required to supply a 5 mgd desalination plant operating at 75 percent recovery. The plant expansion to 8 mgd will require six additional wells. State regulations require a site with a 100-foot radius for each well, and sufficient wells on line to limit pumping of any well to 12 hours per day at average day conditions. The existing publicly-owned property can accommodate 10 well sites. Additional land would be required for expansion. Each well will require flow metering, telemetry, and a small superstructure for equipment protection. Well drilling by a method not utilizing drilling mud will be required, and casing and pump must be constructed of corrosion resistant materials. The probable cost for the well field is shown in Table 5-10.

5.8.5.2 Raw Water Transmission Main. A pipeline approximately 13,500 feet will be required for a field of 10 wells at minimum spacing. The pipe should be of inert material, either PVC, fiberglass, or stainless steel. The probable cost of this initial pipeline is shown in Table 5-10.

5.8.5.3 Pretreatment. The pretreatment required for a groundwater desalination plant is less stringent than for a surface water plant. For the upper part of the Yorktown aquifer to be treated by reverse osmosis, pretreatment for iron removal may be required depending upon the outcome of recommended pilot scale studies. The only other pretreatment required is that normally associated with an RO plant: pH adjustment, chemical addition for scaling control, and 5-micron cartridge filtration.

TABLE 5-10

OPINION OF PROBABLE COST

ALTERNATIVE 3B

BRACKISH GROUNDWATER

Item	Stage 2 ($ x 1,000,000)	Stage 3 ($ x 1,000,000)
Brackish water well field and piping	5.71	4.85
Desalination plant	4.81	4.00
High service pumping	0.25	0.20
5 MG storage at treatment plant		
Standby wells on Roanoke Island	0.50	--
	12.57	10.35

DARE COUNTY, NORTH CAROLINA

WATER SUPPLY ALTERNATIVES STUDY - 1985

BLACK & VEATCH, INC.
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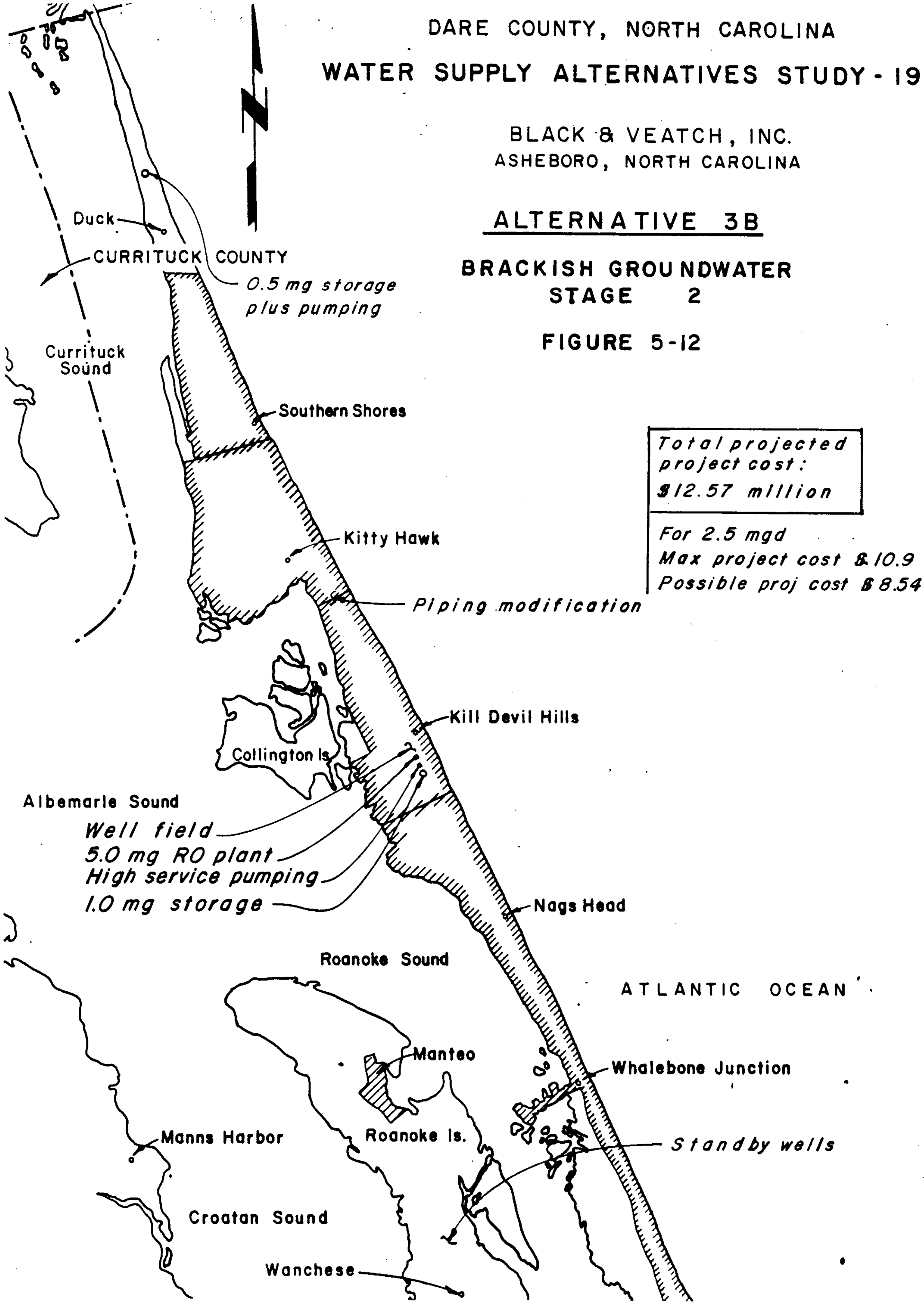
ALTERNATIVE 3B

BRACKISH GROUNDWATER
STAGE 2

FIGURE 5-12



5.8.5.4 Desalination. Available processes for desalination are covered in greater depth in section 7 of this report. The least costly option for the Yorktown aquifer is reverse osmosis. Either a spiral wound or hollow fiber membrane configuration could be utilized. The TDS in the feed water is at the upper range for low pressure membrane utilization.

Quotations were solicited from suppliers of RO and electrodialysis systems for the Yorktown aquifer feedwater, and the probable cost based from these quotations is given in Table 5-10. This cost includes building space to house the process equipment.

5.8.5.5 Other Facilities. The facilities described in paragraphs 5.7.3.5, 5.7.3.6, and 5.7.3.7 for a surface water plant will also be required for a groundwater treatment plant. The exception is pretreated water storage, which will not be required for a groundwater treatment plant.

5.9 COST COMPARISON

Table 5-11 provides a comparison of capital costs for the five alternatives discussed previously in this section. The table covers Stage 2 facilities only.

The table shows that expansion of the Roanoke Island well field and treatment plant is the least expensive alternative, followed by desalination of water on the Outer Banks, followed by mainland groundwater as the most expensive. It must again be emphasized that additional studies need to be completed before Alternative 1, expansion of the Roanoke Island water supply, can be considered feasible.

TABLE 5-11
WATER SUPPLY ALTERNATIVES
CAPITAL COST COMPARISON
STAGE 2

Alternative		Capital Cost ($ x 1,000,000)
1	Roanoke Island Expansion	6.65
2A	Mainland Groundwater	14.13
2B	Mainland Groundwater	22.59
3A	Brackish Surface Water	14.60
3B	Brackish Groundwater	12.57

6.0 EXISTING WATER SYSTEM

6.1 MAJOR SYSTEM COMPONENTS

The existing Dare County water system consists of the following major components.

- Water supply system, consisting of 10 wells on Roanoke Island and associated transmission mains.
- Water Treatment Plant on Roanoke Island, consisting of water softener units rated at 5 mgd, disinfection, and a 2.0 MG ground storage reservoir. This plant will be called the Roanoke Island water plant, to differentiate it from other water plants in the area.
- Transmission system, consisting of high service pumps located at the Roanoke Island water plant and approximately 80,000 feet of 24-inch and 16-inch ductile iron transmission main. The Towns of Nags Head and Kill Devil Hills own and operate their own water distribution systems that are supplied from metered connections to the transmission main.
- Roanoke Island distribution system, consisting of pumps located at the Roanoke Island water plant, elevated storage, distribution mains, and service connections. The Town of Manteo owns and operates its own water distribution system, and receives water from a metered connection to the County's Roanoke Island distribution system.
- Outer Banks distribution system, consisting of elevated and ground storage, pumping, distribution mains, and service connections in the Towns of Kitty Hawk and Southern Shores and in unincorporated areas north of Southern Shores.

The Towns of Manteo, Nags Head, and Kill Devil Hills each operated a water treatment plant prior to the 1980 start-up of the County's Roanoke Island water plant. Both Manteo and Kill Devil Hills have converted parts of their plants to other municipal uses, which has rendered them unavailable for use in the regional water system. The Nag's Head water plant is essentially intact as it was prior to its 1980 shutdown. This plant can be used for incorporating the Fresh Pond water supply into the county system, and is therefore discussed below. The separate water distribution systems of Manteo, Nags Head, and Kill Devil Hills, including all associated pumping, storage, and distribution mains, are beyond the scope of this study and are not considered in this report except where they have a definite bearing on one of the components of the county system.

Each component is described below, together with present operating characteristics and future requirements. Section 8.0 describes the recommended improvements in detail.

6.2 WATER SUPPLY SYSTEM

6.2.1. Existing System

The existing water supply system consists of 10 wells on Roanoke Island and associated transmission mains.

Wells No. 1, 4, 5, 7, 8, 9, 10, and 12 (eight wells) have been operational since 1978, and have intake screens in the principal aquifer. Wells No. 11 and 13 were placed into operation in August 1984. These two wells also have intake screens in the principal aquifer. Table 6-1 gives the principal characteristics of each well. It can be seen from this table that each well has specific operating characteristics. The combination of intake screen depth, pump size and depth, distance from the water treatment plant, and specific capacity produces a different capacity for each well in the present hydraulic system. The rated capacity is the maximum that could be pumped if the well were operating independently of other wells.

The factor limiting rated capacity for the eight original wells is the well pump size. The limiting factor for the two new wells is the pump setting depth. The normal operating capacity of the well field is the summation of the capacities of each well with all other wells operating. With one or two wells shut off, the normal operating capacity for the entire system will be very close to this total, since the remaining wells will each pump at a slightly higher rate.

The raw water transmission system consists of a single transmission line that connects to the discharge pipe of all ten wells. This main was constructed in 1978-79 of 16-inch and 12-inch asbestos-cement pipe. The transmission main originally discharged directly to the water treatment plant without repumping.

TABLE 6-1

EXISTING ROANOKE ISLAND WELL FIELD DATA

Well Data

Well No.	Intake Screen ft	Specific capacity gpm/ft	Motor Size hp	Rated Capacity gpm	Rated Head	Diesel Generator
1	NA	13.0	40	500	181	No
4	170-220	9.0	40	500	174	No
5	168-218	10.4	30	500	162	No
7	165-215	10.7	30	500	157	No
8	162-212	6.2	30	500	146	No
9	120-190(2)	20.8	30	500	148	Yes
10	141-192	8.0	25	500	133	Yes
11	180-192 196-220	18.4	25	600	130	No
12	140-190	19.2	25	500	119	Yes
13	176-216	24.3	30	600	150	No

Notes:

(1) Specific Capacity $= \frac{\text{Well Q}}{\text{Drawdown}}$,based upon test data. This is an approximate linear relationship for all ten wells. The drawdown for a given capacity is computed by the flow by the specific capacity. Thus, for a flow of 500 gpm and a specific capacity of 15, the drawdown will be 500/15 = 33.3 ft.

(2) Screen is intermittent between these depths. Total length is 50 feet.

6.2.2. Operating Characteristics

The water supply system had difficulty meeting water demands prior to August, 1984. The well field could not produce sufficient head to run the plant at greater than 3.5 mgd during the 1983 and 1984 peak seasons, resulting in the plant being bypassed to decrease pressures and thereby increase well yield. The wells and transmission mains are not part of a hydraulic system that includes the Roanoke Island water treatment plant. The system deficiency can be described either in terms of well yield or treatment plant capacity. This problem, together with potential mitigating measures, is discussed in greater detail in Section 6.3 as it relates to the Roanoke Island water plant capacity.

Operating problems have been experienced with wells 1 and 4, due primarily to area private well owners' complaints about their wells being adversely affected by the County's pumping. Although studies have shown that the problems cannot be definitely traced to the County's operation, the continued complaints caused the County to shutdown wells 1 and 4 for extended periods. A potential solution would be to connect all residence in the Wanchese region to the county system, thereby eliminating the need for private wells. This would also increase the area available to the county for additional wells which will be needed to meet future water demands. In 1983 and 1984, private wells were installed for residents whose wells were apparently affected by the County's pumping, which allowed use of wells 1 and 4 to be resumed.

6.2.3. Future Expansion

If the Roanoke Island water supply system is expanded, additional wells and transmission mains will be needed. The size of the system expansion should correspond with concurrent Roanoke Island water plant expansion. The safe yield of the principal aquifer is the limiting factor in expansion of this system. As discussed in Section 5 and Appendix A, additional wells and pipelines can be added to expand the supply system to meet 2005 water demands, pending the successful completion of additional studies.

If an alternative to Roanoke Island groundwater is developed, no further improvements are required, except for the installation of standby wells as discussed in Section 7.

6.3 ROANOKE ISLAND WATER TREATMENT PLANT

6.3.1. Existing Treatment Plant.

The existing Roanoke Island water treatment plant was constructed in 1977-1980. The plant is an ion-exchange softening facility consisting of four pressure softeners, salt regeneration facilities, and appurtenances. The plant uses a blending process to achieve the desired hardness in the water; that is, a portion of the water entering the plant bypasses the softeners and is not treated. This water is combined, or blended, with water that has been treated by the softeners, resulting in a water with an acceptable level of hardness. The process equipment, on-line since 1980, was supplied by Bruner Corporation.

As described in the 1984 MGA report, the treatment plant was analyzed during that study for specific operating problems, adequacy of existing capacity, and for its potential for expansion. The discussion from the previous report is repeated here, with revisions and updates to reflect changed conditions.

6.3.2. Water Treatment Plant Description

The treatment plant's four ion-exchange softeners are fed directly from the raw water transmission main and discharge into the ground storage reservoir. The softeners operate under pressure. A raw water booster pumping station, constructed at the water treatment plant in 1984, provides more pressure for plant operation and thereby increases the capacity of both the well field and the treatment plant.

Each softener is designed for a flow of 875 gpm at 15 psi head loss. For three softeners on-line, the plant softener through-put would be 2,265 gpm, or 3.8 mgd. Four softeners on-line would result in a through-put of 5.0 mgd. Each softener contains 146 cubic feet of ion-exchange resin. According to the original design, as described in the Bruner service manual provided with the treatment plant, each softener can treat 200,000 gallons before regeneration is required. This computes to a hardness removal capacity of 18,000 grains per cubic feet of resin, which is at the low end of Bruner's usual design.

Regeneration of the ion-exchange resin is accomplished by passing salt solution through a softener, which removes the "hardness" from the resin. According to the service manual, each regeneration requires 730 pounds of salt, equivalent to 5 pounds of salt per cubic foot of resin. Again, this is at the very low end of the Bruner recommendations for regeneration. Each regeneration requires four steps and takes 90 minutes to accomplish. It is possible to increase the resin capacity by increasing the salt dosage during regeneration. The brine that results from regeneration is discharged untreated to a lagoon which in turn overflows to Croatan Sound by way of a drainage canal.

Blending is accomplished by means of an 8-inch hydraulic blending valve located downstream of the softeners. This valve adjusts automatically, based on influent flow rate, to maintain the design ratio of treated bypass water. The original design required 25 percent of the raw water to be bypassed, resulting in a finished water hardness of 50 ppm. Three softeners on-line plus a 25 percent bypass equals a total plant flow of 5.0 mgd.

6.3.3. Roanoke Island Plant Capacity Evaluation

As previously stated, the design capacity of the Roanoke Island plant is 5.0 mgd. This is based on both the construction specifications and the manufacturer's information. Determining the actual plant capacity is not that straight forward. There is a variety of opinion among ion-exchange equipment manufacturers and consulting engineers regarding the most significant design parameters for ion-exchange process design. The treatment process design was based on a volumetric loading rate of 6 gpm/cu ft of resin capacity, resulting in a maximum flow rate of 875 gpm per softener. This analysis places no significance on the surface loading rate, which computes to 23 gpm/sq ft. This latter rate is more widely accepted as the most important design parameter, with a maximum recommended rate of 10 gpm/sq ft. Such a limitation would cause each softener to be rated at only 385 gpm, with the resulting plant flow, including the design blending, equal to 2.22 mgd. This discrepancy is quite significant.

Using a surface loading rate of 10 gpm/sq ft as the limiting design parameter will provide a much more conservatively designed plant. The practical realities of current and near future demand levels dictate that this plant continue to be operated based on the volumetric loading rate for the next several years. Before any expansion is considered, a detailed study should be made to determine the long-term capacity of the existing plant based on the installed softener units, actual raw water quality, and desired finished water quality.

Two additional factors relate to existing treatment plant capacity. First, the hardness removal required is less than was assumed when the plant was designed. Raw water hardness is 175 ppm (200 ppm used in design), and the finished water hardness recommended by State regulatory agencies is 75 ppm (50 ppm used in design). These new figures indicate a blending ratio of 60 percent treated to 40 percent bypassed (original design was 75:25). If the volumetric loading rate discussed above is applied, this computes to a plant capacity of 6.3 mgd. If the surface loading rate discussed above is applied, computed plant capacity is 3.7 mgd.

The second factor that affects plant capacity is the desired finished water hardness. The current operation produces finished water with 75 ppm hardness which is a good median level of treatment. However, if a higher hardness can be tolerated, the amount of water bypassed and the finished water produced can be increased. Increasing the finished water to 96 ppm will increase the blending ratio to 1:1. This is not recommended at this time since the current level of treatment produces finished water at the level recommended by the State. However, the County should be aware of the potential of producing more water at a lower quality should this be necessary to meet short term peaks.

The findings of this section are summarized as follows.

(1) The existing Roanoke Island treatment plant can be considered capable of producing 6.3 mgd of finished water for the next several years, as a matter of necessity to meet water demands.

(2) A study should be conducted prior to any plant expansion to more accurately determine existing plant capacity and the additional softener units required to develop overall plant capacity to meet future water needs.

6.3.4. Existing Operating Problems

The water treatment plant has been burdened with a number of significant problems that affect plant capacity and O&M expenses. These problems, which for the most part have occurred regularly since the plant was placed in operation, are discussed below together with possible ways of alleviating them.

The 1984 MGA report listed several operating problems that have been corrected. A problem identified as a combination of insufficient well pump capacity and excessive plant head loss has been corrected by adding a booster pumping station on the raw water line. This station has operated successfully allowing the plant to function at and above design capacity. Problems related to maintaining proper chlorine residuals have been corrected by replacing existing chlorine feeders with new ones that are dedicated to the specific chlorine feed points. Problems with vortexing in the ground storage reservoir and air binding of the transmission pumps has been minimized by installing an anti-vortexing baffle in the ground storage reservoir.

6.3.4.1 Equipment Breakdowns. There are several items of plant equipment that are difficult to maintain in working order. Many of the breakdowns are minor, but add to O&M costs. Safety hazards are also increased by these breakdowns due to water spillage that becomes unavoidable during operation and repair. These nuisance items should be replaced.

The breakdown of the softener meters is more than a nuisance. These meters are used to totalize the flow through each softener and to start automatic regeneration based on preset levels. When a meter breaks down, which occurs approximately 50 percent of the time, it is necessary for the operators to estimate by other flow measurements when a softener should be regenerated. If this estimate is not accurate, the plant will run inefficiently since regenerating too soon will increase costs and regenerating too late will increase finished water hardness. It is essential to plant operation that these meters be replaced with properly functioning meters that accurately indicate the need for regeneration.

6.3.4.2 Softener Through-put. Through-put is defined as the amount of water that a softener can treat before regeneration is required. According to the manufacturer's literature supplied with the plant, each softener is capable of a through-put of 200,000 gallons. Experience has shown that actual through-puts are lower, sometimes as low as 100,000 gallons. This reduction in through-put can be caused by resin damage, excessive hardness leakage, incomplete regeneration, or a combination of these. Testing of the resin and of the product water from each softener is required to determine the cause of reduced through-put.

Permanent resin damage can be caused by a number of substances in the water. Replacement of the resin is required to correct this deficiency. Excessive hardness leakage occurs when resin's removal capacity is exhausted or when improper flow distribution in a softener causes water to pass through the resin without being treated. Correction of this deficiency requires (1) testing to determine the correct loading rate for the existing resin area and depth, inlet distributor type and underdrainage system, and (2) operating the softeners at the correct dosage rate.

The softeners are presently regenerated at a relatively low salt dosage, which is economical in terms of operating costs. However, at this low salt dosage, it is possible for regeneration to be incomplete. This causes the resin's removal capacity to be reduced with time. This process can be reversed by occasionally regenerating with a much higher salt dosage. This has been suggested to plant operators and will be practiced in the future.

6.3.4.3 Electrical. Irregular construction practices have caused operating difficulties by repeated tripping of certain circuit breakers and burnout of motor leads. The electrical system has very little room for expansion. This wiring problem cannot easily be corrected by rewiring of circuits or addition of new circuits. It is recommended that these deficiencies be corrected the next time any electrical loads that require expansion of the motor control center are added to the plant.

6.3.4.4 Telemetry. The existing telemetry system has never worked consistently. This equipment has much higher operating costs than other systems currently available. The Water Department is investigating replacing this system. This investigation should continue focusing on reliability and operating costs, and an improved system should be included in Stage 2 improvements.

6.4 FRESH POND WATER TREATMENT PLANT

The Fresh Pond water treatment plant, owned by the Town of Nags Head, began producing water for the regional water system in November 1985. Improvements were made to this plant in 1985, as recommended in the 1984 MGA report and a December 1983, letter report, to bring the treatment process in line with current regulations. The improvements made are as follows.

- Reconstruction of the floating intake structure.
- Replacement of raw water pumps and construction of a new pumping station superstructure.
- Removal of the microstrainer.
- Construction of a new rapid mix chamber.
- Reconstruction of the wood flocculation cribs.
- Addition of a decant pump to the sedimentation basin.
- Construction of new filters, filter control building, and clearwell.
- Addition of transmission/backwash pumps.
- Construction of a sludge pumping station and sludge drying beds.
- Construction of a wash water recovery system.

Several improvements were deferred due to budget limitations. Also, a study prepared for the Town of Nags Head has revealed several other improvements that are needed for improved reliability and safety. These additional improvements are as follows.

- o Addition of a standby generator.
- o Improvements to the alum and caustic soda storage and feed systems.
- o Construction of new chlorine storage and handling facilities.
- o Construction of sludge drawoff piping in the sedimentation basin.
- o Construction of additional clearwell capacity.

6.5 TRANSMISSION SYSTEM

6.5.1. System Description

The transmission system delivers finished water from the Roanoke Island and Fresh Pond water treatment plants to the points of draw-off for wholesale water users and to the beginning of Dare County's distribution system. The transmission system consists of pumps at each treatment plant, a 24-inch water main from the Roanoke Island water treatment plant to the boundary of Nags Head and Kill Devil Hills, a 16-inch main connecting the Fresh Pond water treatment plant to the transmission system, and a 16-inch main continuing to a master meter in Kill Devil Hills and to the County's distribution system north of Kill Devil Hills. The transmission pumps located at the Roanoke Island water treatment plant are a part of the transmission system when considering design and capacity. However, for constuction and operation, they function as part of the water treatment plant.

The Fresh Pond water treatment plant also feeds into this transmission system.

There are five draw-off points from the transmission system.

- o Nags Head's Gull Street Pumping Station.
- o Nags Head's Eighth Street Reservoir.
- o Kill Devil Hills' Fresh Pond Reservoir.
- o Kill Devil Hills' Municipal Complex Pumping Station.
- o Dare County's Kitty Hawk Pumping Station.

Each location is equipped with a master meter and altitude valve, ground storage reservoir, and high service pumps. Table 6-2 lists the transmission pumps which feed the system.

6.5.2. System Evaluation

Both computer and manual analyses of the transmission system were performed as part of this study. Figures 6-1 and 6-2 present diagrams of this system together with the flow rates required in Stages 1, 2, and 3. Figure 6-1 assumes all water will come from the existing water treatment plants, whereas Figure 6-2 includes a new treatment plant on the Outer Banks. All three phases were analyzed to determine the system operating characteristics and any improvements required. The results of the analyses are described in the following section.

6.5.3. Expansion Of Existing Water Plants

Figure 6-1 is a schematic of the transmission system showing inputs and drawoffs during the three growth stages. All inputs are from the Roanoke Island and Fresh Pond water treatment plants. The transmission system was analyzed for its ability to transmit flows from an expanded Roanoke Island water treatment plant, including the contribution from the Fresh Pond plant.

The very large water demands and flow rates required in Stage 3 cannot be met with the present transmission system. Although the velocities in the transmission mains are reasonable, the long transmission distances result in large pressure losses. There are three basic types of improvements that can be made to correct this situation.

(1) Install new transmission pumps at the water treatment plant that can produce sufficient pressure to deliver water to the farthest point in the transmission system. In this case, the required pressure is almost 240 psi, which greatly exceeds the allowable pressure of the transmission main. It is also best to maintain a high safety factor in the main under Roanoke Sound, since repairs would be extremely costly and would severely cripple the entire system for a considerable time. Therefore, this alternative is not considered feasible.

TABLE 6-2

TRANSMISSION PUMPS

Pumping Units	Capacity gpm	Rated Head ft	
Roanoke Island No. 1 and No. 2	4,000	94	(at high speed)
	2,828	47	(at low speed)
No. 3 and No. 4	700		
Fresh Pond No. 1, No. 2, and No. 3	1,040	75	

NOTES:

1. Only one Roanoke Island pump may be operated at a time due to electrical system limitations.

2. All three Fresh Pond pumps may be operated at the same time.

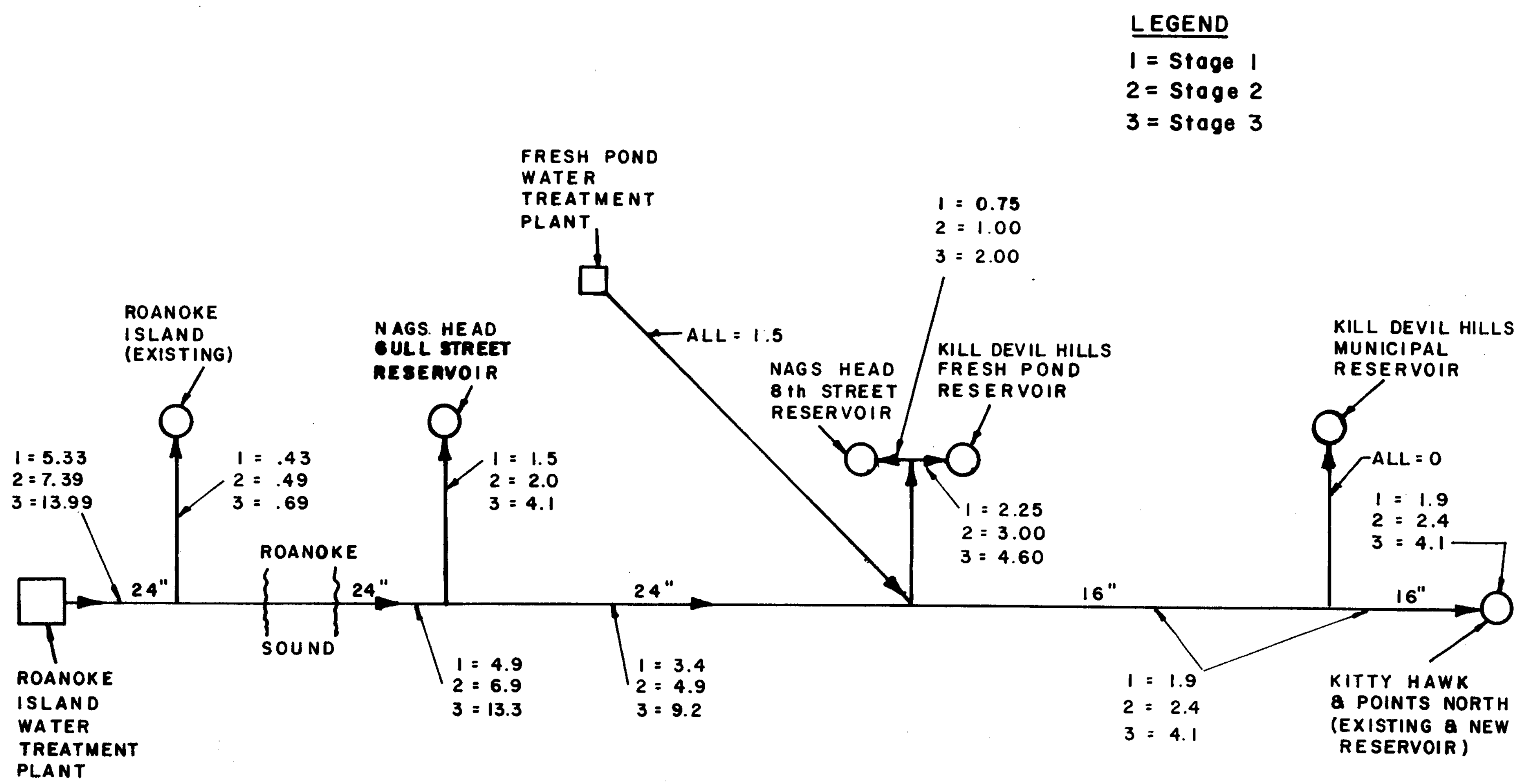


FIGURE 6-1
TRANSMISSION SYSTEM FLOW RATES
(NO DESALINATION PLANT)

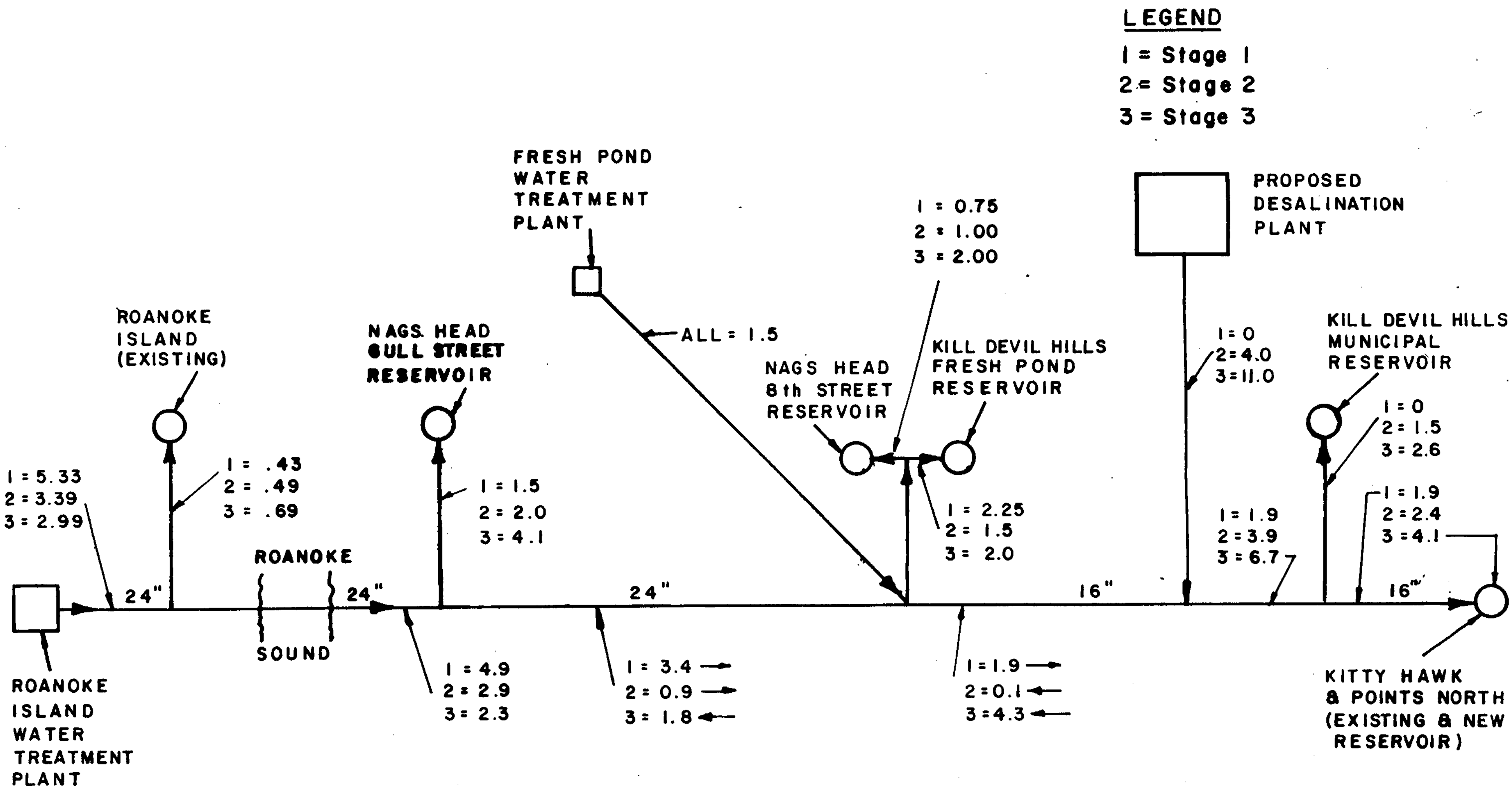


FIGURE 6-2
TRANSMISSION SYSTEM FLOW RATES
(INCLUDING DESALINATION PLANT)

(2) Install additional mains parallel to the existing transmission main. The hydraulic analysis shows that it would be necessary to parallel the main with a new main of equal size. The total length of mains required is 81,000 feet, and the probable construction cost is $4,900,000.

(3) Install booster pumping stations, as required, along the transmission line. The hydraulic analysis determined that three booster stations would be required for Stage 3 flows; one on Roanoke Island just before the Sound Crossing, one at Whalebone Junction, and one at the Nags Head/Kill Devil Hills boundary. These three pumping stations will deliver the water required under Stage 3, while maintaining pressures in the submarine main below 60 psi. However, replacement of the Roanoke Island transmission pumps will be required to handle larger plant flows.

Table 6-3 presents the costs of these three booster stations, including the present worth of power costs, and compares them to the new transmission main alternative. To facilitate the comparison, it was assumed that all improvements would be required in 1995 and the power cost of the booster station alternative is lower than the transmission main alternative by a factor of 4.

