COMPREHENSIVE ENGINEERING REPORT

ON

WATER SYSTEM IMPROVEMENTS

FOR

COUNTY OF DARE

DARE COUNTY, NORTH CAROLINA

BOARD OF COMMISSIONERS

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MARCH, 1984

MOORE, GARDNER & ASSOCIATES, INC.
CONSULTING ENGINEERS
A SUBSIDIARY OF BLACK & VEATCH ENGINEERS-ARCHITECTS

MGA PROJECT NO. 2574

I hereby certify this 1ST day of MARCH 1984, that this report was prepared by me or under my direct supervision.

NORTH CAROLINA
PROFESSIONAL
(SEAL)
SEAL
4649
ENGINEER
Y. ELLIS SPAKE, JR.

By ______________________
Y. Ellis Spake, P.E.
Project Manager

By ______________________
A. Barry Nelson, CPG
Project Geologist

By ______________________
Fred Waters

By ______________________
Dave Todd, P.E.

By ______________________
Larry Pearce, P.E.

Reviewed and Approved by Moore, Gardner & Associates, Inc.

____
Joseph E. Hardee, P.E.
President

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ACKNOWLEDGEMENTS

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1.0 SUMMARY AND RECOMMENDATIONS

The Dare Beaches and Roanoke Island sections of Dare County are experiencing a rapid level of growth and development. Roanoke Island experienced a 36% growth rate in population between 1970 and 1980. The Dare Beaches, in contrast, saw a population growth of 102% during the same period. Since the 1980 Census, growth has continued at an unchecked level. Based on these trends, it is projected that the present population of 88,262 inhabitants (both seasonal and permanent) will increase to approximately 201,000 by the year 2005. This tripling of the population will have a dramatic affect on the ability of the Dare County Water System to provide potable water to these residents.

Examination of the water system components reveals that immediate upgrading is required to enable the system to furnish the peak water demands during the summer of 1984. These are stop gap measures only. Overall expansion, addition and upgrading of the systems beginning in 1985 are required to track the greatly increasing demand for water. These improvements have been targeted for a three phase development plan that will require upgrading of the water treatment plant from the present 5 mgd to 10 mgd and ultimately to 15 mgd, installation of new transmission facilities including booster pump stations and mains, upgrading the raw water system to a total of 22 wells, addition of new service areas in Manteo and Wanchese, and additional storage facilities on Roanoke Island and the Dare Beaches. A long range master plan for development of the water system is given in Section 7.7 of this report. This plan, which cites the needed improvements, and their costs, and the target date for implementation, should serve as a valuable planning tool for managing the water system over the next 20 years. Periodic updating of this plan may be required to determine if any modifications to the original outline are required as water demands and population increases.

An analysis of the present rate structure was conducted to determine the necessity for revision to allow financing of the system upgrading. Flat rate charges were developed for both wholesale and retail customers based on water consumption to recover O&M costs. Capital improvement cost recovery was based on reserved capacity of the water treatment plant for wholesale customers and peak water consumption for retail customers. Rate restructuring also assumed financing Phase I improvements with a $200,000 Clean Water Bond Grant, a FmHA loan and impact fee revenues for the remaining Phase I costs.

Capital improvements for Phase II and III will be recovered through impact assessment and fixed rate charges. Specific rate figures can be found in Section 8.0.

The Engineers recommend the County consider adopting the new rate structure and begin immediate implementation of the Master Development Plan. This will assure that adequate water supplies will be available to Roanoke Island and the Beaches ares of Dare County as the dramatic growth and development activities continue to increase.

2.0 INTRODUCTION

2.1 STUDY PURPOSE AND SCOPE

The Dare County Commissioners, realizing the rapid growth and development occurring in the County and its impact on the existing water supply, contracted with Moore, Gardner & Associates, Inc. to undertake an extensive investigation to determine the immediate and long range needs of the County Water System. This report addresses six major points; County population growth trends, projected water demands, water supply sources, system improvements, financing methodologies, and water rate structure. Although this is a county-wide study, it is mainly concerned with the heavily populated areas of Roanoke Island and the Dare Beaches. No portion of the County's water system exists on the mainland at this time; therefore, a limited discussion of the mainland will be presented in this report. Discussions on the southern part of Hatteras Island, which is served by the Cape Hatteras Water Association (CHWA), also will be limited in this study.

The report will outline alternatives and make recommendations for development of water supplies and related systems in order to provide for future water needs of the County to the year 2005.

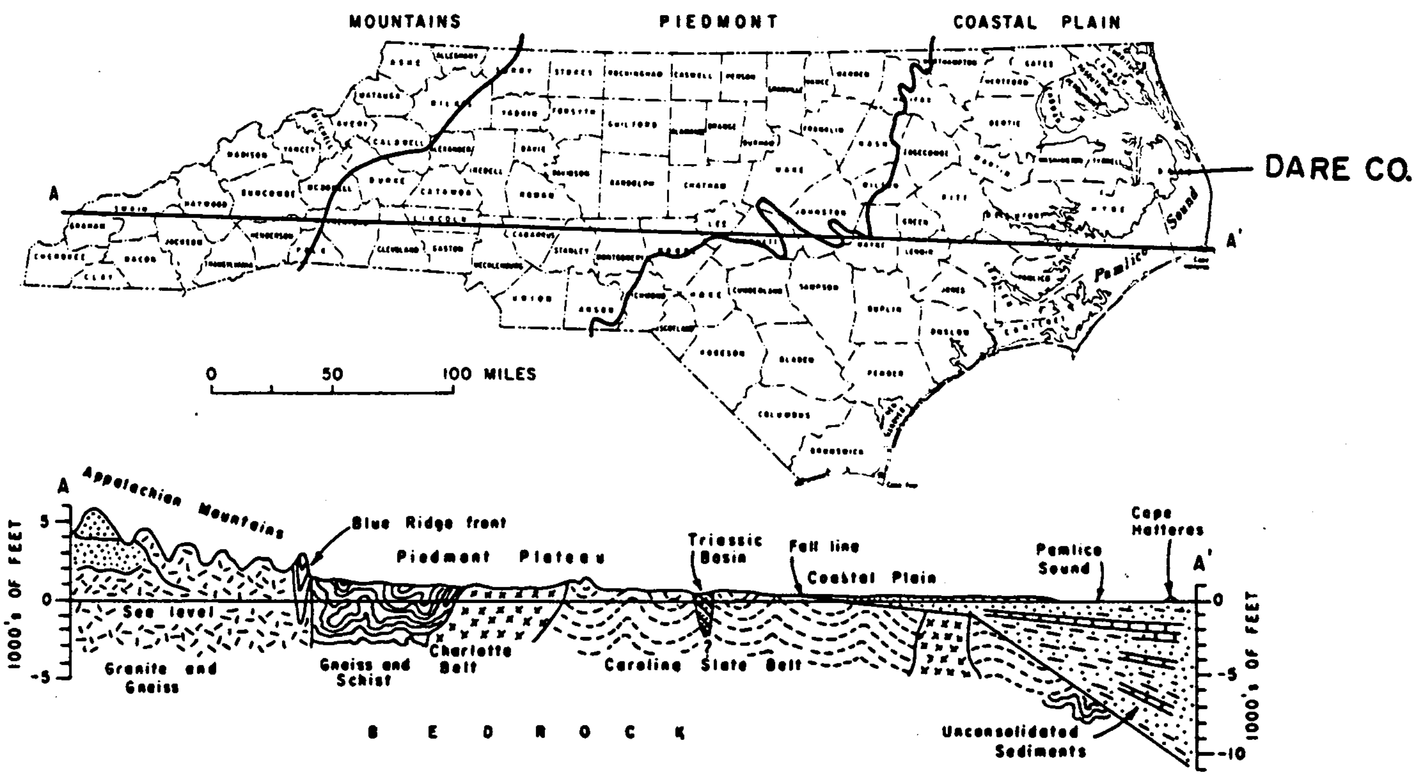

FIGURE 2.1-1 VICINITY MAP.
(AFTER HEATH, 1980.)

3.0 DEMOGRAPHIC EVALUATION

3.1 PAST POPULATION GROWTH TREND

Dare County is undergoing a major change in its growth and development, as illustrated in the 1980 Census. During the period between 1970 and 1980, the County population increased by 91% from 6,995 to 13,377. The largest portion of the growth occurred on the Outer Banks (Dare Beaches). The majority of the growth can be linked to the County's recreational potential which has 120 miles of ocean frontage and 45 square miles of preserved national seashores. In fact, Dare County has the largest percentage of seasonal cottages than any other county in the State.

The number of housing units constructed between 1970 and 1980 also illustrates the growth trend of the County. In 1970, 5,057 units were reported in the census compared to 11,006 in 1980; however, only 6,112 are classified as year-round dwelling units. Based on the number of permanent units and population for 1980, there are 2.19 persons per household. The past population growth trends are illustrated in Table 3.1-1.

3.1.1 Mainland

Growth on the mainland has been slower than in other parts of the County. The Croaton and East Lake Townships are the areas on the mainland where approximately 6% of the total population resides. Growth on the mainland increased by approximately 30% between 1970 and 1980. However, this growth only represents 3% 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.

3.1.2 Roanoke Island

Growth on Roanoke Island has mainly occurred in the vicinity of the Towns of Manteo and Wanchese. The Island is a part of the Nags Head Township. Approximately 30% of the County's population inhabits the Island. The population increased on the Island by 35.6% between 1970 and 1980. This growth represents 16.2% of the total growth in the County.

TABLE 3.1-1

POPULATION TRENDS, DARE COUNTY, NC

	Population				Housing Units			
Township	1970	1980	% Change	% of Total	1970	1980	% Change	% of Total
(3) Atlantic	1,141	3,732	227.5	28	1,883	4,851	157.6	44
(1) Croaton	540	714	32.2	5	214	358	79.9	3
(1) East Lake	88	112	27.3	1	42	67	59.5	1
(3) Hatteras	1,333	2,783	108.8	21	620	1,379	122.4	13
(3) Kennekeet	565	1,060	87.6	8	324	648	100.0	6
(2) Nags Head	3,328	4,971	49.4	37	1,974	3,676	86.2	33
Dare County	6,995	13,372	91.2	100%	5,057	10,979	117.6	100%

(1) Mainland
(2) Roanoke Island and Part of Outer Banks
(3) Outer Banks

Source: 1980 US Census

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3.1.3 Dare Beaches

The Outer Banks of Dare County are divided into two segments which are separated by Oregon Inlet. The southern section, Pea/Hatteras Island, makes up the Kennekeet and Hatteras Townships. Pea/Hatteras Island has 29% of the total population. Growth of the area between 1970 and 1980 increased by 102% which is 30% of the total population.

The northern segment, Bodie Island, consists of the Atlantic and Nags Head Townships. These areas have approximately 35% of the County's population. Growth of Bodie Island between 1970 and 1980 was 205.6% or 50.2% of the total growth in the County. Approximately 65% of all the inhabitants in the County are located on the Outer Banks.

3.2 CURRENT POPULATION GROWTH TRENDS

The current growth trends in Dare County were determined from a review of building permits issued between 1980 and 1982. During this period 4,206 permits were issued. This is an average of approximately 1,402 per year. About 50% of the permits issued were for residential construction, of which 60% were permanent residences. It is estimated that 1,682 permanent residential units have been constructed in the County since 1980. This would account for an increase of 3,684 in permanent population (2.19 per household) since the 1980 Census. The permanent population is increasing at a rate of approximately 7% per year. The current population (1983) is estimated to be 17,063 persons.

Growth is also occurring in the seasonal cottages. In 1980, 44% (4,894 units) were classified in this category. It is estimated that the County issued 1,121 permits for cottage construction which represents an approximate growth rate of 6.2%. Timesharing condos or cooperative ownership represents 20.5% of the construction. In 1980, 116 units were listed, whereas an estimated 310 were in operation in 1983.

3.3 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, covering a total of 811,777 acres of land and water. The County's Land Use Plan estimates that

15% of the total land in the County is suitable for development. The land use in the County is illustrated in Table 3.3-1. It should be noted that this data was taken from the County's 1976 Land Use Plan. The actual acreage in each class may have changed since previous reports have indicated that large tracts of land on 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 indicates a high development potential.

3.3.1 Mainland

The mainland 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 persons per acre.

3.3.2 Roanoke Island

Roanoke Island consists of approximately 12,400 acres, of which 5,116 acres are considered to be developed or developable. The developed areas are centered around the Towns of Manteo and Wanchese. The majority of the residences are permanent year-round dwellers. Approximately 5,120 persons reside on the Island. The population density of the developed area is estimated to be 1.00 person per acre. Based on developable land, the Island is approximately 60% developed.

3.3.3 Dare Beaches

The Outer Banks of Dare County consist of approximately 51,633 acres, 26,502 of which are located north of Oregon Inlet and 25,131 acres to the south. Approximately 16,685 acres of developed or developable land are located to the north of Oregon Inlet compared to only 3,116 acres to the south. This large portion of developable land to the north is an indication why most of the development is occurring to the north. Four municipalities may be linked to the growth in the north, they are Nags Head, Kill Devil Hills, Kitty Hawk, and Southern Shores.

TABLE 3.3-1

LAND CLASSIFICATION

Land Use	Acreage
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

A large concentration of the seasonal population also occupies the northern segment of the Outer Banks. The rapid increase in seasonal and permanent population has created a question as to what the ultimate peak population would be if the area was fully developed.

Assuming that 75% of the total developed and developable land on the Outer Banks and Roanoke Island were 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. This development density is for the Dare Beaches. Actual density on Roanoke Island would be less.

3.4 POPULATION PROJECTION

3.4.1 Dare County

Growth of the County has increased at a rapid rate in both permanent and seasonal populations. Previous calculations determined that the permanent population is increasing at approximately 7% per year, seasonal population is increasing at approximately 5%.

The population projections illustrated in Table 3.4-1 are based on the current growth trends and are projected through the planning period. The projections may not be completely on target due to a variety of economic, social and climatic conditions. Periodic updating of these projections will be necessary in intervals no greater than five years, in order to keep the projections in line with actual occurrences.

To insure that the MGA projections are in line, the NC Office of State Budget and Management was contacted concerning its population projections for Dare County. These projections are shown in Figure 3.4-1 along with the MGA projections.

3.4.2 Existing Service Area

The existing service area populations are shown in Table 3.4.2. As indicated, there are five existing service areas within the County. 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 multi-units by a factor of 3.6 persons per unit.

TABLE 3.4-1

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

Seasonal Accommodations	Units	Occupancy Per units	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	44,330	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 Accommodation)

PERMANENT
POPULATION PROJECTION
FIGURE 3.4-1

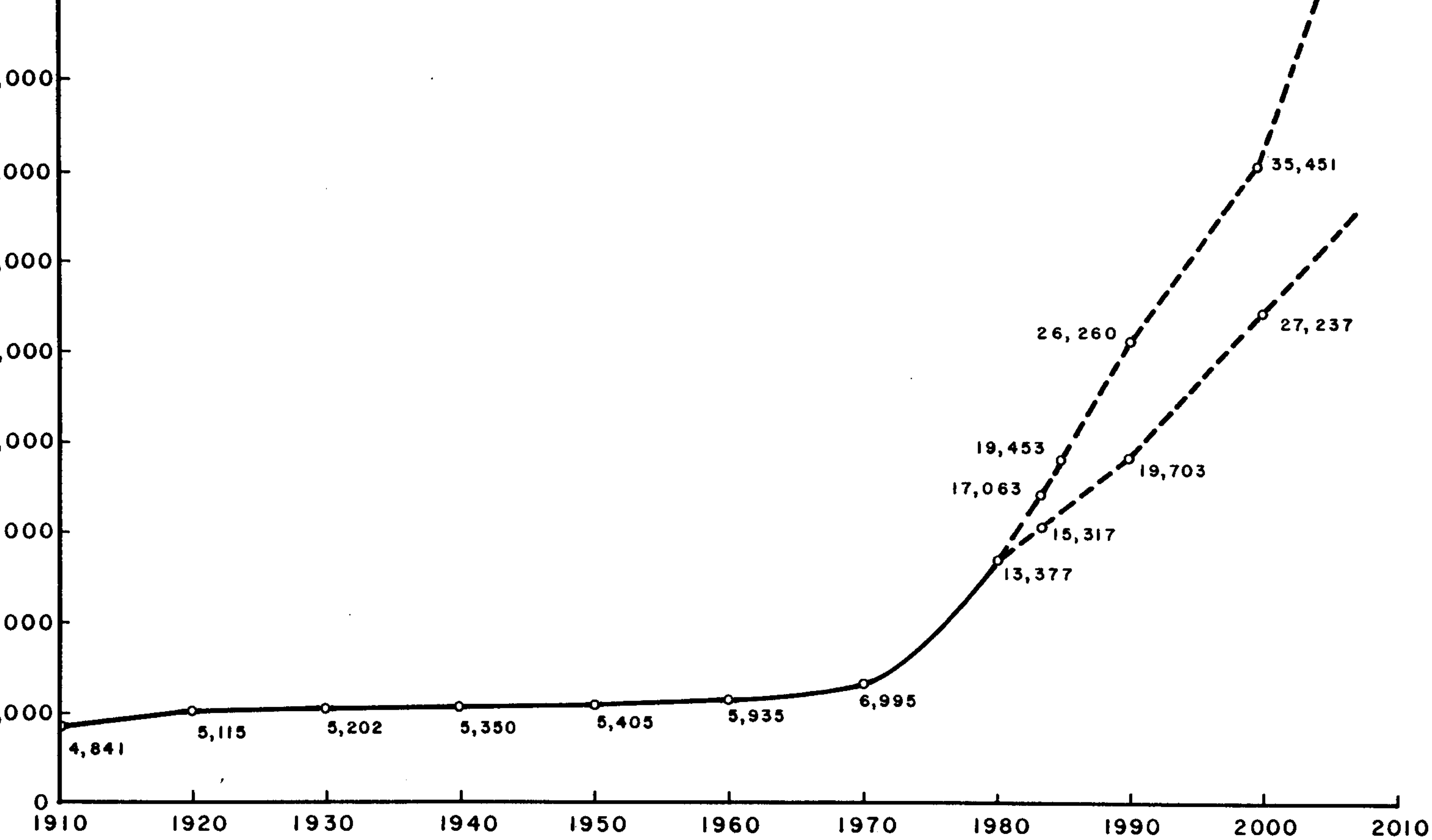

TABLE 3.4-2

SERVICE AREAS POPULATION PROJECTIONS
PERMANENT AND SEASONAL, DARE COUNTY, NC

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
Service from County Water System	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 from County Water System	43,480	54,920*	88,940	121,00	153,020

*Service to new areas, 1985.

3.4.3 Potential New Service Areas

Potential new service areas are identified by analyzing the most densely populated areas that could feasibly support the water system or areas that are in great need of water service. Each new service area is discussed in this report. It should be noted that it may not be feasible to serve all areas from the County system. They are discussed in this report to show the need 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 supply source i.e., CHWA or Dare County Water.

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 service in the areas is predominately provided by individual shallow wells that draw on fresh water aquifers which are recharged by precipitation. The water in these areas is highly susceptible to contamination by flooding (salt water intrusion) and by wastewater discharges from septic tanks. As the population increases, the need for a potable water supply also will increase.

Like other areas of the Outer Banks, the villages have experienced a substantial increase in seasonal population. The population projections illustrated in Table 3.4.2 were taken from the MGA 1982 Engineering Report entitled "Water Supply and Treatment Alternatives of the Rodanthe-Wave-Salvo." That report also projected the population of the County. The methodology used in those projections was based on federal projections. The County projections illustrated in this report, as previously stated, are based on the growth trend that has occurred since 1980.

Roanoke Island

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

The Town of Manteo operates its own water distribution system and purchases treated water from the County. The County supplies water service to approximately 75 customers in the Manteo area. However, the majority of the residential water supplies are provided by individual shallow wells that draw on the fresh water table aquifer. The water is susceptible to contamination by flooding and by wastewater discharge from septic tanks. The County uses the groundwater supplies on Roanoke Island as its water source and will be dependent 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 increases. With increased demand from residents utilizing their own wells and the County obtaining its water from the groundwater sources, the stress on the aquifer system increases.

To eliminate any problems that could result from the competition for water supplies on Roanoke Island, individuals in these potential service areas should be brought onto the County system.

It is estimated that 2,000 persons would be in the service area, which would include Wanchese and areas around Manteo. A total of 540 possible connections have been identified in Wanchese. Population growth of the Island is assumed to be similar to that of Manteo, approximately 3% per year. The projected population of the service areas on the Island are shown in Table 3.4.2.