The Stage 3 transmission system was also analyzed for average day off-season conditions. The analysis showed that a pressure of 138 feet would suffice to deliver water to all users on the system. Thus, by sizing the high service pumps to produce 140 feet of head, the booster pumps in the transmission system will not be required under normal off-season conditions. This will minimize annual operating costs.

The hydraulic analysis for Stage 2 determined that any one of the booster stations required under Stage 3 will suffice to deliver the required water to all users on the transmission system. The pressure requirements of the main below Roanoke Sound effectively prohibit the construction of the Roanoke Island booster station before Stage 3. The Whalebone Junction booster station is better matched to the Stage 2 conditions than the Kill Devil Hills booster station, because it can be installed at a reduced capacity in Stage 2 and expanded in Stage 3. This cannot be done for the Kill Devil Hills station since it will have only two pumps to meet Stage 3 water demands.

TABLE 6-3

TRANSMISSION SYSTEM IMPROVEMENTS COST COMPARISON

Booster Stations	Project Costs	Annual Power Costs	Present Worth
Roanoke Island	$ 190,000	$ 18,000	
Whalebone Junction	220,000	54,000	
Kill Devil Hills	80,000	31,500	
	$ 490,000	$ 103,500	$ 1,126,000
New Transmission Line	$4,950,000		$ 4,950,000

An average day analysis was performed for Stage 2. This revealed that no booster pumping would be required in the off season. This will minimize annual operating costs for the Stage 2 period.

The hydraulic analysis for Stage 1 indicated that the water demand will exceed the transmission main capacity by the 1984 or 1985 peak season if all water comes from Roanoke Island. Reactivation of the Fresh Pond Water Treatment Plant, with a flow rate of 1.0 mgd, allows the transmission system to supply all demands during Stage 1.

6.5.4. Effects Of New Treatment Plant

If a water treatment plant is constructed on the Outer Banks, there will be a significant change in the transmission system flow patterns. This will add a third input to the system, and reduce the amount of water that must be delivered from Roanoke Island. As can be seen in Figure 6-2, two sections of transmission main will have very little flow during Stage 2, and will experience a flow reversal during Stage 3.

This new input would better utilize the existing transmission main that would increase the discharge from the Roanoke Island water treatment plant. Although velocities in some sections of the main are higher than desired, no additional water mains or booster pumps would be required.

6.6 OUTER BANKS DISTRIBUTION SYSTEM

The distribution system supplying the Towns of Kitty Hawk and Southern Shores, and unincorporated areas north of Southern Shores is owned and operated by Dare County. This system has been covered in a separate study and report, Report On Water Distribution System For Dare County, North Carolina, June 1986. The most significant findings and recommendations from that report are as follows.

- Demand in the distribution system area will increase to 2.4 mgd in 1990 and 4.17 mgd in 2005.

- The existing distribution system will not be able to supply the increased demands, nor will adequate firefighting pressures be available in Kitty Hawk Village and Duck.

- Additional water mains will be needed to supply demands. Main improvements will be 12-inch and larger mains.
- Additional elevated storage is required in the Duck area.
- Improvements to the Kitty Hawk pumping station will be required by 1990.

6.7 ROANOKE ISLAND DISTRIBUTION SYSTEM

Dare County owns and operates a distribution system west and north of Manteo on Roanoke Island, serving less than 90 customers. Treated water is pumped from the clearwell at the Roanoke Island plant to a 200,000 gallon elevated tank at the treatment plant site. The pumps are 30 hp, rated 400 gpm at 150 feet. One pump serves as an installed spare, and the firm pumping capacity is 400 gpm. Water from the tank flows through a 12-inch main, through the Town of Manteo, which receives water from a master meter at its old water treatment plant. Water is distributed to Dare County's customers through a mostly non-looped network of 6-inch and 8-inch water mains.

The growth rate of this system has been very slow, and thus far the network of 6-inch and 8-inch mains has performed satisfactorily. No improvements are anticipated for this system, except for minor water main extensions.

There are other areas on Roanoke Island that could support distribution systems. These include areas north and west of the existing distribution system, and the Wanchese community. Residents in either area have expressed no interest in a municipal water supply. Therefore, no futher analysis of these areas is included in this study and report.

7.0 DESALINATION OPTIONS

7.1 GENERAL

Desalination processes are used when water contains dissolved solids that cannot be removed by conventional treatment or when total dissolved solids are present in sufficient quantity to make desalination processes cost effective. Typical dissolved solids which, when present in sufficient quantity, can require to the use of a desalination process include sodium, calcium, magnesium, sulfate, chloride, and bicarbonate. Saline water is a term given to any water with chloride concentration greater than 250 mg/l and total dissolved solids (TDS) concentrations greater than 500 mg/l. Saline waters include salt water, which is undiluted seawater typically containing 35,000 mg/l TDS, and brackish water, containing up to 10,000 mg/l TDS. Brackish waters are highly mineralized groundwaters and diluted seawater.

Dare County has an abundance of saline waters which are potential drinking water supplies if desalination can be economically employed. Sources of saline water near the existing regional water system, include:

- The Atlantic Ocean.
- Currituck, Albemarle, and Roanoke Sounds.
- Mainland surface water, found in canals, creeks, East Lake, and South Lake.
- Outer Banks groundwater, found in several aquifers of both brackish and salt water at depths exceeding 100 feet.
- Deep Roanoke Island groundwater.
- Deep mainland groundwater.

Water treatment costs, both capital and operating and maintenance, are considerably less for brackish water than for salt water. Since brackish water is available, salt water will not be considered further.

7.2 DESALINATION PROCESSES

Typical processes used for removing dissolved solids from brackish water are reverse osmosis (RO) electrodialysis (ED), and ion exchange (IE). Distillation can also be used for more concentrated brackish waters although it is more typically used for salt water desalting. Several other desalination processes are being developed, but are not yet commercially feasible and will not be discussed herein.

A brief description of the four processes mentioned above follows. Cost comparisons for RO and ED are given in Section 7.3. RO, as the selected process, is discussed more fully in Section 7.4.

7.2.1. Reverse Osmosis

Reverse osmosis is a physical process that takes advantage of the natural tendency of water to dilute a concentrated solution. When salt water and fresh water are on opposite sides of a membrane that is permeable to water but not to solids dissolved in the water, dilution of the salt water occurs as water molecules pass through the membrane. When an external pressure is applied to the salt water, the water flow across the membrane can be reversed and pure water is removed from the more concentrated salt solution. This is the process of reverse osmosis.

Figure 7-1 shows the reverse osmosis process in simplified form. Pressure is continously applied to the feed stream by a high pressure pump, while product and brine are continuously withdrawn. Dissolved solids rejected by the membrane are continously flushed from the system in the brine. The brine contains a high level of dissolved solids while the product contains a low level. A flow regulating valve on the brine discharge line controls the percentage of feedwater that is converted to product.

Currently available RO devices are either hollow fiber membrane permeators or spiral wound membrane permeators. The design and manufacture of these devices and of process systems incorporating these devices is highly specialized. Performance of the process depends upon feedwater quality, applied pressure, and the presence of potential fouling or scaling ions which can harm membranes. RO is typically cost-effective at feedwater TDS concentrations of 1,000-8,000 mg/l. The primary operating costs include electric power for the high pressure pumps and membrane replacement.

RO is used mainly for treating groundwater. Surface waters usually require extensive pretreatment for removal of suspended solids and turbidity. Groundwater requires cartridge filtration for removal of fine suspended solids and possibly chemical addition. Other pretreatment requirements are specific to membrane materials and configuration.

RO is a viable process for treating brackish groundwater in Dare County. As discussed in Sections 5.7 and 5.8, treatment of brackish groundwater through desalination will be more cost-effective than treating brackish surface water.

7.2.2. Electrodialysis

Electrodialysis (ED) is a membrane desalination process using an electric field to draw dissolved solids out of feedwater. The feedwater gradually becomes desalted enough to be considered product water. ED was the first membrane process to demonstrate a practical desalination alternative to distillation. The most important components of the ED process are the membranes themselves. The membranes are cast as flat sheets, typically 0.15 inch thick, and are commercially available in sizes up to 20-inches by 60-inches. Membranes are designed and constructed to allow either anions or cations to pass through while rejecting the opposite charge. By placing alternately selective membranes in an electric field between a positive and negative charge, the ions in the feedwater are drawn toward the oppositely charged electrode and will pass through one membrane but be rejected by the next. The solution between one pair of membranes becomes depleted of ions while the solutions on either side become more concentrated. Figure 7-2 is a simplified schematic diagram of the ED process. A commercial ED unit consists of hundreds of paired membranes stacked vertically and clamped between two large oppositely charged electrodes.

The ED process is cost-effective in treating brackish waters up to 3,000 mg/l TDS. It will generally be most competitive against other processes at feedwater concentrations of 500 to 1,500 mg/l TDS. ED does have some disadvantages compared to reverse osmosis.

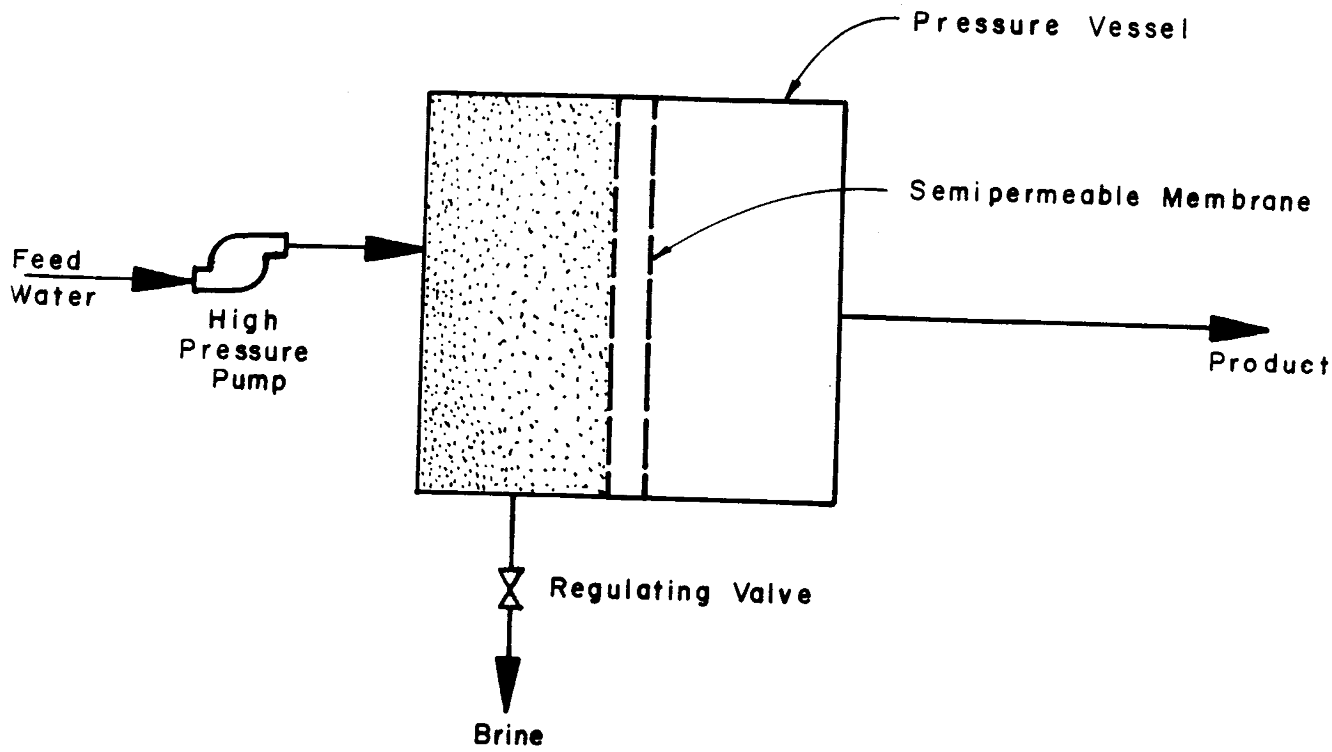


FIGURE 7-1. RO SCHEMATIC

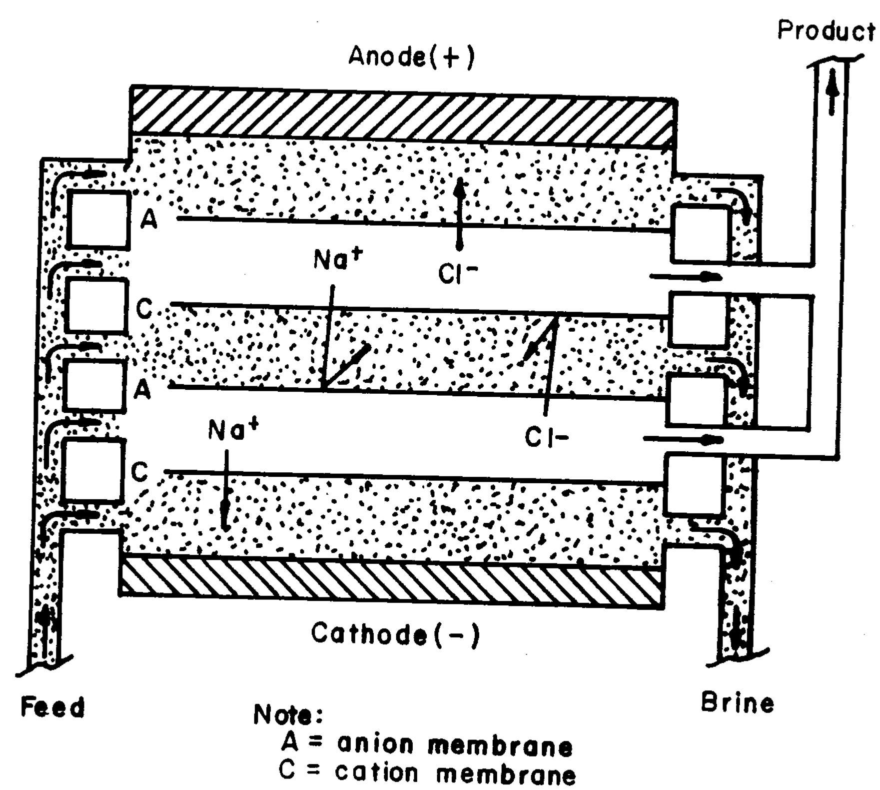


FIGURE 7-2. ED SCHEMATIC

- o Feedwater channels are directly connected to treated water channels. In RO, the feedwater and treated water are separated by the membrane.
- o ED generally has less salt rejection and lower recovery than RO.
- o ED is more labor intensive, as cleaning of membranes is more frequent and requires more manhours per cleaning than the equivalent RO process.

Pretreatment requirements for the two processes are similar. Cartridge filtration to remove suspended matter and removal or sequestering of potential membrane foulants and scalants is required.

Because of its generally favorable characteristics, ED is a viable process for desalination of brackish water for Dare County. Whether the process would be cost effective compared to others would depend upon the specific feedwater quality, and is discussed in Section 7.3.

7.2.3. Ion Exchange

Ion exchange resins are natural or synthetic granular materials that exchange one ion for another. The new ion is held temporarily then released to a regenerating solution.

More commonly considered a demineralizing process than a desalting process, ion exchange (IX) is capable of cost efficient removal of essentially all of the inorganic salts from feedwater containing less than 500 mg/l TDS. The capital and operating costs for IX systems rise significantly with increasing feedwater salinity, and RO or ED is usually more cost-effective when the feedwater salinity exceeds 500 mg/l TDS.

In raw water, dissolved inorganic compounds become dissociated and exist as either positively charged cations or negatively charged anions. The IX process removes virtually all inorganic ions by utilizing two types of ion exchangers: a cation exchanger and an anion exchanger. In each exchanger, an ion contained on the resin is exchanged for one of the ions in solution. Depending on the exchange resin used, the ion contributed will be sodium, hydrogen, or hydroxide. The process will continue until the ability of the resin bed to exchange ions is exhausted. The resin bed must be backwashed to dislodge and remove very fine particles, then treated with a regenerating chemical that will replace the ions that have been lost. The regenerating chemical may be common salt, sulfuric acid, soda ash, or caustic soda. Regeneration is completed by rinsing the bed of all regenerant chemical. The entire regeneration cycle takes about an hour, and the frequency depends upon design capacity and use. Daily regeneration is probably typical for an active system. Disposal of the regeneration waste can be difficult and expensive due to discharge restrictions.

IX equipment will consist of at least two types of resin in separate pressure vessels or in a mixed bed pressure vessel. Regenerating a mixed bed vessel requires separating the resin hydraulically, treating the resin, then remixing it for reuse. Mixed bed systems are frequently used as a final step in a demineralizing system to remove any remaining residual salts. A typical IX vessel is shown in Figure 7-3. Controls for new systems are usually automatic, requiring few operating decisions. Exchanger materials are either plastic, lined steel, or unlined steel. Pretreatment will generally consist of removal of particulate colloidal matter and turbidity by clarification and filtration, removal of oxidizing agents, and reduction of iron and aluminum.

IX has a number of disadvantages that eliminate it from consideration as a desalination process in Dare County. The primary disadvantage is cost. The high level of solids described in sections 5.7 and 5.8 and the relatively high allowed ion limits in product water high when compared to limits for which IX is typically used puts IX out of its most cost effective range. The frequent regeneration requirement is not conducive to a continuously operating plant. The nature of the regenerant waste poses disposal difficulties in the sensitive environment of Dare County.

7.2.4. Distillation

Distillation is a desalination process that removes impurities from feedwater by boiling, collecting the water vapor, and then cooling the vapor until it condenses. The freshly condensed water is virtually free of any contaminants and is called distilled water, distillate, or condensate.

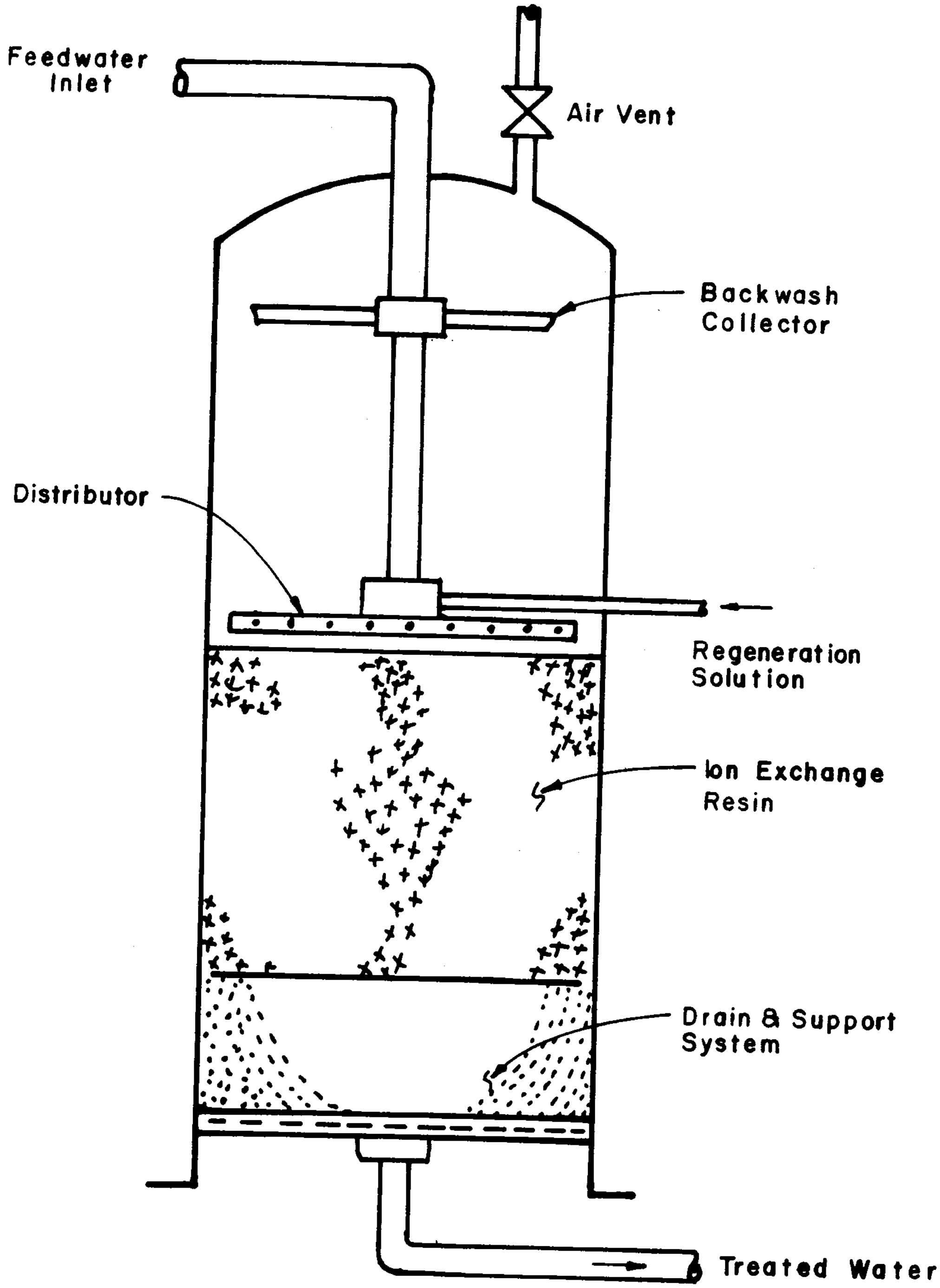


FIGURE 7-3. ION EXCHANGE VESSEL

When water boils it changes to a vapor, leaving behind the non-volatile impurities. When water vapor (steam) is cooled, it changes back to water which is free from impurities. Energy is required to turn water into steam, and energy is released when steam is turned back to water. All modern distillation process designs involve techniques to use the heat released by the vapor, as it condenses, to heat the feedwater. Thus, overall system energy requirements are reduced.

Three of the most common seawater distillation processes are multi-stage flash distillation, vertical and horizontal tube evaporation, and vapor compression. Each type of plant requires special materials of construction to minimize problems of pitting, galvanic action, stress corrosion cracking, and corrosion fatigue. Pretreatment is required to minimize scale formation, to remove oxygen and other gases, to remove suspended solids, to prevent foaming, and for disinfection.

Distillation consumes great quantities of energy to desalt water. It is rarely used for brackish water, and then only on the higher TDS brackish water source. With less than 8,000 mg/l TDS, other desalination processes will be more cost-effective.

7.3 COST COMPARISON AND RECOMMENDATIONS

Of the four available desalination options mentioned in Section 7.2, only reverse osmosis and electrodialysis are given a detailed cost comparison. Feedwater quality is clearly outside of the ranges in which ion exchange and distillation can be cost-effective.

RO and ED treatment systems are normally supplied and constructed by specialty contractors. Because of this, cost histories are not available for various components which make up the specialty construction package. A determination of probable plant cost is very difficult due to the relative newness of these processes (approximately 15 years of commercially supplied plants), and the lack of related experience in North Carolina. Therefore, budgetary quotations were solicited from eight specialty contractors who have experience with plants of the size and type needed for Dare County. The contractors were given the water analysis shown in Table 7-1 on which to base their budgetary costs. The analysis was based on Currituck Sound water, since it was considered a probable worst case water supply for desalination. Five quotations were received, as shown in Table 7-2. As can be seen, the cost provided by the ED contractor, for both capital and O&M costs, is greater than those supplied by any of the RO contractors. This was anticipated, since the TDS concentration of 6,900 mg/l is outside the normal cost-effective range of ED. A second quotation was requested of the ED contractor, with feedwater TDS reduced to 3,000 mg/l. This was done to determine the variation in ED costs with varying TDS as might be encountered in Dare County. As shown in Table 7-2, the ED cost still exceeds the RO costs.

The wide range in cost provided by RO contractors is indicative of the differing approaches these contractors will take towards plant design and also of the fact that the information provided to them was less than is required for a detailed cost estimate. Each contractor was required to make assumptions, and the exact amount of process equipment, pre- and post-treatment equipment, instrumentation, and building costs was certainly different with each contractor. These differences were ascertained as far as possible through correspondence. The opinion of probable plant costs provided in Section 5.7 and 5.8 represents the engineer's judgment of all these factors.

Due to the difference in costs described above, reverse osmosis is recommended as the most feasible desalination alternative.

7.4 RO FACILITY REQUIREMENTS

7.4.1. RO Process Performance

Previous discussions of reverse osmosis in this report have been brief, without giving any thought to describing performance characteristics, or to factors affecting performance. Typical measures of performance are salt rejection, membrane flux (flow through the membrane per unit time), percent recovery, and time between membrane cleanings. Factors which affect performance are operating pressure, feedwater temperature and concentration, recovery control, product pressure, and ionic constituents in the water. Several of these characteristics and factors are discussed below.

TABLE 7-1
PROBABLE FEEDWATER ANALYSIS
FOR COST COMPARISON

Parameters	Pretreated Water
Alkalinity (mg/l)	30.0
Chloride (mg/l)	3380
Fluoride (mg/l)	1.64
Hardness, Calcium (mg/l)	206
Hardness, Total (mg/l)	820
pH (units)	6.6
Total Dissolved Solids (mg/l)	6995
Sulfate (mg/l)	520
Sulfide (mg/l)	<0.1
Turbidity (NTU)	0.43
Calcium (mg/l)	80.9
Iron (mg/l)	0.037
Potassium (mg/l)	68.1
Magnesium (mg/l)	203
Manganese (mg/l)	0.011
Sodium (mg/l)	1628
Silicon (mg/l)	0.35

TABLE 7-2

RO AND ED COST COMPARISON
5 MGD PLANT

Vendor	RO Capital Cost ($ x 10^6)	RO O&M Cost ($ per 1000 gal)	ED Capital Cost ($ x 10^6)	ED O&M Cost ($ per 1000 gal)
A	4.5	0.64	--	--
B	4.0	0.51	--	--
C	6.2	NA	--	--
D	5.0	NA	--	--
E	--	--	6.2	1.41

Revised ED cost for reduced TDS.

E	--	--	5.5	0.95

7.4.1.1. Feedwater Concentration and Recovery. The concentration of dissolved solids in the feedwater is of prime importance in planning RO systems. The higher the concentration, the higher the feed pressure must be to overcome the feedwater's osmotic pressure. At constant pressure, the percent recovery will decrease as TDS concentration increases. Recovery is the percentage of feedwater which passes through the membranes to become product water. This is a critical parameter in determining RO feasibility since higher recoveries reduce the raw water requirements and the amount of RO membranes to be used, and minimize pretreatment costs and the amount of brine to be handled. Usual design is intended to utilize as high a recovery as possible without causing damaging scale formation.

A change in feedwater concentration may adversely affect RO performance. In the Yorktown aquifer in Dare County, there is potential for both upward and horizontal movement of higher TDS concentration water into the well field. Such movement would gradually increase feedwater TDS, resulting in higher osmotic pressures and lower recovery. With time, the plant capacity would be reduced. Proper well field design and operation can minimize salt water migration in the aquifer, but some increase in feedwater salinity can be anticipated with time. It is recommended that this increase be considered in the plant design.

7.4.1.2. Salt Rejection. Salt rejection is a percentage measure of how much salt is removed from (i.e., TDS retained by the membrane) the feedwater. Each ion is rejected to a different extent, and the overall rejection will be a weighted average of the rejection of each constituent. Salt rejection is relatively constant with the age of the membrane, but is reduced if the net operating pressure across the membranes is reduced.

Most RO membranes available today have excellent rejection characteristics for ionic constituents typically found in brackish water. Rejections greater than 90 percent are easily achieved, and 95 percent is possible, depending upon the feed pressure and ionic constituents. Greater salt rejection results in a purer product water. If the feedwater TDS is not too high, the product water will have TDS well below the statutory limits, and blending of some feedwater with product water may be possible. This will reduce both the amount of raw water and the number of RO membranes required.

For Yorktown aquifer water having a nominal TDS of 4,000 mg/l, the product water could be expected to have a concentration of 200 mg/l with 95 percent salt rejection. Bypassing and blending of only 5 to 7 percent of the raw water flow would be possible, which is not economical.

7.4.1.3. Ionic Constituents. Certain ions which are commonly dissolved in water are either difficult to remove or potentially harmful to the membranes. There is a saturation point for ions in solution after which precipitation of the ion occurs. If this occurs in an RO vessel, the precipitate will scale on the membrane, reducing the water flux across the membrane. The most common scalants are calcium carbonate ($CaCO_3$) and calcium sulfate ($CaSO_4$). Other possible scalants are barium sulfate ($BaSO_4$), strontium sulfate ($SrSO_4$), calcium fluoride (CaF_2), and silica (SiO_2). The critical location of scaling is on the brine side of the membrane, where concentrations are greatest. Dissolved iron and aluminum compounds can also be a source of scaling.

Scaling can be controlled by:

(1) Reducing recovery to avoid exceeding solubility limits.

(2) Removing the calcium ion (or other scaling ion) by pretreatment.

(3) Removing the carbonate/bicarbonate ion by adding acid.

(4) Adding a scale-inhibiting chemical.

The analysis of Dare County's Yorktown aquifer water does not indicate any unusual scaling problem. The design should consider pH adjustment and chemical addition for scaling control, as is done at most RO plants. There is some iron present in the test well, which may be naturally in the aquifer or may be coming from the test well's steel casing. It is recommended that a pilot plant be operated to determine the effect of iron from this well on RO membranes.

7.4.2. RO Equipment Requirements

An RO plant to produce water for the Dare County Regional Water System would consist of many RO membranes assembled in vessels and groups of vessels and staged. The major options to choose between on the RO process are as follows.

- Spiral wound permeators, consisting of sheets of membranes and spacers wound around a collection tube. Usually, several permeators are placed in series in a single pressure tube.
- Hollow fiber permeators, consisting of many hair like fibers in a bundle, located inside a pressure tube.
- Parallel single-staged systems, consisting of many permeators connected in parallel.
- Brine-staged systems, wherein the brine from the first stage becomes the feedwater for the second stage. This approach both maximizes recovery and minimizes brine volumes.
- Product-staged systems, where the products from the first stage are passed through a second stage to provide higher quality product water.

All of these options have much in common. Permeators are mounted on racks and connected by manifolds. Recovery is controlled by a valve on the brine manifold. Provisions must be made to sample and replace permeators individually and to clean permeator racks. RO systems usually have parallel racks of permeators that can be operated independently to increase or decrease plant flow. Several on-line instruments are needed to provide performance data for operating decisions.

Materials of construction must be carefully chosen for all parts of the RO system due to the potential for corrosion. Non-metallic materials should be used for all wetted parts wherever practical and economical. The high operating pressures also must be considered in material selection.

Pretreatment and other appurtenant systems are also required.

- Cartridge filtering system, to remove suspended particles down to 5 microns.

- Chemical feed systems as required by detailed process design. The most likely chemicals that will be needed are acid for pH control and a scale inhibitor.

7.4.3. Other Plant Facilities

Other equipment and facilities will be required at the RO water treatment plant. Remote facilities are discussed in sections 5.7 and 5.8. Facilities located at the plant are listed below.

- Treatment Plant Building. This building will house all the RO equipment, pretreatment equipment, chemical feed and storage systems, and transmission and transfer pumps. The building will also house mechanical and electrical equipment, and include personnel facilities such as operations room, laboratory, offices, conference rooms, locker rooms, etc.

- Clearwell. A clearwell will be required for collection of product water and as a source of water for various in-plant uses.

- Storage Reservoir. A ground storage reservoir will be needed to provide a buffer between plant production rate and pumping rate and for times when the treatment plant is shut down. The size of the reservoir should be based on total water demand, and available storage facilities. Since each distribution system has adequate storage facilities, the amount of storage required at the water treatment plant need not be excessive. A 1 MG storage reservoir is recommended.

- Transmission Pumps. Transmission pumps will supply treated water to the transmission main. Studies should be made during the plant design to determine the correct discharge pressure of the pumps, based on them operating alone and together with the Roanoke Island and Fresh Pond transmission pumps.

- Operations Room. There should be one room in the plant for centralized controls where operating decisions are made. This room would have all display instruments for monitoring plant processes, and contain control instruments for implementing process control decisions. The control system will include some microprocessor based controls. The master station of the water system telemetry system, recommended in Section 6.4, could be located here.

The plant should be arranged on site to accommodate future expansion. The initial capacity will be 5 mgd, with a building capacity to house 8 mgd or process units. The plant site will be arranged for a plant capacity of 12 mgd. The potential site on the Baum Tract in Kill Devil Hills will have more than adequate space.

Figures 7-4 and 7-5 provide schematic presentations of an RO plant for treating surface water and groundwater.

7.4.4. Plant Operations

If a desalination plant is constructed on the Outer Banks, it will be one of three plants supplying water to the Dare County regional water system. The effect that this will have on the transmission main is discussed in Section 6.3. The County should be aware that treated waters are not always compatible, due to differing concentrations of varying ions. It is possible that post-treatment will be required at one or more plants to make the waters compatible.

An RO plant composed of several parallel treatment modules can be operated at various flow rates simply by starting or stopping modules. Proper shutdown procedures must be followed to prevent damage to the membranes while inactive. Thus, varying the flow rate should be done in increments and not to match water demands required for short durations. The plant flow rate should be selected and set and maintained for as long as possible. Of the three plants, the RO plant would be the best one to base load during peak season, varying the flow rate at the other two plants to meet short term changes in demand. During the off-peak season, the RO plant should be run at a reduced capacity, rotating the modules daily.

Achieving a good balance between ease of operation and minimizing the total cost of producing water will not come automatically. There will be a period of trial and error during which time plant operators must work with various flow rates from each plant, during both peak and off-peak seasons, to achieve the best possible performance.

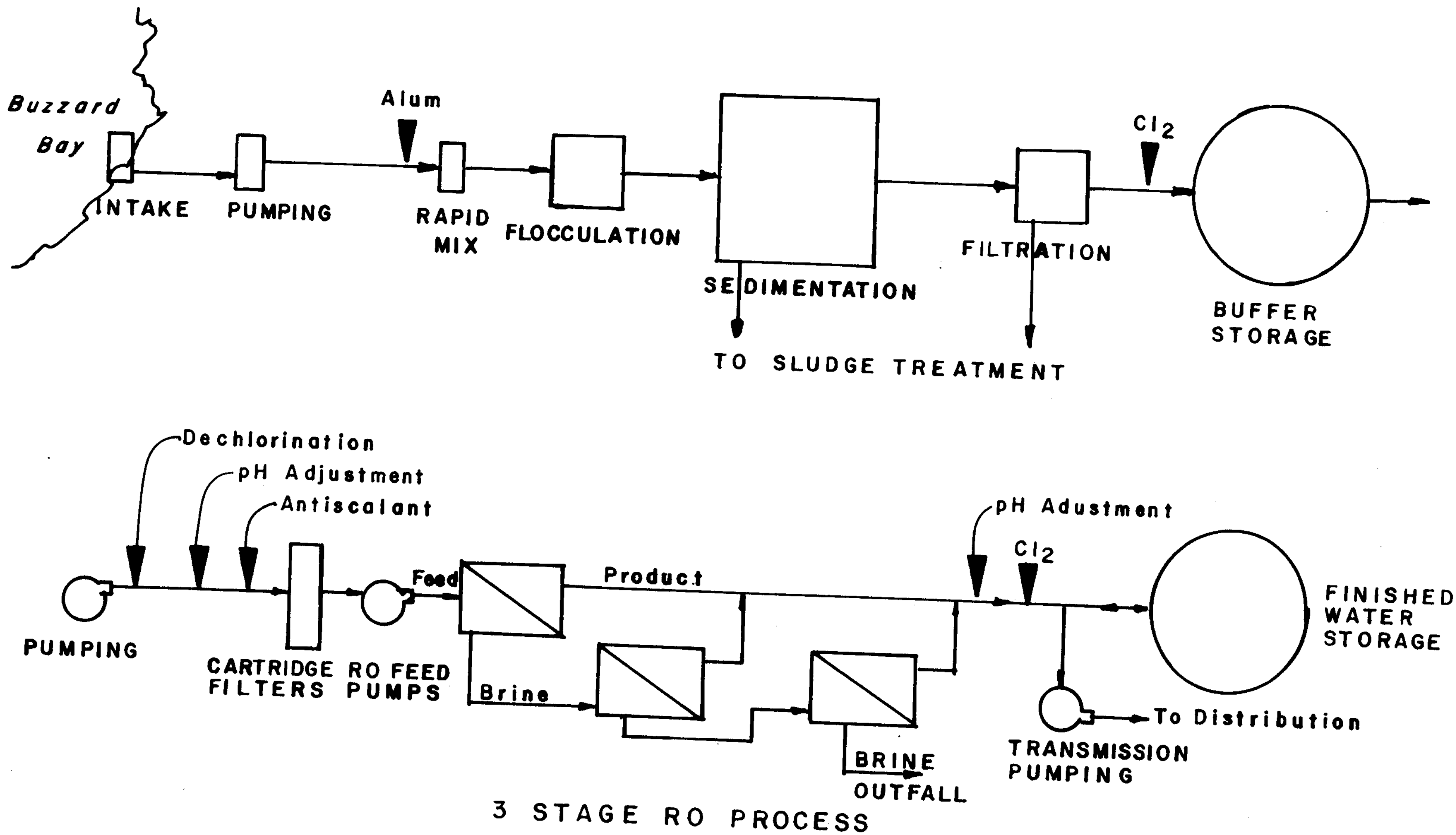


FIGURE 7-4 SCHEMATIC OF SURFACE WATER RO PLANT

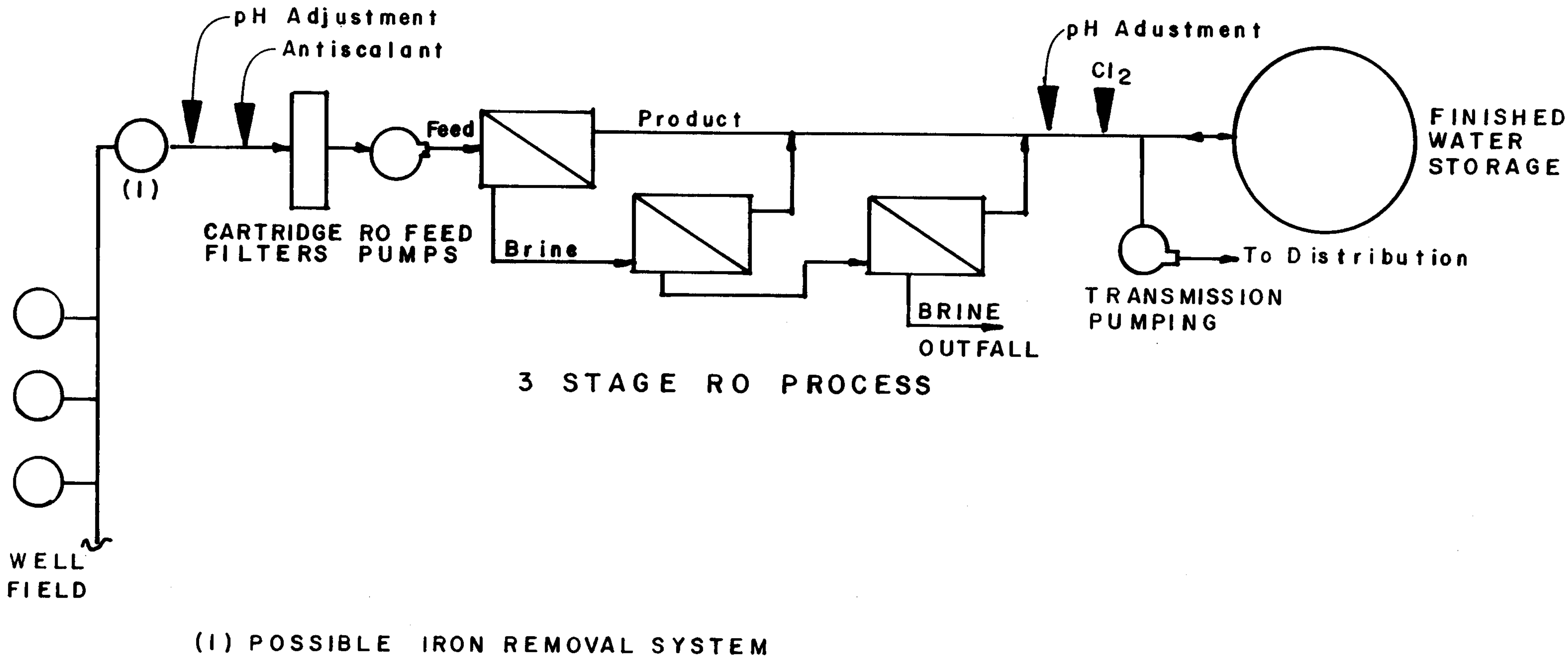


FIGURE 7-5 SCHEMATIC OF GROUND WATER RO PLANT

8.0 RECOMMENDED IMPROVEMENTS

8.1 IMPROVEMENT STAGES

As previously stated, the 1984 Comprehensive Report identified three stages for implementing improvements. The stages were chosen to meet the critical immediate and long-term water needs. Stage 1 improvements were needed to meet immediate deficiencies in the water system, and were scheduled for completion in 1985. Stage 2 improvements covered the period through 1995, and Stage 3 covered through 2005.

Certain Stage 1 improvements were implemented in 1984 and 1985. These were the raw water booster pumping station and upgrading the Fresh Pond Water Treatment Plant. Other improvements recommended were deferred, such as the Duck elevated tank and Wanchese distribution system.

This report continues with the same improvement stages as the 1984 Comprehensive Report. Stage 2 covers the period from the present through 1995, and represents an expansion of the water system from a nominal 6 mgd to 11 mgd capacity. This is the period for which immediate steps should be taken to implement the improvements. Stage 3 covers the period from 1995 through 2005, and includes another nominal 5 mgd expansion.

8.2 IMPROVEMENT OPTIONS

Sections 5, 6, and 7 have described several options that are available to Dare County for expanding the regional water system. The alternatives hinge primarily on the selection of a source of raw water. This report presents three main alternatives of raw water supply: Roanoke Island groundwater, mainland groundwater, and desalination of Outer Banks water. The most expensive alternative, in terms of both capital and life cycle costs, is mainland groundwater. Three different well field locations and two different pipeline routes were analyzed and all proved to be more expensive than the least costly desalination option.

Expansion of the Roanoke Island groundwater supply was analyzed, making certain assumptions. This analysis indicates that expansion of this supply is less costly than desalination on the Outer Banks. The assumptions, however, are critical to the analysis and to the alternative's feasibility. It was assumed that sufficient water could be obtained from the aquifer to meet both Stage 2 and Stage 3 demands. However, the validity of this assumption rests with some past reports and data for which relatively little actual subsurface investigation was done. Recent experience with shallow wells on Roanoke Island indicates that salt water encroachment is occurring. Expansion of this supply should not be undertaken until more subsurface investigation is done and study shows that the aquifer is safe from salt water intrusion. Despite past data, it seems unlikely that expansion beyond Stage 2 demands will be possible. This is discussed further in Section 5.4.

Two desalination alternatives were considered: brackish surface water from Albemarle or Currituck Sound, and brackish groundwater from the Yorktown aquifer. As presented in Sections 5.7 and 5.8, development of the Yorktown aquifer is both less expensive and more reliable than a surface water supply. Groundwater modeling will be ongoing during early project design, but indications are that sufficient groundwater is available for both Stage 2 and Stage 3 development. The least costly desalination process is reverse osmosis.

The desalination of Yorktown aquifer by reverse osmosis is the recommended alternative, based on cost and reliability. The sections which follow describe what improvements must be made to the regional water system to expand for Stage 2 and Stage 3 demands while implementing the recommended alternative. These improvements have been described elsewhere in this report, and are repeated here to provide a concise discussion and to aid Dare County in their implementation.

8.3 WATER SUPPLY

8.3.1 Stage 2

The 5.0 mgd RO plant will require between 5.88 and 7.14 mgd of raw water. This equals 1.18 to 1.43 mgd (820-990 gpm) per million gallons of plant capacity. Two wells per million gallon plant capacity is preferred. Thus, initially, 10 wells will be required. The wells should be constructed of inert materials, preferably PVC or fiberglass, although a high quality stainless steel is also acceptable. The pump should be all stainless steel. Drilling techniques should preclude any drilling mud entering the water production zone. Anticipated well diameter is 10-inches, with intake screen depth of 300-500 feet.

The wells should include a superstructure for housing pump motors, piping, electrical equipment, and instrumentation. Provisions for removing the pump should be included.

Well field layout should make maximum possible use of County and other government owned land on the Baum Tract in Kill Devil Hills. The proper orientation of wells on this tract should be determined during design.

Raw water mains should be constructed of fiberglass pipe. The initial pipe design should make provisions for expansion of the field in Stage 3. Piping design should preclude the entrance of oxygen into the raw water.

Each well should be provided with a flowmeter, conductivity meter for water quality monitoring, remote control starting from the RO plant, and a drawdown gage and recorder.

The well field design should include monitoring wells in critical locations. The most probable direction for salt water intrusion into the brackish water aquifer is from the east. Monitoring wells should be constructed with the same screen depth and length as production wells, between the well field and the ocean. The exact number and location of these wells depends upon final well field layout.

North Carolina public health regulations require that redundant well capacity be installed. The total well field capacity must be sufficient to supply average daily flow in 12 hours, to allow each well to pump half time. Average daily flow is defined in the regulations as 400 gpd per connection. Based upon the existing 9,400 connections receiving water from the regional system, 400 gpd per connection, and 475 gpm average yield for existing Roanoke Island wells, 11 wells are required, whereas only 10 are now existing. If the Stage 2 Yorktown well field will supply twice the number of connections, and the two fields are considered together for computation of redundancy, a total of 22 wells will be required, plus or minus one or two wells depending upon final RO well capacity. Thus two additional wells are required somewhere in the well fields. The recommendation of where these wells should be placed is deferred until early well construction is complete in the Yorktown well field. For determining project costs, they are shown as being constructed in the Roanoke Island well field.

8.3.2. Stage 3

Stage 3 will require increasing by 140 percent the raw water supply for the RO plant. Well design and piping design should be similar to that for Stage 2. Depending upon final development plans for the RO plant, the construction of some of the Stage 3 wells may need to be advanced to the latter part of Stage 2. It is possible for an RO plant to be expanded in modules over several years. For each 1 mgd of additional RO capacity, two more wells will be needed.

There will not be sufficient area on the Baum Tract for the full Stage 3 expansion. Additional sites off the tract should be acquired. Pipelines for these wells may be extensions of Stage 2 pipelines if well sites are conveniently located. Current State regulations require that well sites have an area of at least 31,420 square feet (100 ft radius from the well).

As the well field expands off the Baum Tract, additional monitoring wells will be required.

Initial operation of the Stage 2 well field will provide valuable data relative to well field expansion and possible salt water intrusion. This data may reveal that the well field capacity is limited in the vicinity of the plant, and that expansion must be located at some distance to the north or south. The cost of well sites, constuction, and raw water pipelines should be compared to the cost for developing the brackish surface water supply. Additional factors which would enter into this determination will be any advances in desalination technology, and continued growth possibilities beyond the Stage 3 period.

8.4 WATER TREATMENT

Reverse osmosis is a process that lends itself to expansion in relatively small increments without much lead time, if original plant design allows for this. The required Stage 2 capacity is 5.0 mgd, to be on line by 1989. Stage 3 expansion is 7.0 mgd, required by 1995. However, it is possible to initially design the plant for several 1.0 mgd increments to be added between 1989 and 1995, thus maintaining spare capacity for a longer period of time. The initial design should determine the County's financing capability and expansion preferences and determine the actual size for plant buildings and support systems to allow for incremental expansion.

8.4.1 Stage 2

Stage 2 water demands will require a 5.0 mgd treatment plant. The plant will consist of RO membrane modules, high pressure pumps, cartridge filtration, preand post-treatment chemical systems, chlorination, clearwell, storage reservoir, transmission pumps, instrumentation, operations and control room, employee facilities, offices, storage, laboratory facilities, and provisions for receiving visitors. Plant facilities that are not directly related to treatment should be sized for the entire plant, including Stage 3 expansion.

It is recommended that an installed spare pump be included in the RO plant. Although this involves more piping, the installed spare is much more valuable than a spare pump in storage. The spare pump can also serve as the feed pump for a new RO increment. The lead time for RO permeators is approximately 2-3 months less than for the high pressure pump, and having this installed spare reduces the time required to bring an increment on line. Installation of a new spare pump will lag by the 2-3 months, which is acceptable.

Plant process design should take into account the potential for salt water intrusion into the brackish aquifer. An increase in feedwater salinity should be countered by raising feed pressure, requiring pump modification, or reducing recovery, which effectively reduces plant capacity. The effects of salt water intrusion can be minimized with proper process design.

No recommendation is made at this time concerning exact RO permeators to be used or for pretreatment chemicals. Water analysis does not indicate any advantage to either spiral wound or hollow fiber membranes. Pretreatment needs are somewhat dependent upon the specific permeators supplied and the degree of salt rejection they achieve. The water analyses to date do indicate, however, the need for some degree of pretreatment to control scaling. No recommendation is made relative to pretreatment for iron, pending the results of the pilot plant work.

On-site storage should be a 5.0 million gallon ground storage reservoir, according to Dare County's preferences, as discussed in Section 7.4.3.

The telemetry system discussed in Section 6.3 should be included in the desalination project if it is not implemented earlier. The telemetry system should consist of remote control of all wells and system pumping, and remote monitoring of all critical parameters in the regional water system. The telemetry system previously designed and bid is comprehensive for the water system as it now exists. Additional design will be required to incorporate the RO plant project.

8.4.2 Stage 3

Stage 3 involves expanding the RO plant from 5 mgd to 12 mgd. The three treatment plants will then have a combined rated capacity of 15.9 mgd (see Sections 6.3 and 6.4). The Stage 3 expansion may in part be in 1.0 mgd increments, as discussed above. This expansion to 12 mgd will necessitate expanding chemical and filtration pretreatment, high pressure pumps, RO modules, chlorination capacity, and transmission pumping.

Expansion of on-site storage should not be required. However, Dare County's preferences regarding the amount of storage relative to plant size may dictate the need for an additional 5.0 MG of ground storage.

Stage 3 plant layout and site utilization should consider treatment needs beyond the study period described in this report. An increase or decrease in the rate of growth and ultimate water demand could cause the plant expansion to be revised upward or downward. Also to be considered are any improvements in RO equipment, which could result in new recommendations. Since the RO process and industry are still young relative to other water processes, it is not inconceivable that significant technological improvements could be made before the time for Stage 3 implementation.

8.5 TRANSMISSION SYSTEM

The recommended water system expansion results in minimal improvements needed for the transmission system. The transmission pumps at the RO water treatment plant must be carefully designed to allow each treatment plant to pump at capacity over a wide range of flow conditions.

As mentioned in the 1986 Distribution System report, additional ground storage at the Kitty Hawk pumping station would improve pumping station operation.

8.6 PROJECT COSTS

Table 5-10 shows probable capital costs for the recommended improvements for Stage 2 and for the Stage 3 expansion. Costs of improvements to the Dare County distribution system are not included. The costs are for third quarter 1986 construction conditions, and should be updated prior to implementation.

Not included in the capital cost is membrane replacement. Although a capital improvement, this cost is built into and paid from the water rates.

8.7 FURTHER STUDY

The findings and recommendations of this study and report are valid for the projected population growth and water use rates. If any significant change in either of these rates occurs, the County should conduct an additional study to be certain that the improvements are still necessary and are properly sized.

The pilot plant study for iron pretreatment should be continued until definitive results are obtained.

The County has recently initiated a study of brackish surface water quality in Currituck and Albemarle sounds, to better evaluate them as future water supplies. This study will provide valuable base data, and should be continued to its intended completion.

The behavior of the Yorktown aquifer should be studied as pumping begins and continues. The possibility of salt water intrusion should be assessed through monitoring of water levels and salinities in all production wells and monitoring wells. This data may result in recommendations for changes in groundwater management procedures.

This study should be updated in 1991. The selection of this year is based upon having up-to-date information before implementing Stage 3 improvements. The updating should be comprehensive, covering all aspects of the water system. The distribution system study should also be updated at that time, and possibly incorporated into one report.

APPENDIX A
ROANOKE ISLAND GROUNDWATER

This appendix describes the hydrogeologic system, discusses the systems that influence it, outlines the effects of withdrawals of groundwater, and predicts the effects of increased withdrawals caused by additional groundwater development.

Extensive hydrogeolgical study of aquifers beneath Roanoke Island and surrounding areas has been conducted by the United States Geological Survey and North Carolina Department of Natural Resources and Community Development. These studies have included drilling of boreholes, geophysical logging, and water sampling. The purpose of the studies was to determine the geological conditions and the extent of fresh water available. This appendix does not provide full data required for well field planning and design, but does provide significant guidance. The results of the studies listed in Section 1.5 of this report were used extensively in the preparation of this appendix.

A.1 Hydrogeologic Framework

Roanoke Island is underlain by a series of gently dipping sediment beds that have been deposited over time. Deposition occurred during periods of sea level transgressions, and weathering and erosion processes acted during sea level regressions. The area has been exposed since the last sea level rise during the Pleistocene Epoch when worldwide sea level rose in response to melting of the glacial ice sheets.

Figure A-1 is a stratigraphic section of the sedimentary wedge under eastern North Carolina, including Roanoke Island. This column is based on data from key test wells drilled in the area. For the purpose of this groundwater investigation, only the upper 500 feet of sediments and their water bearing characteristics are of interest.

The Miocene and post Miocene units contain six significant hydrogeologic units: three aquifers and three confining beds. Significant exploratory wells are located on Figure A-2, and shown in cross section on Figures A-3, A-4, and A-5.

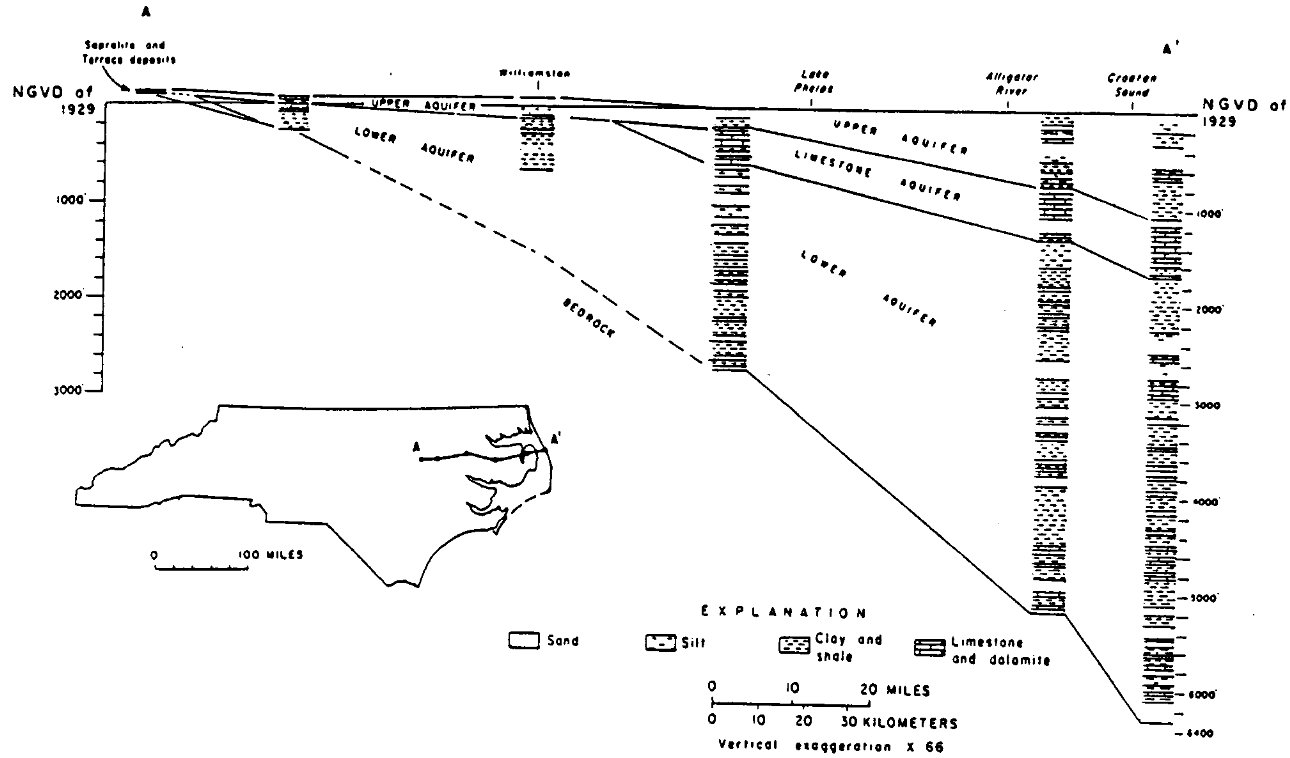
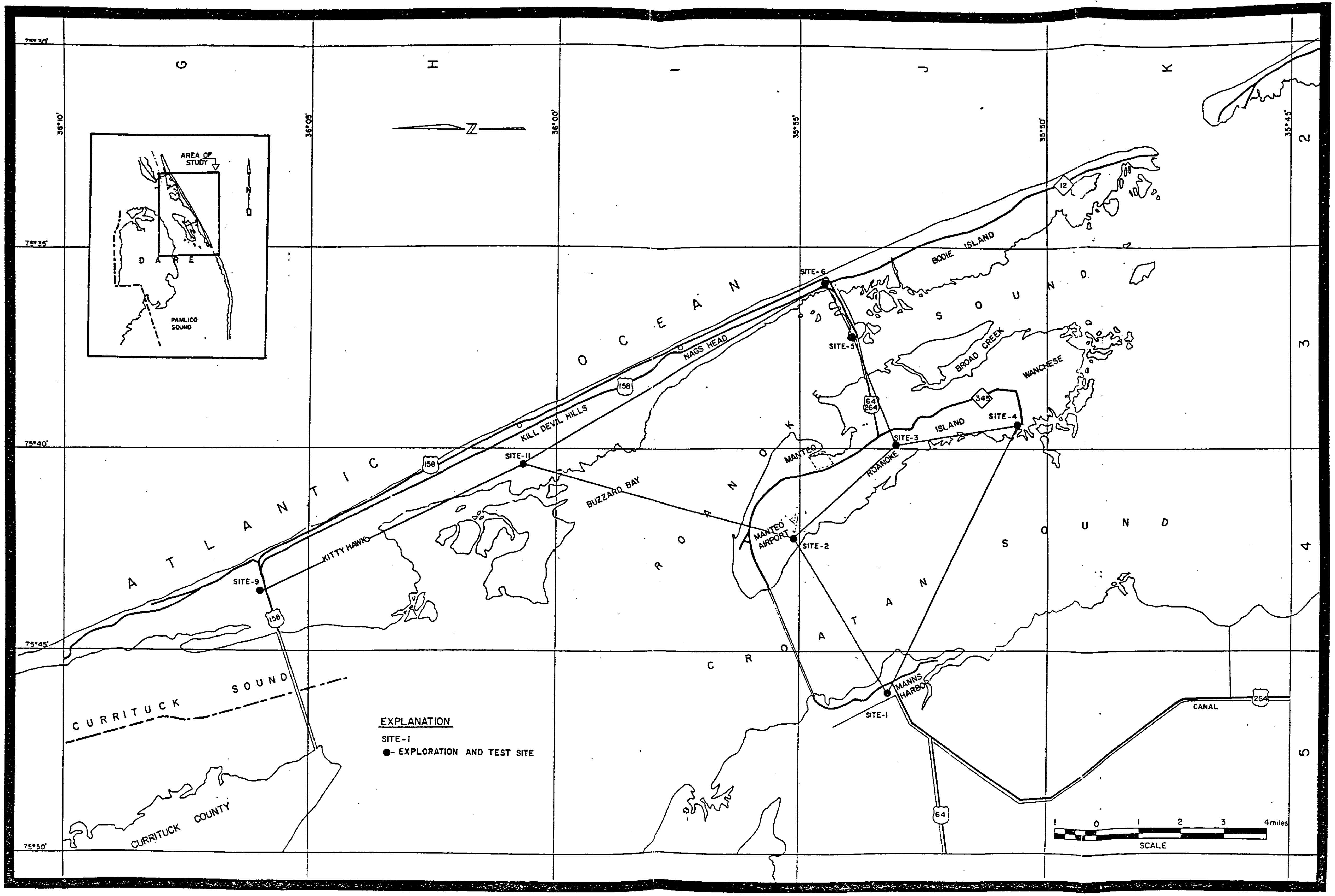


FIGURE A-1 STRATIGRAPHIC FRAMEWORK. (FROM HEATH, 1980)



FROM PEEK, ET AL 1972

FIGURE A-2
MAP SHOWING LOCATION OF STUDY AREA, TEST SITE AND CROSS-SECTION

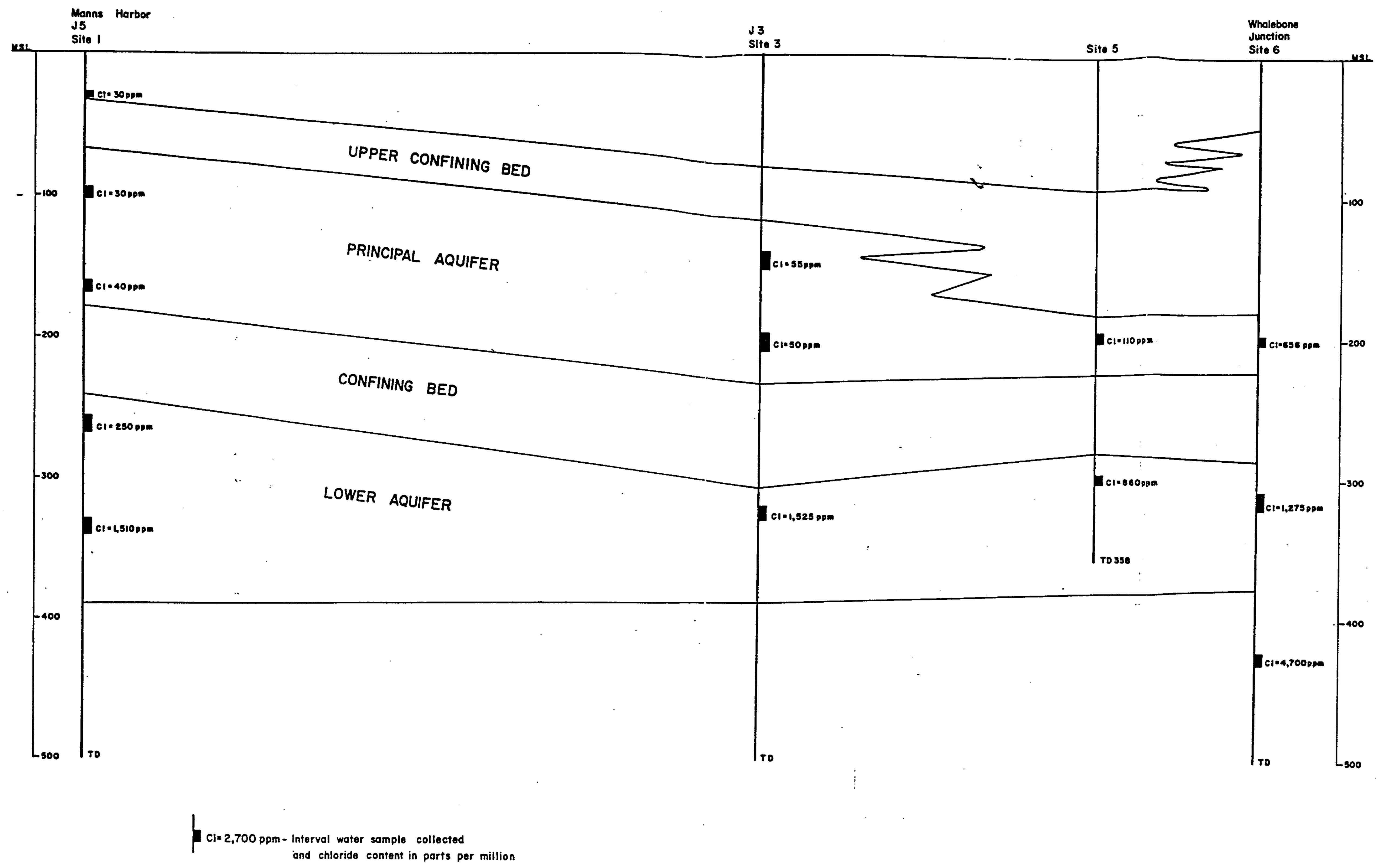


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FIGURE A-3
HYDROGEOLOGIC CROSS-SECTION, SITE I TO SITE 6

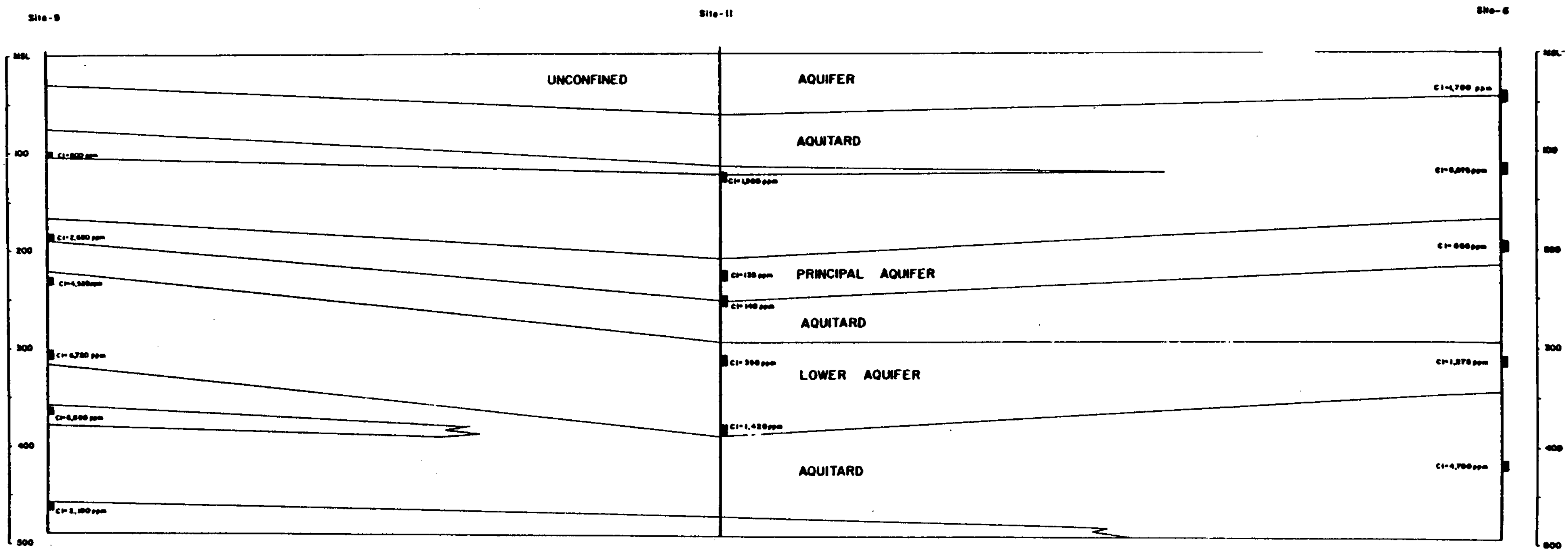

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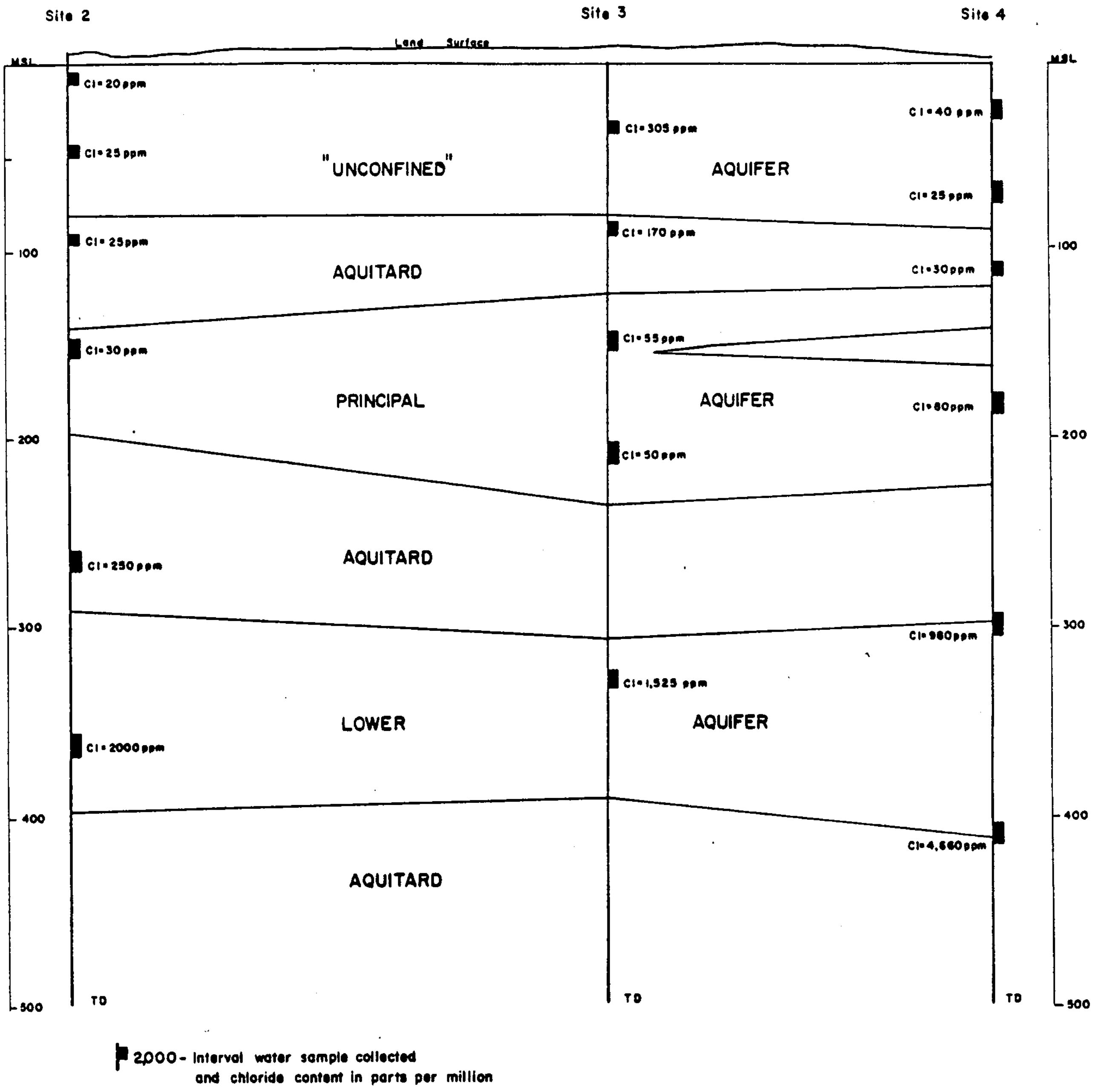
FIGURE A-4

HYDROGEOLOGIC CROSS-SECTION, SITE 9 TO SITE 6



FROM PEEK, ET AL 1972

FIGURE A-5
HYDROGEOLOGIC CROSS-SECTION, SITE 2 TO SITE 4

The uppermost aquifer extends from the surface to depths of 50 feet on Roanoke Island. This aquifer thickens to the east and reaches a maximum thickness of 100 feet at Kitty Hawk. In most places, this aquifer acts under water table conditions, but is semi-confined at some localities and depths due to the presence of clay lenses. The aquifer consists mainly of medium grained sands with some interbedded silts and clays. This aquifer is recharged by precipitation which falls on Roanoke Island. Other than providing water to private wells the significance of this aquifer is to serve as a recharge reservoir for the underlying aquifer system.