4.0 WATER USE EVALUATION

4.1 EXISTING WATER DEMANDS

The County operates a 5 mgd water treatment plant with a raw water production capacity of 7.2 mgd. The system was built in 1980 and began operating at approximately 40% of its rated capacity. Currently, it has reached 90% of its capacity. Past water production is shown in Table 4.1-1. The large variation in minimum and maximum demands represents the typical seasonal and nonseasonal variation in demands. Approximately 90% of the maximum is required to meet the seasonal demands.

Table 4.1.2 illustrates the 1983 peak seasonal demands for the County system. The average seasonal per capita consumption was determined to be 105 gallons per capita per day.

4.2 WATER DEMAND PROJECTIONS

Water demands have increased by approximately 43% since 1980, for an average increase of approximately 15% per year. Both water demands and population growth are increasing at approximately the same rate. The water demand is therefore, expected to continue the present increasing pattern to mirror population growth. Table 4.2-1 and Figure 4.2-2 illustrate the projected water demands for the planning period. They are based on the projected populations and an average water consumption of 100 gallons per capita per day. This table shows a demand of 6.75 mgd by 1985, 8.8 mgd by 1990, 11.95 mgd by 2000 and 15.51 mgd by 2005.

TABLE 4.1-1

HISTORICAL WATER PRODUCTION (MGD)

Water Production	1980	% Increase	1981	% Increase	1982	% Increase	1983
* Minimum Day	.241	39.8	.337	23.1	.415	-15.6	.359
** Maximum Day	2.280	35.4	3.086	17.0	3.256	37.5	4.477
Annual Total	160.256	--	516.072	9.8	566.504	30.3	738.160

* Nonpeak Seasonal

**Peak Seasonal

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TABLE 4.1-2

PRESENT PEAK SEASONAL WATER DEMAND
DARE COUNTY, NC

Service Area	Peak Daily 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

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TABLE 4.2-1

WATER DEMAND PROJECTIONS
DARE COUNTY, NC

Existing Service Area (mgd)

Service Area	1983	1985	1990	2000	2005
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	.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)

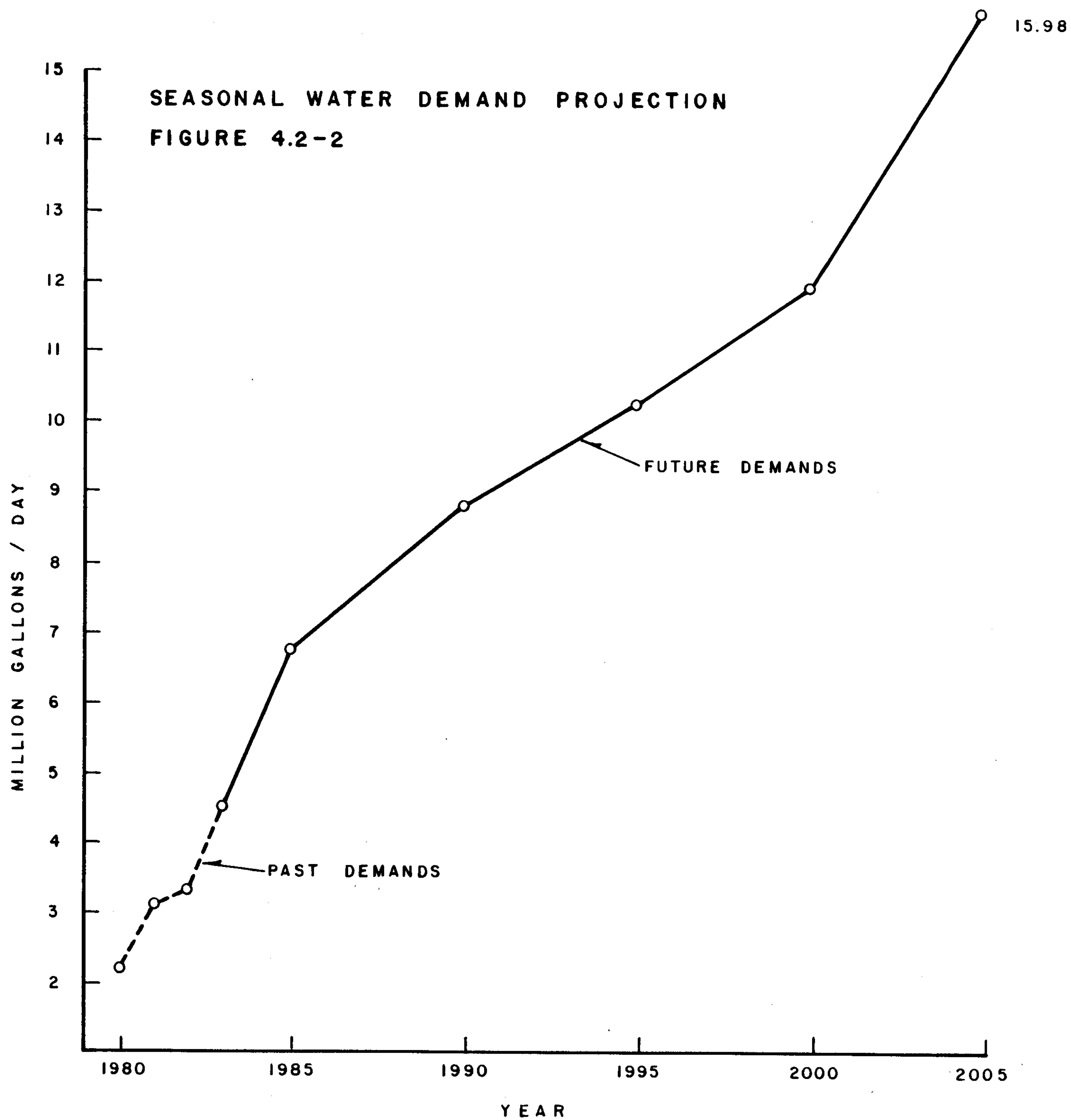
Roanoke Island	___	0.22	0.25	.28	0.33
Future Service Dare County Water	--	6.83	8.89	12.09	15.49
*Rodanthe-Waves Salvo		0.40	0.65	.82	1.0

(1) Miller Williams projections to 1990.

(2) MGA projection (100 gpcd).

(3) MGA projection, 1982 Engineering Report.

* Not feasible to serve from Dare County System.


SEASONAL WATER DEMAND PROJECTION
FIGURE 4.2-2
15.98
FUTURE DEMANDS
PAST DEMANDS
MILLION GALLONS / DAY
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1980
1985
1990
1995
2000
2005
YEAR

5.0 WATER SUPPLY SOURCES

There are five potential sources of water supply for the County of Dare. Three of these sources are considered conventional; two are non-conventional. The conventional sources are groundwater on Roanoke Island, groundwater from the mainland, and the fresh pond at Nags Head. The non-conventional sources are desalinization of sound water and water reuse. Each of these sources is discussed below.

5.1. FRESH POND

Fresh Pond is a 27-acre lake which is situated halfway between the ocean and sound and is bisected by the boundary between Nags Head and Kill Devil Hills. This lake served as a water supply source for the two towns until 1979 when the towns entered into a water purchase agreement with the County.

5.1.1 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, some to runoff and some to discharge of groundwater to the ocean and sounds through lateral movement. Only 25% 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. Under withdrawal by pumping, a "cone of depression" forms around the lake and behaves as a groundwater divide. Investigation of the pumping effects on movement of the divide was conducted by E.O. Floyd of Moore, Gardner & Associates, Inc. in 1979. Floyd found that when withdrawal of water from the pond occurs, the water surface of the pond is lowered and groundwater begins to move to the lake in all directions. The gradient on the lake and 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 center of the pond. This cone is referred to as the cone of depression. Within this cone of depression, groundwater flows toward the lake. The edge of the cone of depression becomes a groundwater divide. As a result, there is inflow of groundwater to the lake and lateral discharge to the sound for groundwater outside the cone. Floyd determined that the areal position of the groundwater divide was not greatly affected by the change from the seasonal low water level to the seasonal high water level. The position of the groundwater divide was relatively stationary. This indicates that the water in the zone of influence remains relatively constant throughout the year.

Since the water supply has not been used in some years, the surface of the lake has risen dramatically. This is because evaporation is the only outlet for water in the zone of influence. Thus, a potential water supply source is still available at Fresh Pond.

5.1.2 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 5.1-1. A data tabulation of some data points is shown in Table 5.1-1. Floyd estimated that during the summer months, pumping from the lake imposed 2 to 3 feet of drawdown on its surface in addition to the evaporation losses. The additional drawdown imposed by pumping (one) mgd from the lake produced the necessary loss of head to induce additional flow of water from the aquifer to the lake. Due to the water storage and transmittal properties of the aquifer, sufficient groundwater flow to balance pumpage 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.

5.1.3 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 performed for a short duration, the volume of withdrawal could be doubled so as to tap the water in storage. Prior to pumping at the higher yield, a determination of the stage should be made in order to figure the volume of water in storage. An estimate of pumping duration would be made

TABLE 5.1-1

STAGE STORAGE DATA AT SELECTED WATER
SURFACE ELEVATIONS

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

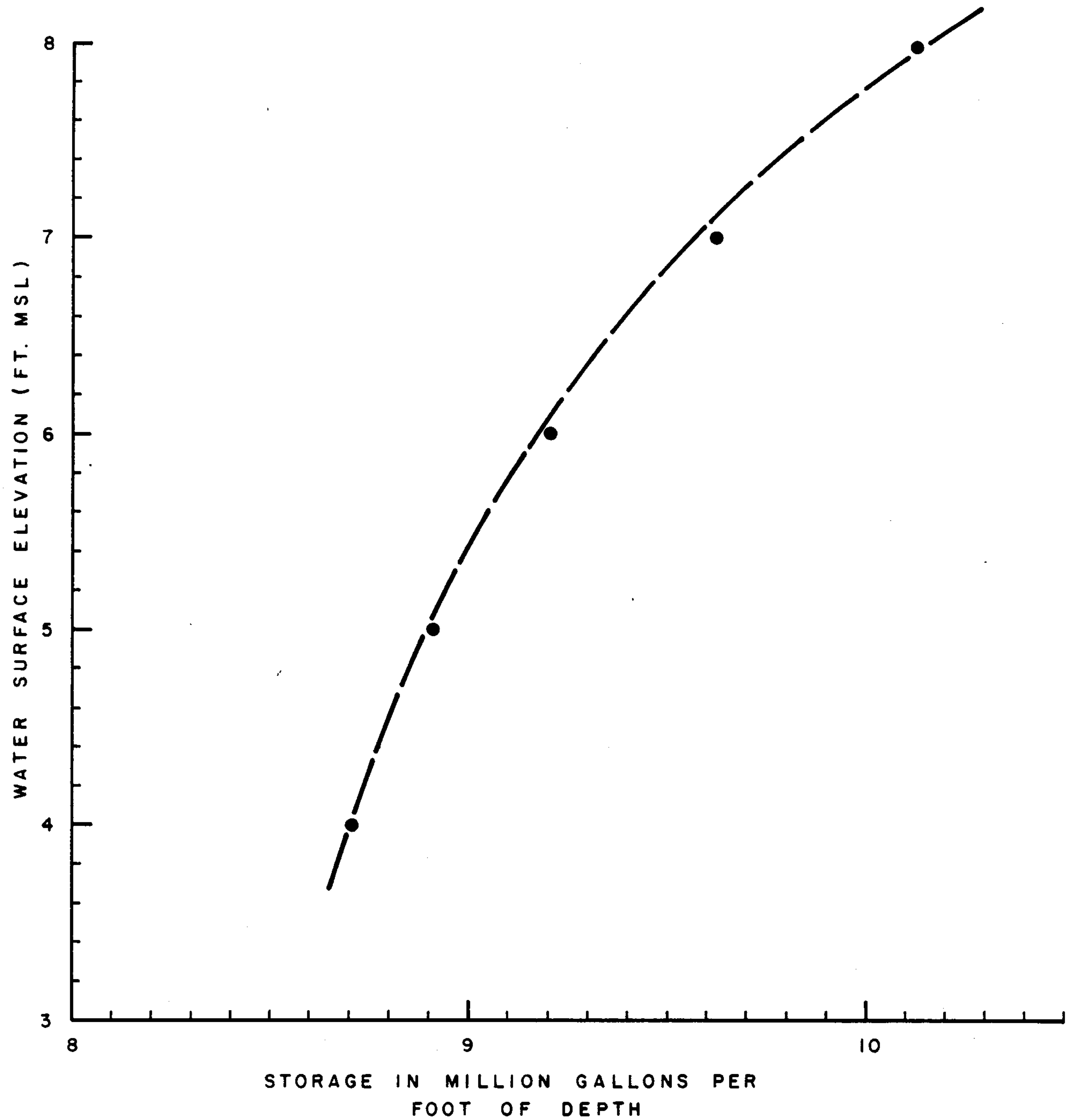


FIGURE 5.1-1 STAGE – STORAGE RELATIONSHIP AT FRESH POND, NAGS HEAD, N.C.

and the total volume of water required would be determined. By checking the stage-storage curve and the daily safe yield data Table 5.1-2, a safe withdrawal rate could be determined. From this data, withdrawal of 1.0-1.5 mgd could be pumped from the Fresh Pond without adversely affecting the supply, for durations of up to one month, throughout the year. Higher rates could be pumped from the pond from January through June.

5.1.4 Improvements Required for Use

In order to use the raw water supply of the Fresh Pond, upgrading of one of the treatment plants at the site will be required. It is recommended that refurbishing of the plant be completed with a capacity to supply a peak day demand of 1.5 million gallons.

The existing plants consist of floating raw water intakes, raw water reservoirs, microstrainers and settling tanks. Treatment during operation consists of microstraining to remove algae, alum and phosphate addition for water conditioning and chlorination for bacterial treatment.

Modifications to one of the treatment plants will be required in order to meet drinking water standards. Proposed modifications (Figure 5.1-2 and Table 5.1-3) include installation of an aeration tower for iron removal, removal of the microstrainer, chemical feed improvements, conversion of the 0.5 million gallon raw water reservoir to a settling basin, installation of a gravity filter with peripheral backwash holding basins, a sump with pumps, water meters, and piping.

The flow and treatment process will be to pump water from the Fresh Pond via a 14 inch suction line to the raw water pumps (to be increased to 1,042 gpm). The raw water will then flow to the aeration tower, through the vault where the microstrainer has been removed and where chemical feed will occur. From this point, flow will be to the settling basin under gravity flow then to the 1.5 mgd gravity filter. The water will then flow to a sump where pumps will boost the finished water into the distribution system. Backwash from the gravity filter will go to a holding pond where it will settle and then be discharged back to the Fresh Pond. An altitude valve will control filling of the finished water reservoir. Meters will be installed in the finished water line and the line from the gravity filter to the finished water reservoir.

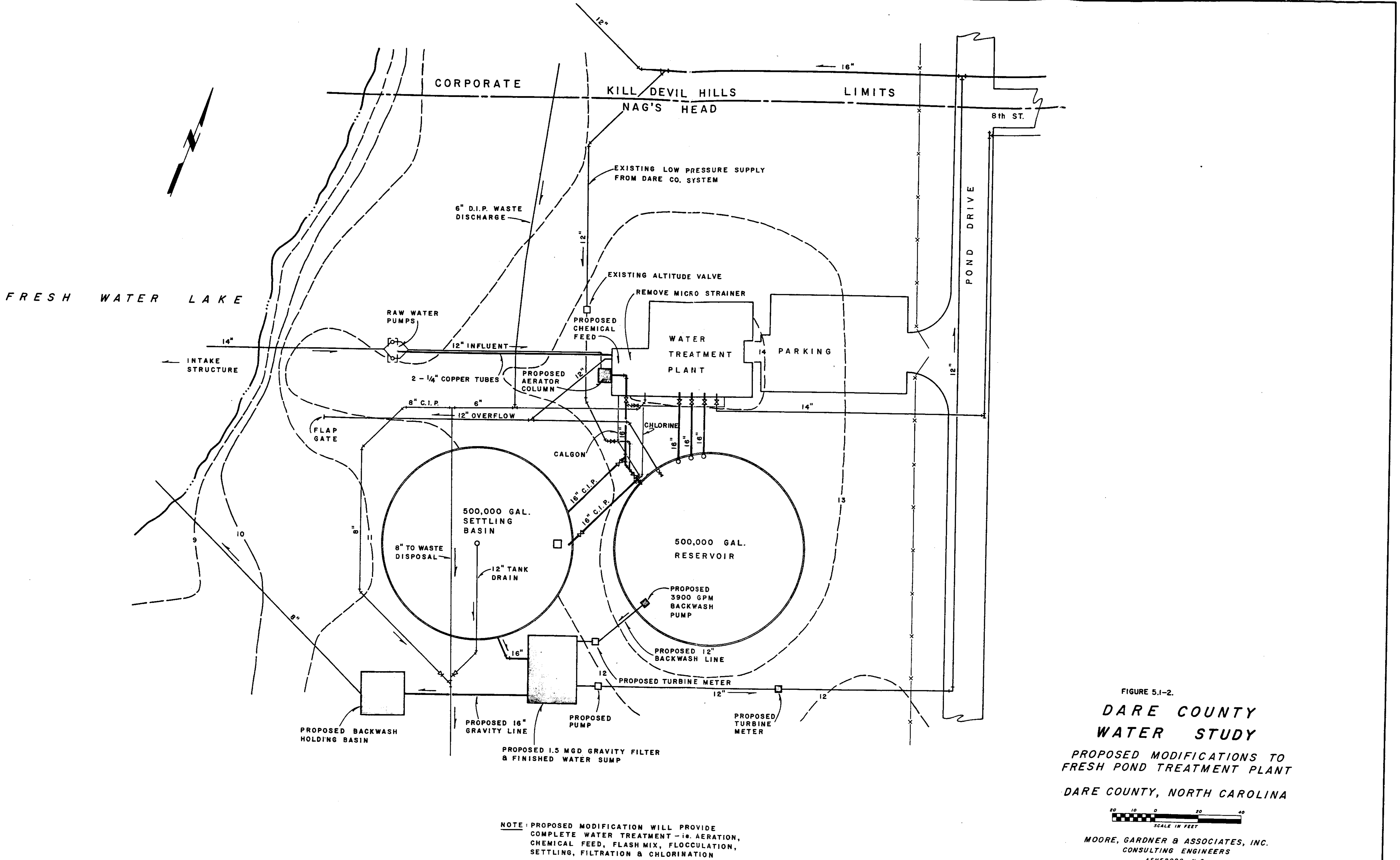


FIGURE 5.1-2.

DARE COUNTY WATER STUDY

PROPOSED MODIFICATIONS TO FRESH POND TREATMENT PLANT

DARE COUNTY, NORTH CAROLINA

MOORE, GARDNER & ASSOCIATES, INC.
CONSULTING ENGINEERS
ASHEBORO, N. C.
A SUBSIDIARY OF BLACK & VEATCH

TABLE 5.1-2

DARE COUNTY POTENTIAL WATER SUPPLY
FRESH POND @ NAGS HEAD

SAFE YIELD BY MONTH

SAFE YIELD

	Climatological Data			Fresh Pond Climatological Data			Safe Yield = Pws-Rws-Efpa	
Month	P (ft.)	R (ft.)	E (ft.)	Pws (MG)*	Rws (MG)	Efpa (MG)	Month (MG)	Day (MG)
1980-Jan.	.49	.02	.18	52.11	2.13	1.64	48.34	1.56
Feb.	.26	.01	.29	27.65	1.06	2.64	23.95	0.83
Mar.	.54	.03	.40	57.42	3.19	3.64	50.59	1.63
Apr.	.39	.02	.63	41.47	2.13	5.74	33.60	1.12
May	.37	.02	.73	39.35	2.13	6.65	30.58	0.99
June	.28	.01	.85	29.77	1.06	7.74	20.97	0.70
July	.37	.02	.87	39.35	2.13	7.92	29.30	0.95
Aug.	.21	.01	.74	22.33	1.06	6.74	14.53	0.47
Sept.	.42	.02	.57	44.66	2.13	5.19	37.34	1.24
Oct.	.29	.01	.38	30.84	1.06	3.46	26.32	0.85
Nov.	.33	.02	.30	35.09	2.13	2.73	30.41	1.01
Dec.	.34	.02	.28	36.15	2.13	2.55	31.47	1.02
	.36	.02	.52	38.02	2.13	4.72	31.45	1.03

Watershed (WS) = 326.19 acres
Fresh Pond Area (FPA) = 27.93 acres

59B/BB
6

TABLE 5.1-3

UPGRADING OF FRESH POND WTP

Preliminary Opinion of Probable Project Cost

Construction Cost	\$400,000
Legal, Contingency, Administrative, Engineering, and Construction Cost	\$100,000
TOTAL PROJECT COST	\$500,000

Utilization of either plant will be dependent upon negotiations between the towns and county concerning ownership and operation. However, the anticipated low cost for producing the water should serve as a strong incentive for cooperation. This plant could serve as a reserve supply in non-peak months and could be used to augment the existing system capacity during peak months.

5.2 ROANOKE ISLAND GROUNDWATER

5.2.1 Introduction

The groundwater system of Roanoke Island is a complex hydrogeologic feature affected by a myriad of physical, meteorologic and oceanographic factors. This section will describe the hydrogeologic system, discuss the systems that influence it, outline the affects of withdrawals of groundwater and predict the affects of increased withdrawals caused by additional groundwater development.

5.2.2 Hydrogeologic Framework

Roanoke Island is underlain by a series of gently dipping beds of sediments that have been deposited over millions of years. Deposition has occurred during periods of sea level transgressions and the area has been exposed to weathering and erosional processes 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 5.2-1 is a stratigraphic column of the sedimentary wedge under Roanoke Island. This column is based on data from key test wells drilled in the area. Brown (1972), described the formations and chronostratigraphic units shown in the section. For the purpose of this groundwater investigation, we will be interested only in the first 500 feet of sediments and principally in their water bearing characteristics.

The Miocene and post Miocene units contain six significant hydrogeologic units (Peek 1972). Data from exploratory wells (Figure 5.2-2), have shown three aquifers and three confining beds. These units are shown in cross section on Figures 5.2-3 through 5.2-5.

The uppermost aquifer extends from the land surface to depths of 50 feet on Roanoke Island. This unit 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 due to the presence of clay lenses, it is confined at some localities and depths. The unit consists mainly of medium grained sands with some interbedded silts and clays. This aquifer

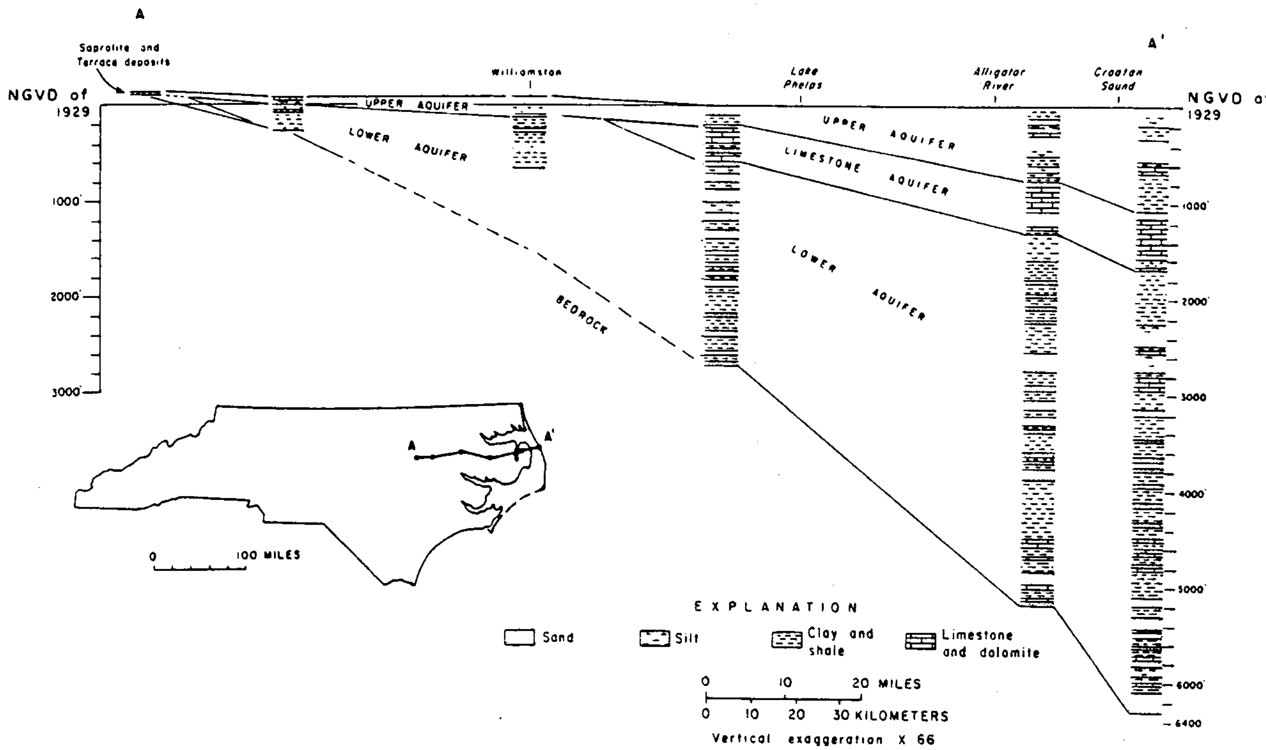
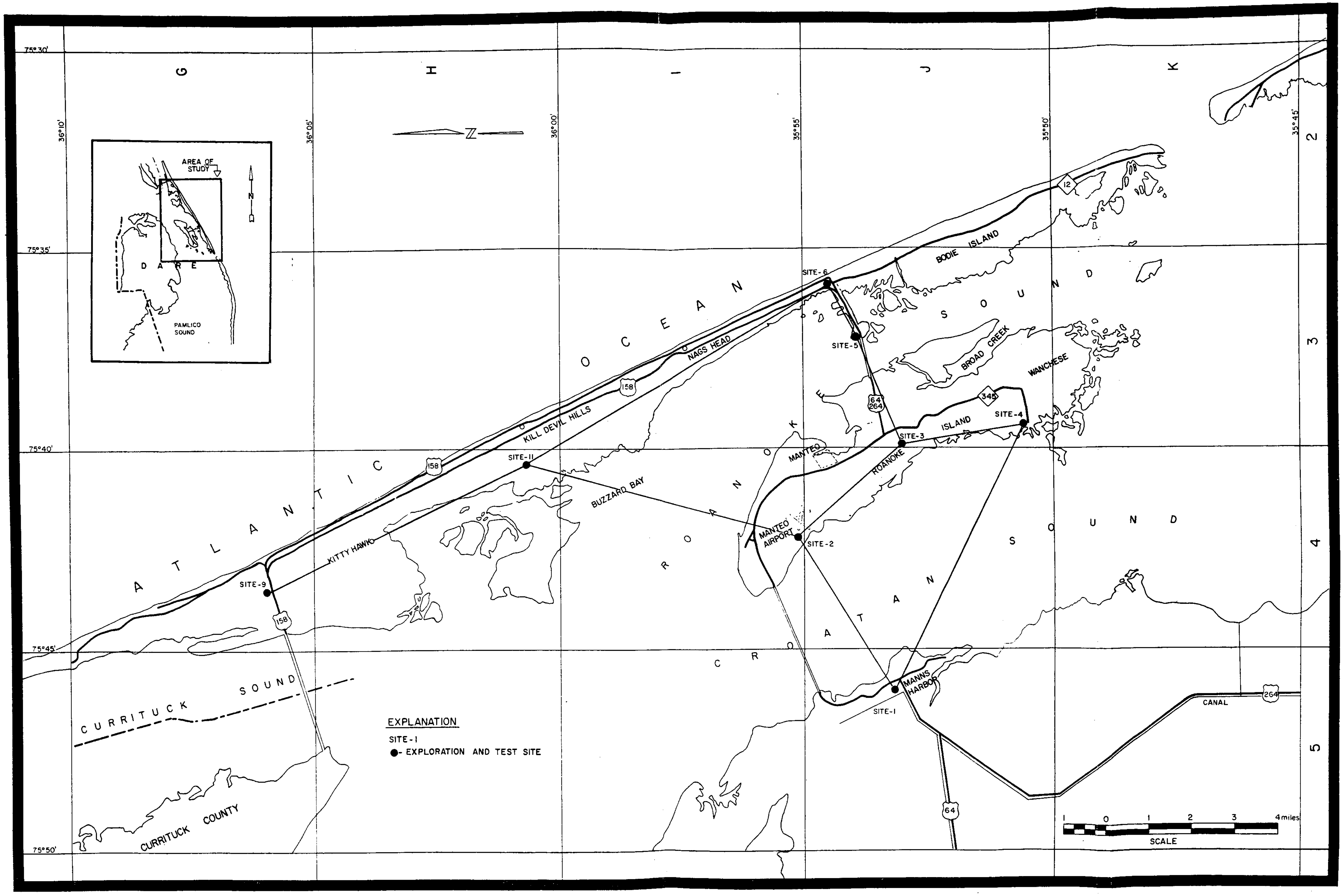
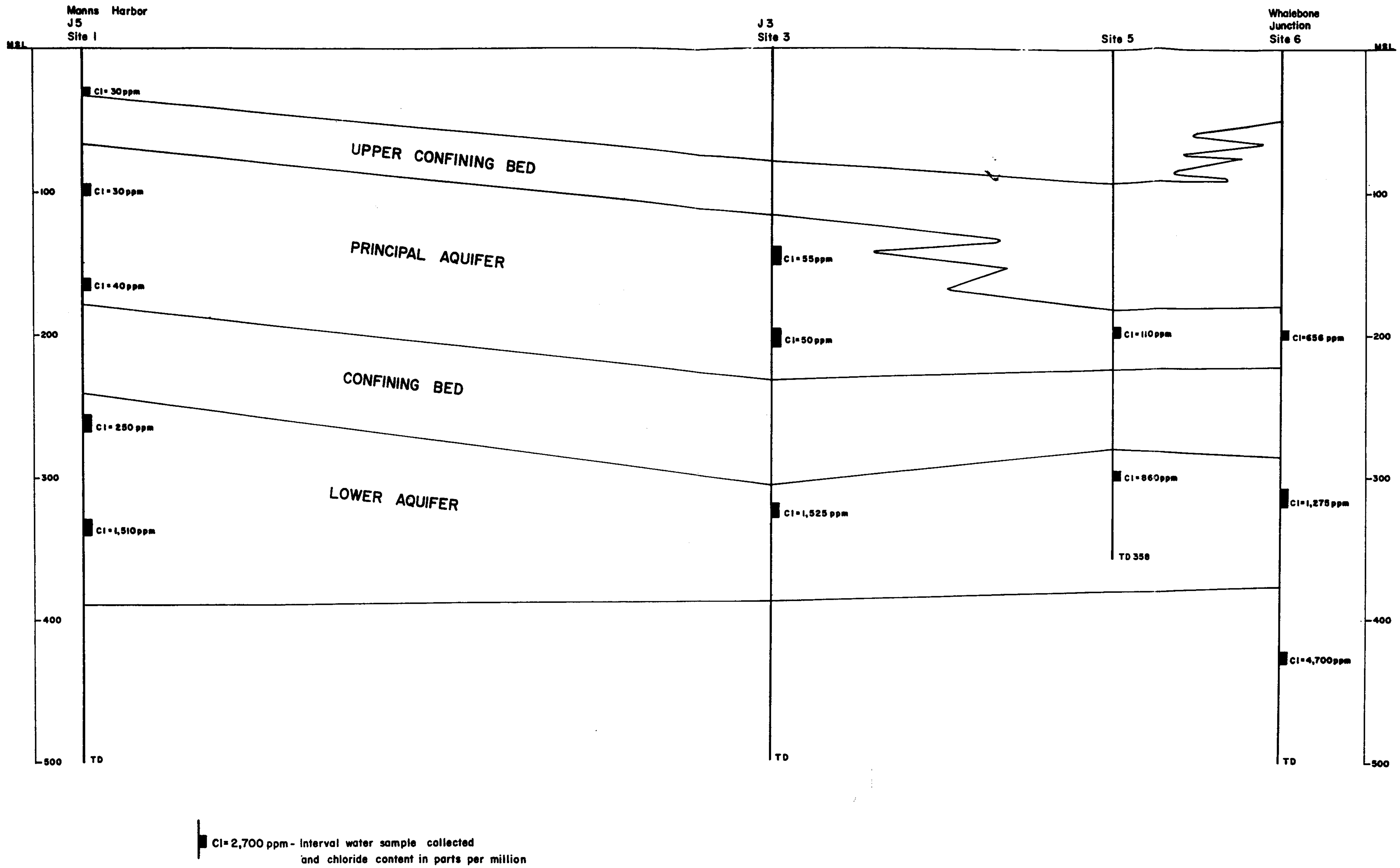


FIGURE 5.2-1 STRATIGRAPHIC FRAMEWORK. (FROM HEATH, 1980)



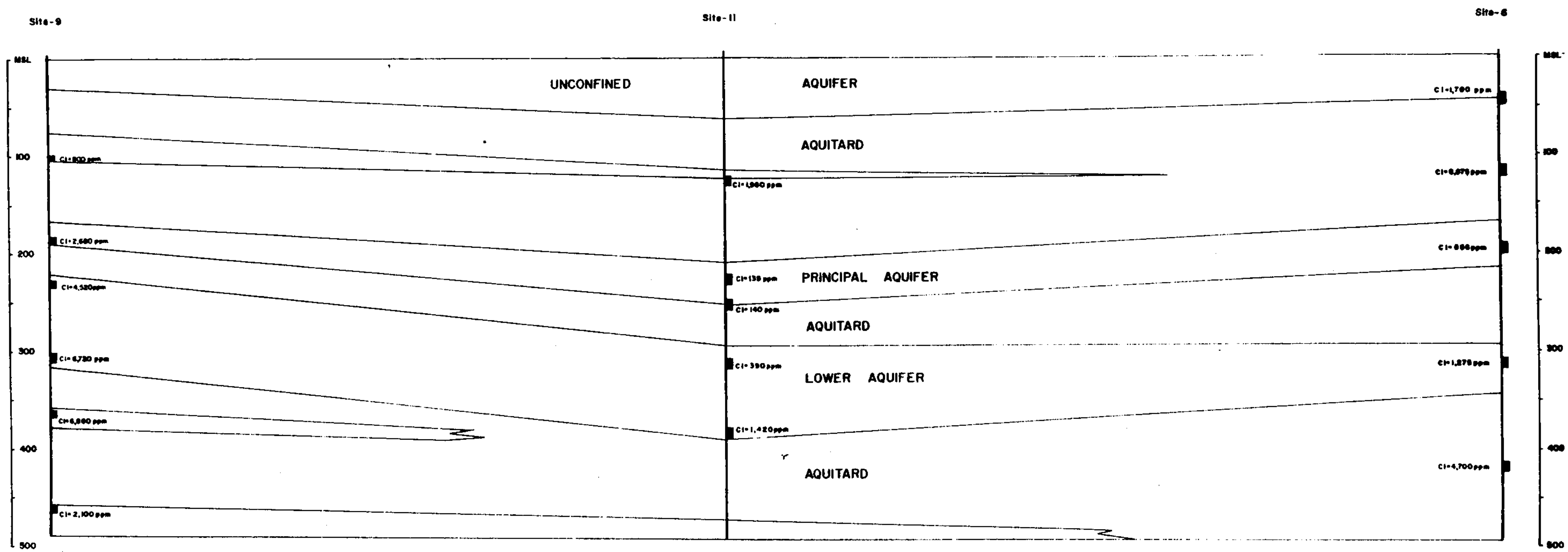
FROM PEEK, ET AL 1972

FIGURE 5.2-2
MAP SHOWING LOCATION OF STUDY AREA, TEST SITE AND CROSS-SECTION



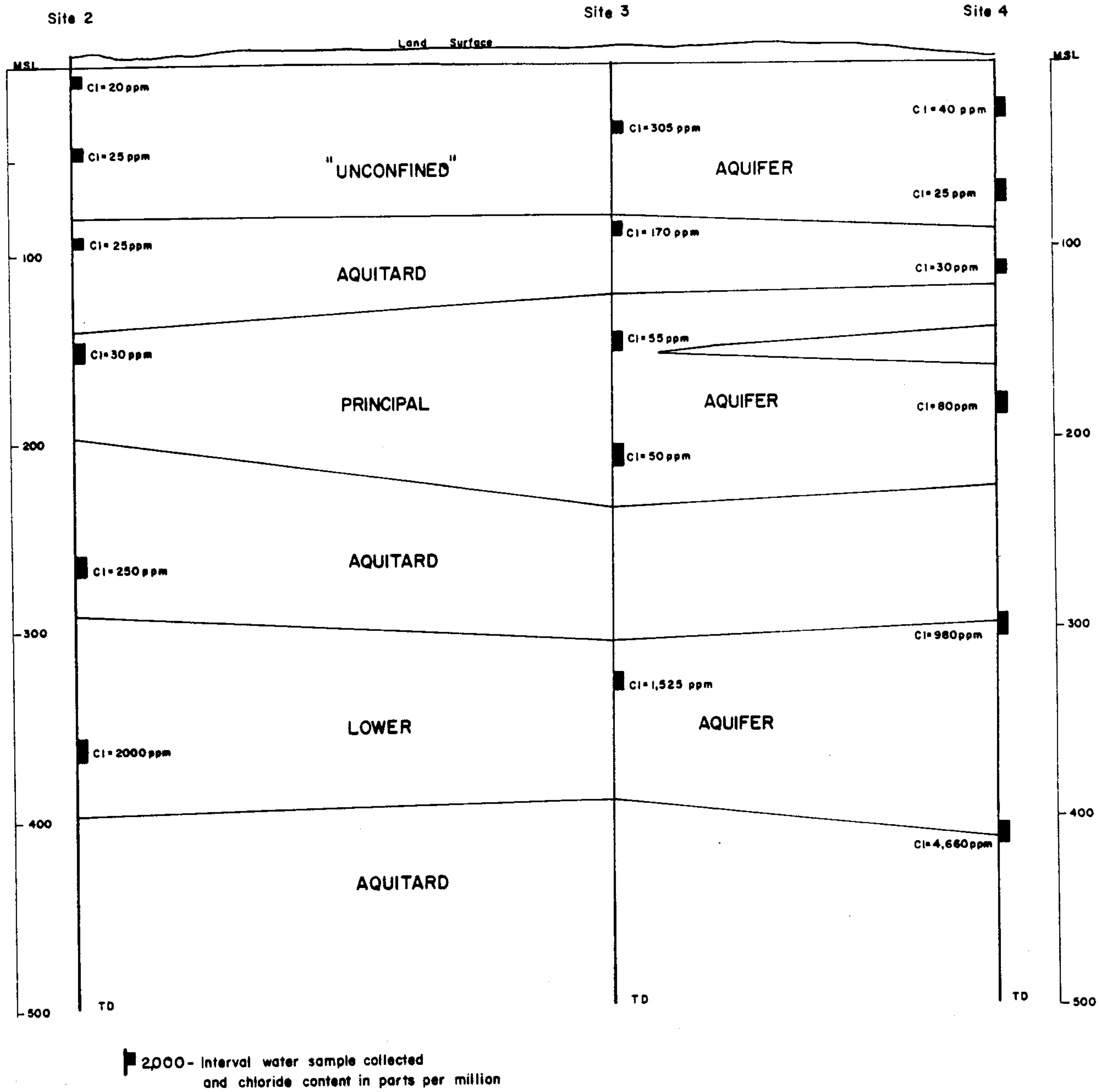
FROM PEEK, ET AL 1972

FIGURE 5.2-3
HYDROGEOLOGIC CROSS-SECTION, SITE 1 TO SITE 6



FROM PEEK, ET AL 1972

FIGURE 5.2-4
HYDROGEOLOGIC CROSS-SECTION, SITE 9 TO SITE 6



FROM PEEK, ET AL 1972

FIGURE 5.2-5
HYDROGEOLOGIC CROSS-SECTION, SITE 2 TO SITE 4

is recharged by precipitation which falls on Roanoke Island. Other than providing water to some domestic wells to residents of Wanchese, the significance of this aquifer is to serve as a recharge reservoir for the underlying aquifer system.

The water table aquifer is underlain by clays and interbedded clays and sand. This unit is not homogeneous and should be envisioned as a series of lenticular beds that allow transmission of water in a vertical direction. The thickness of this unit ranges from 35 to 100 feet. It serves as a semi-confining bed between the upper water bearing zone and the semi-confined aquifer which Peek (1972), calls the principal aquifer.