The upper aquifer is underlain by clays and interbedded clays and sand which varies in thickness from 35 to 100 feet. This stratum is not homogeneous and should be envisioned as a series of lenticular beds that allow transmission of water in a vertical direction. It serves as a semi-confining bed between the upper aquifer and the principal aquifer beneath it.

The principal aquifer is a water bearing zone which varies in thickness from 110 feet on Roanoke Island to only 45 feet at Nags Head. This aquifer consists of fine to coarse sand with a predominance of medium grained sand with some shell fragments and clay lenses. The principal aquifer serves as the source of water to the Dare County water system and domestic and industrial wells on Roanoke Island. The depth to the top of the principal aquifer varies from about 100 feet on Roanoke Island to greater than 200 feet on the Outer Banks.

A clay layer 60 to 100 feet thick separates the principal aquifer from a lower confined aquifer. The depth to the top of the clay layer varies from 175 to 230 feet. This stratum is predominately a homogeneous clay which acts as a good aquitard between the principal aquifer and the lower confined aquifer.

The lower confined aquifer is a fine to medium grained sand with interbedded clays and some shell fragments. The top of this unit is 240 to 300 feet deep and is from 90 to 150 feet thick. Permeability of this unit decreases with depth and it is not a known water supply source.

Below the lower confined aquifer is a unit consisting of interbedded clays and fine sands of low permeability. This aquitard varies in thickness from 10 to 100 feet, and is found below 315 feet to 400 feet. Figure A-6 shows the hydrogeologic framework.

A.2 Aquifer Hydraulics

The ability of aquifers to transmit water to wells is of prime importance to the suitability of the aquifer as a water supply. Analysis of the hydraulic characteristics of the Roanoke Island strata was based on pumping test data from over 16 supply wells and test wells. Hydraulic conductivity, transmissivity and storativity were determined to evaluate aquifers. Table A-1 is a summary of the principal hydraulic characteristics of the unconfined upper aquifer and the semi-confined principal aquifer. This data indicates a wide variation in transmissivity in the principal aquifer, which is caused in part by differences in aquifer thickness and permeability. Figure A-7 shows that higher transmissivities occur to the southeast, where aquifer thickness is greatest. Transmissivities in the upper aquifer range from 20,000 to 70,000 gal/day/ft. The major factors influencing transmissivity are the variation in aquifer material and the presence of clays. No hydraulic data is available for the lower confined aquifer; however, stem tests conducted during test well drilling indicate a yield of 30 to 40 gpm can be obtained at most sites on Roanoke Island in the lower aquifer.

The upper aquifer and the principal aquifer appear to be hydraulically connected, with the upper aquifer serving as a source of recharge to the principal aquifer. Plots of 16 aquifer tests in the principal aquifer indicate that recharge occurs as soon as 30 minutes after pumping begins. It is possible that recharge is occurring almost instantaneously in some wells, as indicated by no deflection in the drawdown curve.

Recharge from the upper aquifer to the principal aquifer occurs through the semi-permeable clay lenses which separate them. This vertical recharge from the upper unit to the lower is commonly referred to as leakage. Leakage in the aquifer system creates hydraulic characteristics which cause the lower

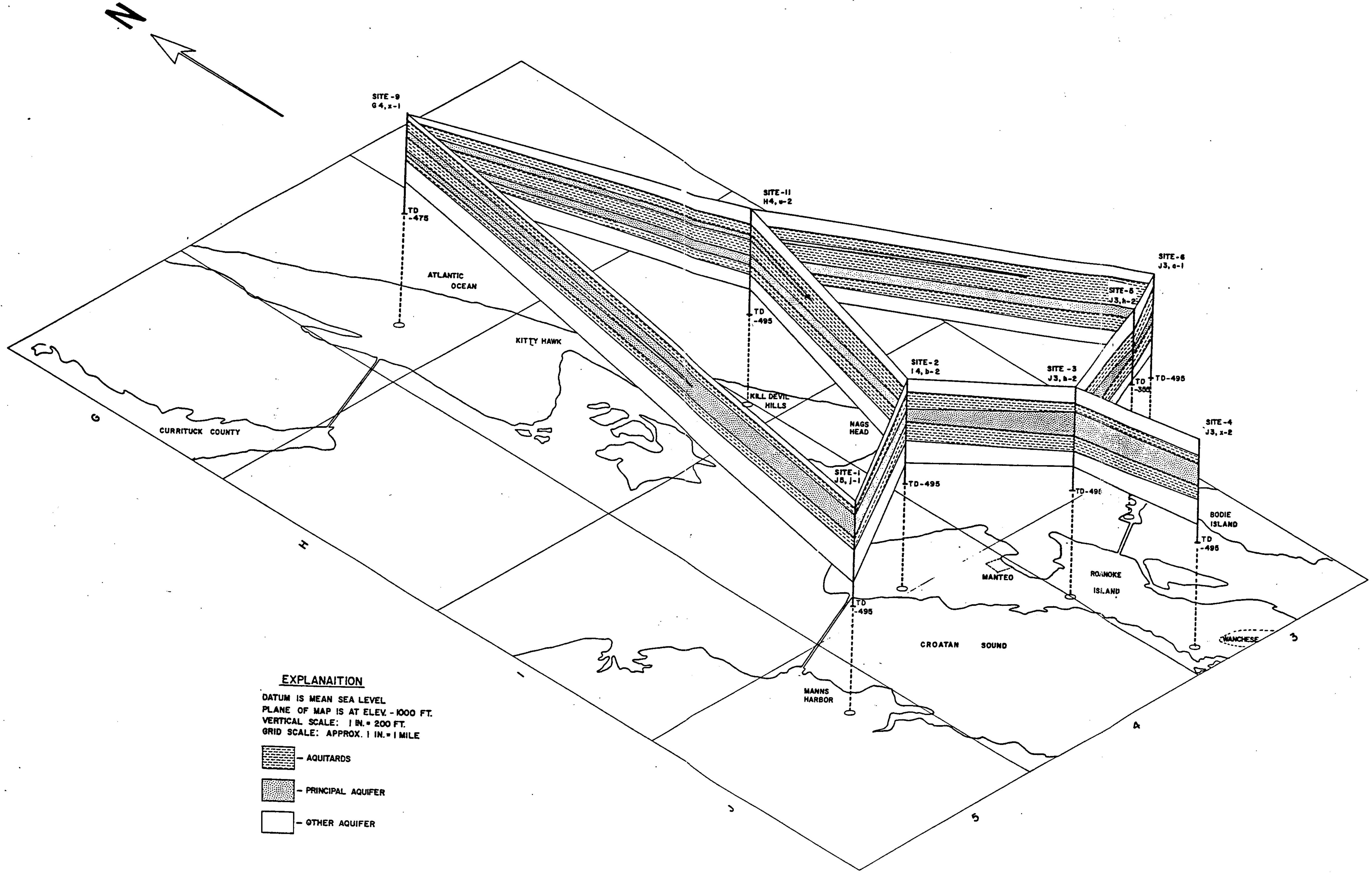


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FIGURE A-6
HYDROGEOLOGIC DIAGRAM, DARE COUNTY BEACHES

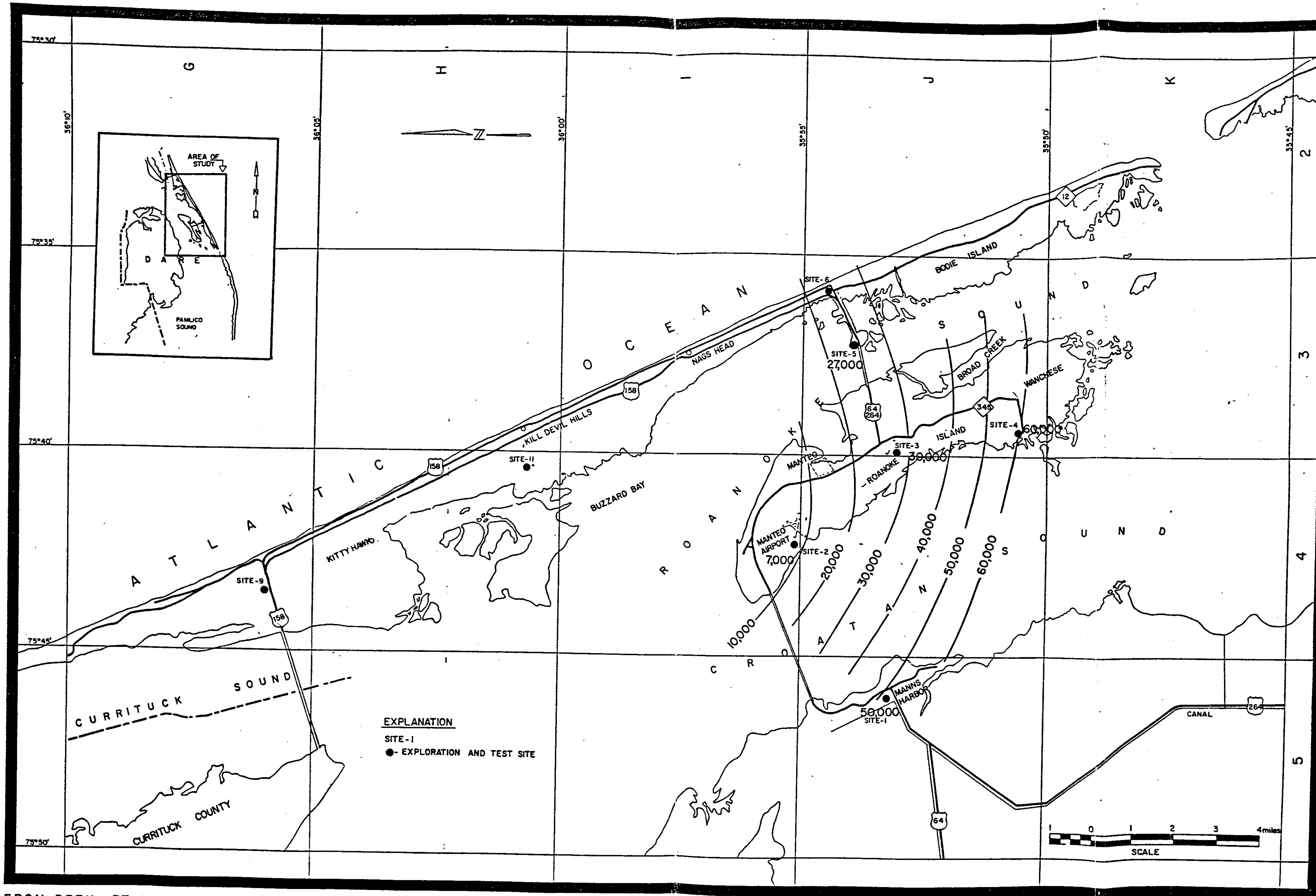

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FIGURE A-7
TRANSMISSIVITY OF PRINCIPAL AQUIFER

TABLE A-1

AQUIFER HYDRAULIC CHARACTERISTICS

Strata	Depth to Top feet	Average Thickness feet	Average Permeability gal/day/ft^2	Transmissivity gal/day/ft	Storativity Coefficient
Water Table Aquifer	0	50	6,000	20,000-70,000	.003
Aquitard	70	40	20	1,000	—
Principal Aquifer	110	50	8,213	20,000-150,000	.027

principal aquifer to behave under unconfined conditions rather than confined or semiconfined conditions. Calculations of the volume of leakage, known as leakance, indicate a downward leakance of 19.7 gal/day/ft^2. This volume of water is significant in that it can be recovered by wells in the principal aquifer and is therefore available as water supply to the Dare County system.

A.3 Water Quality

Water quality data for the upper aquifer, the principal aquifer, and the lower confined aquifer are shown in Table A-2. For evaluating aquifers as fresh water supplies, the chloride content of each water bearing zone is of utmost importance. Chloride concentration in excess of 250 mg/l indicates the aquifer is unsuitable for water supply development unless desalination is provided. It should be noted that both vertical and horizontal locations of water bearing zones greatly affect the chloride concentrations. The difference in salinity is generally caused by variation in permeabilities and hydrologic conditions at each site.

The chloride content of the upper aquifer varies from 15 to 8000 mg/l. High cloride concentrations are common along coastlines where the zone of mixing between fresh water and brackish water from the sounds is present. Figure A-8 shows relative chloride concentrations in the upper aquifer.

The chloride content in the principal aquifer is more uniform. It varies from 30 to 140 mg/l on Roanoke Island but increases east of the island.

The chloride concentrations in the lower aquifer are generally related to aquifer depth. In the upper part of the aquifer, concentrations from 250 to 1500 mg/l are common. In the lower portion of the aquifer, concentrations range from 1500 to more than 4600 mg/l.

Only water from the upper aquifer and the principal aquifer are suited for conventional water treatment and supply. Generally, the water in these aquifers is high in hardness but can be treated to provide potable water. In some areas, iron may be present in sufficient concentration to require its removal.

TABLE A-2

COMPREHENSIVE WATER ANALYSIS OF PRINCIPAL AQUIFERS IN NORTHEASTERN DARE COUNT

(Analyses made by U.S. Geol. Survey. Results of Chemical Analyses in Milligrams Per Liter)

Site	Well	Screen	Silica (Sio(2))	Aluminum (Al)	Iron (Fe)	Manganese (Mn)	Calcium (Ca)	Magnesium (Mg)	Sodium (Na)	Potassium (K)	Carbonate (Co(3))
1	J-5, j-1a	332-342	14	.370	.000	.014	27	69	1030	51	0
	1b	258-268	15	.204	.000	.012	22	27	245	22	0
	1c	162-172	24	.165	.096	.000	68	7.8	66	8.5	0
	j-2a	34- 39	5.3	.000	.000	.000	25	3.8	16	.9	0
	2b	92-102	13	.048	.000	.016	29	2.3	14	1.2	0
2	I-4, v-1	10- 14	7.9	.000	.000	.014	6.7	.7	6.9	.9	0
	v-2a	360-370	18	.830	1.132	.016	21	73	1340	55	0
	2b	264-274	9.9	.374	.081	.015	8.3	8	350	13	0
	2c	150-160	34	.176	.040	.006	12	6.7	94	13	0
	v-3a	50- 55	15	.096	.000	.021	40	2.7	29	1.4	0
	3b	92- 97	17	.195	.109	.008	28	2.0	41	3.8	0
3	J-3, f-2a	40- 45	12	.199	.000	.014	139	12.0	116	3.0	0
	f-2b	84- 89	19	.043	.000	.008	47	8.4	64	3.0	0
	f-2c	324-334	16	.366	.198	.026	22	36	1070	37.0	0
	f-2d	200-210	17	.000	.000	.014	61	8.0	34	5.8	0
	f-3a	134-144	32	.239	.248	.000	71	10	34	8.9	0
4	J-3, y-2a	21- 31	6.9	.038	.000	.008	27	8.6	23	1.4	0
	y-2b	63- 73	13	.139	.050	.012	32	4.5	46	2.3	0
	y-2e	189-199	21	.067	.000	.052	43	5.8	56	7.5	0
	y-3a	105-115	37	.122	.000	.022	43	11	42	9.2	0
5	h-2b	294-304	16	.294	.295	.004	8.7	12	750	23	0
	h-2c	197-207	22	.215	.078	.000	24	3.4	167	10	0
9	G-4, x-1a	461-471	2.6	.240	.000	.046	47	37	1260	47	0
	x-1b	363-373	2.2	.667	.000	.204	90	184	3860	136	0
	x-1c	305-315	6.2	.864	.000	.051	99	198	3980	126	0
	x-1d	228-238	12	.696	.000	.040	58	155	2840	78	0
	x-1e	187-197	1.6	.474	.059	.014	45	96	1740	50	0
	x-1f	101-106	15	.428	.381	.014	65	60	475	30	0

TABLE A-2 (continued)

Carbonate (HCO(3))	Sulfate (SO(4))	Chloride (CL)	Fluouride (F)	Nitrate (NO(3))as N	Phosphate (PO(4))	Dissolved Solids (Sum)	Hardness (as CaCO(3))	pH	Color	Alk (CaCO(3))	Spec. Cond (umhos @ 2 degress C
548	87	1470	.2	3.30	.038	3040	370	8.2	10	548	5000
448	.6	244	.4	1.70	.054	780	165	8.1	10	367	1300
398	1.6	29	.2	.20	.000	405	202	7.7	10	326	650
90	13	17	.1	1.20	.000	131	78	7.2	0	74	230
106	.6	18	.1	.10	.000	131	82	7.4	0	87	220
20	.2	15	.1	.0	.000	48	20	6.4	0	16	85
518	15	2050	.3	3.00	.000	3850	354	8.1	15	425	6500
594	13	238	.8	2.00	.120	948	54	8.2	140	487	1550
315	.8	19	.5	.07	.190	339	58	7.6	30	258	525
178	8.2	20	.1	.00	.000	205	111	7.4	0	146	350
162	10.0	20	.3	.07	.000	203	78	7.3	0	133	330
316	2.4	283	.2	.10	.000	726	395	7.8	25	259	1280
104	3.4	152	.1	.07	.000	349	152	7.7	0	85	720
600	11	1480	.6	1.0	.066	2980	202	8.1	20	492	5000
252	6.2	43	.2	.00	.000	301	185	7.8	0	206	500
299	.6	42	.2	.00	.000	349	218	7.6	5	189	550
131	7.4	30	.1	.02	.000	169	103	7.5	0	107	300
164	26	32	.1	.05	.000	238	99	7.6	10	134	390
250	3.0	44	.2	.00	.043	306	132	7.7	5	205	500
197	25	45	.3	.2	.000	311	152	7.7	10	162	500
668	2.4	828	1.1	2.9	.15	1990	70	7.9	50	548	3200
412	1.8	90	.3	.05	.093	525	74	7.8	40	338	900
294	9.4	1970	.4	.7	.000	3520	268	7.7	10	241	5800
234	74	6480	.5	.2	.000	10900	938	7.6	10	192	18000
476	46	6640	.4	.09	.000	11300	1060	7.9	20	390	17000
436	35	4710	.7	2.8	.076	8120	773	7.7	30	358	12200
381	27	2900	.5	.07	.000	5050	506	8.2	30	312	9000
451	2.2	780	.5	.7	.093	1660	407	8.1	30	370	2800

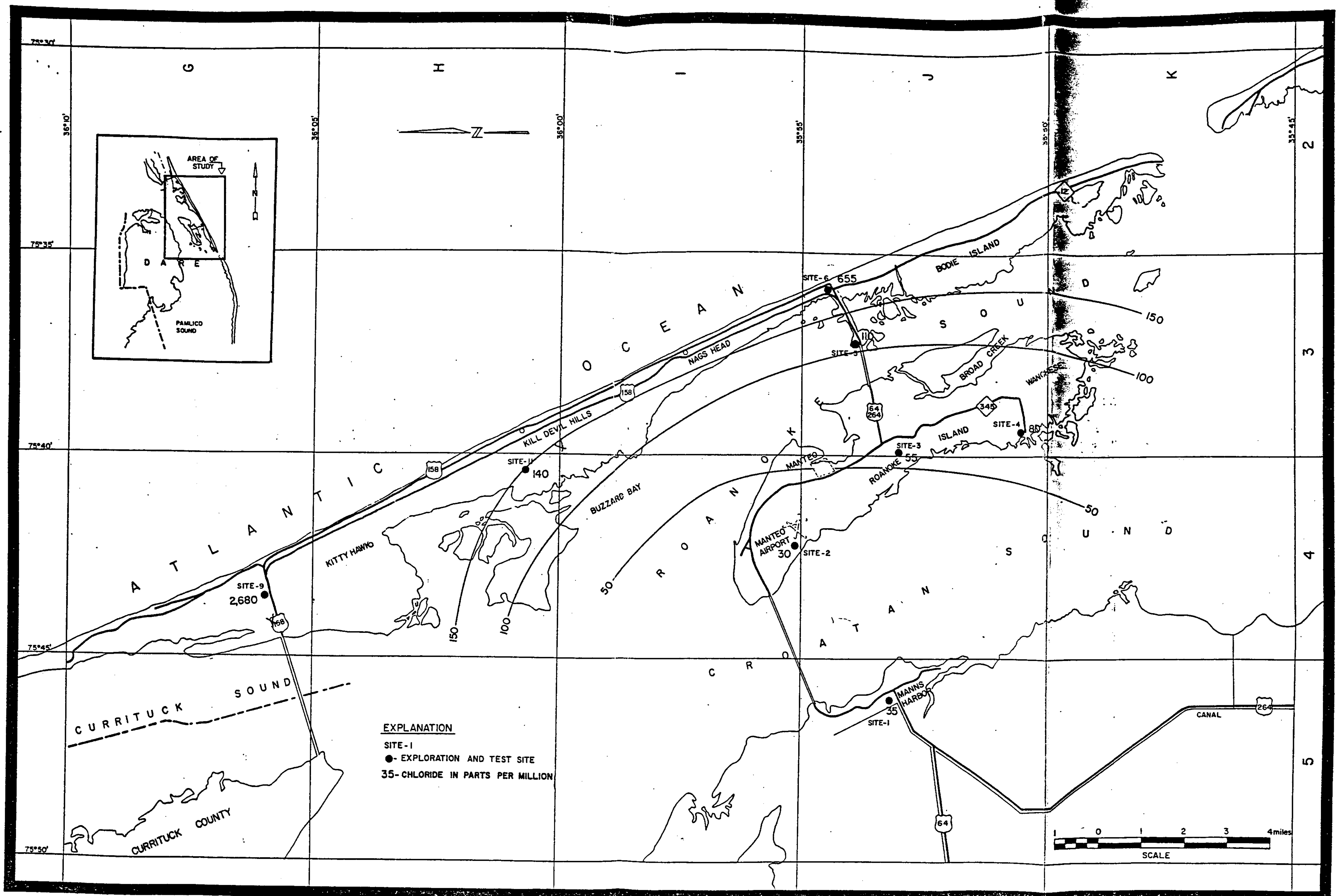

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FIGURE A-8
CHLORIDE CONTENT OF WATER IN PRINCIPAL AQUIFER

A.4. Recharge and Discharge

The recharge area for the upper aquifer is the 12,400 acre land surface of Roanoke Island. Based on an annual precipitation of 52 inches per year and a 33 percent recharge, the annual water available for recharge to the water table aquifer is 5,777 million gallons. Not all of this water leaks to the principal aquifer. Some is lost due to lateral discharge to the sounds.

Analysis of the potentiometric data of the principal aquifer indicates that the recharge area for the principal aquifer is on the mainland of Dare County. The exact extent of the recharge area cannot be determined with existing data. An estimate of its extent is shown on Figure A-9. Assuming this recharge area is accurate, precipitation on 42,000 acres recharges the principal aquifer, providing a total annual recharge of 19,439 million gallons.

The leakage from the upper to the lower aquifer is 19.7 gpd/acre, which is approximately 244,280 gallons per day. Tables A-3 and A-4 show monthly water balances for recharge, precipitation and withdrawals from the upper aquifer alone, the lower aquifer alone, and the lower aquifer plus leakance from the upper aquifer.

A.5 Effects of Withdrawals

The pumping of wells on Roanoke Island will result in a lowering of the water level in both the principal aquifer and the upper aquifer. The drawdown of both aquifers occurs because of the hydraulic connection of the aquifers previously discussed. This drawdown may affect wells in close proximity to a well being pumped. The drawdown may also cause a reversal in the movement of water laterally into the sounds, resulting in the potential for salt water intrusion into fresh water. Careful management of pumping duration and placement of future wells is necessary to prevent irreversible salt water encroachment. Recent data gathered by the North Carolina Division of Enviornmental Management suggests that pumping of County wells has caused several shallow wells in the southern part of Wanchese to turn salty. While this data is as yet unsubstantiated, it does verify that good management of the aquifer is necessary. Pumping wells 18 hours or less per day help the aquifer to recover.

TABLE A-3

UPPER AQUIFER
ROANOKE ISLAND GROUNDWATER EVALUATION

Potential Water Supply
Recharge by Month

Month	Precipitation feet	Recharge Rate feet	Recharge Volume 10^6 gal
January (1980)	.49	.16	647
February	.26	.09	364
March	.54	.18	727
April	.39	.13	525
May	.37	.12	485
June	.28	.09	364
July	.37	.12	485
August	.21	.07	283
September	.42	.14	566
October	.29	.10	404
November	.33	.10	404
December	.34	.11	445
Average	.36	.12	475

Roanoke Island Area - 12,400 acres
Assume 33% recharge rate

032387

TABLE A-4

LOWER PRINCIPAL AQUIFER
MAINLAND GROUNDWATER EVALUATION

Groundwater Divide from Manns Harbor to Stumpy Point

Month	Precipitation feet	Recharge Rate feet	Recharge Volume 10^6 gal
January (1980)	.49	.16	2,191
February	.26	.09	1,232
March	.54	.18	2,464
April	.39	.13	1,779
May	.37	.12	1,643
June	.28	.09	1,232
July	.37	.12	1,643
August	.21	.07	958
September	.42	.14	1,916
October	.29	.10	1,369
November	.33	.11	1,506
December	.34	.11	1,506

Recharge Area = 42,000 acres

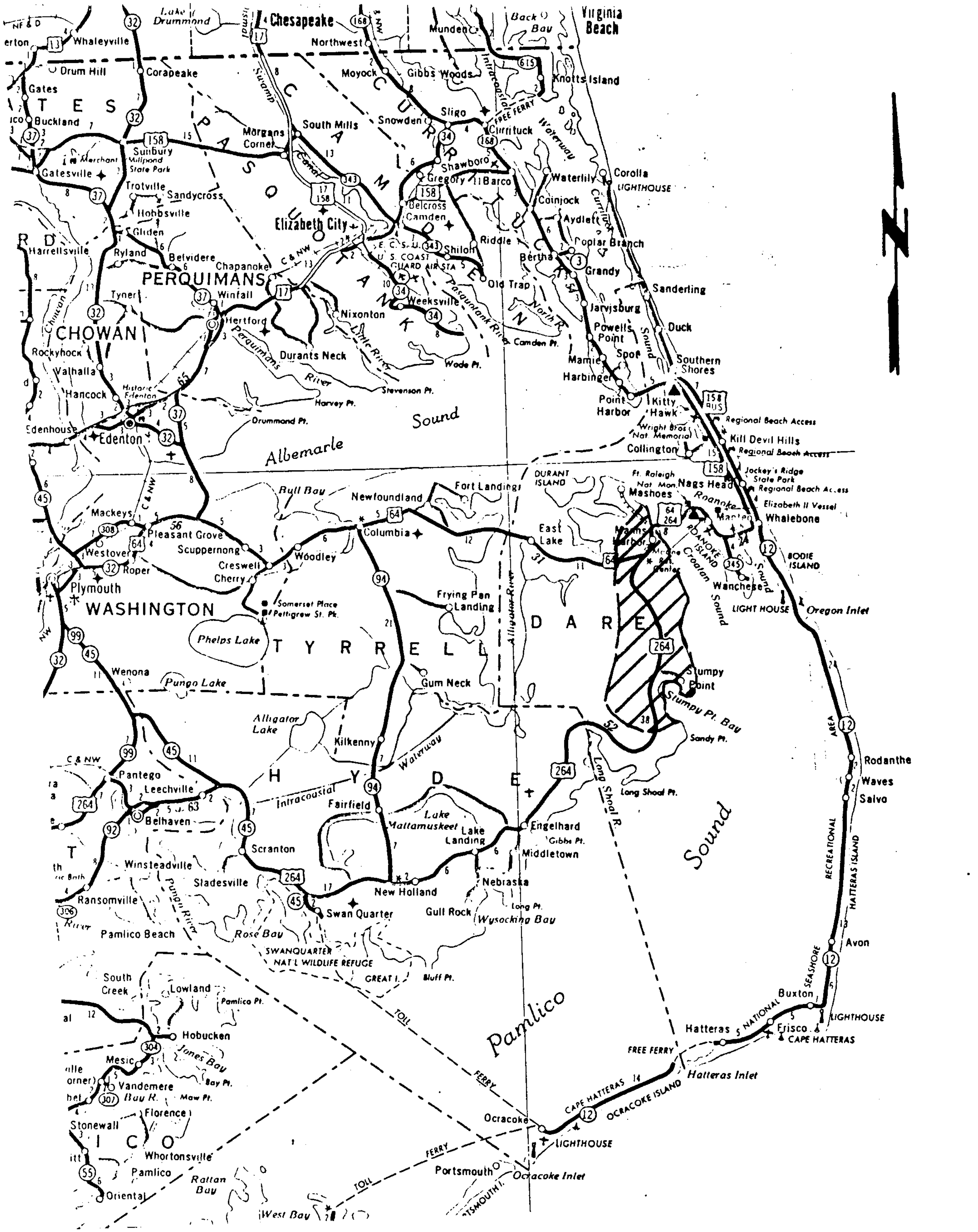

FIGURE A-10 APPROXIMATE LOCATION OF RECHARGE AREA OF PRINCIPAL AQUIFER.

A.6 Long-Term Development

The potential for long range groundwater development and the maximum safe yield of the aquifer system is based on recharge and withdrawal. Total annual recharge is 19,439 million gallons, which is well below annual withdrawal. The major problem is recovery of water without adversely stressing the aquifer. In order to prevent this and to maximize the recovery of groundwater, a long range aquifer management plan is needed. The key points in this plan are as follows.

1. Require residents of Wanchese to connect to the County water system.
2. Install or repair all well head water meters.
3. Construct additional wells as required to allow staggered pumping of wells in the well field by cyclic pumping.
4. Construct additional wells along the spine of the island no closer than 1,000 feet from each other.
5. Never pump a well longer than 18 hours per day. If additional water is required, begin pumping other wells.
6. Install a nest of groundwater monitoring wells at key locations.
7. Monitor wells weekly for water levels and chloride content of the water during the months from May through September. Monthly monitoring during the off season will suffice.
8. Install at least one continuous water level recorder at one of the monitoring wells.
9. Keep accurate logs of water pumped, time pumped, water levels and chloride content for wells and monitoring wells.
10. Submit this data annually to a groundwater hydrologist for review and recommendations regarding any modifications required in pumping or in future well development.

APPENDIX B

BRACKISH GROUNDWATER

Dare County undertook an exploration program to determine the groundwater resources that are available on the Outer Banks. The program included the following activities:

(1) Drill two test wells to sufficient depth to locate the Castle Hayne aquifer.

(2) Obtain water samples from the top and bottom of each aquifer encountered.

(3) Analyse each sample for a wide range of characteristics which are important in reverse osmosis.

(4) Obtain an accurate geophysical log of each well.

(5) Select an aquifer with suitable characteristics for water supply development.

(6) Construct a screened well in the selected aquifer.

(7) Conduct a pump test program consisting of step drawdown test, 24 hour test, and 30 day test.

(8) Evaluate aquifer characteristics from data gathered, and determine well field design parameters.

Two wells were drilled, as discussed in Section 5.0 and located on Figure 5-8. Test Well No. 1 was drilled to 1,610 feet and encountered three aquifers, including the Yorktown aquifer, which is suitable for desalination. Test Well No. 2 encountered only the water table aquifer underlain by thick clay layers, and drilling was terminated at 700 feet.

Samples were obtained by placing 10 feet of 4-inch diameter well screen and a 4-inch diameter pipe into the drilled hole to the appropriate depth, isolating the zone to be sampled with packers above and below, and using a submersible pump to pump a clear sample.

A report was prepared by the well drilling contractor describing the test well, presenting the test data, and recommending well field design parameters. Their report follows as a part of this appendix.