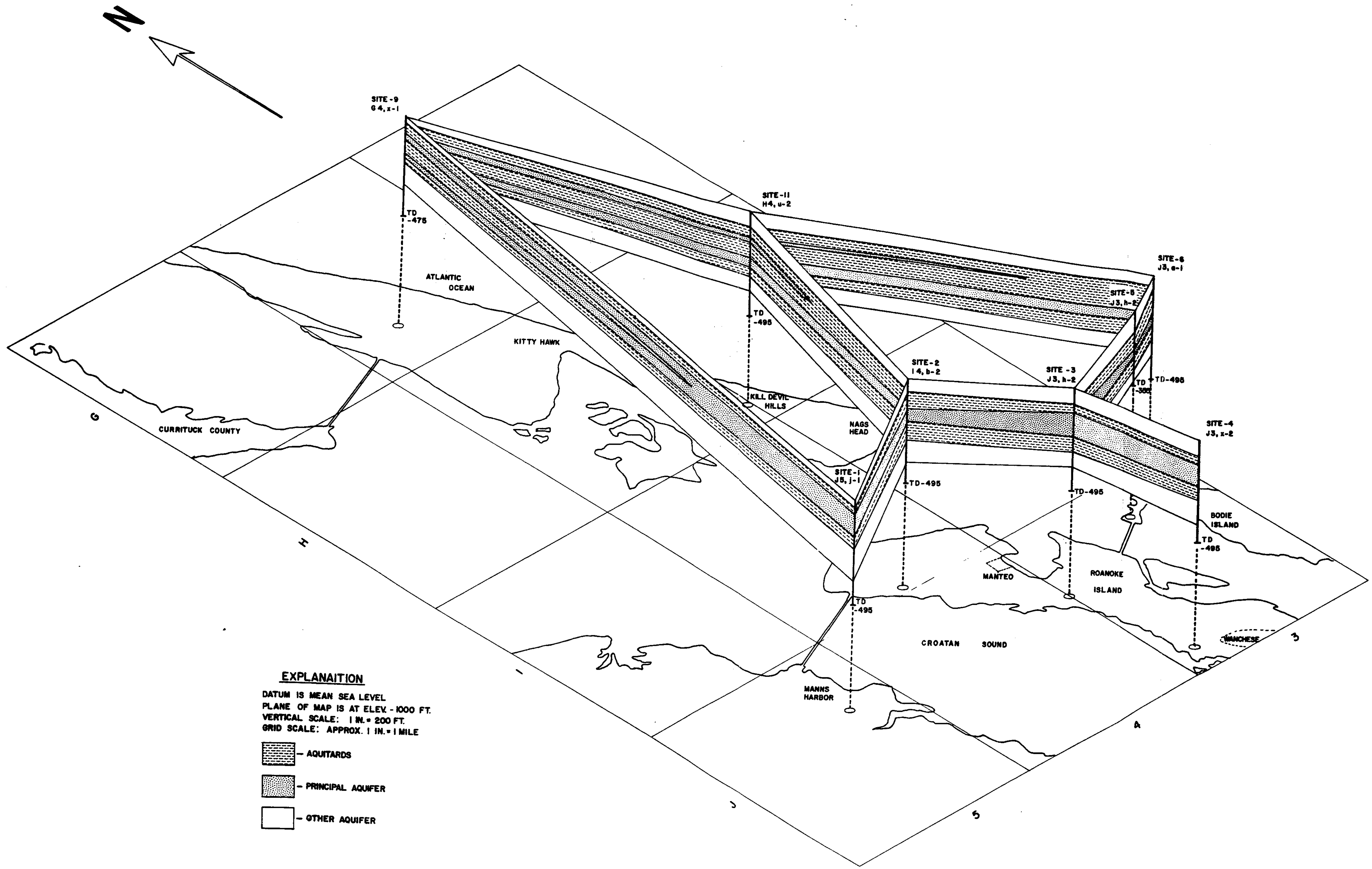
The so-called principal aquifer is a water bearing zone which ranges in thickness from 110 feet on Roanoke Island to only 45 feet at Nags Head. This pinching out of the aquifer to the east is caused by a gradual increase in clays and silts in the upper portion of the aquifer in the vicinity of the Outer Banks. 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 several domestic and industrial wells on Roanoke Island. The depth to the top of the principal aquifer ranges from about 100 feet on Roanoke Island to greater than 200 feet on the Outer Banks.

Below the principal aquifer, 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 ranges from 175 to 230 feet below sea level. This unit is predominantely 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 lies from 240 to 300 feet below sea level 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 ranges in thickness from 10 to 50 to 100 feet, and is found below 315 feet to 400 feet.

Figure 5.2-6 is a fence diagram showing the hydrogeologic framework.



FROM PEEK, ET AL 1972

FIGURE 5.2-6
HYDROGEOLOGIC DIAGRAM, DARE COUNTY BEACHES

5.2.3 Aquifer Hydraulics

The ability of the aquifers to transmit water to wells which penetrate them is of prime importance to the suitability of the aquifer as a water supply. Analysis of the hydraulic characteristics of the units was based on pumping test data from over 16 wells and test wells on Roanoke Island. Determinations of aquifer hydraulic conductivity, transmissivity and storativity were used to evaluate the chief water bearing zones. Table 5.2-1 is a summary of the principal hydraulic characteristics of the upper unconfined aquifer and the semi-confined principal aquifer. Analyses of this data indicate a wide range of transmissivity in the principal aquifer. This is caused in part by differences in aquifer thickness and varying permeabilities. Figure 5.2-7 is a map showing average transmissivities across Roanoke Island. This figure shows that higher transmissivities occur to the southeast, since the thickness of the aquifer increases in this direction. Transmissivities in the upper aquifer range from 7,000 to 10,000 gal/day/ft. The major factor influencing transmissivity in the upper aquifer is the variation in aquifer material and the presence of clays which decrease the permeability of the aquifer material. 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 gallons per minute (gpm) is obtainable at most sites on Roanoke Island.

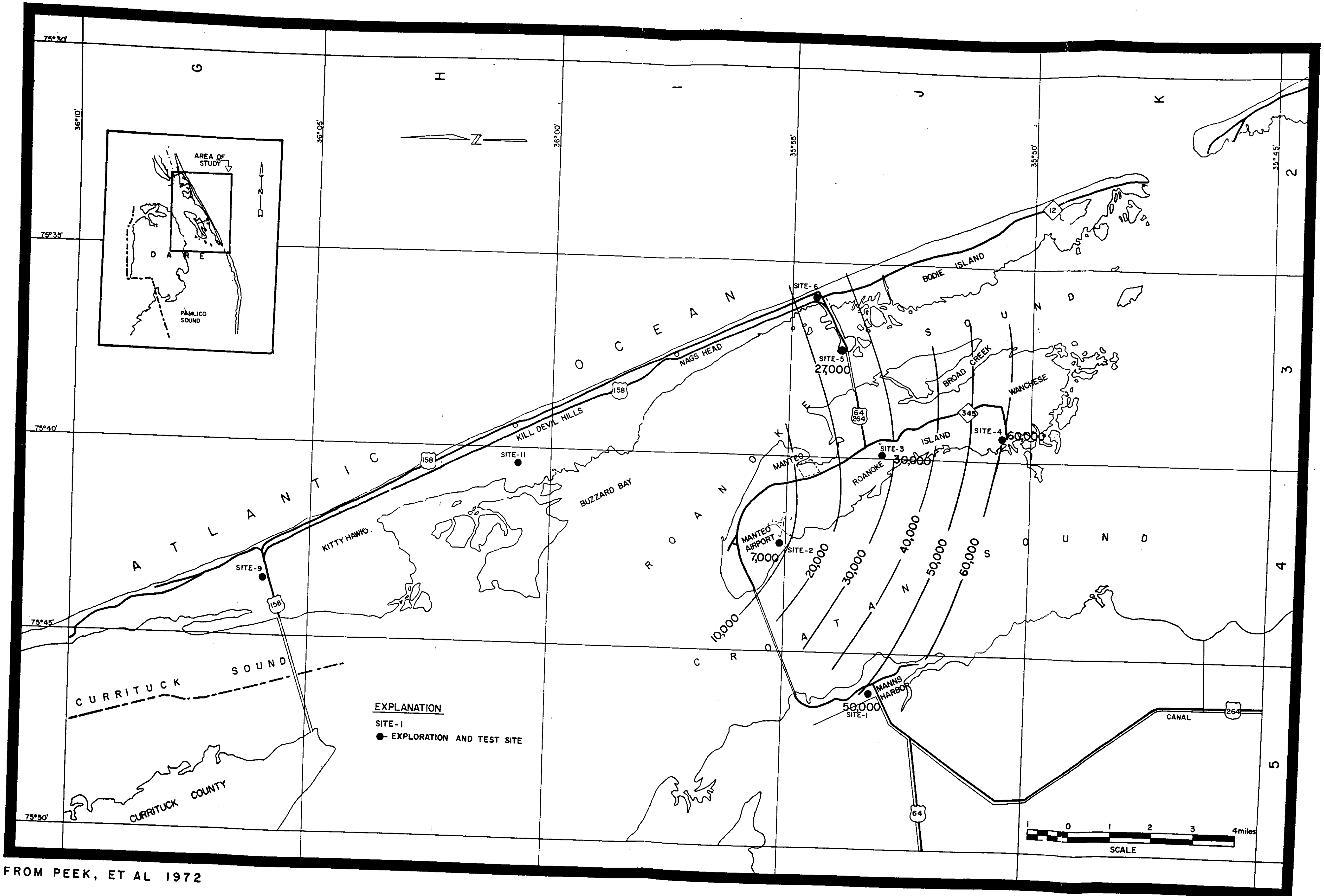
Peek (1972), believed that the upper water table aquifer and the principal aquifer were hydraulically connected and that the significance of the upper water table aquifer was to serve as a source of recharge to the principal aquifer. However, his perception of two aquifer systems acting independently of each other cannot be validated by analysis of pumping test data.

By plotting drawdown versus time on semi-logarithmic graph paper according to the Cooper-Jacob method, a straight line should be formed after the drawdown has reached steady state conditions. Deflection of this line towards the horizontal indicates that recharge to the aquifer is occurring (Figure 5.2-8). Plots of 16 aquifer tests in the principal aquifer indicate that recharge is occurring in as little as 30 minutes after pumping begins. Furthermore, it is quite possible that recharge is

TABLE 5.2-1

HYDRAULIC CHARACTERISTICS OF UNITS
UNDERLAYING ROANOKE ISLAND

Unit	Depth to Top	Thickness	Permeability K	T	S.C
Water Table Aquifer	0 feet	50 feet	6,000 gal/day/ft^2	20-70000 gal/day/ft	.003
Aquitard	70 feet	40 feet	20 gal/day/ft^2	<1,000 gal/day/ft	--
Principal Aquifer	110 feet	50 feet	8,213 gal/day/ft^2	20-150,000 gal/day/ft	.027



FROM PEEK, ET AL 1972

FIGURE 5.2-7
TRANSMISSIVITY OF PRINCIPAL AQUIFER

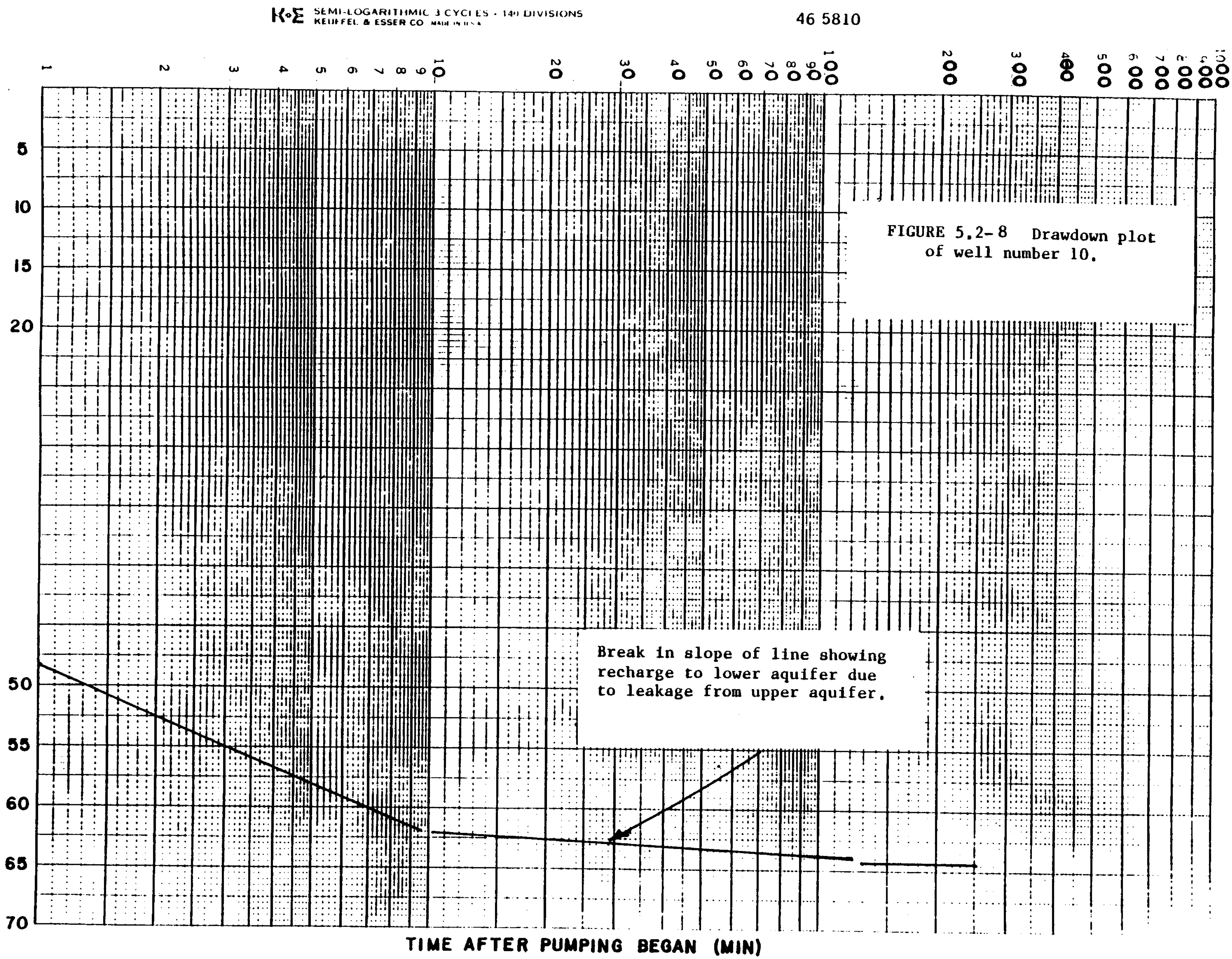


FIGURE 5.2-8 Drawdown plot of well number 10.

occurring almost instantaneously with initiation of pumping in those wells that did not show deflection of the drawdown curve. Wyrick and Floyd (in press), show the importance of micro-time measurements during the first few minutes of a pumping test in detecting true aquifer hydraulics. Their studies indicate that transmissivities can be in error by as much as 100% due to the lack of sufficient data points during the first few minutes of a pumping test. If this lack of data occurs during the first few minutes of the pumping tests analyzed during this investigation, it is quite possible that recharge affects could not be determined.

Recharge from the upper water table aquifer to the principal aquifer must occur through the semi-permeable clay lenses which separate the two water bearing units. This vertical recharge from the upper unit to the lower is commonly referred to as leakage. The presence of leakage in the aquifer system creates hydraulic characteristics which cause the lower principal aquifer to behave under water table conditions rather than confined or semi-confined conditions. A further break in the semi-log plot of the drawdown curve indicates when leakage from the upper water table aquifer ceases and dewatering of the principal aquifer begins. Calculations of the volume of water, known as leakance for well tests in this study, indicate a downward leakance of 20 gal/day/ft^2. This volume of water is significant in that it increases the available water volume which is recoverable by wells in the principal aquifer and is therefore available as water supply to the Dare County system.

5.2.4 Water Quality

Water quality data for the upper water table aquifer, the principal aquifer and the lower confined aquifer, are shown in Table 5.2-2. For the purpose of evaluating the suitability of the aquifers for use as a water supply, the chloride content of each water bearing zone is of utmost importance. Chloride concentrations in excess of 250 mg/l indicate the aquifer is unsuitable for water supply development. 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 variations in permeabilities and varying hydrologic conditions at each site.

TABLE 5.2-2 COMPREHENSIVE WATER ANALYSIS OF PRINCIPAL AQUIFERS IN NORTHEASTERN DARE COUNTY

(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)	Bicarbonate (HCO_3)	Sulfate (SO_4)	Chloride (CL)	Fluoride (F)	Nitrate (NO_3) as N	Phosphate (PO_4)	Dissolved Solids (Sum)	Hardness (as $CaCO_3$)	pH	Color	Alk ($CaCO_3$)	Spec. Cond. (µmhos @ 25° C)
1	J-5, j-1a	332-342	14	.370	.000	.014	27	69	1030	51	0	548	87	1470	.2	3.30	.038	3040	370	8.2	10	548	5000
	1b	258-268	15	.204	.000	.012	22	27	245	22	0	448	.6	244	.4	1.70	.054	780	165	8.1	10	367	1300
	1c	162-172	24	.165	.096	.000	68	7.8	66	8.5	0	398	1.6	29	.2	.20	.000	405	202	7.7	10	326	650
	j-2a	34- 39	5.3	.000	.000	.000	25	3.8	16	.9	0	90	13	17	.1	1.20	.000	131	78	7.2	0	74	230
	j-2b	92-102	13	.048	.000	.016	29	2.3	14	1.2	0	106	.6	18	.1	.10	.000	131	82	7.4	0	87	220
2	I-4, v-1	10- 14	7.9	.000	.000	.014	6.7	.7	6.9	.9	0	20	.2	15	.1	.0	.000	48	20	6.4	0	16	85
	v-2a	360-370	18	.830	1.132	.016	21	73	1340	55	0	518	15	2050	.3	3.00	.000	3850	354	8.1	15	425	6500
	2b	264-274	9.9	.374	.081	.015	8.3	.8	350	13	0	594	13	238	.8	2.00	.120	948	54	8.2	140	487	1550
	2c	150-160	34	.176	.040	.006	12	6.7	94	13	0	315	.8	19	.5	.07	.190	339	58	7.6	30	258	525
	v-3a	50- 55	15	.096	.000	.021	40	2.7	29	1.4	0	178	8.2	20	.1	.00	.000	205	111	7.4	0	146	350
	3b	92- 97	17	.195	.109	.008	28	2.0	41	3.8	0	162	10.0	20	.3	.07	.000	203	78	7.3	0	133	330
3	J-3, f-2a	40- 45	12	.199	.000	.014	139	12.0	116	3.0	0	316	2.4	283	.2	.10	.000	726	395	7.8	25	259	1280
	f-2b	84- 89	19	.043	.000	.008	47	8.4	64	3.0	0	104	3.4	152	.1	.07	.000	349	152	7.7	0	85	720
	f-2c	324-334	16	.366	.198	.026	22	36	1070	37.0	0	600	11	1480	.6	1.0	.066	2980	202	8.1	20	492	5000
	f-2d	200-210	17	.000	.000	.014	61	8.0	34	5.8	0	252	6.2	43	.2	.00	.000	301	185	7.8	0	206	500
	f-3a	134-144	32	.239	.248	.000	71	10	34	8.9	0	299	.6	42	.2	.00	.000	349	218	7.6	5	189	550
4	J-3, y-2a	21- 31	6.9	.038	.000	.008	27	8.6	23	1.4	0	131	7.4	30	.1	.02	.000	169	103	7.5	0	107	300
	y-2b	63-73	13	.139	.050	.012	32	4.5	46	2.3	0	164	26	32	.1	.05	.000	238	99	7.6	10	134	390
	y-2e	189-199	21	.067	.000	.052	43	5.8	56	7.5	0	250	3.0	44	.2	.00	.043	306	132	7.7	5	205	500
	y-3a	105-115	37	.122	.000	.022	43	11	42	9.2	0	197	25	45	.3	.2	.000	311	152	7.7	10	162	500
5	h-2b	294-304	16	.294	.295	.004	8.7	12	750	23	0	668	2.4	828	1.1	2.9	.15	1990	70	7.9	50	548	3200
	h-2c	197-207	22	.215	.078	.000	24	3.4	167	10	0	412	1.8	90	.3	.05	.093	525	74	7.8	40	338	900
9	G-4, x-1a	461-471	2.6	.240	.000	.046	47	37	1260	47	0	294	9.4	1970	.4	.7	.000	3520	268	7.7	10	241	5800
	x-1b	363-373	2.2	.667	.000	.204	90	184	3860	136	0	234	74	6480	.5	.2	.000	10900	938	7.6	10	192	18000
	x-1c	305-315	6.2	.864	.000	.051	99	198	3980	126	0	476	46	6640	.4	.09	.000	11300	1060	7.9	20	390	17000
	x-1d	228-238	12	.696	.000	.040	58	155	2840	78	0	436	35	4710	.7	2.8	.076	8120	773	7.7	30	358	12200
	x-1e	187-197	1.6	.474	.059	.014	45	96	1740	50	0	381	27	2900	.5	.07	.000	5050	506	8.2	30	312	9000
	x-1f	101-106	15	.428	.381	.014	65	60	475	30	0	451	2.2	780	.5	.7	.093	1660	407	8.1	30	370	2800

The chloride content of the upper water table aquifer varies from 15 to 8000 ppm. High chloride concentrations are common along coastlines where the zone of mixing between freshwater and saltwater from the sounds is present. Figure 5.2-9 shows relative chloride concentrations in the upper aquifer.

In the principal aquifer, the chloride content of the water is relatively uniform. It ranges from 30 to 140 ppm on Roanoke Island but increases significantly east of the island.

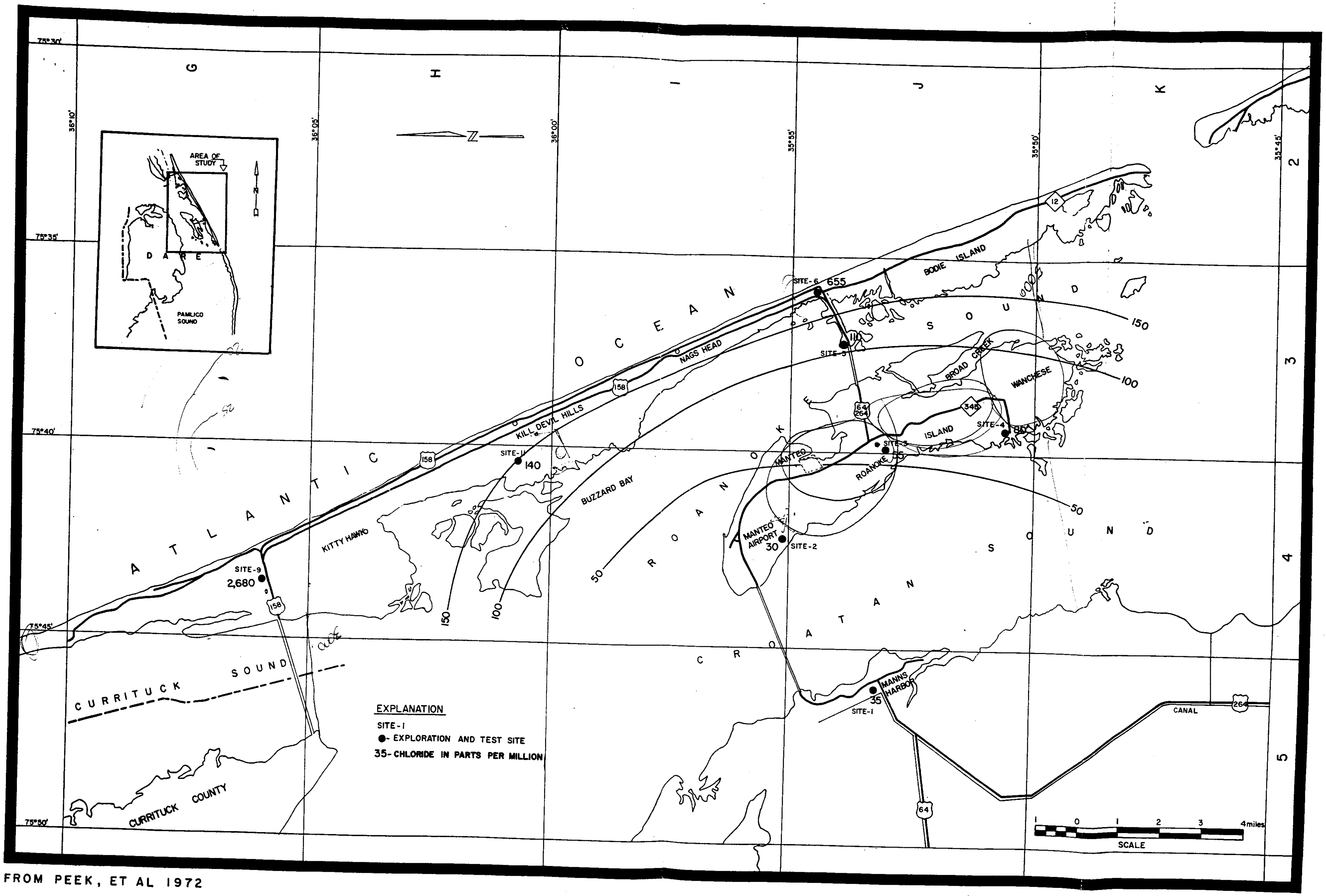
The chloride concentrations in the lower aquifer are generally related to aquifer depth. In the upper part of the unit, concentrations from 250 to 1500 ppm are common. In the lower portion of the aquifer, concentrations range from 1500 to more than 4600 ppm.

Only water from the upper water table aquifer and the principal aquifer are suited for water supply development on Roanoke Island. Generally, the water quality characteristics of these two aquifers are good. The water is hard but can be treated to provide potable water. In some areas, iron may be present in objectionable quantities (7.3 ppm), and iron removal may be necessary. The water can be classified as A according to the system devised by Heath (1981).

5.2.5 Recharge - Discharge

The recharge area for the upper water table aquifer is confined to the land surface of Roanoke Island. This is a total of 12,400 acres. Based on an annual precipitation of 52 inches per year and a 33% recharge volume of the precipitation, the annual water available for recharge to the upper aquifer is 5,699 million gallons. Not all of this water leaks to the principal aquifer. Some of the water is lost due to lateral discharge to the sounds. Winner (1975), estimates a head loss of .03 feet per day due to lateral discharge. This is an annual loss of 343 million gallons per square mile. Some discharge could be recovered by careful pumping.

Analysis of the potentiometric data of the principal aquifer indicates that the recharge area for the principal aquifer is on the mainland portion of Dare County and on Roanoke Island. The exact extent of the recharge area cannot be determined with the existing data. An estimate of its extent is shown on Figure 5.2-10. This is based on gradients on the mainland



FROM PEEK, ET AL 1972

FIGURE 5.2-9
CHLORIDE CONTENT OF WATER IN PRINCIPAL AQUIFER

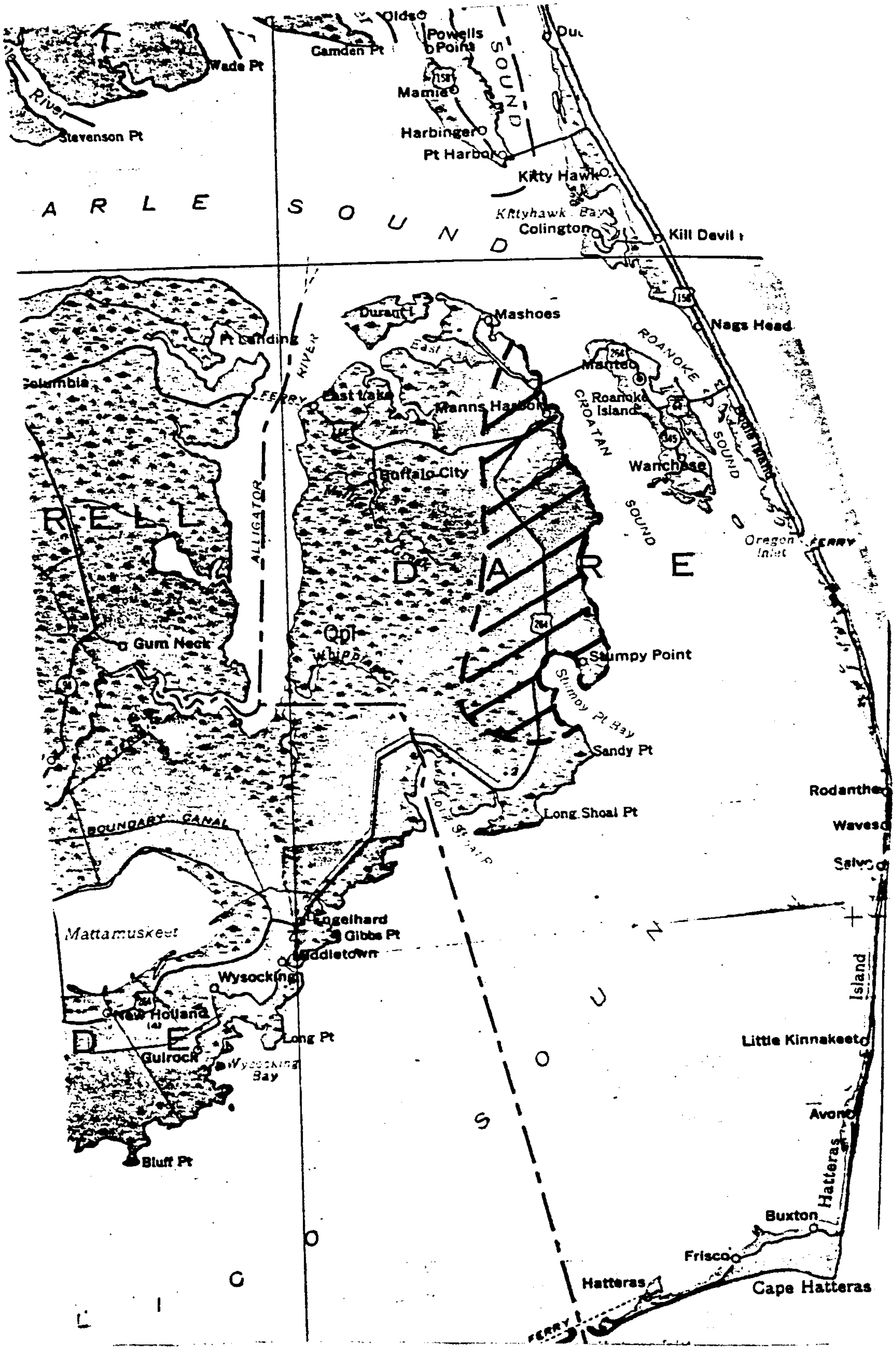


FIGURE 5.2-10 APPROXIMATE LOCATION OF RECHARGE AREA OF PRINCIPAL AQUIFER.

and discharge points. Assuming this recharge area is accurate, a total area of 42,000 acres recharges the principal aquifer on the mainland. This is a total annual recharge of 19,439 million gallons.

As stated above, the upper aquifer/lower aquifer system is a leaky aquifer. The leakance from the upper to the lower aquifer is 19.7 gpd/ft^2. This is approximately 244,280 gallons per day. Tables 5.2-3 and 5.2-4 show monthly water balances for recharge, precipitation and withdrawals from the upper aquifer alone, the upper aquifer plus the fresh pond, the lower aquifer alone and the lower aquifer plus the leakance from the upper aquifer.

Based on this analysis, it is evident that sufficient water supplies are available from the Roanoke Island Aquifer System to supply 15 mgd to the county water system. The operation of the water system must be controlled and optimized in order to provide this large water volume with no adverse effects.

5.2.6 Affect of Withdrawals

When a well or series of wells are pumped, water in the pore spaces of the sediments is drawn to the pumping source. This drawdown takes the shape of an inverted cone which has its vertex at the well intake (Figure 5.2-11). On Roanoke Island, the multi-aquifer system will respond to pumping in a similar fashion. Once pumping starts, water is immediately withdrawn from the principal aquifer; this creates a head differential between the upper and lower aquifers. After about the first 30 minutes of pumping, leakage from the upper aquifer to the lower aquifer occurs. This can be seen in almost all semi-log time drawdown plots from aquifer tests on Roanoke Island (Figure 5.2-8). After leakage begins, the cone of depression expands in the upper aquifer at a fairly constant rate until the vertical extent of the cone is lowered below the potentiometric surface of the upper aquifer. At this point, the upper aquifer has been "dewatered" to its maximum extent. Leakage from the upper aquifer slows and the major water volume is coming from the principal aquifer. Because of its high transmissivity, there are only small effects of pumping on the principal aquifer. The maximum lateral extent of the cone of depression in the upper aquifer ranges from 1,000 to 5,000 feet. As such, some overlapping of cones of

TABLE 5.2-3

UPPER AQUIFER
ROANOKE ISLAND GROUNDWATER EVALUATION

Potential Water Supply
Recharge by Month

	Climatological Data				Water Supply Month		
Month	P(ft)	R(.33XP)	Rvol (milgal)	Rvol + FP	15 mgd	10 mgd	5 mgd
Jan. (1980)	.49	.16	647	694	465	310	155
Feb.	.26	.09	364	406	*420	280	140
Mar.	.54	.18	727	774	465	310	155
Apr.	.39	.13	525	570	450	300	150
May	.37	.12	485	532	465	310	155
June	.28	.09	364	409	*450	300	150
July	.37	.12	485	532	465	310	155
Aug.	.21	.07	283	330	*465	310	155
Sept.	.42	.14	566	611	450	300	150
Oct.	.29	.10	404	451	*465	310	155
Nov.	.33	.10	404	449	*450	300	150
Dec.	.34	.11	445	492	*465	310	155
Average	.36	.12	475				

Roanoke Island Area (A) = 12,400 acres

Assume 33% recharge rate

FP - Fresh Pond

59B/BB
4

TABLE 5.2-4

LOWER PRINCIPAL AQUIFER
MAINLAND GROUNDWATER EVALUATION
Groundwater Divide from Manns Harbor to Stumpy Point

	Climatological Data			Water Supply Month		
Month	P(ft)	R(.33XP)	Rvol (milgal)	15 mgd	10 mgd	5 mgd
Jan. (1980)	.49	.16	2,191	465	310	155
Feb.	.26	.09	1,232	420	280	140
Mar.	.54	.18	2,464	465	310	155
Apr.	.39	.13	1,779	450	300	150
May	.37	.12	1,643	465	310	155
June	.28	.09	1,232	450	300	150
July	.37	.12	1,643	465	310	155
Aug.	.21	.07	958	465	310	155
Sept.	.42	.14	1,916	450	300	150
Oct.	.29	.10	1,369	465	310	155
Nov.	.33	.11	1,506	450	300	150
Dec.	.34	.11	1,506	465	310	155

Mainland Area 42,000 acres

R = Recharge

Rvol = Recharge Volume

59B/BB
5

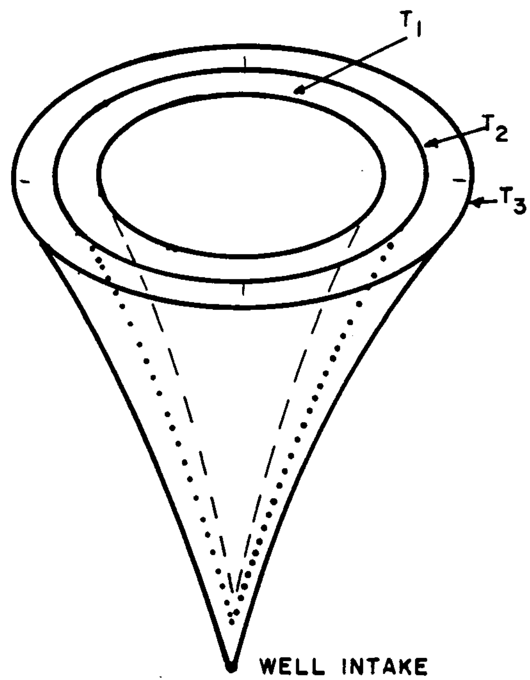


FIGURE 5.2-11. CONE OF DEPRESSION WITH CONSTANT PUMPING RATE AND VARIABLE TIME.

depression occur between mutually pumping wells. This produces a cumulative drawdown which is equal to the sum of the drawdowns of the wells. Based on graphical analysis of pumping affects, this well interference can result in increased vertical drawdowns in the upper aquifer of 6-10 feet.

This lowering of the potentiometric surface and the lateral migration of the cone of depression toward the sounds is of prime importance in system operation. Overpumping of the aquifer by long-term pumping and dewatering of the upper aquifer could result in a reversal of the natural hydraulic gradient from the inland portions of the island outward to the sounds. If this reversal occurs, water from the sounds could be made to flow towards the well intakes, resulting in saltwater encroachment. The possibility of this saltwater encroachment is magnified greatly by increased pumping, long duration pumping and increasing the number of wells.

An assessment of the performance of the wells and the outward extension of the cone of depression under cyclic pumping was performed. This analysis indicated that during pumping periods of up to 18 hours, only minimal effects on lateral migration of the cone of depression occurred. During the down time, the aquifer recovered and returned to its natural discharge equilibrium i.e., lateral migration of water laterally to the sounds. This allowed any movement of saline water inward to the wells to be flushed back out of the aquifer system by natural discharge. This concept is of prime importance in long-term management of the groundwater resources.

5.2.7 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. Based on a total annual recharge of 19,439 million gallons and an annual withdrawal of 5,475 million gallons (15 mgd), it can be seen that more than adequate water is available as a groundwater resource. The major problem is recovery of the resource 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:

1. Require residents of Wanchese to hook up to the County water system.

2. Install or repair all well head water meters.

3. Develop additional wells as required to allow staggered pumping of wells in the well field by cyclic pumping.

4. Develop additional wells along the spine of the island no closer than 1,000 feet from each other (Figure 5.2-12).

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 (Figure 5.2-13) at the locations shown on Figure 5.2-12.

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 as to any modifications required in the pumping scheme or in future well development.

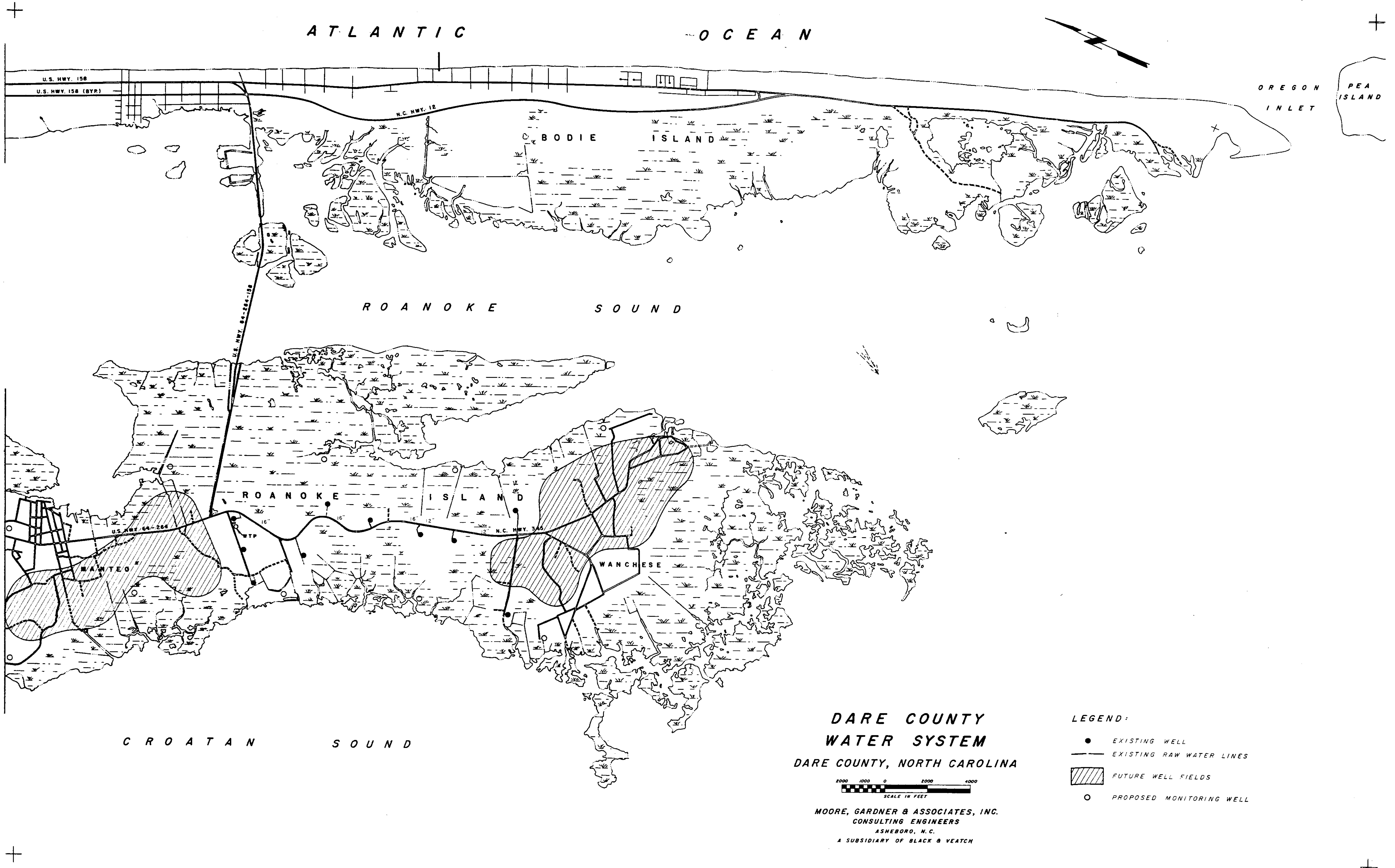

ATLANTIC OCEAN
U.S. HWY. 158
U.S. HWY. 158 (BYP.)
N.C. HWY. 12
BODIE ISLAND
OREGON INLET
PEA ISLAND
ROANOKE SOUND
U.S. HWY. 64-264-158
ROANOKE ISLAND
U.S. HWY. 64-264
N.C. HWY. 345
WTP
MANTEO
WANCHESE
16"
12"
CROATAN SOUND
DARE COUNTY
WATER SYSTEM
DARE COUNTY, NORTH CAROLINA
2000 1000 0 2000 4000
SCALE IN FEET
MOORE, GARDNER & ASSOCIATES, INC.
CONSULTING ENGINEERS
ASHEBORO, N.C.
A SUBSIDIARY OF BLACK & VEATCH
LEGEND:
EXISTING WELL
EXISTING RAW WATER LINES
FUTURE WELL FIELDS
PROPOSED MONITORING WELL