Kill Devil Hills Island

Well No. KDH-DT

Test Analysis Report

December 4, 1986

Prepared for:

Black & Veatch, Inc.
110 West Walker Avenue
Asheboro, North Carolina 27203

Prepared by:

Groundwater Management, Inc.
610 South 38th Street
Kansas City, Kansas 66106

GMI Specialized Groundwater Engineering Services

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Kill Devil Hills Island

Well No. KDH-DT

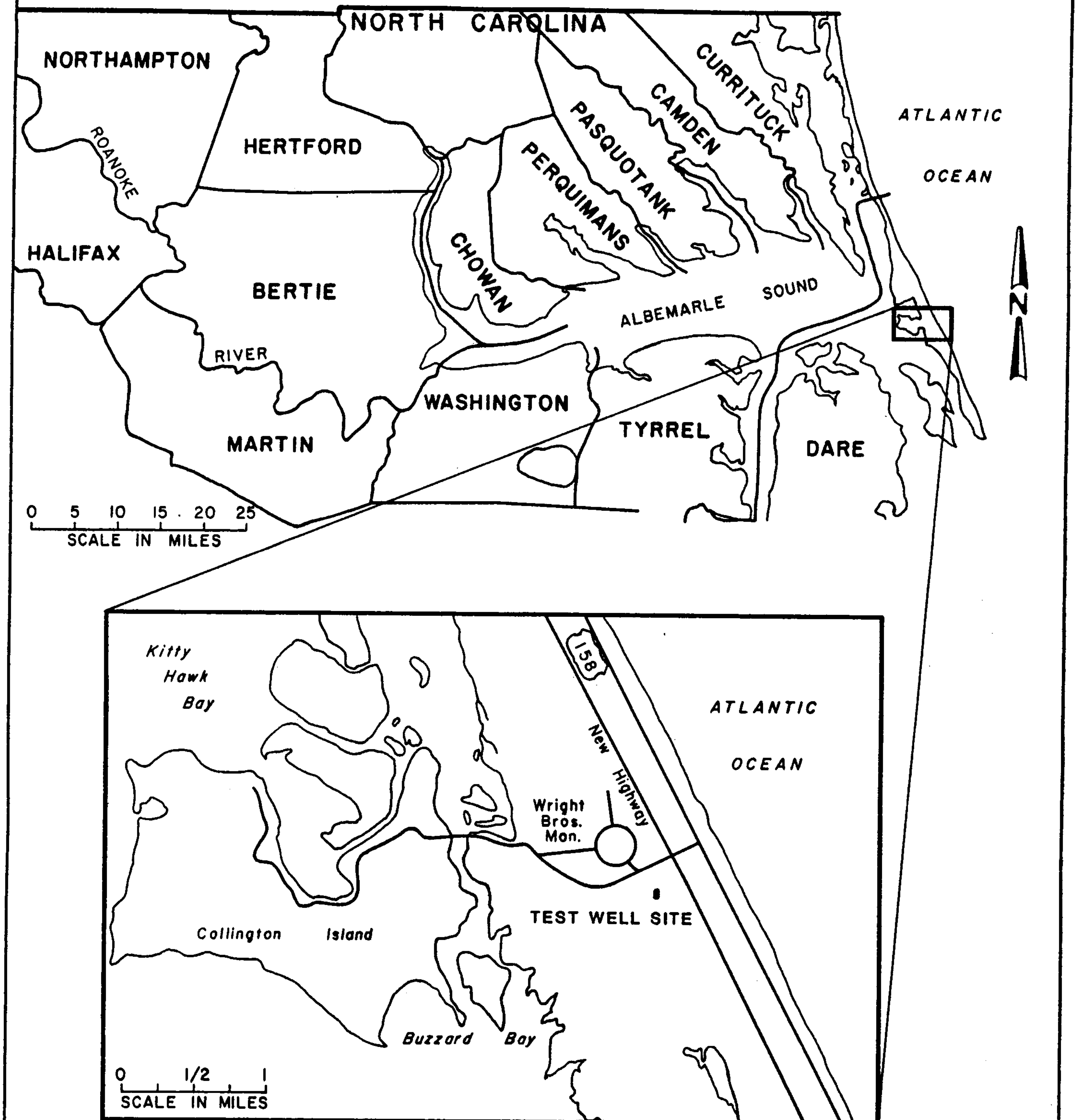
Test Analysis Report

INTRODUCTION

A study is presently being performed to evaluate the possibility of developing a saline water supply on Kill Devil Hills Island. The saline water would be treated by the Reverse Osmosis (RO) technique for potable water supply use. The engineering firm of Black and Veatch, Inc. is coordinating the study.

One of the portions of the study is the evaluation of the water bearing capability of the groundwater formations beneath the island. The Norfolk district office of Layne-Atlantic Company drilled a test hole and developed a test well near Kill Devil Hills. A series of tests was performed on the test well. Figure 1 shows the location of the test well site.

The test data was forwarded to Groundwater Management, Inc. (GMI) for analysis. This report has been prepared to present the results of the analyses of the test data. This report also presents preliminary recommendations for well field development. Additional drilling and well field design will be necessary in order to maximize the use of this water supply source.



GROUNDWATER MANAGEMENT, INC.

FIGURE 1

LOCATION OF TEST WELL SITE

JOB NO: HY-0147 DATE: SEPT. 8, 1986

The well testing analyses and well field design were directed toward water supply only. No water quality data was presented and water quality was not addressed in the work by GMI.

AQUIFER PUMPING TESTS

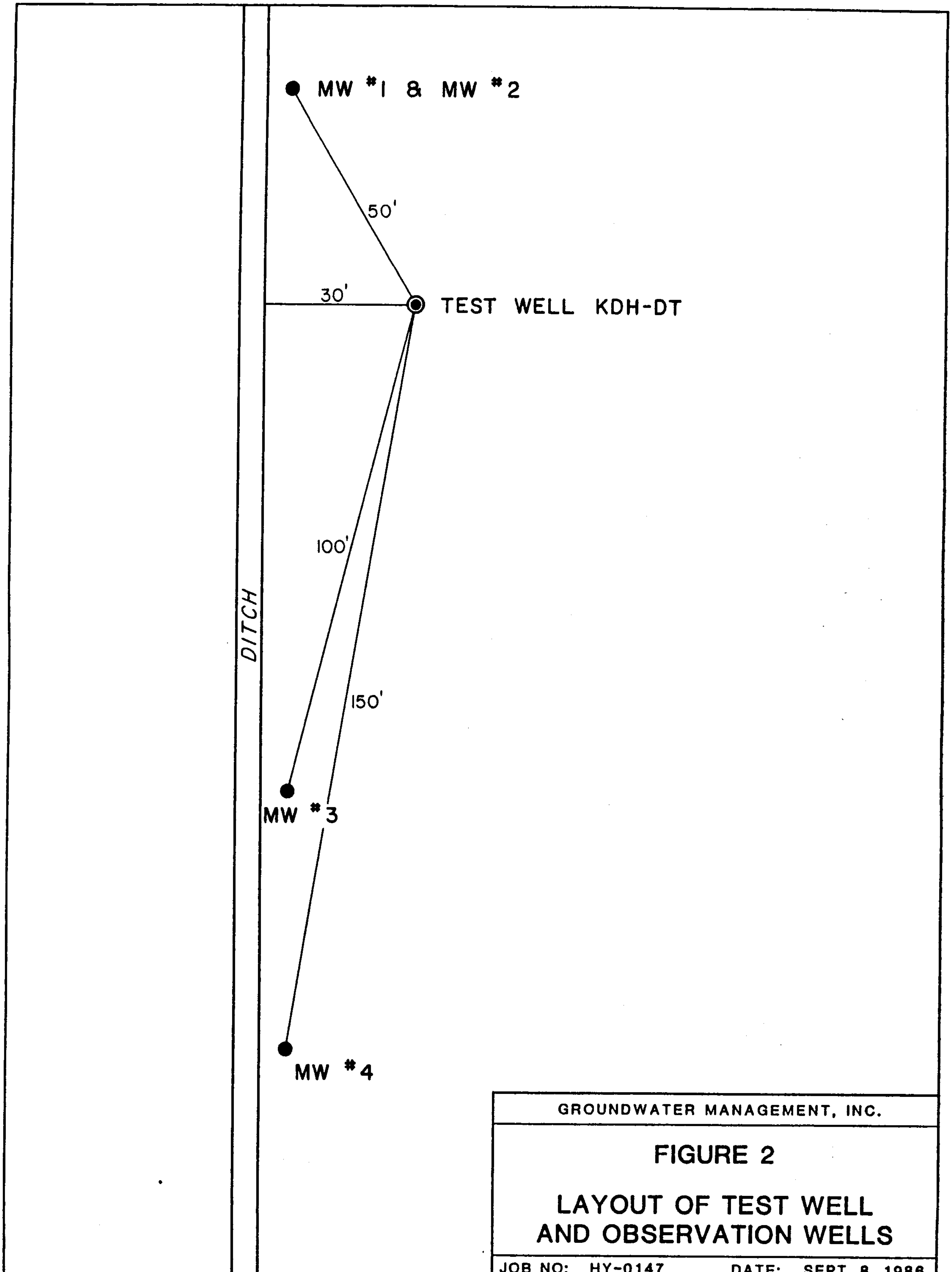
Layout of Wells

One phase of this study consisted of the drilling of a test hole on Kill Devil Hills Island. The test hole was located in the central portion of the island, approximately one-quarter mile south of the Wright Brothers Memorial. The site is on property owned by Dare County.

The test hole was drilled to a total depth of 1610 feet below ground surface. Based on the geophysical logs of the test hole and the data collected during the drilling, a section was selected for aquifer testing. The zone tested was located between depths of approximately 320 feet to 660 feet below ground surface.

The test well was screened between the depths of 330 feet to 480 feet. Six-inch diameter screen was used in the test well. Temporary pumping equipment was installed for the test.

Four monitoring wells were installed to allow water level measurements during the aquifer pumping tests. Figure 2 shows the layout of the test well and observation wells. Monitor wells (MW) #1, #3, and #4 were screened between the depths of 400



GROUNDWATER MANAGEMENT, INC.

FIGURE 2

LAYOUT OF TEST WELL AND OBSERVATION WELLS

JOB NO: HY-0147 DATE: SEPT. 8, 1986

feet to 410 feet. These monitor wells are completed in the same formation as the test well. MW #2 was screened between the depths of 100 feet and 110 feet. This monitor well was constructed in order to evaluate the effect of pumping the saline aquifer on the upper fresh water aquifer.

24-hour Aquifer Pumping Test

A 24-hour aquifer pumping test of the test well was started on April 15, 1986. The test well was pumped at a constant rate of 450 gallons per minute (gpm) during the 24-hour test. Water level measurements were collected in the test well and in the four monitoring wells.

Following the completion of the test, water level measurements were made for 24-hours as the water levels recovered. These recovery measurements were collected in the test well and the four monitoring wells. All data was collected by Layne-Atlantic personnel.

40-day Test

On May 1, 1986, a long term pumping evaluation was begun of the test well. The test well was pumped at a rate of 450 gpm from May 1st until June 11th. Water level measurements were made in the test well and in the monitoring wells.

The purpose of the 40-day test was to evaluate the possible effects of aquifer boundaries on the performance of the well. The data was forwarded to GMI for analysis. Copies of the test data are presented in Appendix I.

TEST ANALYSES

Technique

Standard groundwater analysis techniques were used to evaluate the test data of the 24-hour and 40-day tests. The Modified Theis technique was used to determine the aquifer characteristics. The data from the tests were plotted on semi-logarithimic graphs for the analysis. Copies of these plots are presented in Appendix II.

Two main properties are typically determined from the analysis on an aquifer pumping test. The two properties are the coefficients of transmissivity and storativity. In general, transmissivity relates to the ease with which water moves through a formation. Storativity relates to the volume of water that can be removed from a volume of water bearing formation. The determination of these coefficients is necessary to predict the long term performance of a well and to design an efficient well.

Transmissivity

Two values of transmissivity were calculated in the analysis of the data of the two tests. A short term transmissivity value was calculated based on the 24-hour test. This value represents the aquifer transmissivity in the immediate vicinity of the test well. A long term transmissivity value was calculated based on an analysis of data combined from the 24-hour and 40-day tests.

The short term aquifer transmissivity was calculated to be 107,000 gallons per day per foot of aquifer thickness (gpd/ft). This value is an average of the values determined in the analyses of data from the individual wells. The drawdown data and the recovery data were used in this analysis.

The long term aquifer transmissivity was calculated to be 127,000 gpd/ft. This increase indicates that either the aquifer is more prolific at some distance away from the test well or that the cone of influence has reached a source of recharge. The recharge may be vertical leakage from the overlying aquifer.

Permeability

The value of permeability is a measure of the rate of water movement through a unit area of aquifer material. Transmissivity is equal to permeability times the thickness of the aquifer.

A review was made of the geophysical logs performed at the test hole in order to approximate the thickness of the water bearing formation being tested. Generally, sand formations occur at this site from the ground surface down to a depth of 660 feet. Some portions of the formation contain some fine grained material which act somewhat as confining layers. The zone between depths 320 feet and 580 feet was selected as the best estimate of the aquifer being tested. Based on this thickness of 260 feet, the permeability in the vicinity of the test well was calculated to be approximately 400 gallons per day per square foot of aquifer.

Storativity

The data of the 24-hour test was used to calculate a storativity value of 0.0004 (nondimensional). This value is an average of the values calculated for the three deep monitoring wells.

The long term storativity was calculated to be 0.0001. This value was calculated based on the analysis of combined data from the 24-hour and 40-day tests.

Effect of Pumpage on the Shallow Aquifer

Layne-Atlantic personnel reported that the water levels did not decrease in MW #2 during the 24-hour test or the following recovery period thereby, demonstrating little hydraulic connection to the deeper aquifer. The finer grained materials between the two aquifers are acting to limit the leakage of water downward. Although the vertical movement of water is restricted, the upper aquifer will act to recharge the saline aquifer as water levels decline due to pumpage.

Safe Yield Estimate

The recovery data of the 24-hour test was plotted on semi-log graphs to approximate the time when full recovery would occur. The recovery data of the three deep monitoring wells indicated that full recovery would occur within approximately eight hours. That is, the wells recovered within one-third of the time that pumping occurred. This infers that recharge is occurring and the test well was not pumped in excess of its safe yield. The safe yield of a well at this site may be as much as 700 gpm or more.

Radius of Influence

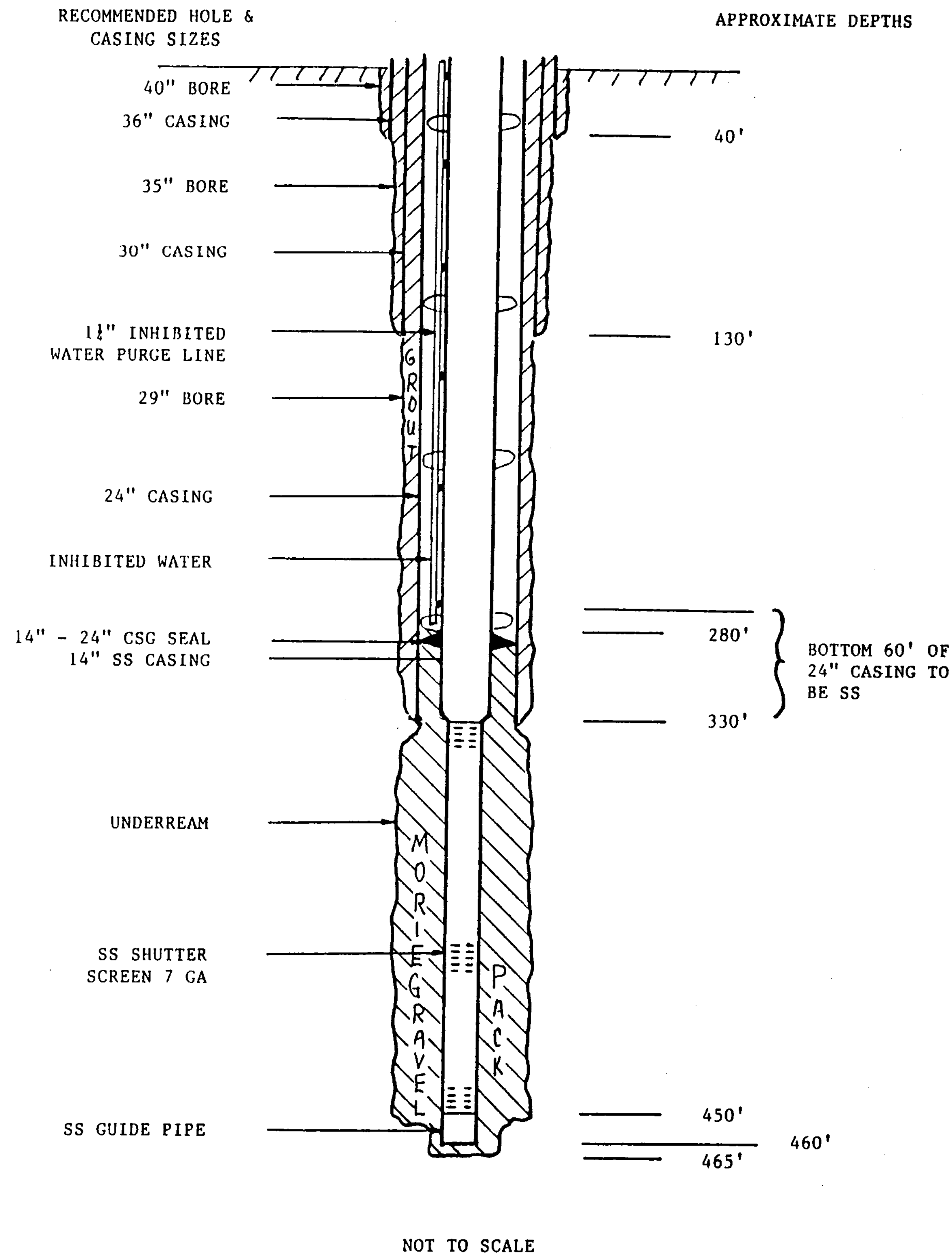
The radius of influence was determined at the end of the 24-hour and the 40-day tests. The radius of influence was 2000 feet after 24 hours of pumpage at 450 gpm. The radius of influence increased to 5000 feet after 40 days.

WELL DESIGN

The results of the analysis of the test well pumping test were used to develop preliminary criteria for the design of individual wells. Based on the permeability of formation and the desired pumping rate of not greater than 500 gpm, 150 feet of screen would be acceptable. The screen should be placed in the upper portion of the formation in order to limit the contribution of potentially poorer quality water.

A large diameter, underreamed borehole is desirable because it allows for a thick gravel pack to be installed which results in lower water velocity from the formation to the gravel pack. This low velocity greatly reduces the chance of silt or sand being drawn into the well. Another important advantage is that a screen with more open area can be installed and thereby reduce the screen entrance velocity. At least a 24-inch or larger diameter borehole should be drilled.

Figure 3 illustrates a generalized well construction design. The design of gravel pack material, screen slot size, and screen diameter would be based on an analysis of the actual formation material. Such an analysis was outside of GMI's scope of work.



GROUNDWATER MANAGEMENT, INC.

FIGURE 3

PROPOSED WELL CONSTRUCTION DESIGN

JOB NO: HY-0147 DATE: SEPT. 8, 1986

Well Materials

Due to the fact that water high in chlorides is going to be pumped, a special design is required for well materials. The screen and well casing materials should be selected so that the saline water of the aquifer will not cause excessive corrosion. The well should be properly grouted so that saline water is not allowed to migrate upward into the freshwater aquifer. Two piece well construction should be used.

Heavy wall casings, properly installed and grouted throughout their length, must be used in order to protect the freshwater aquifer in the area and provide for a long well life. To further increase the life of the well, larger than normal production casing and screen diameters are highly recommended. Stainless steel is the preferred material for these products as well as for the bottom portion of the outer protective casing.

In order to protect the inner walls of the outer protective casing it is recommended that the annular space between it and the stainless steel production casing be filled with an inhibited water solution. The annular space should also contain a small diameter PVC or stainless steel purge line in order to flush and rejuvenate the inhibited water.

The life of the screened portion of the well can be further extended by the use of a 7 gauge or heavier shutter type screen. This screen has excellent inlet capabilities over long lengths, is considerably stronger than wire wrapped screens, has greater resistance to corrosion, and can withstand vigorous agitation during any future redevelopment work which may be required.

Drilling Method

While the surface casings and the outer protective casing may be drilled by the conventional mud rotary method, it is recommended that the screened portion of the well be drilled by the reverse circulation method. Reverse circulation is desired because this method does not use artificial drilling additives which may cause development problems.

In addition, portable steel tanks should be used because of the need to provide an artificial head in order to control the natural borehole fluid pressure. Use of a saltwater drilling fluid and elevation of the rig and drilling platform will be required to further aid in controlling the borehole fluid pressure.

Pump and Motor Selection

Careful attention should be given to material selection and operating speed when selecting the pumping unit. Due to the nature of the water to be pumped special materials should be incorporated into the pump design as follows:

a) Discharge head with protective lining.

b) Stainless steel column pipe.

c) Stainless steel pump shafts with stainless steel sleeves.

d) Stainless steel bowl assemblies and impellers.

Because the wells are located in a humid environment saturated with saltwater vapor and blowing sand, the use of a totally enclosed motor is recommended. For greater life expectancy a 1200 RPM motor is desirable.

WELL FIELD DESIGN

Preliminary Design

GMI has prepared a proposed well field design based on the results of the aquifer pumping tests analysis and design criteria specified by Black & Veatch. Black & Veatch, the consulting engineer for Dare County, performed water quality sampling of the aquifer and decided that wells should not be screened deeper than 500 feet below ground surface. The water quality is poorer in the lower portion than in the upper portion of the aquifer.

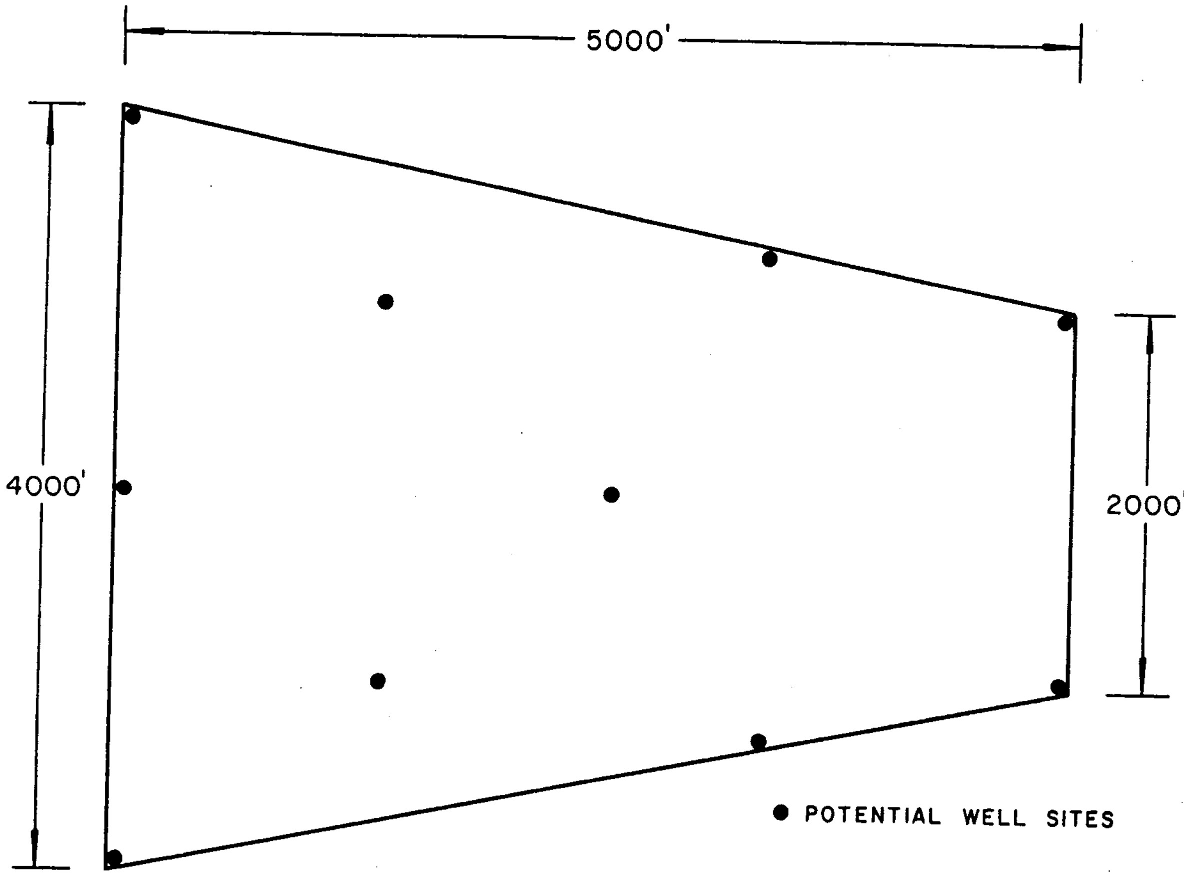
Black & Veatch also requested that the well field be designed within presently available property, if feasible. Black & Veatch reported that Dare County owns or has the rights to use approximately 320 acres in the vicinity of the test well. The property is approximately 5000 feet wide in an east-west direction. The property is nearly 2000 feet along on the east side and nearly 4000 feet long on the west side.

As a first approximation, GMI designed a well field of ten equally producing wells spread evenly throughout the available property. The use of ten wells resulted in minimum well spacings of approximately 1400 feet. The production rates of the wells were selected based on anticipated requirements. Black & Veatch reported that the proposed plant would produce five million gallons per day (MGD) of potable water. A raw water supply of 6.7 MGD would be needed because only 75% of the raw water supply is recovered by reverse osmosis. The ten wells were designed to produce 463 gpm each continuously.

Evaluation of Well Field Design

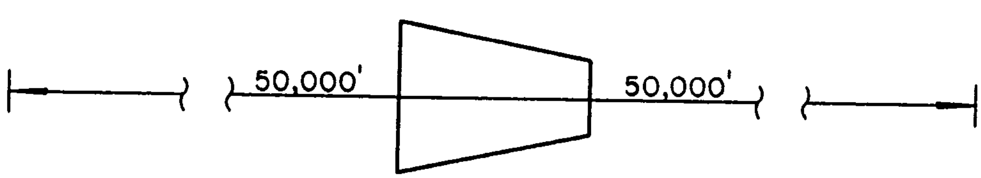
Based on the above well field design, an analytical model was performed to predict the drawdowns caused by pumping. This model was used to predict theoretical drawdowns based on standard groundwater equations. The effects of recharge or of aquifer boundaries were not taken into account in this model. The results of this modeling effort are conservative approximations which have allowed GMI to verify that the property available to Dare County can be developed to meet present production requirements of the RO plant.

Figure 4 presents the layout of the proposed well field and shows the cross section line used to illustrate projected drawdowns. The model was used to illustrate two pumping conditions. The first condition was for a continuous daily production of five MGD. This value was selected as being representative of average daily production. The second condition was for a continuous production of 6.7 MGD. This condition would be the maximum production of the RO plant.



APPROXIMATE WELL FIELD LAYOUT

CROSS SECTION USED IN MODELING CALCULATIONS



GROUNDWATER MANAGEMENT, INC.

FIGURE 4

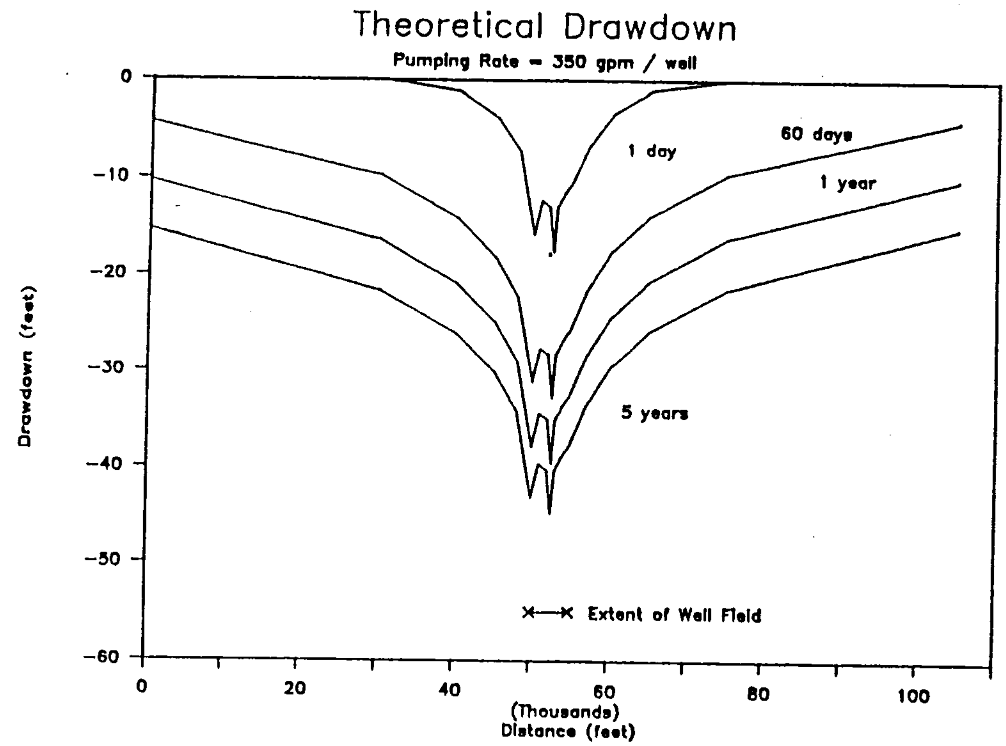
WELL FIELD LAYOUT AND MODEL CROSS SECTION

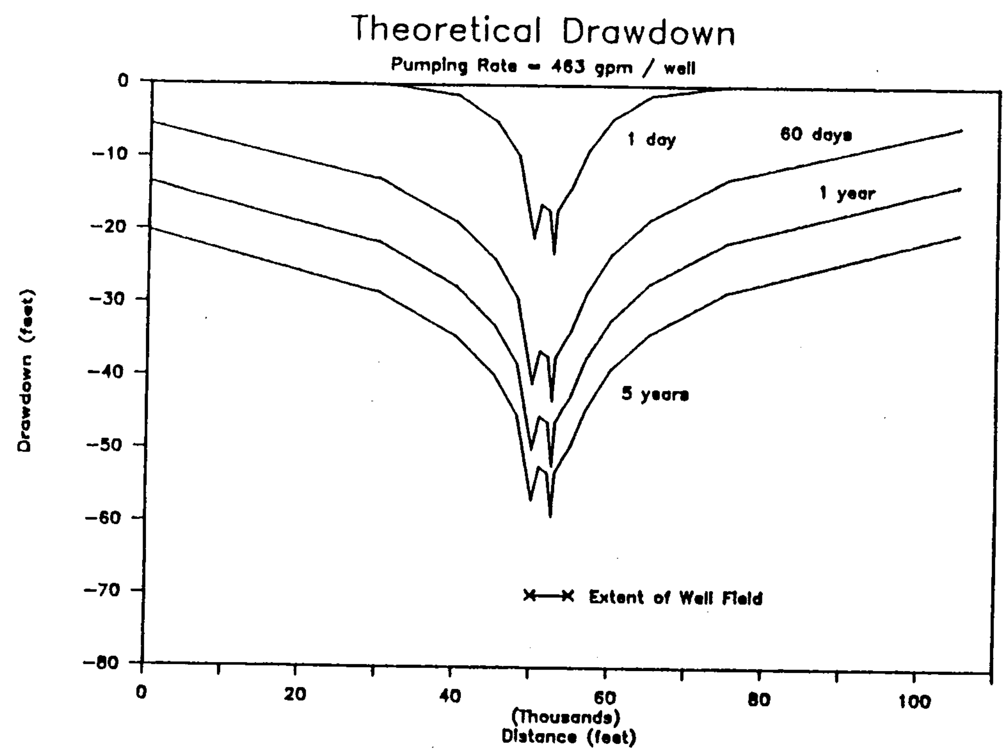
JOB NO: HY-0147 DATE: SEPT. 8, 1986

Figure 5 presents graphs illustrating these two pumping conditions. The top graph of Figure 5 illustrates that the production of five MGD continuously for five years would cause the water levels in the vicinity of the well field to decline approximately forty feet. At a continuous production of 6.7 MGD the decline would be approximately 55 feet. The drawdown of individual wells would be greater due to the well losses associated with water entering the well screens. Both graphs indicate that Dare County should be able to develop a well field on available property capable of supplying the demand requirements of the RO plant.

Monitoring Wells

A series of monitoring wells should be established in and around the new well field as it is developed. The monitoring wells should be used to evaluate the effect of well pumpage on the groundwater levels. The wells should also be used for the collection of water quality samples.





GROUNDWATER MANAGEMENT, INC.