FIGURE 5.2-12

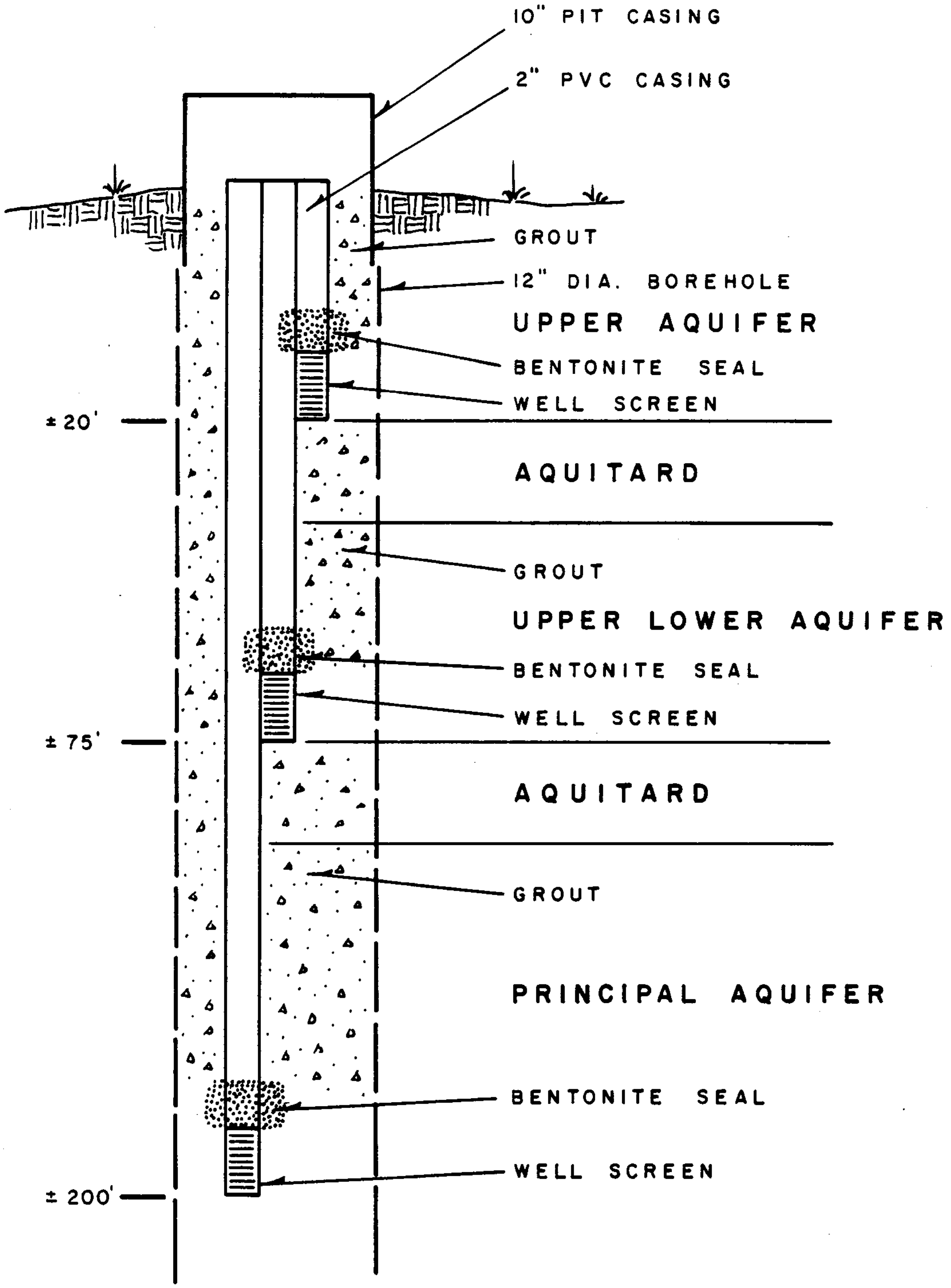


FIGURE 5.2-13 TYPICAL MONITORING WELL CLUSTER CONFIGURATION

5.3 MAINLAND GROUNDWATER

The mainland portion of Dare County has the largest land area in the County. However, this area is not developed and is not suitable for providing water via a large scale distribution network because of the limitation of groundwater supplies on Roanoke Island. It may be necessary to develop water supply on the mainland, if demand surpasses 15 mgd.

5.3.1 Hydrogeologic Framework

The hydrogeology of the mainland section of Dare County is very similar to that of Roanoke Island. The stratigraphic sequence along the Eastern margin of the mainland correlates directly to the sequence on Roanoke Island. Also, the principal water bearing zones present on Roanoke Island are present near the Manns Harbor area.

5.3.2 Safe Yield

Test well drilling and pumping tests conducted near Manns Harbor by the NC Division of Groundwater indicate that ample groundwater supplies exist. These test wells were drilled to depths of ±500 feet and encountered the principal aquifer at depths ranging from 50-100 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 the test wells is 4.58 gal/min/ft of drawdown and 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 500-800 gallons per minute. Wells should be spaced at least 2,000 feet apart to avoid pumping interference.

5.3.3 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 5.2-2 gives the results of water quality testing conducted by the Water Resources Division of the US Geological Survey on the water recovered from the principal aquifer at the test well sites. This analysis indicates that the groundwater is hard, but has little iron or manganese, and is low in chlorides. The groundwater is typical of the groundwater in the Miocene aquifer in Eastern North Carolina.

5.3.4 Impacts of Proposed Land Uses on Aquifer

Several large scale commercial farming operations have proposed development of tracts of land on the Dare County mainland. In order to farm, drainage of large swampy areas is required. These tracts serve as the recharge area for the Miocene aquifer. Should drainage canals be constructed in the recharge areas, both water quantity and water quality would probably be adversely impacted. It is imperative that steps be taken to introduce land use controls in the recharge areas of Dare County in order to protect the County's water supply. Possible actions include designating the area as a capacity use zone or invoking powers under the Water Use Act which would limit withdrawals from the recharge area.

5.3.5 Facilities Description and Cost Estimate

Development of groundwater resources on the Dare County mainland would require construction of raw water facilities, water treatment facilities and transmission mains.

In order to supply an additional 5 mgd to the County water system, at least seven new wells producing 500 gallons per minute (gpm) would be needed. A 5 mgd capacity treatment plant consisting of water softening, chlorination and blending would also be required. A transmission main across Roanoke Sound would be needed to tie the new water supply into the existing County system. Table 5.3-1 is a breakdown of the system components and their probable costs in 1983 dollars. Because of the high cost for sound crossing, development of this groundwater source is capital intensive.

5.4 DESALINIZATION

Desalinization, or the recovery of potable freshwater from brackish water, is an emerging technology that has been developed over the past 15 years. The most common form of desalinization is the reverse osmosis (RO) procedure. This technique utilizes permeator modules and a compact cylinder, containing hollow fibers which reject salts by permitting only purified

TABLE 5.3-1

DARE COUNTY WATER SUPPLY - 5.0 mgd WTP
CROSSING THE CROATON SOUND

Preliminary Opinion of Probable Project Cost

Item	Quantity	Unit	Unit Cost	Total Cost
1.0 Site Work				
Site Preparation		LS	---	50,000
Clearing & Grubbing	10	acre	2,000	20,000
2.0 Raw Water System				
Well & Pumphouse	7	LS	120,000	840,000
6" Water Line	2,500	lin ft	9.00	22,500
8" Water Line	2,500	lin ft	12.00	30,000
12" Water Line	5,000	lin ft	18.00	90,000
3.0 Finished Water				
Water Treatment Plant (5.0 MGD)		LS	---	6,000,000
Ground Storage Tank	1	LS	---	350,000
18" Water Line	13,500	lin ft	27.00	364,500
18" River Crossing	21,500	lin ft	175.00	3,762,500
4.0 Other Appurtenances				50,000
Construction Cost				$11,579,500
Land & Right-of-Way				50,000
Legal & Administrative				116,000
Engineering Design				695,000
Construction Observation				168,000
Contingency				1,157,500
				$13,766,000

water to penetrate into each hollow center. The desalted water then flows to a collection chamber at the end of each module. The permeators produce freshwater from seawater at a single pass rather than by the older system of recirculation. This greatly reduces the power usage which is the major operating cost in original units. If RO was utilized in Dare County, deep wells would most likely be used to supply the raw water. Because the chlorides and TDS in this water would be lower than seawater, the power costs would be reduced.

The RO process is the most efficient means of producing freshwater from seawater. Other methods such as vapor compression distillation processes require twice the electric power of an RO unit. The flash distillation process requires four or more times as much energy as required by reverse osmosis. Corrosion is also minimized with an RO unit. The premeator and peripheral equipment are constructed of inert material, corrosion resistent plastics and stainless steels. Also, the seawater is processed at ambient temperatures and no heat is required. A schematic of a typical RO unit is shown in Figure 5.4-1. The treatment scheme normally consists of pH neutralization followed by sand filtration prior to flow through the permeator. Finished water goes to storage/additional treatment and the wastewater is rejected.

The average size of most RO plants is less than 1 mgd. The largest plant in the western hemisphere is the 3 mgd RO plant at Key West, Florida. Capital and operating costs for a desalinization plant are higher than conventional water treatment facilities. Costs of the 3 mgd Key West plant are provided in Table 5.4-1 as a comparative tool.

Desalinization is a costly method of supplying water. Unless groundwater sources on Roanoke Island are not capable of providing a 15 mgd safe yield, it would not be cost effective to develop an RO plant.

5.5 WATER REUSE

Water reuse involves treating wastewaters so that they become potable. The County will have to go to a wastewater collection and treatment system in the near future. Because of the high costs of other raw water supplied, additional raw water treatment equipment incorporated into the wastewater treatment process could prove to be the most cost effective means for providing long-term quantities of potable water.

FIGURE 5.4-1

Plant Configuration and Design

Figures 1 and 2 show the plant layout and process flow. Seawater containing 38,000 parts per million (ppm, mg/liter) dissolved solids (TDS) is pumped into the plant from two seawells.

Figure 1
Plant Layout Schematic

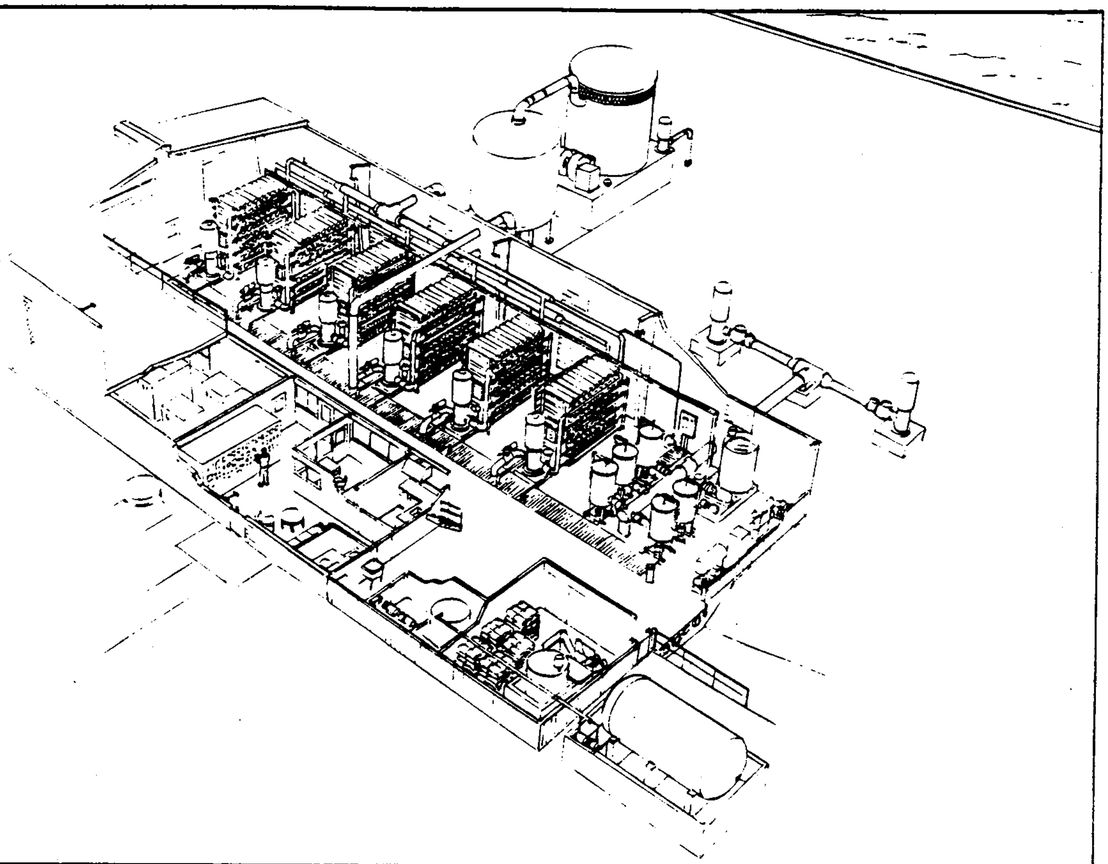


Figure 2
Simplified Process Flow Diagram

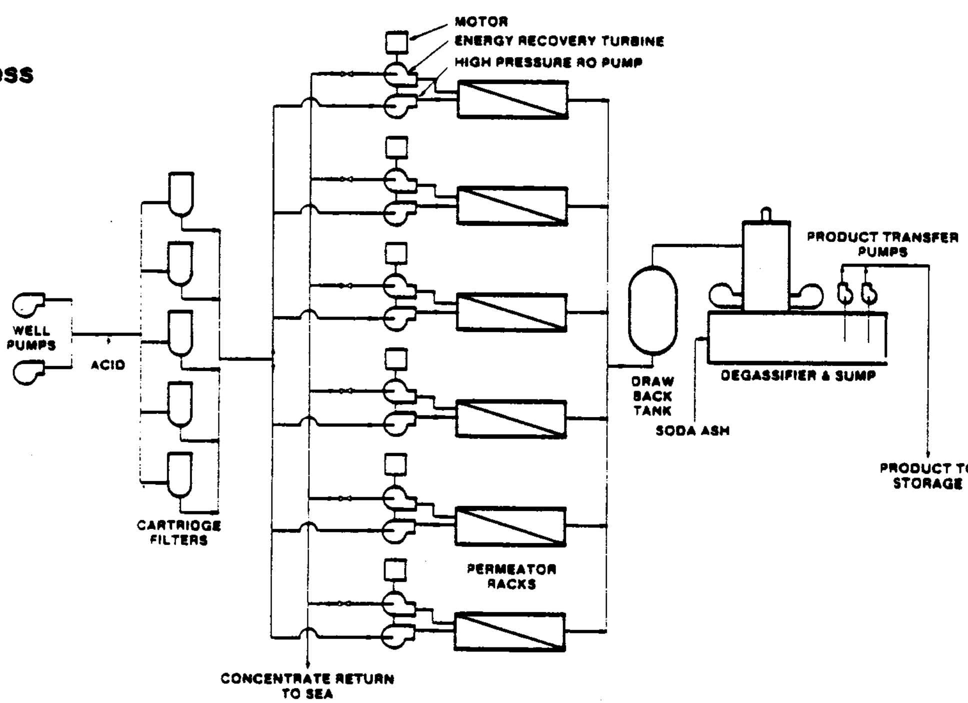


TABLE 5.4-1

COSTS OF 3 mgd
KEY WEST, FLORIDA RO PLANT

Capital Costs (installed)	$11,250,000	($3.75/gpd)
Fixed operating costs (labor payroll, fringes, transportation, office and supplies)	427,050	($0.39/1000 gal)
Variable operating costs	87,000	($0.08/1000 gal)
Power (@ $0.08/Kwhr)	2,222,950	($2.03/1000 gal)
Equipment	32,950	($0.03/1000 gal)

The Engineers realize that this alternative has several logistic and social problems to overcome; however, technology does currently exist that could make this alternative attractive.

6.0 SYSTEM COMPONENT EVALUATION

6.1 RAW WATER SYSTEM

The existing raw water system is comprised of 10 deep wells located on Roanoke Island (Figure 6.1-1). These wells are screened in the principal aquifer which is semi-confined. Well depths average 200-250 feet in depth. Yields range from 300-600 gpm with most wells capable of producing 500 gpm. Because of the high total dynamic head in the raw water collection mains and in the treatment plant, the pumps are operating below their designed optimum performance condition. At optimum conditions, the well field can produce 5,000 gpm (7.2 mgd).

The problem with the low efficiency of the raw water system can be relieved either by paralleling some of the raw water lines with larger diameter pipe or by installing a booster pump ahead of the water softeners. The life cycle costs of these items is such that the most cost-effective alternate is the booster pump. This item will be included in cost estimates and discussed further below.

6.2 DISTRIBUTION SYSTEM EVALUATION

In order to explain the operations of the distribution system (Figure 6.1-1), it is necessary to divide the system into two parts. The first part is the low pressure transmission system that pumps water approximately 23 miles along the Outer Banks. The transmission network consists of 2-750 and 2-4000 gpm high service pumps at the water plant and 24 inch and 16 inch water mains. Water is pumped from the water plant through the 24 inch main that runs to the town limits of Nags Head and Kill Devil Hills (vicinity of Fresh Pond) and connects to the 16 inch main that delivers water to Kitty Hawk and areas to the north. No service connections are made on the transmission main; only the taps where the towns take out their supply. Water flows out of the transmission main into several ground storage tanks owned by the Town and/or County at Kitty Hawk. Future plans of the Towns include adding new withdrawal taps on the transmission mains. However, depending on the exact location, the take out points may have a significant

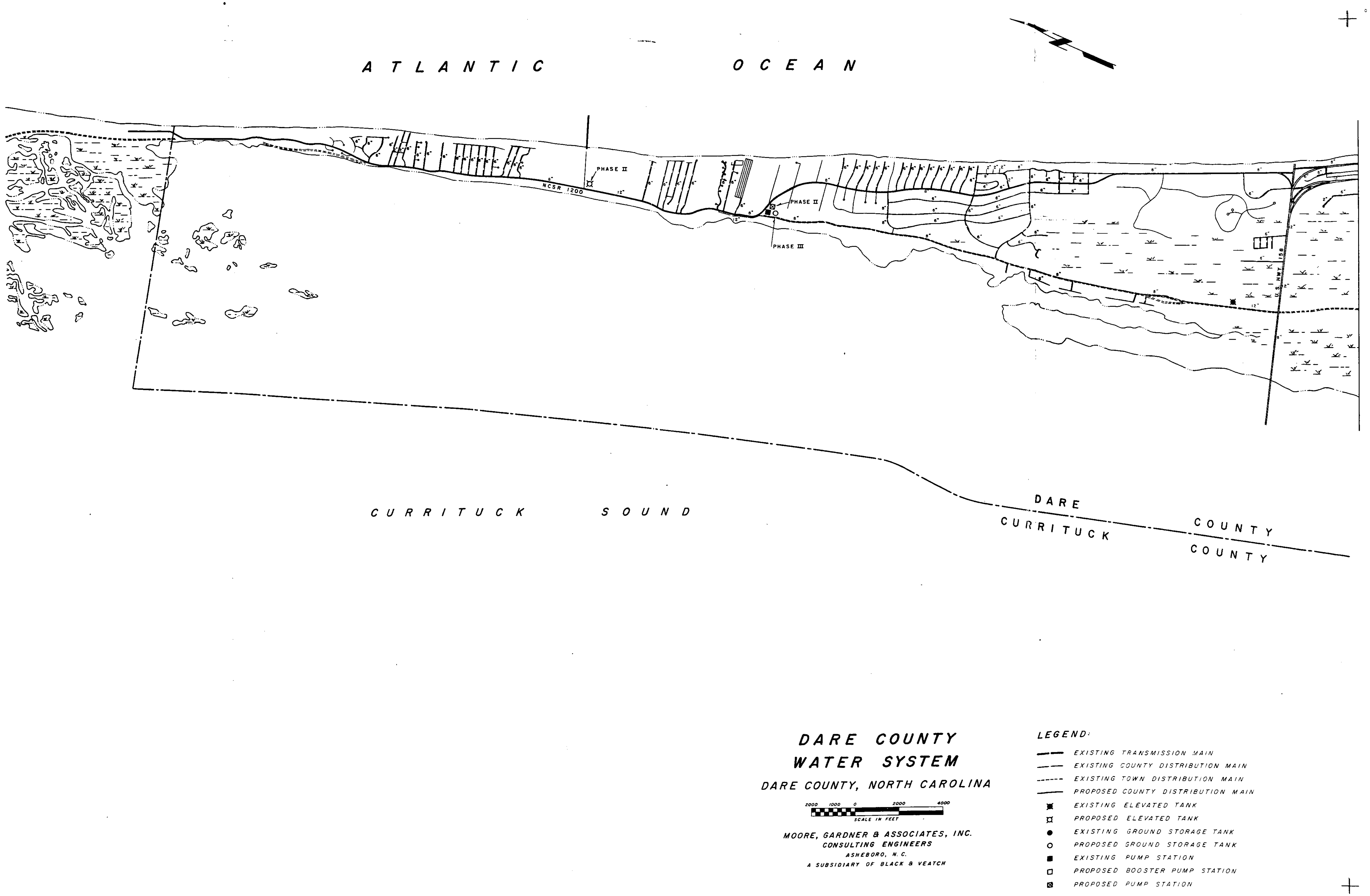

ATLANTIC OCEAN
CURRITUCK SOUND
DARE COUNTY
CURRITUCK COUNTY
NCSR 1200
U.S. HWY 158
PHASE II
PHASE II
PHASE III
DARE COUNTY
WATER SYSTEM
DARE COUNTY, NORTH CAROLINA
2000 1000 0 2000 4000
SCALE IN FEET
MOORE, GARDNER & ASSOCIATES, INC.
CONSULTING ENGINEERS
ASHEBORO, N. C.
A SUBSIDIARY OF BLACK & VEATCH
LEGEND:
EXISTING TRANSMISSION MAIN
EXISTING COUNTY DISTRIBUTION MAIN
EXISTING TOWN DISTRIBUTION MAIN
PROPOSED COUNTY DISTRIBUTION MAIN
EXISTING ELEVATED TANK
PROPOSED ELEVATED TANK
EXISTING GROUND STORAGE TANK
PROPOSED GROUND STORAGE TANK
EXISTING PUMP STATION
PROPOSED BOOSTER PUMP STATION
PROPOSED PUMP STATION

FIGURE 6.1 - 1A

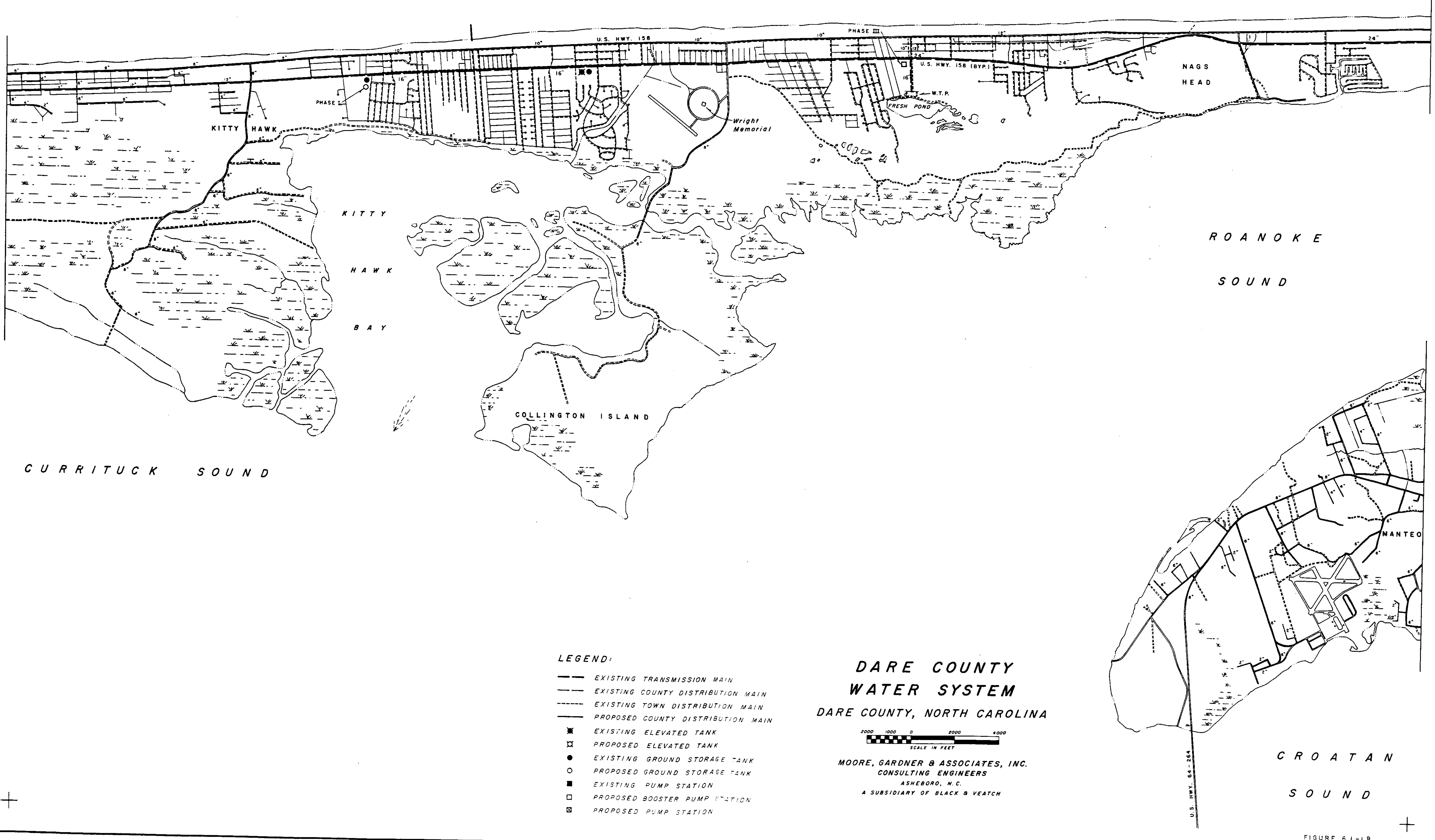

ATLANTIC OCEAN
U.S. HWY. 158
U.S. HWY. 158 (BYP.)
PHASE I
PHASE III
W.T.P.
FRESH POND
Wright Memorial
KITTY HAWK
NAGS HEAD
KITTY HAWK BAY
COLLINGTON ISLAND
CURRITUCK SOUND
ROANOKE SOUND
CROATAN SOUND
MANTEO
U.S. HWY. 64-264
LEGEND:
EXISTING TRANSMISSION MAIN
EXISTING COUNTY DISTRIBUTION MAIN
EXISTING TOWN DISTRIBUTION MAIN
PROPOSED COUNTY DISTRIBUTION MAIN
EXISTING ELEVATED TANK
PROPOSED ELEVATED TANK
EXISTING GROUND STORAGE TANK
PROPOSED GROUND STORAGE TANK
EXISTING PUMP STATION
PROPOSED BOOSTER PUMP STATION
PROPOSED PUMP STATION
DARE COUNTY
WATER SYSTEM
DARE COUNTY, NORTH CAROLINA
2000 1000 0 2000 4000
SCALE IN FEET
MOORE, GARDNER & ASSOCIATES, INC.
CONSULTING ENGINEERS
ASHEBORO, N.C.
A SUBSIDIARY OF BLACK & VEATCH

FIGURE 6.1-1B

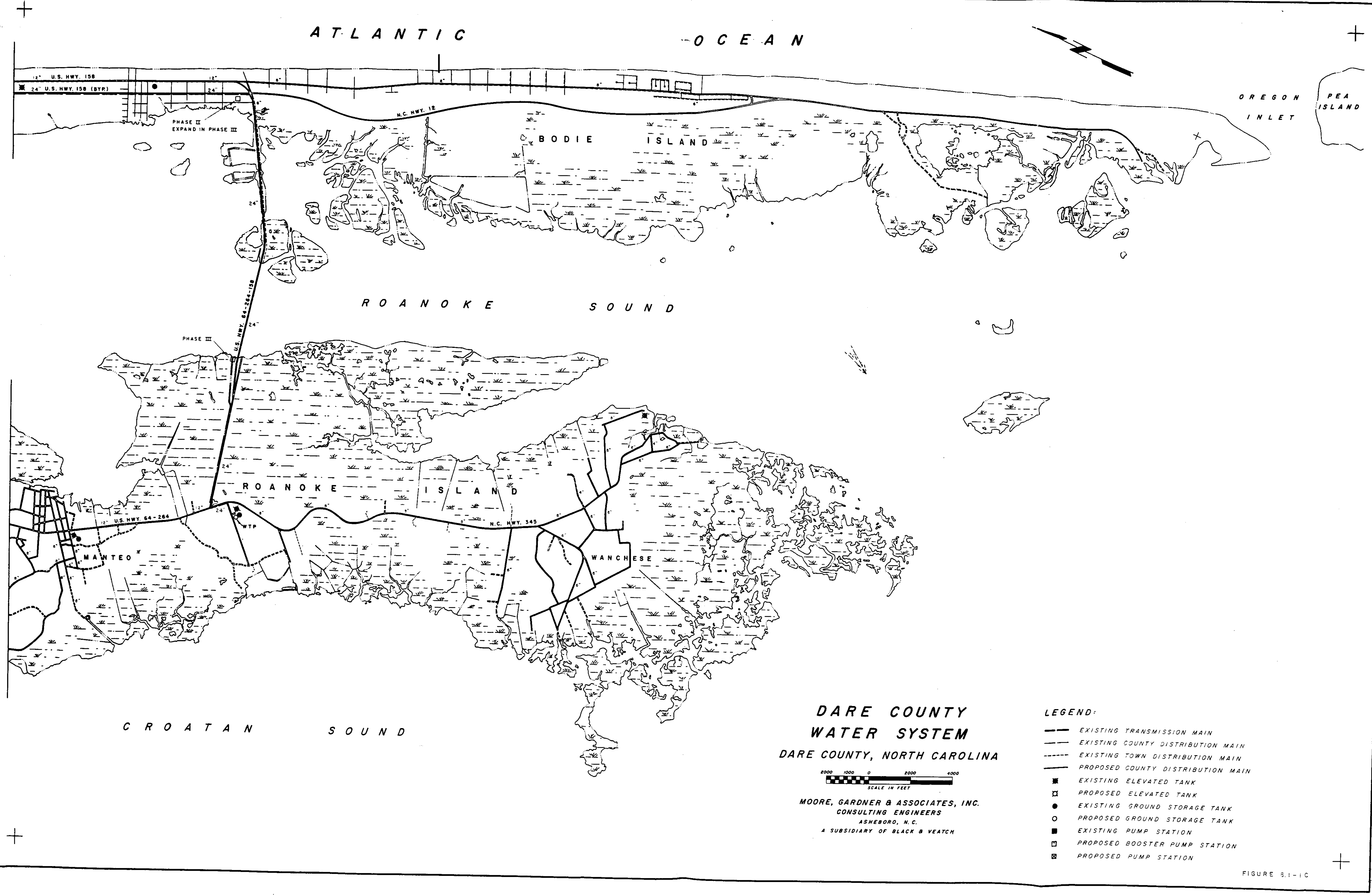

ATLANTIC OCEAN
U.S. HWY. 158
U.S. HWY. 158 (BYP.)
PHASE II EXPAND IN PHASE III
N.C. HWY. 12
BODIE ISLAND
OREGON INLET
PEA ISLAND
ROANOKE SOUND
PHASE III
U.S. HWY. 64-264-158
ROANOKE ISLAND
U.S. HWY. 64-264
N.C. HWY. 345
MANTEO
WTP
WANCHESE
CROATAN SOUND
DARE COUNTY
WATER SYSTEM
DARE COUNTY, NORTH CAROLINA
SCALE IN FEET
MOORE, GARDNER & ASSOCIATES, INC.
CONSULTING ENGINEERS
ASHEBORO, N.C.
A SUBSIDIARY OF BLACK & VEATCH
LEGEND:
EXISTING TRANSMISSION MAIN
EXISTING COUNTY DISTRIBUTION MAIN
EXISTING TOWN DISTRIBUTION MAIN
PROPOSED COUNTY DISTRIBUTION MAIN
EXISTING ELEVATED TANK
PROPOSED ELEVATED TANK
EXISTING GROUND STORAGE TANK
PROPOSED GROUND STORAGE TANK
EXISTING PUMP STATION
PROPOSED BOOSTER PUMP STATION
PROPOSED PUMP STATION

FIGURE 6.1-1C

impact on delivering water to the northern beach area (Kitty Hawk). A hydraulic analysis was made by computer modeling to determine the affects of additional take out points. The results of the modeling is discussed below.

6.2.1 Nags Head

The proposed ground storage tank (take out) would be located approximately 4 miles from the water plant and approximately 6 miles from the existing ground storage tank, which is 10 miles from the County water plant (Figure 6.2-1). The construction of this tank would benefit the County by reducing the delivery time required to transport the water to the existing location. The withdrawal at this new location would increase the flow to the northern section due to less head loss in the system. This inturn would decrease the head on the pumps and result in an increased pumping rate.

The location of the proposed take out could provide the County with some flexibility during peak seasonal demand periods. An agreement with the Town of Nags Head to use its new take out location only during peak demands, would enable the County to deliver more water to Kitty Hawk and areas to the north.

6.2.2 Kill Devil Hills

Existing ground storage is located 10 miles from the County water plant; the proposed tank is located approximately 13 miles from the County water plant (Figure 6.2-2). The construction would have an adverse effect on the system since it would require longer delivery time to the areas north of Nags Head than is required under current conditions. The effect would be the reverse of that caused by the Nags Head tap, i.e., the head on the pump would increase and the flow in the system would decrease.

Although the proposed take out location would produce some constraints on delivering water to the Kitty Hawk area, an agreement with the Town of Kill Devil Hills to close its new take out point during peak demand periods would reduce the impact.

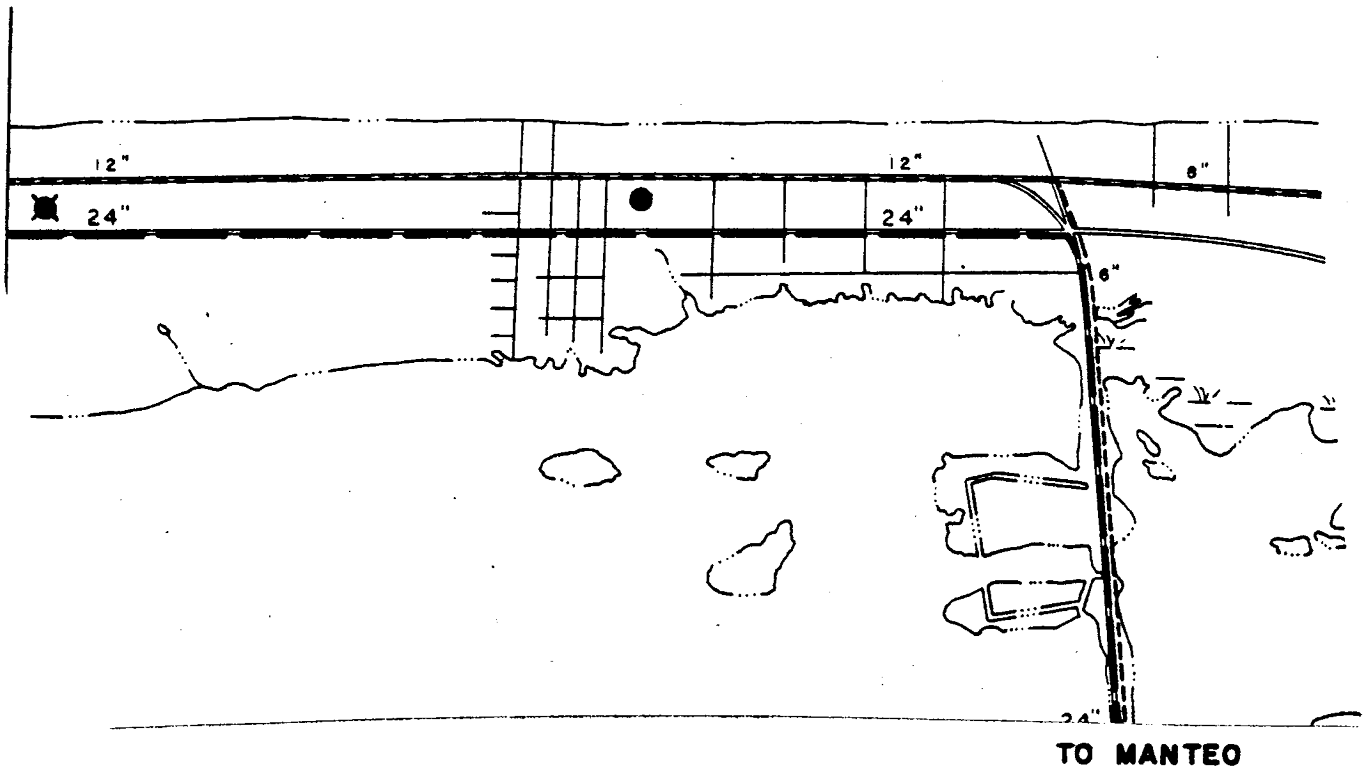


FIGURE 6.2-1. NAGS HEAD TAP LOCATION.

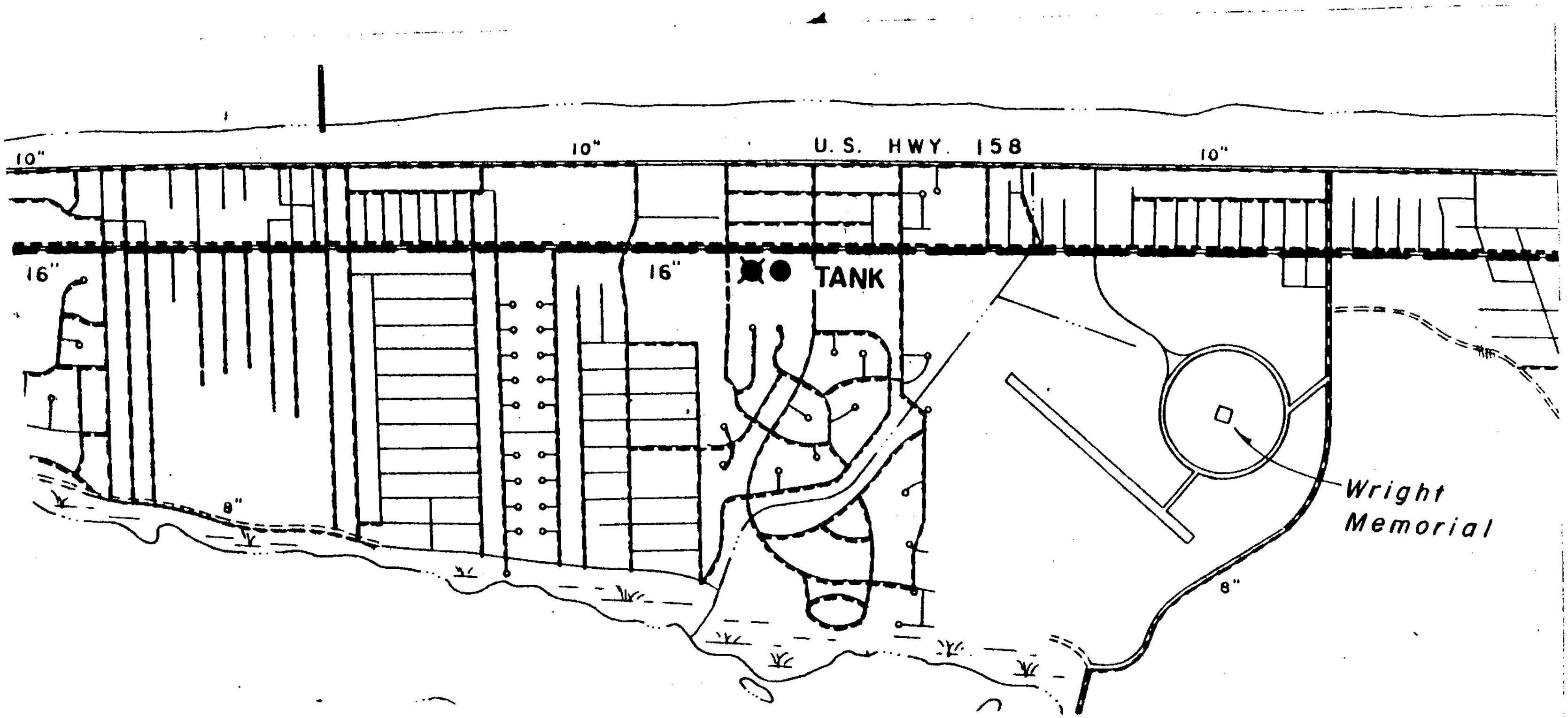


FIGURE 6.2-2. PROPOSED TANK LOCATION

6.2.3 Kitty Hawk and Areas North

Delivering water to Kitty Hawk and areas to the north is a major problem with the existing transmission system. The distance away from the water plant and the taps at Nags Head and Kill Devil Hills impose problems in delivering the required flow during the peak demand period. Computer modeling indicates that the maximum flow that can be delivered to the north is 780 gpm, when the Nags Head and Kill Devil Hills systems are withdrawing water. This is substantiated by the information received from the water plant operator concerning the flow in the ground storage tank at Kitty Hawk. When the 1,500 gpm pump at the ground storage tank is operating, the withdrawal from the tank is greater than the inflow, as indicated in the computer modeling. The computer modeling also shows that the maximum water quantity that can be delivered to Kitty Hawk area with no other withdrawal is approximately 3.39 mgd.