FIGURE 5
THEORETICAL DRAWDOWN GRAPHS

JOB NO: HY-0147 DATE: SEPT. 8, 1986

APPENDIX A

Aquifer Pumping Test Data

AQUIFER TEST DATA

Name: Kill Devil Hills 24-hr Test of KDH DT
Location: Dare County, NC Well No: KDH DT

Date: APR 15, 1986
Job No: HY-0147

Date	Time of Day	Elapsed Time (Min)	Water Level Below Ms Point (Feet)	Drawdown (Feet)	Pumping Rate (GPM)
15-Apr-86	800	0	11.79	0.00	0
15-Apr-86	801	1	37.66	25.87	450
15-Apr-86	802	2	38.81	27.02	450
15-Apr-86	803	3	39.35	27.56	450
15-Apr-86	804	4	39.62	27.83	450
15-Apr-86	805	5	39.74	27.95	450
15-Apr-86	806	6	39.93	28.14	450
15-Apr-86	807	7	39.97	28.18	450
15-Apr-86	808	8	39.97	28.18	450
15-Apr-86	809	9	40.02	28.23	450
15-Apr-86	810	10	40.11	28.32	450
15-Apr-86	820	20	40.41	28.62	450
15-Apr-86	830	30	40.46	28.67	450
15-Apr-86	840	40	40.64	28.85	450
15-Apr-86	850	50	40.68	28.89	450
15-Apr-86	900	60	40.78	28.99	450
15-Apr-86	1000	120	41.01	29.22	450
15-Apr-86	1100	180	41.08	29.29	450
15-Apr-86	1200	240	41.36	29.57	450
15-Apr-86	1300	300	41.38	29.59	450
15-Apr-86	1400	360	41.56	29.77	450
15-Apr-86	1500	420	41.82	30.03	450
15-Apr-86	1600	480	41.89	30.10	450
15-Apr-86	1700	540	41.98	30.19	450
15-Apr-86	1800	600	42.05	30.26	450
15-Apr-86	1900	660	42.05	30.26	450
15-Apr-86	2000	720	42.05	30.26	450
15-Apr-86	2100	780	42.05	30.26	450
15-Apr-86	2200	840	42.05	30.26	450
15-Apr-86	2300	900	42.05	30.26	450
15-Apr-86	2400	960	42.05	30.26	450
16-Apr-86	100	1020	42.05	30.26	450
16-Apr-86	200	1080	42.05	30.26	450
16-Apr-86	300	1140	42.05	30.26	450
16-Apr-86	400	1200	42.05	30.26	450
16-Apr-86	500	1260	42.16	30.37	450
16-Apr-86	600	1320	42.28	30.49	450
16-Apr-86	700	1380	42.28	30.49	450
16-Apr-85	800	1440	42.28	30.49	450

AQUIFER TEST DATA

Name: Kill Devil Hills 24-hr Test of KDH DT   Date: APR 16, 1986
Location: Dare County, NC   Well No: KDH DT   Job No: HY-0147

Date	Time of Day	Elapsed Time (Min)	Water Level Below Ms Point (Feet)	Residual Drawdown (Feet)	Pumping Rate (GPM)
15-Apr-86	800	0	11.79	0.00	0
16-Apr-86	801	1	14.51	2.72	0
16-Apr-86	802	2	14.49	2.70	0
16-Apr-86	803	3	14.47	2.68	0
16-Apr-86	804	4	13.45	1.66	0
16-Apr-86	805	5	13.40	1.61	0
16-Apr-86	806	6	13.34	1.55	0
16-Apr-86	807	7	13.17	1.38	0
16-Apr-86	808	8	13.10	1.31	0
16-Apr-86	809	9	13.06	1.27	0
16-Apr-86	810	10	13.04	1.25	0
16-Apr-86	820	20	13.01	1.22	0
16-Apr-86	830	30	12.99	1.20	0
16-Apr-86	840	40	12.97	1.18	0
16-Apr-86	850	50	12.94	1.15	0
16-Apr-86	900	60	12.92	1.13	0
16-Apr-86	1000	120	12.50	0.71	0
16-Apr-86	1100	180	12.43	0.64	0
16-Apr-86	1200	240	12.41	0.62	0
16-Apr-86	1300	300	12.40	0.61	0
16-Apr-86	1400	360	12.27	0.48	0
16-Apr-86	1500	420	12.27	0.48	0
16-Apr-86	1600	480	12.27	0.48	0
16-Apr-86	1700	540	12.25	0.46	0
16-Apr-86	1800	600	12.25	0.46	0
16-Apr-86	1900	660	12.25	0.46	0
16-Apr-86	2000	720	12.25	0.46	0
17-Apr-86	800	1440	11.80	0.01	0

AQUIFER TEST DATA

Name: Kill Devil Hills 24-hr Test of KDH DT
Location: Dare County, NC Well No: MW #1
Date: APR 15, 1986
Job No: HY-0147

Date	Time of Day	Elapsed Time (Min)	Water Level Below Ms Point (Feet)	Drawdown (Feet)	Pumping Rate (GPM)
15-Apr-86	800	0	7.15	0.00	0
15-Apr-86	801	1	8.53	1.38	450
15-Apr-86	802	2	9.02	1.87	450
15-Apr-86	803	3	9.25	2.10	450
15-Apr-86	804	4	9.41	2.26	450
15-Apr-86	805	5	9.57	2.42	450
15-Apr-86	806	6	9.67	2.52	450
15-Apr-86	807	7	9.74	2.59	450
15-Apr-86	808	8	9.83	2.68	450
15-Apr-86	809	9	9.83	2.68	450
15-Apr-86	810	10	9.90	2.75	450
15-Apr-86	820	20	10.29	3.14	450
15-Apr-86	830	30	10.48	3.33	450
15-Apr-86	840	40	10.66	3.51	450
15-Apr-86	850	50	10.75	3.60	450
15-Apr-86	900	60	10.91	3.76	450
15-Apr-86	1000	120	11.08	3.93	450
15-Apr-86	1100	180	11.21	4.06	450
15-Apr-86	1200	240	11.19	4.04	450
15-Apr-86	1300	300	11.35	4.20	450
15-Apr-86	1400	360	11.51	4.36	450
15-Apr-86	1500	420	11.56	4.41	450
15-Apr-86	1600	480	11.68	4.53	450
15-Apr-86	1700	540	11.77	4.62	450
15-Apr-86	1800	600	11.84	4.69	450
15-Apr-86	1900	660	11.82	4.67	450
15-Apr-86	2000	720	11.82	4.67	450
15-Apr-86	2100	780	11.84	4.69	450
15-Apr-86	2200	840	11.84	4.69	450
15-Apr-86	2300	900	11.86	4.71	450
15-Apr-86	2400	960	11.86	4.71	450
16-Apr-86	100	1020	11.86	4.71	450
16-Apr-86	200	1080	11.88	4.73	450
16-Apr-86	300	1140	11.88	4.73	450
16-Apr-86	400	1200	11.88	4.73	450
16-Apr-86	500	1260	11.88	4.73	450
16-Apr-86	600	1320	11.88	4.73	450
16-Apr-86	700	1380	11.88	4.73	450
16-Apr-86	800	1440	11.88	4.73	450

AQUIFER TEST DATA

Name: Kill Devil Hills 24-hr Test of KDH DT   Date: APR 16, 1986
Location: Dare County, NC   Well No: MW #1   Job No: HY-0147

Date	Time of Day	Elapsed Time (Min)	Water Level Below Ms Point (Feet)	Residual Drawdown (Feet)	Pumping Rate (GPM)
15-Apr-86	800	0	7.15	0	0
16-Apr-86	801	1	9.69	2.54	0
16-Apr-86	802	2	9.30	2.15	0
16-Apr-86	803	3	9.11	1.96	0
16-Apr-86	804	4	8.95	1.8	0
16-Apr-86	805	5	8.88	1.73	0
16-Apr-86	806	6	8.81	1.66	0
16-Apr-86	807	7	8.67	1.52	0
16-Apr-86	808	8	8.63	1.48	0
16-Apr-86	809	9	8.60	1.45	0
16-Apr-86	810	10	8.58	1.43	0
16-Apr-86	820	20	8.19	1.04	0
16-Apr-86	830	30	7.98	0.83	0
16-Apr-86	840	40	7.91	0.76	0
16-Apr-86	850	50	7.73	0.58	0
16-Apr-86	900	60	7.63	0.48	0
16-Apr-86	1000	120	7.26	0.11	0
16-Apr-86	1100	180	7.22	0.07	0
16-Apr-86	1200	240	7.20	0.05	0
16-Apr-86	1300	300	7.17	0.02	0
16-Apr-86	1400	360	7.08	-0.07	0
16-Apr-86	1500	420	7.03	-0.12	0
16-Apr-86	1600	480	6.80	-0.35	0
16-Apr-86	1700	540	6.80	-0.35	0
16-Apr-86	1800	600	6.80	-0.35	0
16-Apr-86	1900	660	6.80	-0.35	0
16-Apr-86	2000	720	6.80	-0.35	0
17-Apr-86	800	1440	6.57	-0.58	0

AQUIFER TEST DATA

Name: Kill Devil Hills 24-hr Test of KDH DT
Location: Dare County, NC Well No: MW #3
Date: APR 15, 1986
Job No: HY-0147

Date	Time of Day	Elapsed Time (Min)	Water Level Below Ms Point (Feet)	Drawdown (Feet)	Pumping Rate (GPM)
15-Apr-86	800	0	9.07	0.00	0
15-Apr-86	801	1	10.04	0.97	450
15-Apr-86	802	2	10.27	1.20	450
15-Apr-86	803	3	10.50	1.43	450
15-Apr-86	804	4	10.61	1.54	450
15-Apr-86	805	5	10.73	1.66	450
15-Apr-86	806	6	10.84	1.77	450
15-Apr-86	807	7	10.96	1.89	450
15-Apr-86	808	8	11.01	1.94	450
15-Apr-86	809	9	11.08	2.01	450
15-Apr-86	810	10	11.12	2.05	450
15-Apr-86	820	20	11.61	2.54	450
15-Apr-86	830	30	11.68	2.61	450
15-Apr-86	840	40	11.79	2.72	450
15-Apr-86	850	50	11.91	2.84	450
15-Apr-86	900	60	12.02	2.95	450
15-Apr-86	1000	120	12.25	3.18	450
15-Apr-86	1100	180	12.37	3.30	450
15-Apr-86	1200	240	12.44	3.37	450
15-Apr-86	1300	300	12.55	3.48	450
15-Apr-86	1400	360	12.65	3.58	450
15-Apr-86	1500	420	12.79	3.72	450
15-Apr-86	1600	480	12.83	3.76	450
15-Apr-86	1700	540	12.95	3.88	450
15-Apr-86	1800	600	13.04	3.97	450
15-Apr-86	1900	660	13.04	3.97	450
15-Apr-86	2000	720	13.04	3.97	450
15-Apr-86	2100	780	13.04	3.97	450
15-Apr-86	2200	840	13.04	3.97	450
15-Apr-86	2300	900	13.04	3.97	450
15-Apr-86	2400	960	13.04	3.97	450
16-Apr-86	100	1020	13.04	3.97	450
16-Apr-86	200	1080	13.04	3.97	450
16-Apr-86	300	1140	13.04	3.97	450
16-Apr-86	400	1200	13.04	3.97	450
16-Apr-86	500	1260	13.04	3.97	450
16-Apr-86	600	1320	13.04	3.97	450
16-Apr-86	700	1380	13.04	3.97	450
16-Apr-86	800	1440	13.04	3.97	450

AQUIFER TEST DATA

Name: Kill Devil Hills 24-hr Test of KDH DT
Location: Dare County, NC Well No: MW #3
Date: APR 16, 1986
Job No: HY-0147

Date	Time of Day	Elapsed Time (Min)	Water Level Below Ms Point (Feet)	Residual Drawdown (Feet)	Pumping Rate (GPM)
15-Apr-86	800	0	9.07	0.00	0
16-Apr-86	801	1	11.77	2.70	0
16-Apr-86	802	2	11.49	2.42	0
16-Apr-86	803	3	11.36	2.29	0
16-Apr-86	804	4	11.19	2.12	0
16-Apr-86	805	5	11.21	2.14	0
16-Apr-86	806	6	10.98	1.91	0
16-Apr-86	807	7	10.96	1.89	0
16-Apr-86	808	8	10.89	1.82	0
16-Apr-86	809	9	10.87	1.80	0
16-Apr-86	810	10	10.75	1.68	0
16-Apr-86	820	20	10.43	1.36	0
16-Apr-86	830	30	10.22	1.15	0
16-Apr-86	840	40	10.04	0.97	0
16-Apr-86	850	50	9.97	0.90	0
16-Apr-86	900	60	9.83	0.76	0
16-Apr-86	1000	120	9.53	0.46	0
16-Apr-86	1100	180	9.37	0.30	0
16-Apr-86	1200	240	9.30	0.23	0
16-Apr-86	1300	300	9.27	0.20	0
16-Apr-86	1400	360	9.25	0.18	0
16-Apr-86	1500	420	9.23	0.16	0
16-Apr-86	1600	480	9.23	0.16	0
16-Apr-86	1700	540	9.14	0.07	0
16-Apr-86	1800	600	9.14	0.07	0
16-Apr-86	1900	660	9.14	0.07	0
16-Apr-86	2000	720	9.14	0.07	0
17-Apr-86	800	1440	9.07	0.00	0

AQUIFER TEST DATA

Name: Kill Devil Hills 24-hr Test of KDH DT Date: APR 15, 1986
Location: Dare County, NC Well No: MW #4 Job No: HY-0147

Date	Time of Day	Elapsed Time (Min)	Water Level Below Ms Point (Feet)	Drawdown (Feet)	Pumping Rate (GPM)
15-Apr-86	800	0	6.20	0.00	0
15-Apr-86	801	1	6.80	0.60	450
15-Apr-86	802	2	7.03	0.83	450
15-Apr-86	803	3	7.29	1.09	450
15-Apr-86	804	4	7.38	1.18	450
15-Apr-86	805	5	7.50	1.30	450
15-Apr-86	806	6	7.56	1.36	450
15-Apr-86	807	7	7.66	1.46	450
15-Apr-86	808	8	7.73	1.53	450
15-Apr-86	809	9	7.77	1.57	450
15-Apr-86	810	10	7.80	1.60	450
15-Apr-86	820	20	8.17	1.97	450
15-Apr-86	830	30	8.40	2.20	450
15-Apr-86	840	40	8.44	2.24	450
15-Apr-86	850	50	8.60	2.40	450
15-Apr-86	900	60	8.67	2.47	450
15-Apr-86	1000	120	8.91	2.71	450
15-Apr-86	1100	180	9.00	2.80	450
15-Apr-86	1200	240	8.91	2.71	450
15-Apr-86	1300	300	9.14	2.94	450
15-Apr-86	1400	360	9.30	3.10	450
15-Apr-86	1500	420	9.37	3.17	450
15-Apr-86	1600	480	9.46	3.26	450
15-Apr-86	1700	540	9.53	3.33	450
15-Apr-86	1800	600	9.57	3.37	450
15-Apr-86	1900	660	9.57	3.37	450
15-Apr-86	2000	720	9.57	3.37	450
15-Apr-86	2100	780	9.57	3.37	450
15-Apr-86	2200	840	9.57	3.37	450
15-Apr-86	2300	900	9.57	3.37	450
15-Apr-86	2400	960	9.57	3.37	450
16-Apr-86	100	1020	9.57	3.37	450
16-Apr-86	200	1080	9.57	3.37	450
16-Apr-86	300	1140	9.57	3.37	450
16-Apr-86	400	1200	9.57	3.37	450
16-Apr-86	500	1260	9.57	3.37	450
16-Apr-86	600	1320	9.57	3.37	450
16-Apr-86	700	1380	9.57	3.37	450
16-Apr-86	800	1440	9.57	3.37	450

AQUIFER TEST DATA

Name: Kill Devil Hills 24-hr Test of KDH DT
Location: Dare County, NC Well No: MW #4

Date: APR 16, 1986
Job No: HY-0147

Date	Time of Day	Elapsed Time (Min)	Water Level Below Ms Point (Feet)	Residual Drawdown (Feet)	Pumping Rate (GPM)
15-Apr-86	800	0	6.20	0.00	0
16-Apr-86	801	1	9.04	2.84	0
16-Apr-86	802	2	8.58	2.38	0
16-Apr-86	803	3	8.42	2.22	0
16-Apr-86	804	4	8.37	2.17	0
16-Apr-86	805	5	8.21	2.01	0
16-Apr-86	806	6	8.19	1.99	0
16-Apr-86	807	7	8.12	1.92	0
16-Apr-86	808	8	8.03	1.83	0
16-Apr-86	809	9	7.98	1.78	0
16-Apr-86	810	10	7.98	1.78	0
16-Apr-86	820	20	7.68	1.48	0
16-Apr-86	830	30	7.43	1.23	0
16-Apr-86	840	40	7.26	1.06	0
16-Apr-86	850	50	7.15	0.95	0
16-Apr-86	900	60	7.03	0.83	0
16-Apr-86	1000	120	6.69	0.49	0
16-Apr-86	1100	180	6.50	0.30	0
16-Apr-86	1200	240	6.50	0.30	0
16-Apr-86	1300	300	6.46	0.26	0
16-Apr-86	1400	360	6.43	0.23	0
16-Apr-86	1500	420	6.39	0.19	0
16-Apr-86	1600	480	6.39	0.19	0
16-Apr-86	1700	540	6.39	0.19	0
16-Apr-86	1800	600	6.39	0.19	0
16-Apr-86	1900	660	6.39	0.19	0
16-Apr-86	2000	720	6.39	0.19	0
17-Apr-86	800	1440	6.57	0.37	0

AQUIFER TEST DATA

Name: Kill Devil Hills 40-day Test of KDH DT   Date: MAY 2, 1986
Location: Dare County, NC   Well No: KDH DT   Job No: HY-0147

Date	Time of Day	Elapsed Time (Min)	Water Level Below Ms Point (Feet)	Drawdown (Feet)	Pumping Rate (GPM)
02-May-86	700	0	8.58	0.00	0
02-May-86	1500	480	38.73	30.15	450
02-May-86	2300	960	39.72	31.14	450
03-May-86	700	1440	39.77	31.19	450
03-May-86	1500	1920	39.95	31.37	450
03-May-86	2300	2400	40.25	31.67	450
04-May-86	700	2880	40.11	31.53	450
04-May-86	1500	3360	40.35	31.77	450
04-May-86	2300	3840	40.48	31.90	450
05-May-86	700	4320	40.41	31.83	450
05-May-86	1500	4800	40.58	32.00	450
05-May-86	2300	5280	40.67	32.09	450
06-May-86	700	5760	40.58	32.00	450
06-May-86	1500	6240	40.67	32.09	450
06-May-86	2300	6720	39.95	31.37	450
07-May-86	700	7200	40.00	31.42	450
07-May-86	1500	7680	40.00	31.42	450
07-May-86	2300	8160	40.00	31.42	450
08-May-86	700	8640	40.11	31.53	450
08-May-86	1500	9120	40.25	31.67	450
08-May-86	2300	9600	40.02	31.44	450
09-May-86	700	10080	39.88	31.30	450
09-May-86	1500	10560	40.11	31.53	450
09-May-86	2300	11040	39.88	31.30	450
10-May-86	700	11520	40.23	31.65	450
10-May-86	1500	12000	40.35	31.77	450
10-May-86	2300	12480	40.23	31.65	450
11-May-86	700	12960	40.69	32.11	450
11-May-86	1500	13440	40.14	31.56	450
11-May-86	2300	13920	40.23	31.65	450
12-May-86	700	14400	40.69	32.11	450
12-May-86	1500	14880	40.65	32.07	450
12-May-86	2300	15360	40.44	31.86	450
13-May-86	700	15840	40.81	32.23	450
13-May-86	1500	16320	40.02	31.44	450
13-May-86	2300	16800	39.95	31.37	450
14-May-86	700	17280	40.23	31.65	450
14-May-86	1500	17760	40.11	31.53	450
14-May-86	2300	18240	40.11	31.53	450
15-May-86	700	18720	40.46	31.88	450
15-May-86	1500	19200	40.21	31.63	450

AQUIFER TEST DATA

Name: Kill Devil Hills 40-day Test of KDH DT
Location: Dare County, NC Well No: KDH DT

Date: MAY 2, 1986
Job No: HY-0147

Date	Time of Day	Elapsed Time (Min)	Water Level Below Ms Point (Feet)	Drawdown (Feet)	Pumping Rate (GPM)
15-May-86	2300	19680	40.41	31.83	450
16-May-86	700	20160	40.58	32.00	450
16-May-86	1500	20640	40.25	31.67	450
16-May-86	2300	21120	40.44	31.86	450
17-May-86	700	21600	40.46	31.88	450
17-May-86	1500	22080	40.23	31.65	450
17-May-86	2300	22560	40.11	31.53	450
18-May-86	700	23040	40.65	32.07	450
18-May-86	1500	23520	40.46	31.88	450
18-May-86	2300	24000	40.69	32.11	450
19-May-86	700	24480	40.58	32.00	450
19-May-86	1500	24960	40.58	32.00	450
19-May-86	2300	25440	40.81	32.23	450
20-May-86	700	25920	40.58	32.00	450
20-May-86	1500	26400	40.81	32.23	450
20-May-86	2300	26880	40.81	32.23	450
21-May-86	700	27360	40.67	32.09	450
21-May-86	1500	27840	40.81	32.23	450
21-May-86	2300	28320	40.72	32.14	450
22-May-86	700	28800	40.58	32.00	450
22-May-86	1500	29280	40.90	32.32	450
22-May-86	2300	29760	40.67	32.09	450
23-May-86	700	30240	40.69	32.11	450
23-May-86	1500	30720	41.38	32.80	450
23-May-86	2300	31200	40.90	32.32	450
24-May-86	700	31680	41.13	32.55	450
24-May-86	1500	32160	41.18	32.60	450
24-May-86	2300	32640	40.81	32.23	450
25-May-86	700	33120	41.15	32.57	450
25-May-86	1500	33600	41.38	32.80	450
25-May-86	2300	34080	40.69	32.11	450
26-May-86	700	34560	41.11	32.53	450
26-May-86	1500	35040	41.38	32.80	450
26-May-86	2300	35520	40.97	32.39	450
27-May-86	700	36000	41.39	32.81	450
27-May-86	1500	36480	41.22	32.64	450
27-May-86	2300	36960	41.04	32.46	450
28-May-86	700	37440	41.36	32.78	450
28-May-86	1500	37920	41.04	32.46	450
28-May-86	2300	38400	41.18	32.60	450
29-May-86	700	38880	41.39	32.81	450

AQUIFER TEST DATA

Name: Kill Devil Hills 40-day Test of KDH DT
Location: Dare County, NC Well No: KDH DT Date: MAY 2, 1986
Job No: HY-0147

Date	Time of Day	Elapsed Time (Min)	Water Level Below Ms Point (Feet)	Drawdown (Feet)	Pumping Rate (GPM)
29-May-86	1500	39360	41.15	32.57	450
29-May-86	2300	39840	41.27	32.69	450
30-May-86	700	40320	41.25	32.67	450
30-May-86	1500	40800	41.04	32.46	450
30-May-86	2300	41280	41.18	32.60	450
31-May-86	700	41760	40.78	32.20	450
31-May-86	1500	42240	41.04	32.46	450
31-May-86	2300	42720	40.92	32.34	450
01-Jun-86	700	43200	40.81	32.23	450
01-Jun-86	1500	43680	40.65	32.07	450
01-Jun-86	2300	44160	41.20	32.62	450
02-Jun-86	700	44640	40.92	32.34	450
02-Jun-86	1500	45120	40.65	32.07	450
02-Jun-86	2300	45600	41.15	32.57	450
03-Jun-86	700	46080	40.90	32.32	450
03-Jun-86	1500	46560	40.90	32.32	450
03-Jun-86	2300	47040	41.18	32.60	450
04-Jun-86	700	47520	41.18	32.60	450
04-Jun-86	1500	48000	41.15	32.57	450
04-Jun-86	2300	48480	41.04	32.46	450
05-Jun-86	700	48960	41.06	32.48	450
05-Jun-86	1500	49440	41.18	32.60	450
05-Jun-86	2300	49920	40.95	32.37	450
06-Jun-86	700	50400	41.04	32.46	450
06-Jun-86	1500	50880	40.61	32.03	450
06-Jun-86	2300	51360	40.81	32.23	450
07-Jun-86	700	51840	40.92	32.34	450
07-Jun-86	1500	52320	40.97	32.39	450
07-Jun-86	2300	52800	40.72	32.14	450
08-Jun-86	700	53280	40.97	32.39	450
08-Jun-86	1500	53760	40.92	32.34	450
08-Jun-86	2300	54240	40.69	32.11	450
09-Jun-86	700	54720	40.77	32.19	450
09-Jun-86	1500	55200	40.48	31.90	450
09-Jun-86	2300	55680	40.46	31.88	450
10-Jun-86	700	56160	40.95	32.37	450
10-Jun-86	1500	56640	40.92	32.34	450
10-Jun-86	2300	57120	40.69	32.11	450
11-Jun-86	700	57600	40.95	32.37	450

AQUIFER TEST DATA

Name: Kill Devil Hills 40-day Test of KDH DT
Location: Dare County, NC Well No: MW #1
Date: MAY 2, 1986
Job No: HY-0147

Date	Time of Day	Elapsed Time (Min)	Water Level Below Ms Point (Feet)	Drawdown (Feet)	Pumping Rate (GPM)
02-May-86	700	0	6.10	0.00	0
02-May-86	1500	480	10.67	4.57	450
02-May-86	2300	960	11.34	5.24	450
03-May-86	700	1440	11.32	5.22	450
03-May-86	1500	1920	11.53	5.43	450
03-May-86	2300	2400	11.81	5.71	450
04-May-86	700	2880	11.57	5.47	450
04-May-86	1500	3360	11.97	5.87	450
04-May-86	2300	3840	12.04	5.94	450
05-May-86	700	4320	11.90	5.80	450
05-May-86	1500	4800	12.06	5.96	450
05-May-86	2300	5280	12.01	5.91	450
06-May-86	700	5760	11.99	5.89	450
06-May-86	1500	6240	12.01	5.91	450
06-May-86	2300	6720	12.01	5.91	450
07-May-86	700	7200	11.97	5.87	450
07-May-86	1500	7680	11.97	5.87	450
07-May-86	2300	8160	11.78	5.68	450
08-May-86	700	8640	11.90	5.80	450
08-May-86	1500	9120	12.04	5.94	450
08-May-86	2300	9600	11.81	5.71	450
09-May-86	700	10080	11.97	5.87	450
09-May-86	1500	10560	12.01	5.91	450
09-May-86	2300	11040	11.76	5.66	450
10-May-86	700	11520	12.01	5.91	450
10-May-86	1500	12000	12.13	6.03	450
10-May-86	2300	12480	11.81	5.71	450
11-May-86	700	12960	12.13	6.03	450
11-May-86	1500	13440	12.13	6.03	450
11-May-86	2300	13920	11.97	5.87	450
12-May-86	700	14400	12.24	6.14	450
12-May-86	1500	14880	12.13	6.03	450
12-May-86	2300	15360	11.99	5.89	450
13-May-86	700	15840	12.22	6.12	450
13-May-86	1500	16320	12.06	5.96	450
13-May-86	2300	16800	11.94	5.84	450
14-May-86	700	17280	12.22	6.12	450
14-May-86	1500	17760	12.13	6.03	450
14-May-86	2300	18240	12.01	5.91	450
15-May-86	700	18720	12.36	6.26	450
15-May-86	1500	19200	12.13	6.03	450

AQUIFER TEST DATA

Name: Kill Devil Hills 40-day Test of KDH DT   Date: MAY 2, 1986
Location: Dare County, NC   Well No: MW #1   Job No: HY-0147

Date	Time of Day	Elapsed Time (Min)	Water Level Below Ms Point (Feet)	Drawdown (Feet)	Pumping Rate (GPM)
15-May-86	2300	19680	12.24	6.14	450
16-May-86	700	20160	13.51	7.41	450
16-May-86	1500	20640	12.13	6.03	450
16-May-86	2300	21120	12.27	6.17	450
17-May-86	700	21600	12.36	6.26	450
17-May-86	1500	22080	12.27	6.17	450
17-May-86	2300	22560	12.48	6.38	450
18-May-86	700	23040	12.20	6.10	450
18-May-86	1500	23520	12.36	6.26	450
18-May-86	2300	24000	12.43	6.33	450
19-May-86	700	24480	12.24	6.14	450
19-May-86	1500	24960	12.24	6.14	450
19-May-86	2300	25440	13.51	7.41	450
20-May-86	700	25920	12.24	6.14	450
20-May-86	1500	26400	12.47	6.37	450
20-May-86	2300	26880	12.45	6.35	450
21-May-86	700	27360	12.17	6.07	450
21-May-86	1500	27840	12.29	6.19	450
21-May-86	2300	28320	12.24	6.14	450
22-May-86	700	28800	12.13	6.03	450
22-May-86	1500	29280	12.45	6.35	450
22-May-86	2300	29760	13.35	7.25	450
23-May-86	700	30240	12.22	6.12	450
23-May-86	1500	30720	12.52	6.42	450
23-May-86	2300	31200	12.45	6.35	450
24-May-86	700	31680	12.45	6.35	450
24-May-86	1500	32160	12.71	6.61	450
24-May-86	2300	32640	12.06	5.96	450
25-May-86	700	33120	12.45	6.35	450
25-May-86	1500	33600	12.54	6.44	450
25-May-86	2300	34080	12.20	6.10	450
26-May-86	700	34560	12.36	6.26	450
26-May-86	1500	35040	12.66	6.56	450
26-May-86	2300	35520	12.24	6.14	450
27-May-86	700	36000	12.64	6.54	450
27-May-86	1500	36480	12.45	6.35	450
27-May-86	2300	36960	12.24	6.14	450
28-May-86	700	37440	12.61	6.51	450
28-May-86	1500	37920	12.29	6.19	450
28-May-86	2300	38400	13.51	7.41	450
29-May-86	700	38880	12.66	6.56	450

AQUIFER TEST DATA

Name: Kill Devil Hills 40-day Test of KDH DT Date: MAY 2, 1986
Location: Dare County, NC Well No: MW #1 Job No: HY-0147

Date	Time of Day	Elapsed Time (Min)	Water Level Below Ms Point (Feet)	Drawdown (Feet)	Pumping Rate (GPM)
29-May-86	1500	39360	12.36	6.26	450
29-May-86	2300	39840	12.57	6.47	450
30-May-86	700	40320	13.51	7.41	450
30-May-86	1500	40800	13.49	7.39	450
30-May-86	2300	41280	12.36	6.26	450
31-May-86	700	41760	12.48	6.38	450
31-May-86	1500	42240	13.49	7.39	450
31-May-86	2300	42720	12.68	6.58	450
01-Jun-86	700	43200	12.48	6.38	450
01-Jun-86	1500	43680	13.28	7.18	450
01-Jun-86	2300	44160	12.34	6.24	450
02-Jun-86	700	44640	12.50	6.40	450
02-Jun-86	1500	45120	12.24	6.14	450
02-Jun-86	2300	45600	12.66	6.56	450
03-Jun-86	700	46080	12.45	6.35	450
03-Jun-86	1500	46560	12.59	6.49	450
03-Jun-86	2300	47040	12.68	6.58	450
04-Jun-86	700	47520	12.82	6.72	450
04-Jun-86	1500	48000	12.71	6.61	450
04-Jun-86	2300	48480	12.68	6.58	450
05-Jun-86	700	48960	12.66	6.56	450
05-Jun-86	1500	49440	12.82	6.72	450
05-Jun-86	2300	49920	12.59	6.49	450
06-Jun-86	700	50400	12.68	6.58	450
06-Jun-86	1500	50880	12.84	6.74	450
06-Jun-86	2300	51360	12.48	6.38	450
07-Jun-86	700	51840	12.48	6.38	450
07-Jun-86	1500	52320	12.57	6.47	450
07-Jun-86	2300	52800	12.36	6.26	450
08-Jun-86	700	53280	12.48	6.38	450
08-Jun-86	1500	53760	12.52	6.42	450
08-Jun-86	2300	54240	12.29	6.19	450
09-Jun-86	700	54720	12.86	6.76	450
09-Jun-86	1500	55200	12.45	6.35	450
09-Jun-86	2300	55680	12.41	6.31	450
10-Jun-86	700	56160	12.82	6.72	450
10-Jun-86	1500	56640	12.82	6.72	450
10-Jun-86	2300	57120	12.54	6.44	450
11-Jun-86	700	57600	12.89	6.79	450

AQUIFER TEST DATA

Name: Kill Devil Hills 40-day Test of KDH DT   Date: MAY 2, 1986
Location: Dare County, NC   Well No: MW #2   Job No: HY-0147

Date	Time of Day	Elapsed Time (Min)	Water Level Below Ms Point (Feet)	Drawdown (Feet)	Pumping Rate (GPM)
02-May-86	700	0	4.07	0.00	0
02-May-86	1500	480	4.02	-0.05	450
02-May-86	2300	960	4.07	0.00	450
03-May-86	700	1440	4.14	0.07	450
03-May-86	1500	1920	4.14	0.07	450
03-May-86	2300	2400	4.23	0.16	450
04-May-86	700	2880	4.14	0.07	450
04-May-86	1500	3360	4.09	0.02	450
04-May-86	2300	3840	4.21	0.14	450
05-May-86	700	4320	4.21	0.14	450
05-May-86	1500	4800	4.04	-0.03	450
05-May-86	2300	5280	4.18	0.11	450
06-May-86	700	5760	4.14	0.07	450
06-May-86	1500	6240	4.07	0.00	450
06-May-86	2300	6720	4.25	0.18	450
07-May-86	700	7200	4.25	0.18	450
07-May-86	1500	7680	4.14	0.07	450
07-May-86	2300	8160	4.25	0.18	450
08-May-86	700	8640	4.21	0.14	450
08-May-86	1500	9120	4.21	0.14	450
08-May-86	2300	9600	4.48	0.41	450
09-May-86	700	10080	4.25	0.18	450
09-May-86	1500	10560	4.25	0.18	450
09-May-86	2300	11040	4.25	0.18	450
10-May-86	700	11520	4.25	0.18	450
10-May-86	1500	12000	4.25	0.18	450
10-May-86	2300	12480	4.28	0.21	450
11-May-86	700	12960	4.25	0.18	450
11-May-86	1500	13440	4.23	0.16	450
11-May-86	2300	13920	4.25	0.18	450
12-May-86	700	14400	4.25	0.18	450
12-May-86	1500	14880	4.23	0.16	450
12-May-86	2300	15360	4.28	0.21	450
13-May-86	700	15840	4.25	0.18	450
13-May-86	1500	16320	4.25	0.18	450
13-May-86	2300	16800	4.23	0.16	450
14-May-86	700	17280	4.21	0.14	450
14-May-86	1500	17760	4.21	0.14	450
14-May-86	2300	18240	4.23	0.16	450
15-May-86	700	18720	4.23	0.16	450
15-May-86	1500	19200	4.14	0.07	450

AQUIFER TEST DATA

Name: Kill Devil Hills 40-day Test of KDH DT
Location: Dare County, NC Well No: MW #2

Date: MAY
Job No:

Date	Time of Day	Elapsed Time (Min)	Water Level Below Ms Point (Feet)	Drawdown (Feet)
15-May-86	2300	19680	4.25	0.18
16-May-86	700	20160	4.25	0.18
16-May-86	1500	20640	4.14	0.07
16-May-86	2300	21120	4.23	0.16
17-May-86	700	21600	4.25	0.18
17-May-86	1500	22080	4.25	0.18
17-May-86	2300	22560	4.25	0.18
18-May-86	700	23040	4.25	0.18
18-May-86	1500	23520	4.25	0.18
18-May-86	2300	24000	4.25	0.18
19-May-86	700	24480	4.25	0.18
19-May-86	1500	24960	4.14	0.07
19-May-86	2300	25440	4.25	0.18
20-May-86	700	25920	4.25	0.18
20-May-86	1500	26400	4.21	0.14
20-May-86	2300	26880	4.25	0.18
21-May-86	700	27360	4.25	0.18
21-May-86	1500	27840	4.21	0.14
21-May-86	2300	28320	4.14	0.07
22-May-86	700	28800	4.14	0.07
22-May-86	1500	29280	4.11	0.04
22-May-86	2300	29760	4.23	0.16
23-May-86	700	30240	4.14	0.07
23-May-86	1500	30720	4.14	0.07
23-May-86	2300	31200	4.25	0.18
24-May-86	700	31680	4.25	0.18
24-May-86	1500	32160	4.25	0.18
24-May-86	2300	32640	4.28	0.21
25-May-86	700	33120	4.23	0.16
25-May-86	1500	33600	4.23	0.16
25-May-86	2300	34080	4.25	0.18
26-May-86	700	34560	4.25	0.18
26-May-86	1500	35040	4.28	0.21
26-May-86	2300	35520	4.25	0.18
27-May-86	700	36000	4.28	0.21
27-May-86	1500	36480	4.30	0.23
27-May-86	2300	36960	4.37	0.30
28-May-86	700	37440	4.25	0.18
28-May-86	1500	37920	4.25	0.18
28-May-86	2300	38400	4.28	0.21
29-May-86	700	38880	4.28	0.21

AQUIFER TEST DATA

Name: Kill Devil Hills 40-day Test of KDH DT Date: MAY 2, 1986
Location: Dare County, NC Well No: MW #2 Job No: HY-0147

Date	Time of Day	Elapsed Time (Min)	Water Level Below Ms Point (Feet)	Drawdown (Feet)	Pumping Rate (GPM)
29-May-86	1500	39360	4.25	0.18	450
29-May-86	2300	39840	4.34	0.27	450
30-May-86	700	40320	4.30	0.23	450
30-May-86	1500	40800	4.25	0.18	450
30-May-86	2300	41280	4.28	0.21	450
31-May-86	700	41760	4.28	0.21	450
31-May-86	1500	42240	4.28	0.21	450
31-May-86	2300	42720	4.28	0.21	450
01-Jun-86	700	43200	4.37	0.30	450
01-Jun-86	1500	43680	4.37	0.30	450
01-Jun-86	2300	44160	4.25	0.18	450
02-Jun-86	700	44640	4.18	0.11	450
02-Jun-86	1500	45120	4.18	0.11	450
02-Jun-86	2300	45600	4.25	0.18	450
03-Jun-86	700	46080	4.04	-0.03	450
03-Jun-86	1500	46560	4.04	-0.03	450
03-Jun-86	2300	47040	4.14	0.07	450
04-Jun-86	700	47520	4.04	-0.03	450
04-Jun-86	1500	48000	4.14	0.07	450
04-Jun-86	2300	48480	4.21	0.14	450
05-Jun-86	700	48960	4.04	-0.03	450
05-Jun-86	1500	49440	4.14	0.07	450
05-Jun-86	2300	49920	4.21	0.14	450
06-Jun-86	700	50400	4.04	-0.03	450
06-Jun-86	1500	50880	4.02	-0.05	450
06-Jun-86	2300	51360	4.14	0.07	450
07-Jun-86	700	51840	4.07	0.00	450
07-Jun-86	1500	52320	3.97	-0.10	450
07-Jun-86	2300	52800	4.00	-0.07	450
08-Jun-86	700	53280	4.02	-0.05	450
08-Jun-86	1500	53760	4.14	0.07	450
08-Jun-86	2300	54240	4.02	-0.05	450
09-Jun-86	700	54720	3.91	-0.16	450
09-Jun-86	1500	55200	4.04	-0.03	450
09-Jun-86	2300	55680	4.07	0.00	450
10-Jun-86	700	56160	4.04	-0.03	450
10-Jun-86	1500	56640	4.04	-0.03	450
10-Jun-86	2300	57120	4.21	0.14	450
11-Jun-86	700	57600	4.04	-0.03	450

AQUIFER TEST DATA

Name: Kill Devil Hills 40-day Test of KDH DT   Date: MAY 2, 1986
Location: Dare County, NC   Well No: MW #3   Job No: HY-0147

Date	Time of Day	Elapsed Time (Min)	Water Level Below Ms Point (Feet)	Drawdown (Feet)	Pumping Rate (GPM)
02-May-86	700	0	6.91	0.00	0
02-May-86	1500	480	10.37	3.46	450
02-May-86	2300	960	11.04	4.13	450
03-May-86	700	1440	10.92	4.01	450
03-May-86	1500	1920	11.22	4.31	450
03-May-86	2300	2400	11.69	4.78	450
04-May-86	700	2880	11.46	4.55	450
04-May-86	1500	3360	11.73	4.82	450
04-May-86	2300	3840	11.80	4.89	450
05-May-86	700	4320	11.69	4.78	450
05-May-86	1500	4800	11.94	5.03	450
05-May-86	2300	5280	11.92	5.01	450
06-May-86	700	5760	11.73	4.82	450
06-May-86	1500	6240	11.85	4.94	450
06-May-86	2300	6720	11.76	4.85	450
07-May-86	700	7200	11.73	4.82	450
07-May-86	1500	7680	11.76	4.85	450
07-May-86	2300	8160	11.62	4.71	450
08-May-86	700	8640	11.62	4.71	450
08-May-86	1500	9120	11.85	4.94	450
08-May-86	2300	9600	11.53	4.62	450
09-May-86	700	10080	11.73	4.82	450
09-May-86	1500	10560	11.76	4.85	450
09-May-86	2300	11040	11.50	4.59	450
10-May-86	700	11520	11.62	4.71	450
10-May-86	1500	12000	11.92	5.01	450
10-May-86	2300	12480	11.53	4.62	450
11-May-86	700	12960	11.96	5.05	450
11-May-86	1500	13440	11.73	4.82	450
11-May-86	2300	13920	11.78	4.87	450
12-May-86	700	14400	12.08	5.17	450
12-May-86	1500	14880	11.94	5.03	450
12-May-86	2300	15360	11.73	4.82	450
13-May-86	700	15840	11.96	5.05	450
13-May-86	1500	16320	11.94	5.03	450
13-May-86	2300	16800	11.69	4.78	450
14-May-86	700	17280	11.99	5.08	450
14-May-86	1500	17760	11.92	5.01	450
14-May-86	2300	18240	11.85	4.94	450
15-May-86	700	18720	12.20	5.29	450
15-May-86	1500	19200	11.92	5.01	450

AQUIFER TEST DATA

Name: Kill Devil Hills 40-day Test of KDH DT
Location: Dare County, NC Well No: MW #3

Date: MAY 2, 1986
Job No: HY-0147

Date	Time of Day	Elapsed Time (Min)	Water Level Below Ms Point (Feet)	Drawdown (Feet)	Pumping Rate (GPM)
15-May-86	2300	19680	11.99	5.08	450
16-May-86	700	20160	12.23	5.32	450
16-May-86	1500	20640	11.92	5.01	450
16-May-86	2300	21120	12.08	5.17	450
17-May-86	700	21600	12.20	5.29	450
17-May-86	1500	22080	12.08	5.17	450
17-May-86	2300	22560	11.96	5.05	450
18-May-86	700	23040	12.20	5.29	450
18-May-86	1500	23520	11.99	5.08	450
18-May-86	2300	24000	12.20	5.29	450
19-May-86	700	24480	12.08	5.17	450
19-May-86	1500	24960	12.06	5.15	450
19-May-86	2300	25440	12.31	5.40	450
20-May-86	700	25920	12.01	5.10	450
20-May-86	1500	26400	12.19	5.28	450
20-May-86	2300	26880	12.24	5.33	450
21-May-86	700	27360	11.96	5.05	450
21-May-86	1500	27840	12.17	5.26	450
21-May-86	2300	28320	12.10	5.19	450
22-May-86	700	28800	11.92	5.01	450
22-May-86	1500	29280	12.19	5.28	450
22-May-86	2300	29760	11.96	5.05	450
23-May-86	700	30240	11.96	5.05	450
23-May-86	1500	30720	12.43	5.52	450
23-May-86	2300	31200	11.92	5.01	450
24-May-86	700	31680	12.20	5.29	450
24-May-86	1500	32160	12.19	5.28	450
24-May-86	2300	32640	11.69	4.78	450
25-May-86	700	33120	12.20	5.29	450
25-May-86	1500	33600	12.43	5.52	450
25-May-86	2300	34080	11.99	5.08	450
26-May-86	700	34560	12.17	5.26	450
26-May-86	1500	35040	12.45	5.54	450
26-May-86	2300	35520	11.96	5.05	450
27-May-86	700	36000	12.43	5.52	450
27-May-86	1500	36480	12.24	5.33	450
27-May-86	2300	36960	12.10	5.19	450
28-May-86	700	37440	12.43	5.52	450
28-May-86	1500	37920	12.17	5.26	450
28-May-86	2300	38400	12.24	5.33	450
29-May-86	700	38880	12.47	5.56	450

AQUIFER TEST DATA

Name: Kill Devil Hills 40-day Test of KDH DT   Date: MAY 2, 1986
Location: Dare County, NC   Well No: MW #3   Job No: HY-0147

Date	Time of Day	Elapsed Time (Min)	Water Level Below Ms Point (Feet)	Drawdown (Feet)	Pumping Rate (GPM)
29-May-86	1500	39360	12.19	5.28	450
29-May-86	2300	39840	12.38	5.47	450
30-May-86	700	40320	12.40	5.49	450
30-May-86	1500	40800	12.08	5.17	450
30-May-86	2300	41280	12.24	5.33	450
31-May-86	700	41760	12.31	5.40	450
31-May-86	1500	42240	12.17	5.26	450
31-May-86	2300	42720	12.43	5.52	450
01-Jun-86	700	43200	12.31	5.40	450
01-Jun-86	1500	43680	12.17	5.26	450
01-Jun-86	2300	44160	12.20	5.29	450
02-Jun-86	700	44640	12.43	5.52	450
02-Jun-86	1500	45120	12.01	5.10	450
02-Jun-86	2300	45600	12.24	5.33	450
03-Jun-86	700	46080	12.20	5.29	450
03-Jun-86	1500	46560	12.45	5.54	450
03-Jun-86	2300	47040	12.52	5.61	450
04-Jun-86	700	47520	12.61	5.70	450
04-Jun-86	1500	48000	12.61	5.70	450
04-Jun-86	2300	48480	12.45	5.54	450
05-Jun-86	700	48960	12.54	5.63	450
05-Jun-86	1500	49440	12.68	5.77	450
05-Jun-86	2300	49920	12.43	5.52	450
06-Jun-86	700	50400	12.45	5.54	450
06-Jun-86	1500	50880	12.59	5.68	450
06-Jun-86	2300	51360	12.24	5.33	450
07-Jun-86	700	51840	12.31	5.40	450
07-Jun-86	1500	52320	12.24	5.33	450
07-Jun-86	2300	52800	12.17	5.26	450
08-Jun-86	700	53280	12.40	5.49	450
08-Jun-86	1500	53760	12.43	5.52	450
08-Jun-86	2300	54240	12.08	5.17	450
09-Jun-86	700	54720	12.39	5.48	450
09-Jun-86	1500	55200	12.31	5.40	450
09-Jun-86	2300	55680	12.20	5.29	450
10-Jun-86	700	56160	12.63	5.72	450
10-Jun-86	1500	56640	12.66	5.75	450
10-Jun-86	2300	57120	12.38	5.47	450
11-Jun-86	700	57600	12.66	5.75	450

AQUIFER TEST DATA

Name: Kill Devil Hills 40-day Test of KDH DT Date: MAY 2, 1986
Location: Dare County, NC Well No: MW #4 Job No: HY-0147

Date	Time of Day	Elapsed Time (Min)	Water Level Below Ms Point (Feet)	Drawdown (Feet)	Pumping Rate (GPM)
02-May-86	700	0	4.85	0.00	0
02-May-86	1500	480	7.66	2.81	450
02-May-86	2300	960	8.49	3.64	450
03-May-86	700	1440	8.40	3.55	450
03-May-86	1500	1920	8.49	3.64	450
03-May-86	2300	2400	9.07	4.22	450
04-May-86	700	2880	8.73	3.88	450
04-May-86	1500	3360	8.96	4.11	450
04-May-86	2300	3840	9.21	4.36	450
05-May-86	700	4320	9.14	4.29	450
05-May-86	1500	4800	9.12	4.27	450
05-May-86	2300	5280	9.21	4.36	450
06-May-86	700	5760	9.16	4.31	450
06-May-86	1500	6240	9.00	4.15	450
06-May-86	2300	6720	9.19	4.34	450
07-May-86	700	7200	9.14	4.29	450
07-May-86	1500	7680	8.96	4.11	450
07-May-86	2300	8160	9.07	4.22	450
08-May-86	700	8640	9.07	4.22	450
08-May-86	1500	9120	9.19	4.34	450
08-May-86	2300	9600	8.96	4.11	450
09-May-86	700	10080	9.19	4.34	450
09-May-86	1500	10560	9.14	4.29	450
09-May-86	2300	11040	8.96	4.11	450
10-May-86	700	11520	9.21	4.36	450
10-May-86	1500	12000	9.16	4.31	450
10-May-86	2300	12480	8.77	3.92	450
11-May-86	700	12960	9.30	4.45	450
11-May-86	1500	13440	9.23	4.38	450
11-May-86	2300	13920	9.16	4.31	450
12-May-86	700	14400	9.42	4.57	450
12-May-86	1500	14880	9.19	4.34	450
12-May-86	2300	15360	9.19	4.34	450
13-May-86	700	15840	9.40	4.55	450
13-May-86	1500	16320	9.19	4.34	450
13-May-86	2300	16800	9.14	4.29	450
14-May-86	700	17280	9.42	4.57	450
14-May-86	1500	17760	9.84	4.99	450
14-May-86	2300	18240	9.26	4.41	450
15-May-86	700	18720	9.63	4.78	450
15-May-86	1500	19200	9.16	4.31	450

AQUIFER TEST DATA

Name: Kill Devil Hills 40-day Test of KDH DT   Date: MAY 2, 1986
Location: Dare County, NC   Well No: MW #4   Job No: HY-0147

Date	Time of Day	Elapsed Time (Min)	Water Level Below Ms Point (Feet)	Drawdown (Feet)	Pumping Rate (GPM)
15-May-86	2300	19680	9.44	4.59	450
16-May-86	700	20160	9.53	4.68	450
16-May-86	1500	20640	9.16	4.31	450
16-May-86	2300	21120	9.47	4.62	450
17-May-86	700	21600	9.65	4.80	450
17-May-86	1500	22080	9.51	4.66	450
17-May-86	2300	22560	9.67	4.82	450
18-May-86	700	23040	9.70	4.85	450
18-May-86	1500	23520	9.07	4.22	450
18-May-86	2300	24000	9.65	4.80	450
19-May-86	700	24480	9.47	4.62	450
19-May-86	1500	24960	9.33	4.48	450
19-May-86	2300	25440	9.67	4.82	450
20-May-86	700	25920	9.44	4.59	450
20-May-86	1500	26400	9.65	4.80	450
20-May-86	2300	26880	9.70	4.85	450
21-May-86	700	27360	9.42	4.57	450
21-May-86	1500	27840	9.42	4.57	450
21-May-86	2300	28320	9.53	4.68	450
22-May-86	700	28800	9.37	4.52	450
22-May-86	1500	29280	9.63	4.78	450
22-May-86	2300	29760	9.44	4.59	450
23-May-86	700	30240	9.42	4.57	450
23-May-86	1500	30720	9.67	4.82	450
23-May-86	2300	31200	9.65	4.80	450
24-May-86	700	31680	9.65	4.80	450
24-May-86	1500	32160	9.63	4.78	450
24-May-86	2300	32640	9.42	4.57	450
25-May-86	700	33120	9.65	4.80	450
25-May-86	1500	33600	9.70	4.85	450
25-May-86	2300	34080	9.44	4.59	450
26-May-86	700	34560	9.47	4.62	450
26-May-86	1500	35040	9.83	4.98	450
26-May-86	2300	35520	9.47	4.62	450
27-May-86	700	36000	10.00	5.15	450
27-May-86	1500	36480	9.62	4.77	450
27-May-86	2300	36960	9.60	4.75	450
28-May-86	700	37440	10.00	5.15	450
28-May-86	1500	37920	9.44	4.59	450
28-May-86	2300	38400	9.77	4.92	450
29-May-86	700	38880	10.00	5.15	450

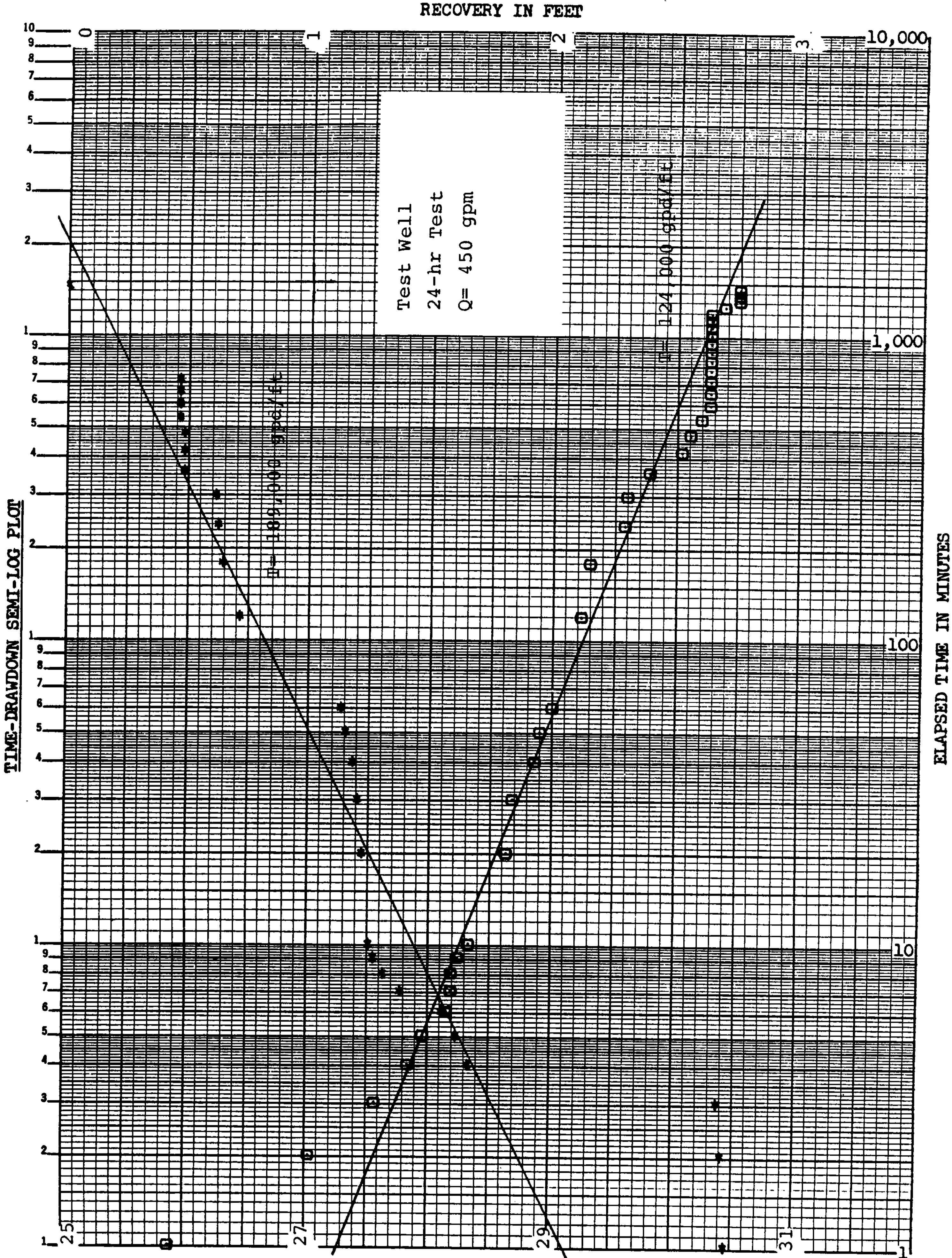
AQUIFER TEST DATA

Name: Kill Devil Hills 40-day Test of KDH DT
Location: Dare County, NC Well No: MW #4
Date: MAY 2, 1986
Job No: HY-0147

Date	Time of Day	Elapsed Time (Min)	Water Level Below Ms Point (Feet)	Drawdown (Feet)	Pumping Rate (GPM)
29-May-86	1500	39360	9.44	4.59	450
29-May-86	2300	39840	9.90	5.05	450
30-May-86	700	40320	9.88	5.03	450
30-May-86	1500	40800	9.40	4.55	450
30-May-86	2300	41280	9.65	4.80	450
31-May-86	700	41760	9.77	4.92	450
31-May-86	1500	42240	9.42	4.57	450
31-May-86	2300	42720	9.88	5.03	450
01-Jun-86	700	43200	9.77	4.92	450
01-Jun-86	1500	43680	9.63	4.78	450
01-Jun-86	2300	44160	9.63	4.78	450
02-Jun-86	700	44640	9.77	4.92	450
02-Jun-86	1500	45120	9.47	4.62	450
02-Jun-86	2300	45600	9.65	4.80	450
03-Jun-86	700	46080	9.72	4.87	450
03-Jun-86	1500	46560	9.90	5.05	450
03-Jun-86	2300	47040	10.07	5.22	450
04-Jun-86	700	47520	10.04	5.19	450
04-Jun-86	1500	48000	9.95	5.10	450
04-Jun-86	2300	48480	10.07	5.22	450
05-Jun-86	700	48960	10.07	5.22	450
05-Jun-86	1500	49440	10.07	5.22	450
05-Jun-86	2300	49920	9.90	5.05	450
06-Jun-86	700	50400	10.00	5.15	450
06-Jun-86	1500	50880	10.07	5.22	450
06-Jun-86	2300	51360	9.77	4.92	450
07-Jun-86	700	51840	9.83	4.98	450
07-Jun-86	1500	52320	9.86	5.01	450
07-Jun-86	2300	52800	9.65	4.80	450
08-Jun-86	700	53280	9.65	4.80	450
08-Jun-86	1500	53760	9.77	4.92	450
08-Jun-86	2300	54240	9.60	4.75	450
09-Jun-86	700	54720	9.92	5.07	450
09-Jun-86	1500	55200	9.65	4.80	450
09-Jun-86	2300	55680	9.70	4.85	450
10-Jun-86	700	56160	10.16	5.31	450
10-Jun-86	1500	56640	10.07	5.22	450
10-Jun-86	2300	57120	9.88	5.03	450
11-Jun-86	700	57600	10.16	5.31	450

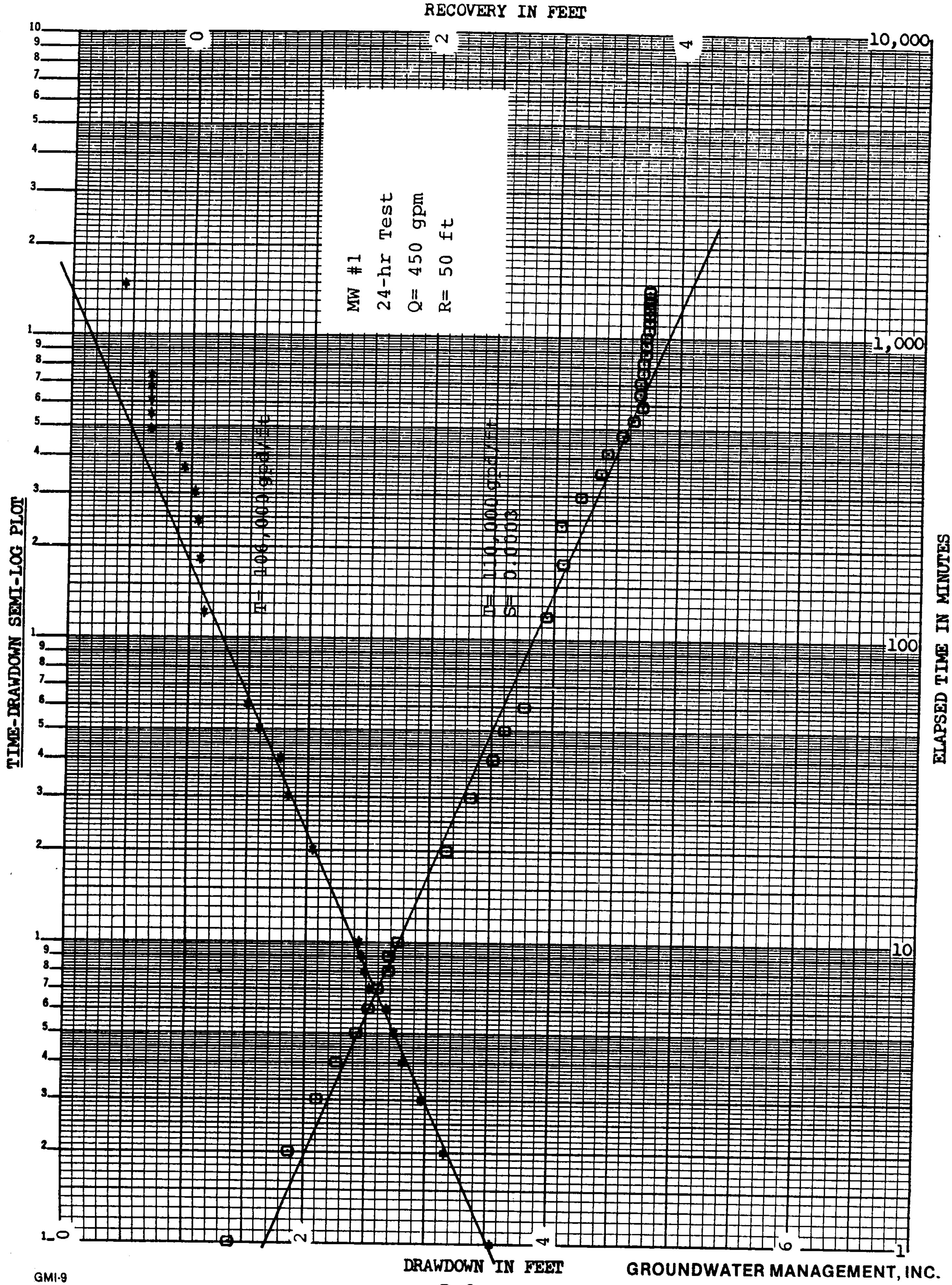
APPENDIX B

Test Analysis Plots

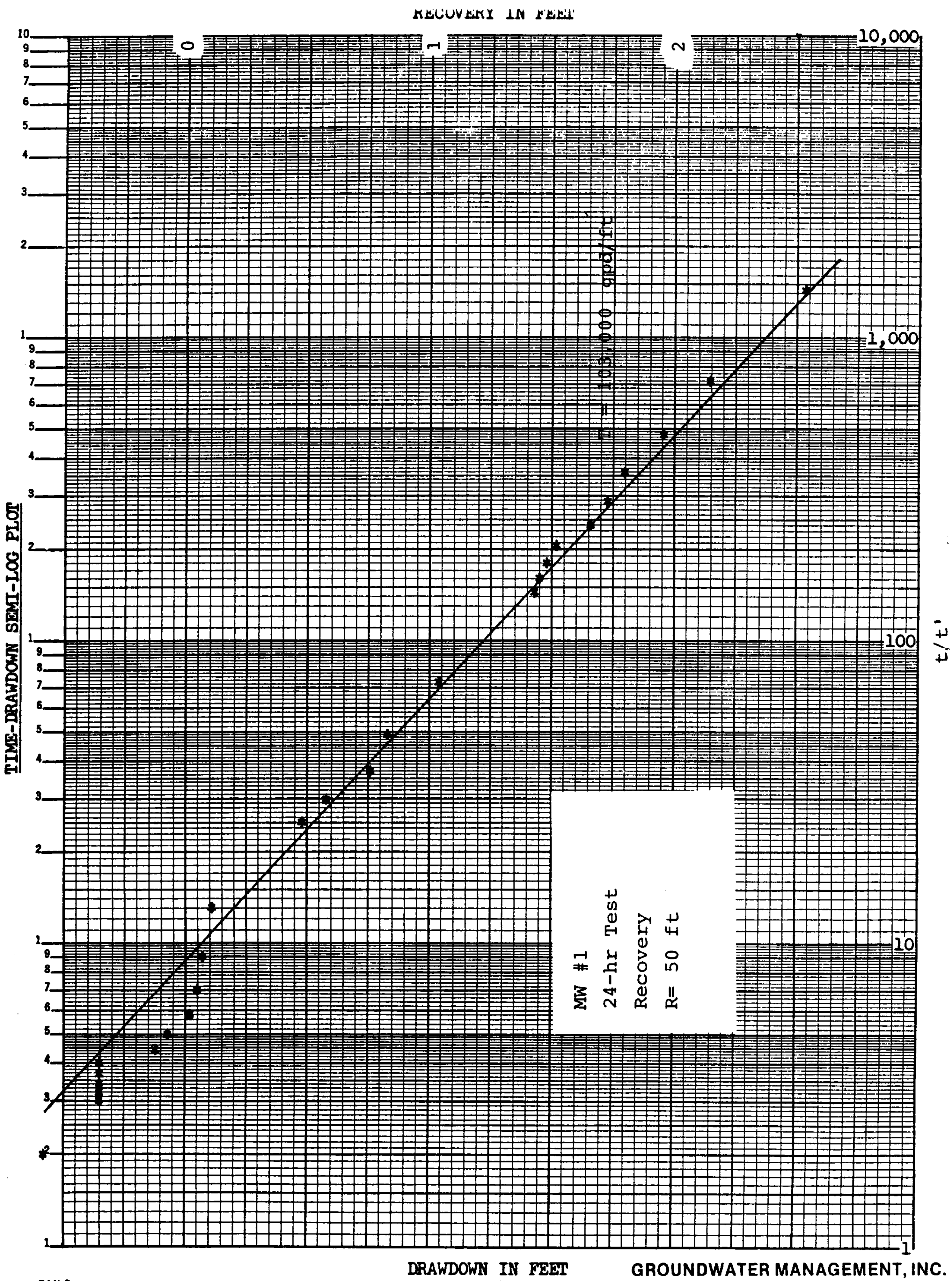


GROUNDWATER MANAGEMENT, INC.

GMI-9

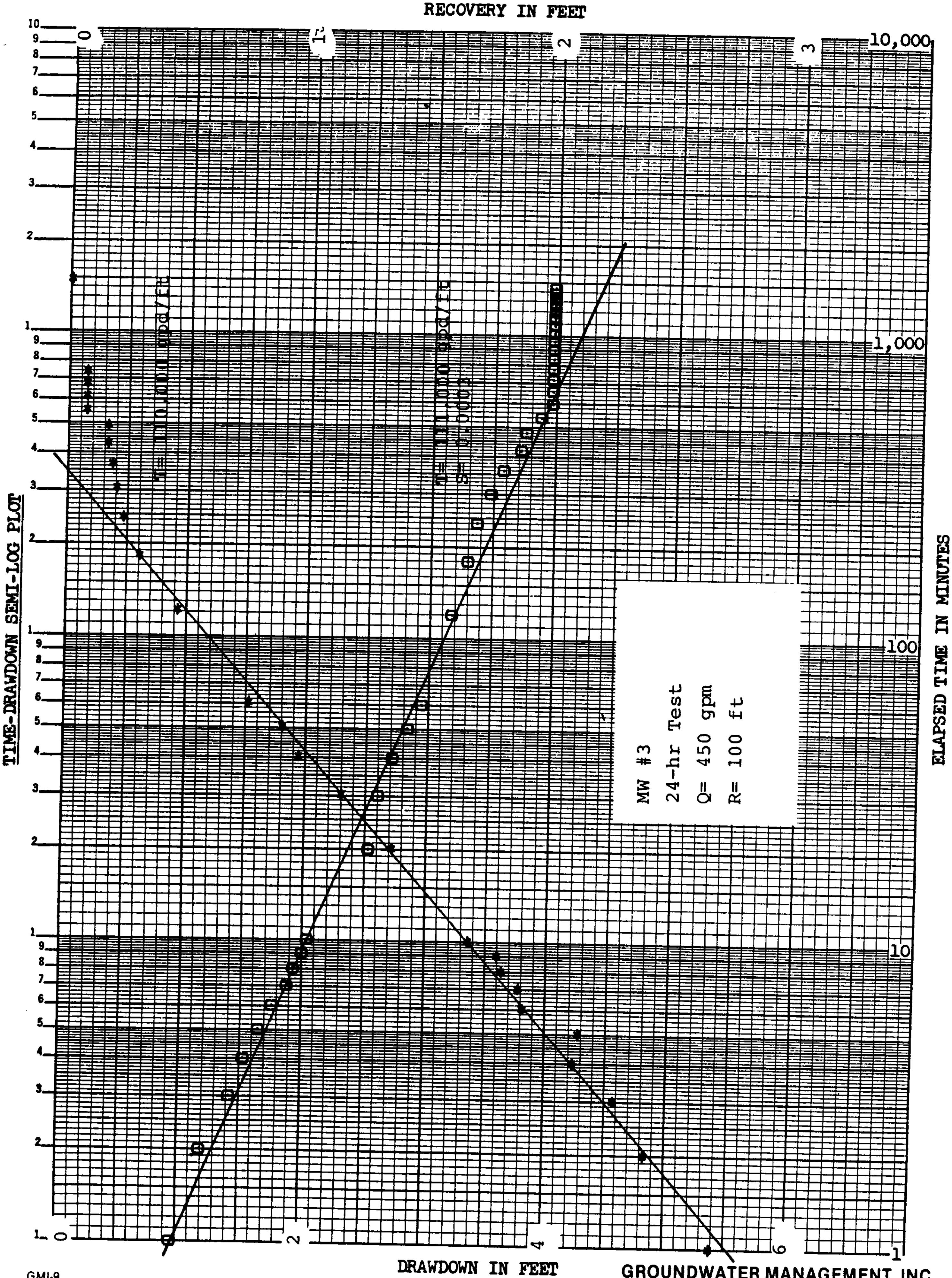


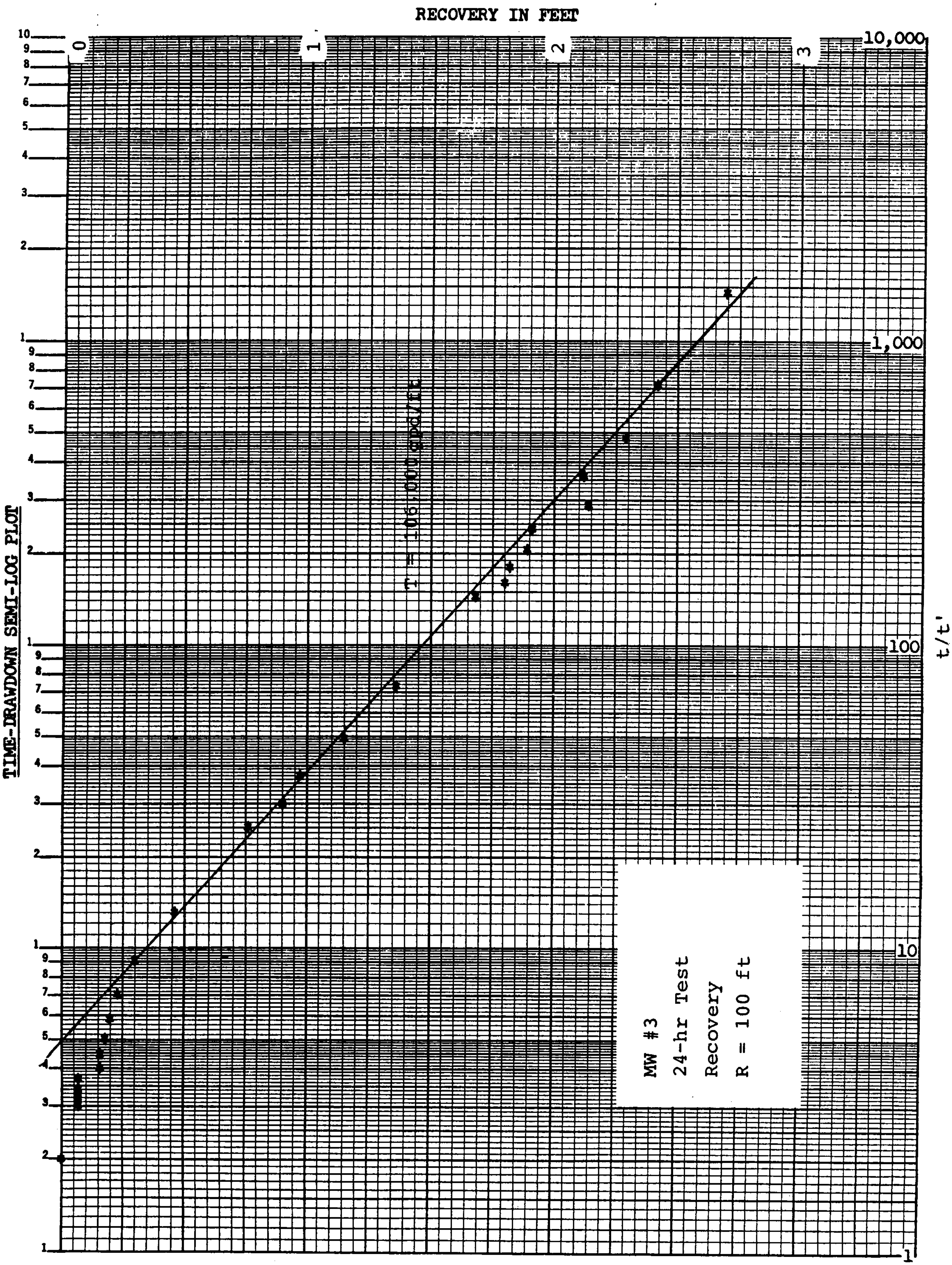
RECOVERY IN FEET
TIME-DRAWDOWN SEMI-LOG PLOT
MW #1
24-hr Test
Q= 450 gpm
R= 50 ft
T= 106,000 gpd/ft
T= 110,000 gpd/ft
S= 0.0003
ELAPSED TIME IN MINUTES
DRAWDOWN IN FEET
GROUNDWATER MANAGEMENT, INC.



GMI-9

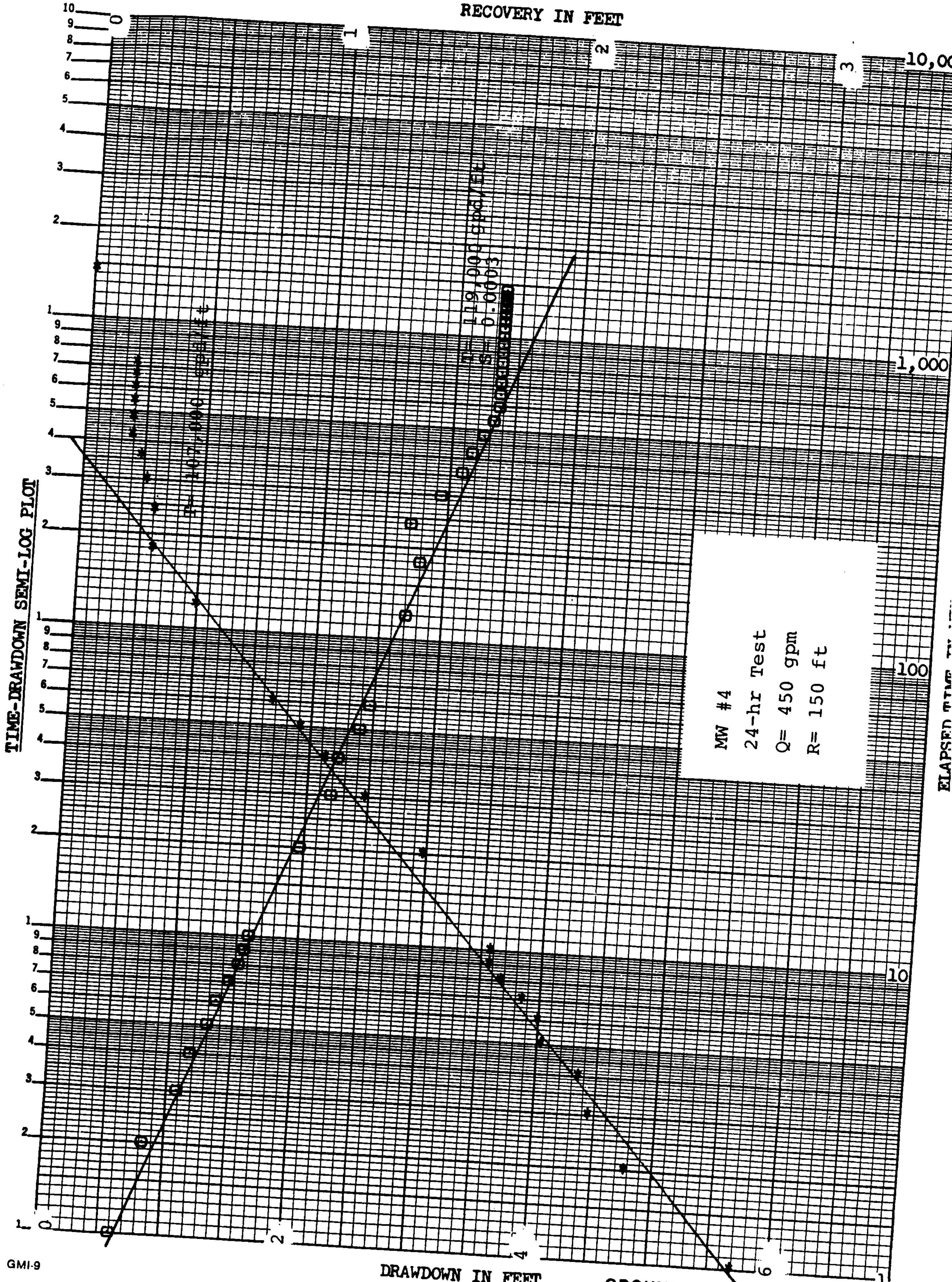
GROUNDWATER MANAGEMENT, INC.


RECOVERY IN FEET
TIME-DRAWDOWN SEMI-LOG PLOT
T= 110,000 gpd/ft
T= 111,000 gpd/ft
S= 0.0003
MW #3
24-hr Test
Q= 450 gpm
R= 100 ft
ELAPSED TIME IN MINUTES
DRAWDOWN IN FEET
GROUNDWATER MANAGEMENT, INC.
GMI-9



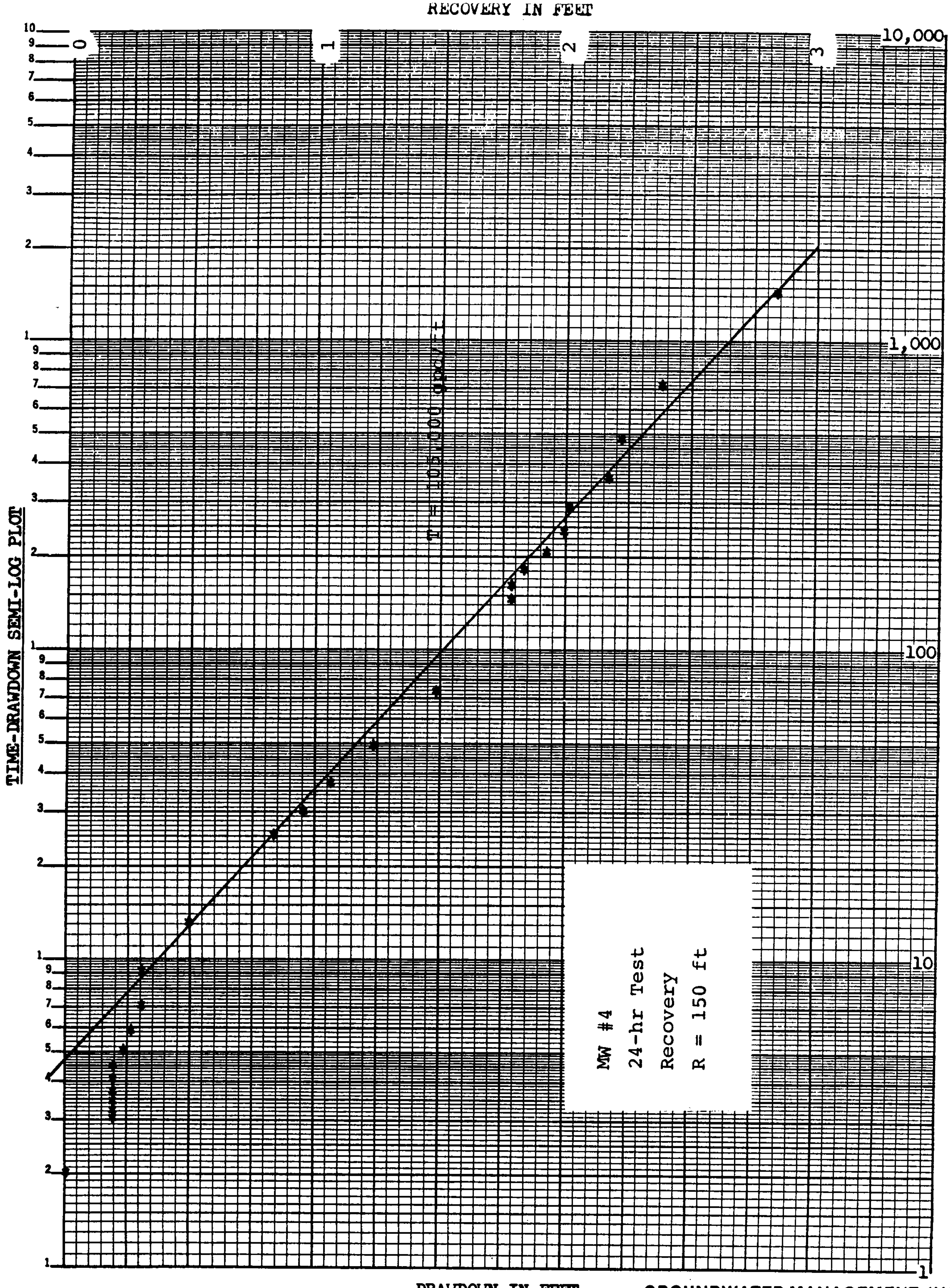
GROUNDWATER MANAGEMENT, INC.

GMI-9



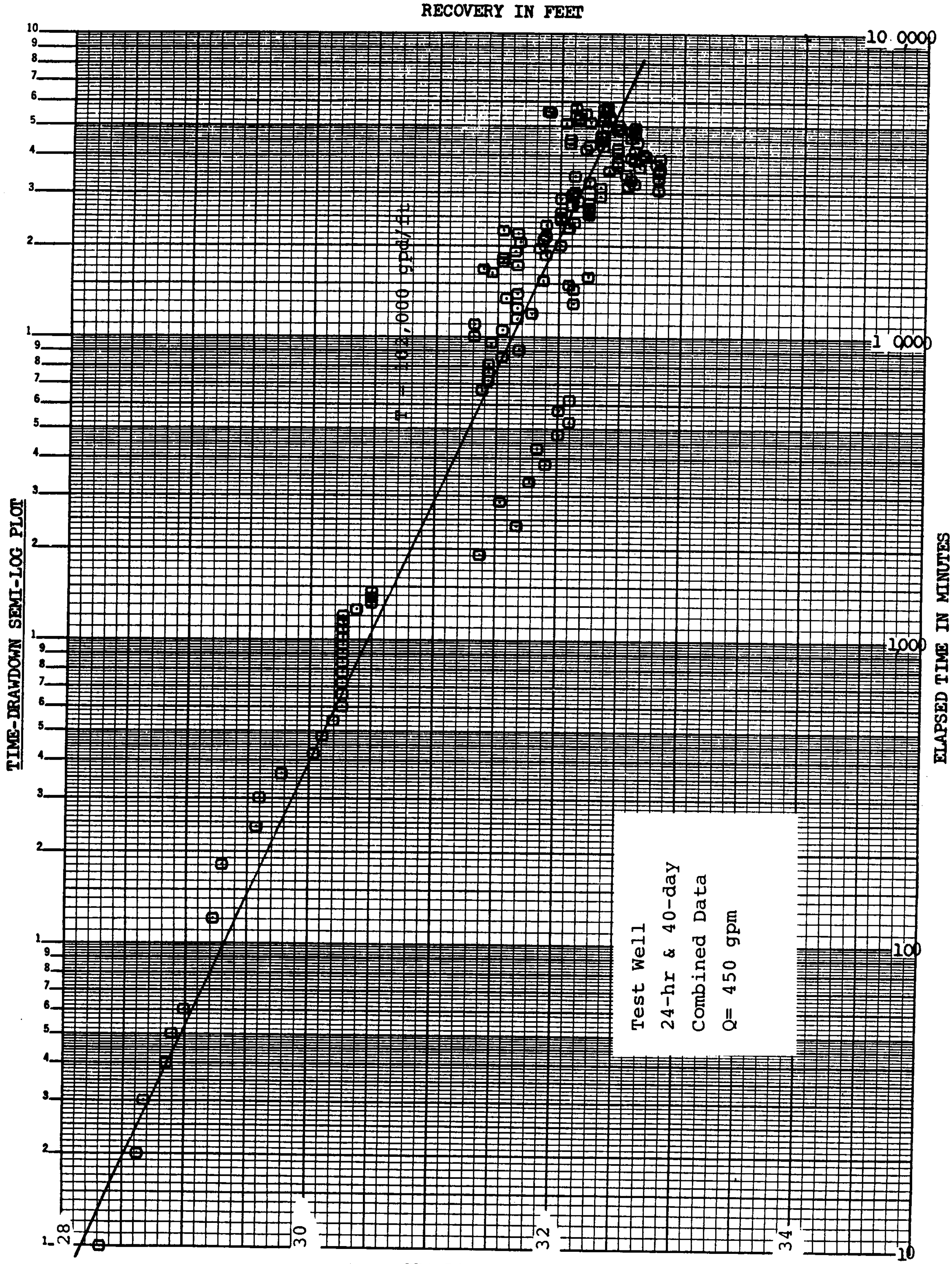
RECOVERY IN FEET
TIME-DRAWDOWN SEMI-LOG PLOT
T= 107,000 gpd/ft
T= 119,000 gpd/ft
S= 0.0003
MW #4
24-hr Test
Q= 450 gpm
R= 150 ft
10,000
1,000
100
10
1
DRAWDOWN IN FEET

GMI-9

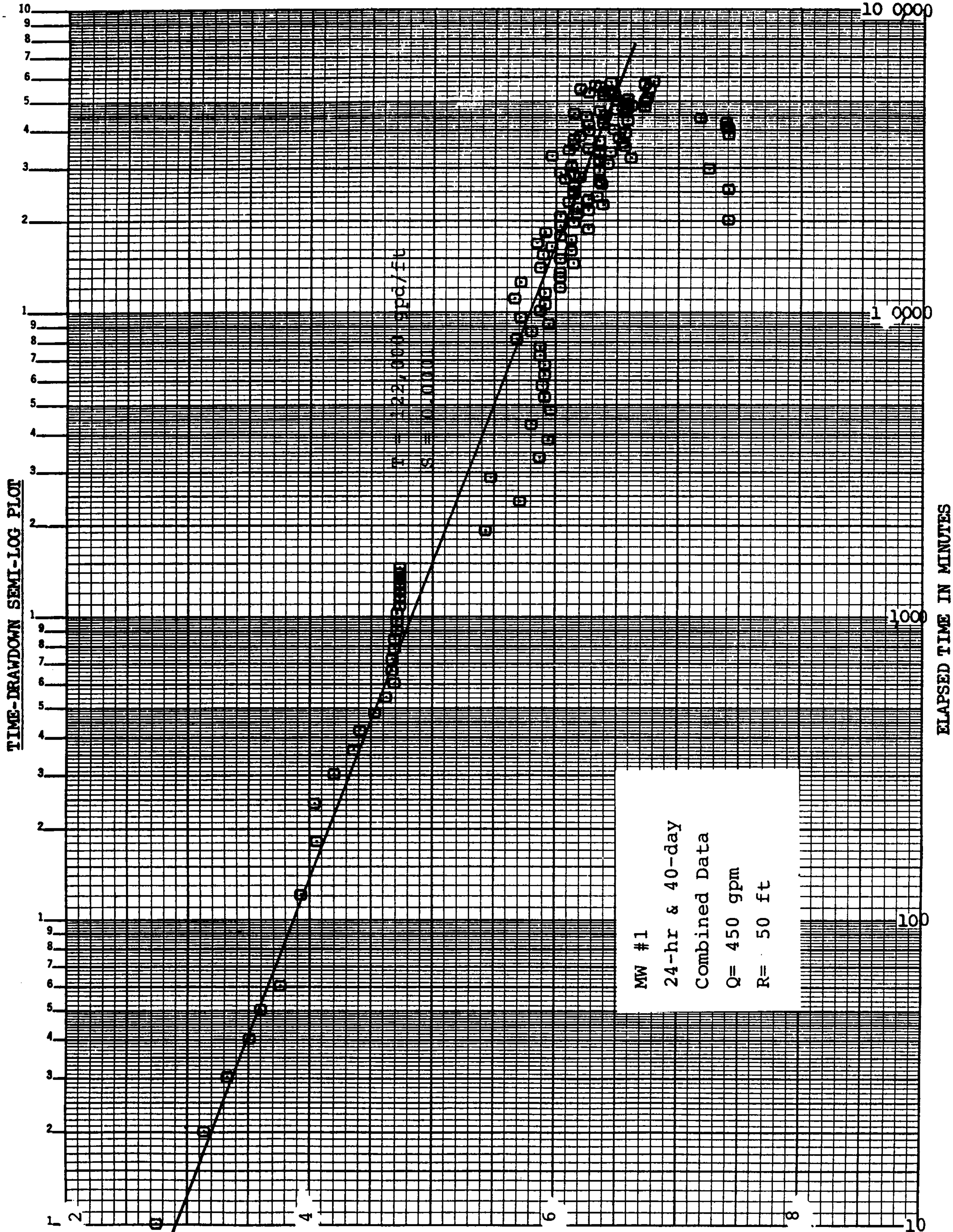


GMI-9

GROUNDWATER MANAGEMENT, INC.


RECOVERY IN FEET
TIME-DRAWDOWN SEMI-LOG PLOT
T = 102,000 gpd/ft
Test Well
24-hr & 40-day
Combined Data
Q= 450 gpm
ELAPSED TIME IN MINUTES
10
100
1000
1 0000
10 0000
28
30
32
34
DRAWDOWN IN FEET

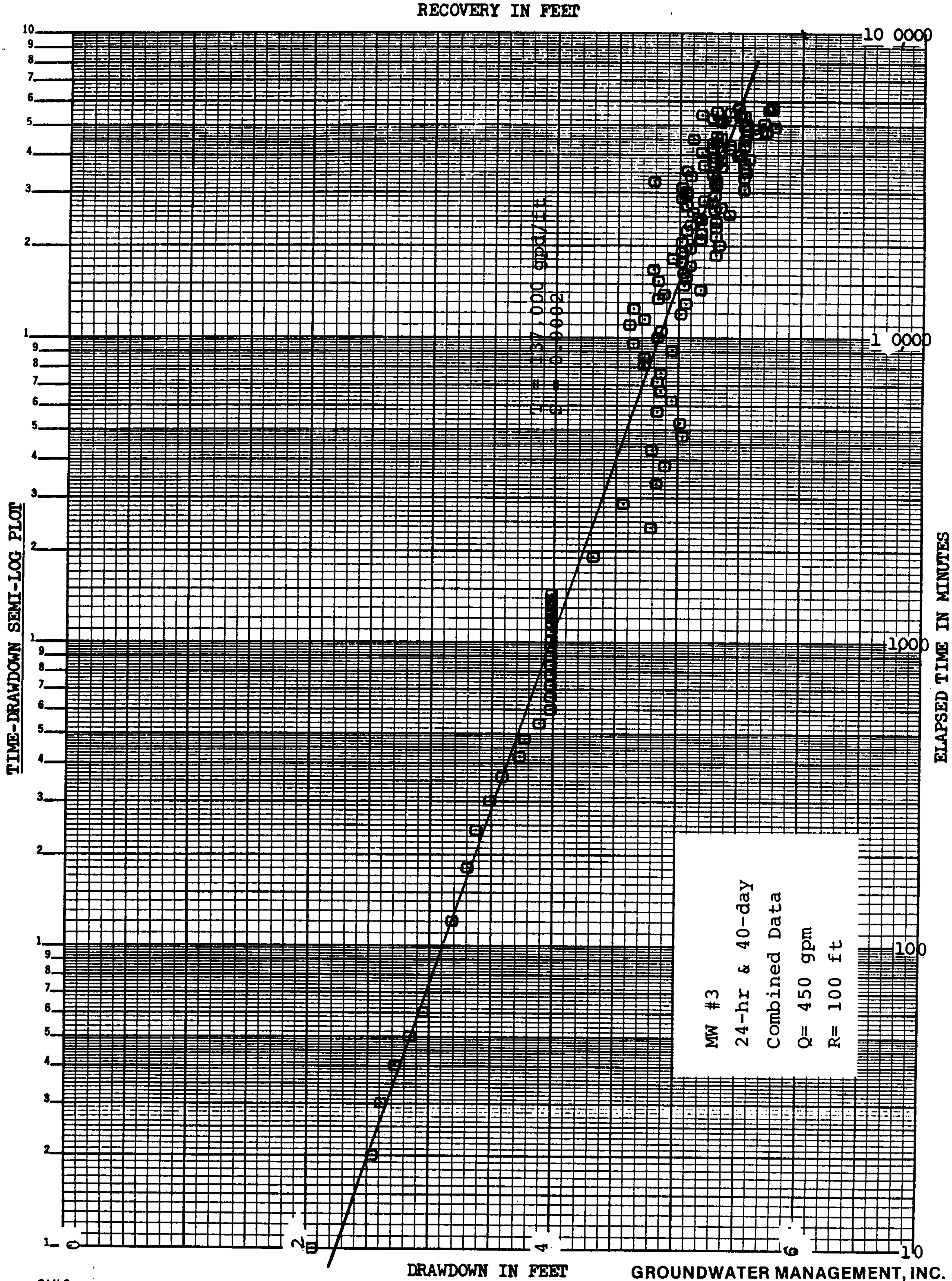
RECOVERY IN FEET



DRAWDOWN IN FEET

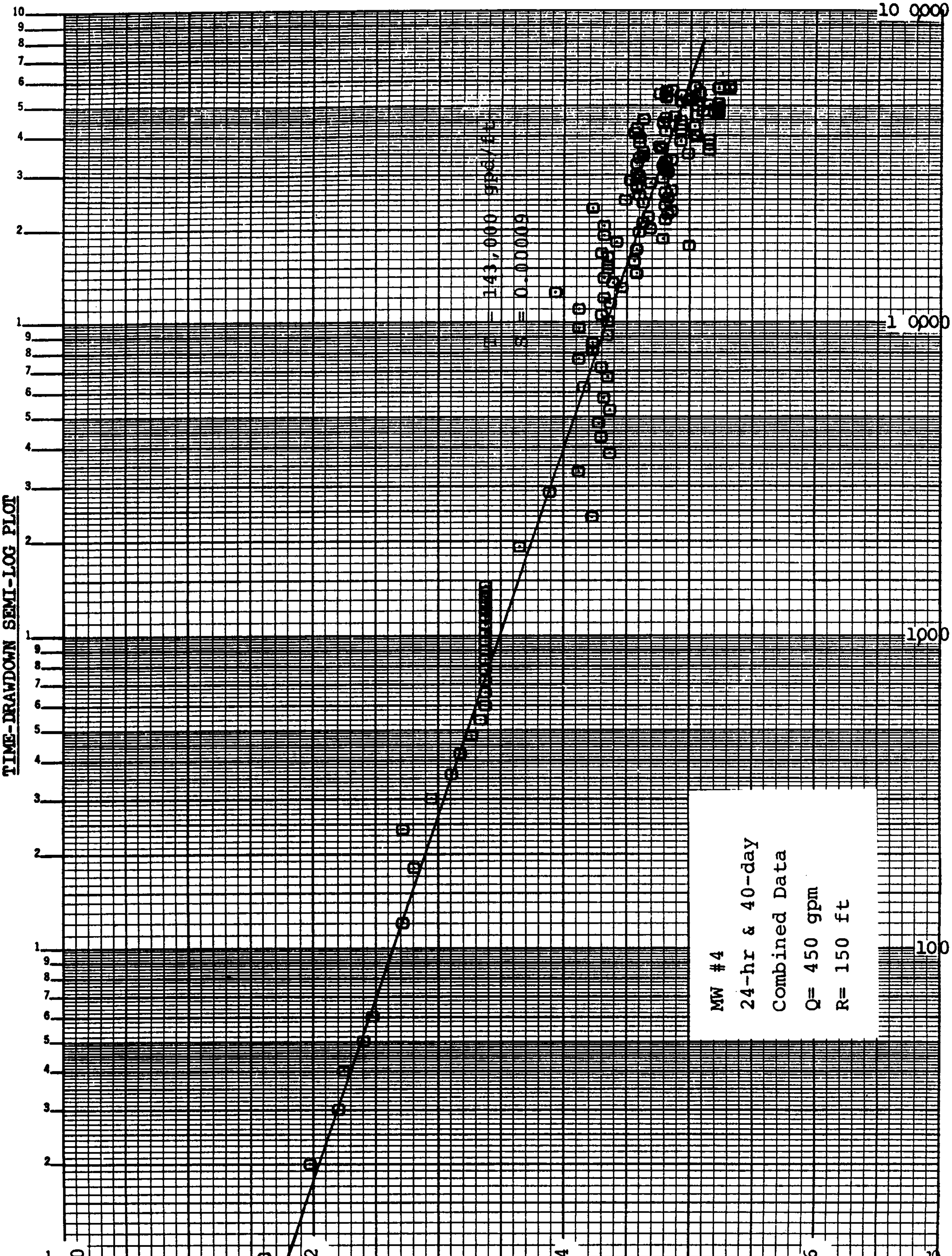
GROUNDWATER MANAGEMENT, INC.

GMI-9


RECOVERY IN FEET
TIME-DRAWDOWN SEMI-LOG PLOT
ELAPSED TIME IN MINUTES
DRAWDOWN IN FEET
T = 137,000 gpd/ft
S = 0.0002
MW #3
24-hr & 40-day
Combined Data
Q= 450 gpm
R= 100 ft
GMI-9
GROUNDWATER MANAGEMENT, INC.

RECOVERY IN FEET

TIME-DRAWDOWN SEMI-LOG PLOT

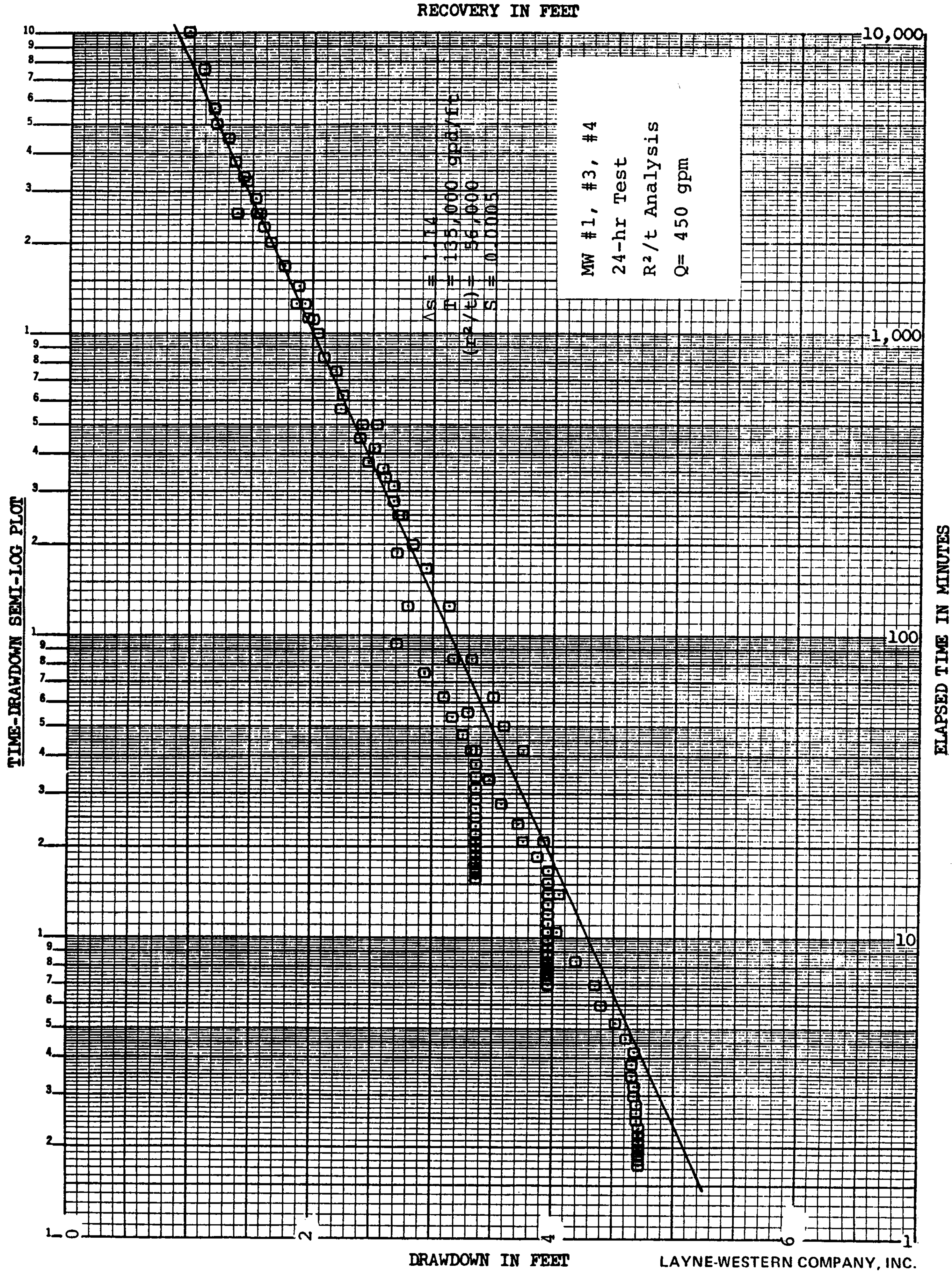


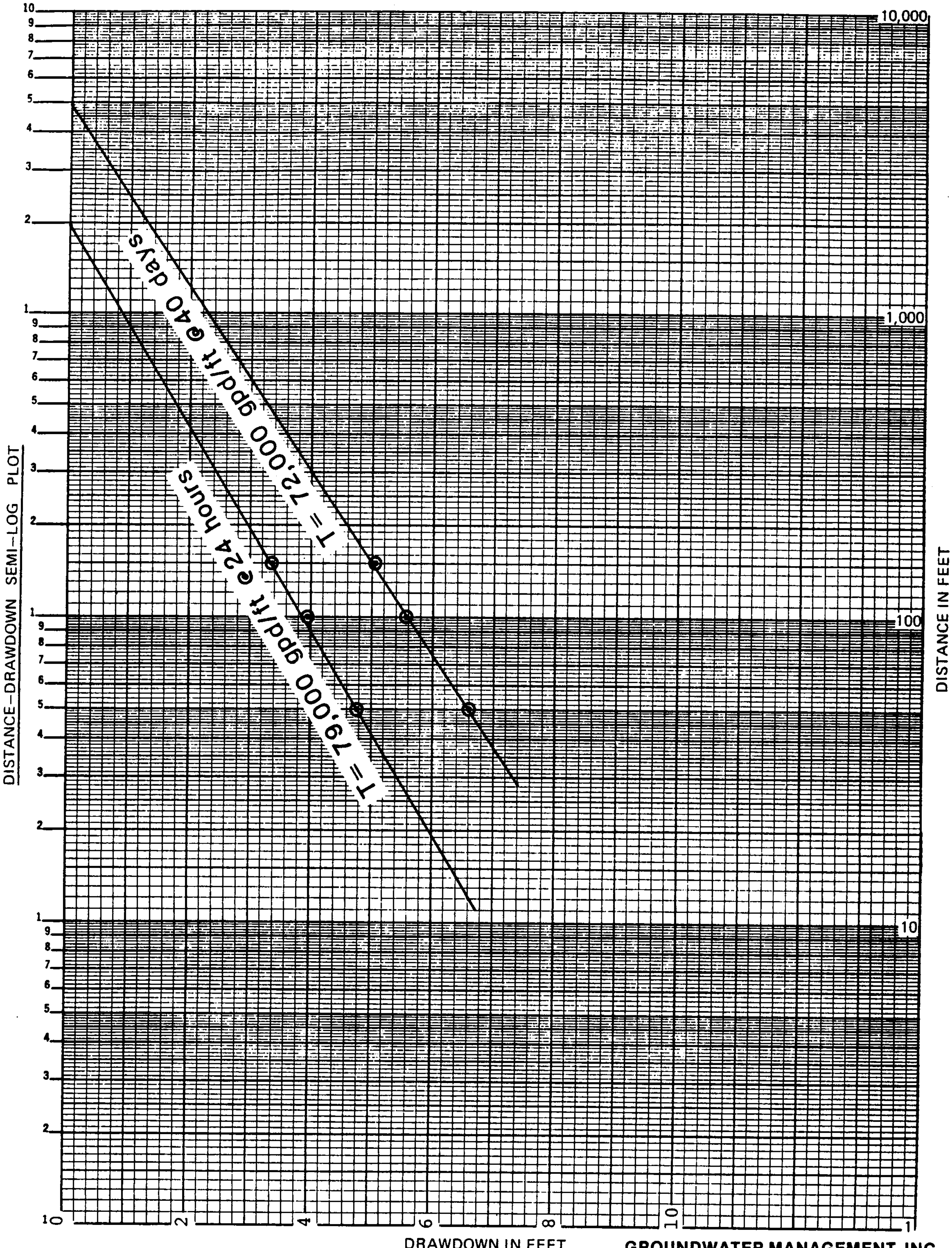
ELAPSED TIME IN MINUTES

DRAWDOWN IN FEET

GROUNDWATER MANAGEMENT, INC.

GMI-9


RECOVERY IN FEET
TIME-DRAWDOWN SEMI-LOG PLOT
ELAPSED TIME IN MINUTES
DRAWDOWN IN FEET
10,000
1,000
100
10
1
0
2
4
6
MW #1, #3, #4
24-hr Test
R²/t Analysis
Q= 450 gpm
Δs = 1.14
T = 135,000 gpd/ft
(r²/t)₀ = 56,000
S = 0.0005
LAYNE-WESTERN COMPANY, INC.



GMI-10

GROUNDWATER MANAGEMENT, INC.