The second part of the distribution systems is the County's service area consisting of water mains ranging from 16 inches to 2 inches. The service distribution system is located mainly in areas north of Kill Devil Hills. The residents of the Towns of Kitty Hawk and Southern Shores are provided water service by the County as retail customers. The Towns do not own any part of their water system. Adequate pressure is available to supply peak demands and for emergency purposes including fire protection. However, future needs indicate that an additional elevated storage tank should be provided in the Duck area to provide adequate pressure.

6.2.4 Storage Capacity Evaluation

The County's service areas to the north of Kill Devil Hills is located approximately 23 miles from the water plant. This distance, requires that adequate storage be available for emergency purposes such as a major breakage on the transmission main or high fire flows.

It is suggested that the County maintain a one day supply of storage in the northern area. Currently, the County has .5 million gallon elevated and .5 million gallon ground storage available. During the peak season of 1983, the water demands reached 1.3 mgd in the area. The demands are expected to increase to approximately 1.9 mgd by 1985. It is recommended that initially, additional storage of 1 million gallon be provided and finally, to increase the storage to 3.5 million gallons by the end of the planning period.

6.3 TRANSMISSION SYSTEM

6.3.1 System Description

The transmission system delivers finished water from the water treatment plant to the point of draw-off for wholesale water users and to the beginning of the distribution system for retail customers. At present, the transmission system consists of a 24 inch water main from the water treatment plant to the ground storage reservoirs at the boundary of Nags Head and Kill Devil Hills. The continuing 16 inch main serves only the water distribution system north of Kill Devil Hills and is really a part of the distribution system. It is discussed in this section because its operation affects the transmission system analysis and design, and has no direct affect on the distribution system. The high service pumps at the water treatment plant are considered a part of the water treatment plant, in terms of construction and operation; however, the design of these pumps is determined both by the treatment plant capacity and by the flow rates and pressure requirements on the transmission system. These requirements are discussed in this section.

6.3.2 System Evaluation

Both computer and manual analyses of the transmission system were performed as part of this study. Figure 6.3-1 presents a diagram of this system together with the flow rates required in each of Phases I, II, and III.

All three phases were analyzed to determine the system operating characteristics and any improvements required. The results of the analyses are described in the section which follows. Phase III is discussed first, since all improvements developed in Phases I and II must be consistent with those required for the ultimate condition.

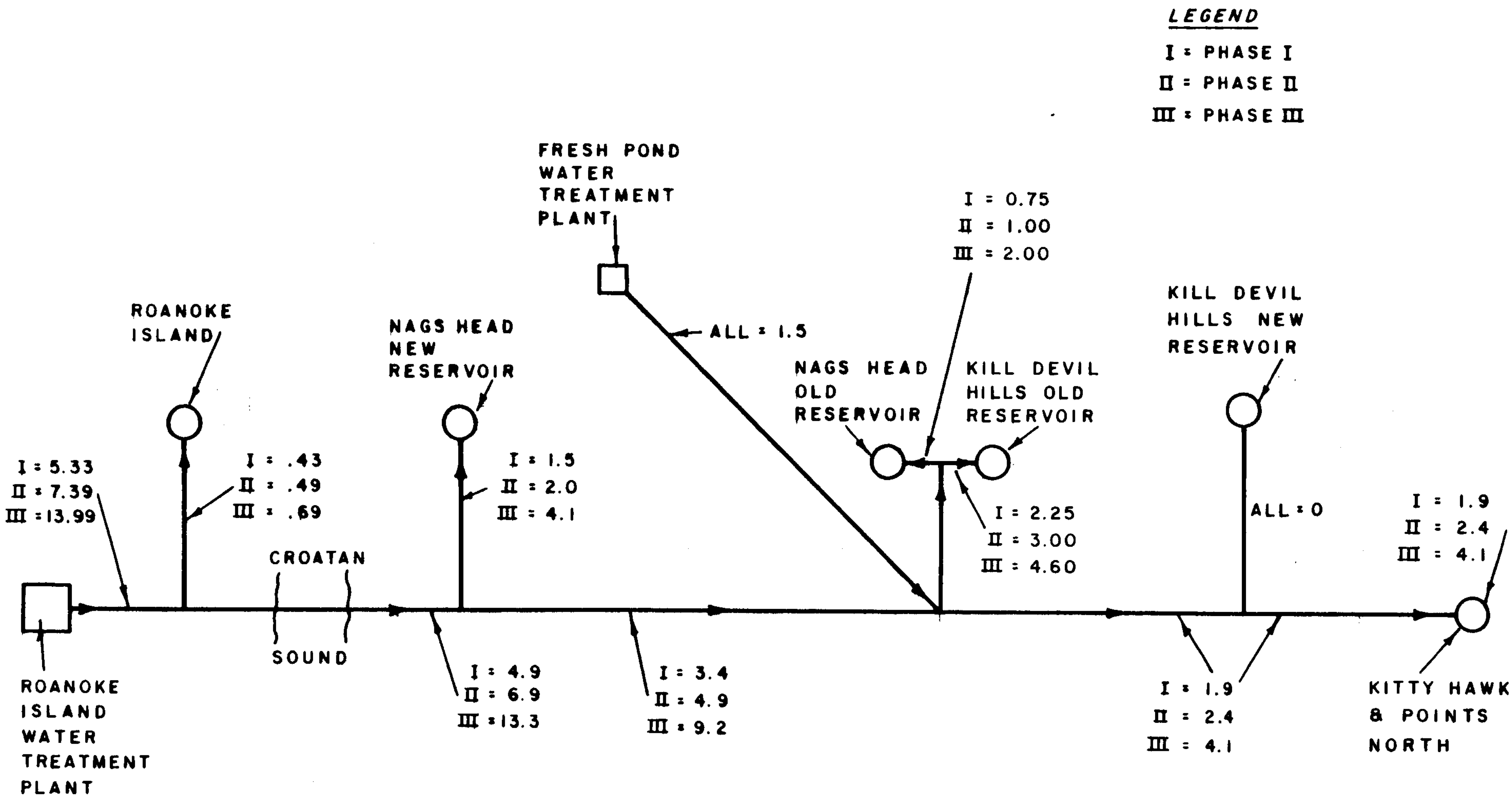


FIGURE 6.3-1
TRANSMISSION SYSTEM FLOW RATES

6.3.3 Phase III

The very large water demands and flow rates required in Phase III 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 high service 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 best to maintain a high factor of safety in the main under Roanoke Sound since repairs to this main would be extremely costly and would severely cripple the entire system for a considerable time. Therefore, this alternative is not considered feasible.

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 pump stations, as required, along the transmission line. The hydraulic analysis determined that three booster stations would be required; one on Roanoke Island, adjacent to Roanoke School; one at Whalebone Junction and, one at the Nags Head/Kill Devil Hills boundary.

   These three pump stations, in conjunction with the existing transmission line, will deliver the water required under Phase III, including maintaining pressures in the subaqueous main below 60 psi.

   Table 6.3-1 presents the costs of these three booster stations, including the present worth of power costs, and compares them to the new transmission

TABLE 6.3-1

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

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 Phase III 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 for producing 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.

6.3.4 Phase II

The hydraulic analysis for Phase II determined that any one of the booster stations required under Phase III 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 Phase III. The Whalebone Junction booster station is better matched to the Phase II conditions than does the Kill Devil Hills booster station, in that it can be installed at a reduced capacity in Phase II and expanded in Phase III. This cannot be done for the Kill Devil Hills station since it will have only two pumps in the ultimate condition.

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

6.3.5 Phase I

The hydraulic analysis for Phase I indicated that the water demand will exceed the transmission main capacity by the 1984 or 1985 peak season. There are two alternative ways, consistent with the Phase III improvements, to correct this situation.

Alt. I. Improve the Nags Head Fresh Pond treatment plant and bring it on-line as soon as possible. This will reduce the amount of water that must be pumped from the Roanoke Island treatment plant. It has the added benefit of increasing the total amount of water available for use.

Alt. II. Install the Whalebone Junction booster station as soon as possible. This will increase the pressure available for all water users on the distribution system.

Only Alternative I is viable, since the maximum capacity of the Roanoke Island water treatment plant will be exceeded by the 1985 peak season, based on current use projections. Alternative II, while it provides adequate pressures at a lower initial costs, fails to meet the water treatment needs of the system.

6.4 WATER TREATMENT

The existing water treatment plant is an ion-exchange softening plant, consisting of four pressure softeners, salt regeneration facilities, six high service pumps, and associated piping and appurtenances. The plant employs a blending process to achieve the desired hardness level in water; that is, a portion of the water entering the plant bypasses the softeners and is not treated. This is combined or blended with water that was treated by the softeners, resulting in a water with an acceptable level of hardness. The plant, on-line since 1980, was supplied by Bruner Corporation.

As part of this study, the treatment plant was analyzed for specific operational problems, for existing capacity and for its potential for future expansion.

6.4.1 Required Treatment Plant Capacity

Table 6.4-1 provides a summary of projected water demands and treatment plant capacities. Alternative I is the immediate improvement under Phase I of the water treatment plant on-line in 1985. Alternative II is the delay of this improvement until Phase II, sometime between 1985 and 1990.

TABLE 6.4-1

FUTURE WATER DEMANDS AND PLANT CAPACITIES

	1985	1988	1990	2000	2005
(1) Existing System Demand	6.61	7.88	8.64	11.81	15.16
Alt. I - Fresh Pond Plant (available)	1.50	1.5	1.5	1.5	1.5
Roanoke Island Plant (req'd)	5.11	6.38	7.14	10.31	13.66
Alt. II - Fresh Pond Plant (available)	____	1.5	1.5	1.5	1.5
Roanoke Island Plant (req'd)	6.61	6.38	7.14	10.31	13.66
(2) Including Wanchese Area	6.83	8.07	8.89	12.09	15.49
Alt. II - Fresh Pond Plant (available)	1.5	1.5	1.5	1.5	1.5
Roanoke Island Plant (req'd)	5.33	6.57	7.39	10.59	13.99

NOTES:

(1) Alternative I calls for Fresh Pond Plant to be on-line in 1985

(2) Alternative II calls for Fresh Pond to be on-line as required between 1985 and 1990.

(3) Design capacity of Roanoke Island Plant is 5.0 mgd; maximum process capacity is 6.3 mgd.

The existing treatment plant on Roanoke Island was designed as a 5.0 mgd plant. However, with certain minor improvements discussed in a later section, this plant can produce 6.3 mgd. This represents a "stressed" condition, and operations at this level should be considered as a stop-gap measure until additional capacity can be constructed. Thus, Alternative II is really not a practical alternative since the Roanoke Island Plant cannot supply the 1985 demand to the existing service areas alone. Based on Alternative I, the Roanoke Island Plant should be expanded to 10.0 mgd by 1988 and to 15.0 mgd by 1997-99.

6.4.2. Water Treatment Plant Description

The treament plant has four ion-exchange softeners which are fed directly from the raw water transmission main and discharged into the ground storage reservoir. The softeners operate under pressure.

Each softener is designed for a flow of 875 gpm at 15 psi head loss. For three softener on-line, the plant treating capacity would be 2,265 gpm or 3.8 mgd. Four softeners on-line would result in a capacity of 5.0 mgd. Each softener contains 146 cu ft 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 gal before regeneration is required. This computes to a removal capacity of 18,000 grains per cu ft of resin, which is at the lowest end of Bruner's usual design.

Regeneration of the ion-exchange resin is accomplished by running a salt solution through a softener, thereby removing the "hardness" from the resin. As per the job service manual, each regeneration requires 730 pounds of salt, equivalent to 5 pounds of salt per cu ft of resin. Again, this is at the very lowest 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.

Blending is accomplished automatically 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 called for 25% 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.

6.4.3 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 considerable variety of opinion among ion-exchange equipment manufacturers and among consulting engineers of what are the most significant design parameters for ion-exchange process design. The treatment process was designed based on a volumetric loading rate of 6 gpm/cu ft of resin capacity, resulting in a maximum flow rate per softener of 875 gpm. This analysis places no significance on the surface loading rate which computes to 23 gpm/sq ft. This later 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, especially given the critical water treatment needs discussed in Section 6.4.1.

Using a surface loading rate of 10 gpm/sq ft as the limiting design parameter will provide a much more conservatively designed plant and will require more treatment capacity to be added. 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 the plant is expanded under Phase II; however, a detailed study should be made to determine the long-term capacity of the existing plant, based on the installed softener units, the actual raw water quality, and the desired finished water quality.

Two additional factors relate to the existing treatment plant capacity. First, the treatment level required is less than was assumed when the plant was designed. Raw water hardness is 175 ppm (200 ppm used in design), and the required finished water hardness is 80 ppm (50 ppm used in design). These new figures compute to a blending ration of 60 percent treated: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.

The second factor affecting plant capacity, is the desired finished water hardness. The current operation produces finished water with 80 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 since the current level of treatment produces finished water at the accepted optimum hardness. However, the County should be aware of the potential of producing more water at a lower quality should this be necessary.

The recommendations of this section can be summarized as follows.

1. The existing Roanoke Island treatment plant should be considered capable of producing 6.3 mgd of finished water for the next several years.
2. A study should be conducted prior to the Phase II expansion to more accurately determine existing plant capacity and the additional softener units required in order to develop the overall plant capacity to meet future water needs.

6.4.4 Existing Operational 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 first operated, are discussed below together with possible ways of alleviating them.

1. Plant Capacity. The water treatment plant supposedly has a capacity of 5.0 mgd. However, experience has shown that the maximum amount of water that the plant can treat, including designed bypass, is 3.0 to 3.5 mgd. Above this range, the softener plant has been shut down and all water routed through a bypass line directly to the storage reservoir. This deficiency is a result of the large amount of pressure that is required for the softening system to run at capacity. Pressure or head loss, is the limiting factor with regard to capacity.

The softeners themselves are adequately sized, as is the regeneration system, to produce 5.0 mgd.

A model of the hydraulic system from the storage reservoir to the farthest well was developed. Both computer and manual calculations were performed on this model under various flow conditions. These calculations predict how the system will function under several options that are available for plant operation.

The model results, summarized in Table 6.4-2, indicate that the raw water wells are not capable of pumping water at a high enough pressure to allow the plant to operate at design capacity. At the head required for design capacity, the wells could only deliver 2,570 gpm (3.7 mgd) of water. A plant flow of 3.5 mgd with its resulting pressure requirement can be supplied by the wells. This is approximately the balanced condition, and in fact corresponds with the operator's experience.

When the softeners are bypassed, as is now done in peak seasons, the model predicts that the wells can deliver 3,826 gpm 5.5 mgd, which is also verified by plant operators.

The problem of reduced plant capacity due to excessive head losses can be alleviated in one or two practical ways. One alternative is to make modifications to the plant and raw water system to reduce head loss and increase available pressure. This alternative, called Alternative A, will require the replacement of certain high head loss items in the plant with lower head loss items. These changes can be made relatively quickly and inexpensively. In addition to the plant changes, it will be necessary to add an additional transmission main to carry part of the raw water from existing wells. A new main would ultimately be required to deliver the flow from the new Wanchese area wells, so the construction of a portion of this main immediately will fit in well with future needs. Figure 6.4-1 shows these improvements.

The second way to alleviate the problem is to add a booster pump upstream of the softeners. This pump could also be installed relatively quickly. The existing electrical system at the plant is not capable of handling the extra load of the booster pump, and additions to the motor control center will be required. This pump can be worked in

TABLE 6.4-2

Total Finished Water (mgd)	Softener Flow (mgd)	Bypass Flow (mgd)	Well Capability (vs. Plant Pressure) gpm	mgd
5.0	3.8	1.2	2,570	3.7
3.5	2.6	.9	2,640	3.8
5.5	0.0	5.5	3,826	5.5

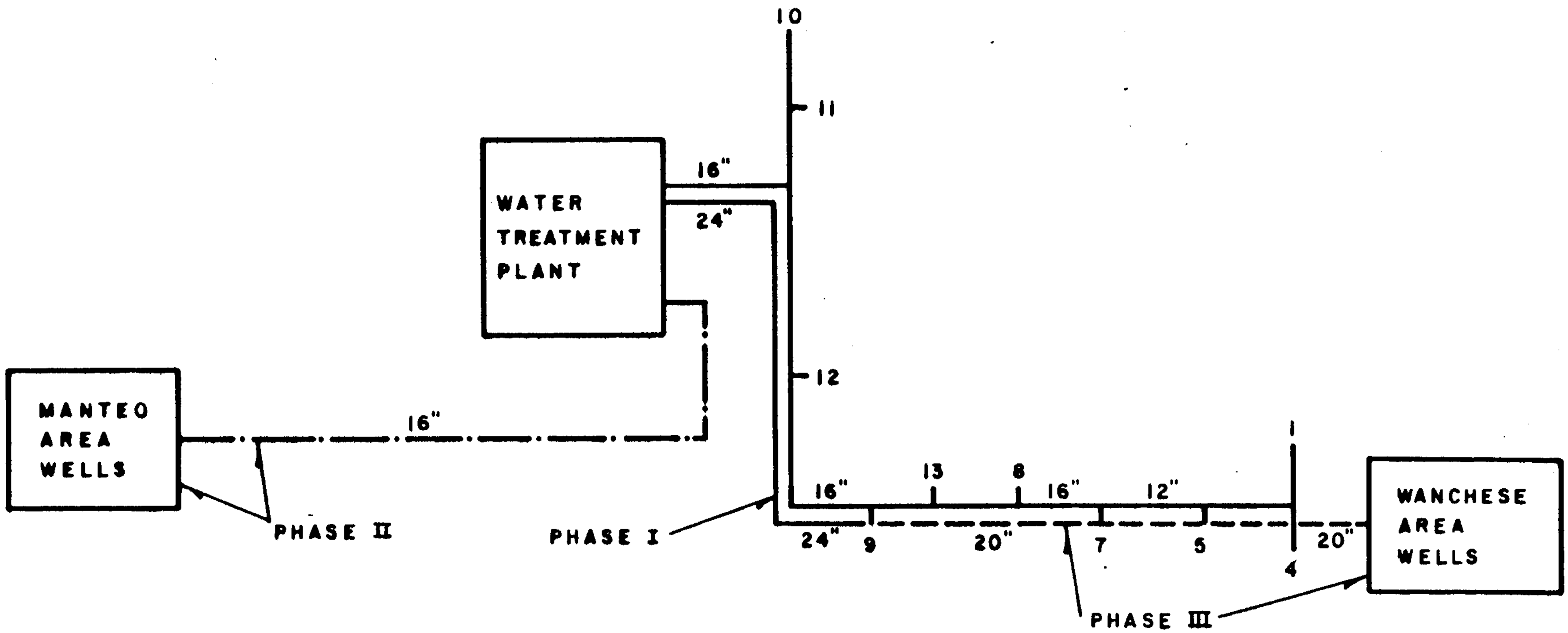


FIGURE 6.4-1 RAW WATER SYSTEM SCHEMATIC
ALTERNATIVE A

with future improvements, because future wells and raw water mains from Wanchese and Manteo can be designed to include booster pumping. Figure 6.4-2 shows the improvements of Alternative B.

The alternative to these improvements would be to downgrade plant capacity, say to 3.0 mgd. This is clearly an unacceptable alternative considering the peak demands expected during the next several years (the subject of plant capacity is discussed further in the section on water treatment plant expansion).

A life cycle analysis of the relative costs of the alternatives was performed and the results are shown in Table 6.4-3. Alternative B has the lower life cycle costs and is recommended for construction in Phase I.

2. Equipment Breakdowns. There are many pieces of plant equipment that are difficult to maintain in working order. Many of these break downs are minor in nature and add to O&M costs by the parts required and the repair time. Safety hazards are also increased by these break downs due to the water spillage that becomes unavoidable during operation and repair. These nuisance items should be permanently replaced when the plant is expanded.

   The break down of the softener meters is more than a nuisance. These meters are used to total the flow through each softener and to start automatic regeneration based on preset levels. When a meter breaks down, which occurs approximately 50% of the time, it is necessary for the operators to estimate when a softener should be regenerated based on other flow measurements. 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 will accurately indicate the need for regeneration.

3. Softener Throughput. Throughput 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

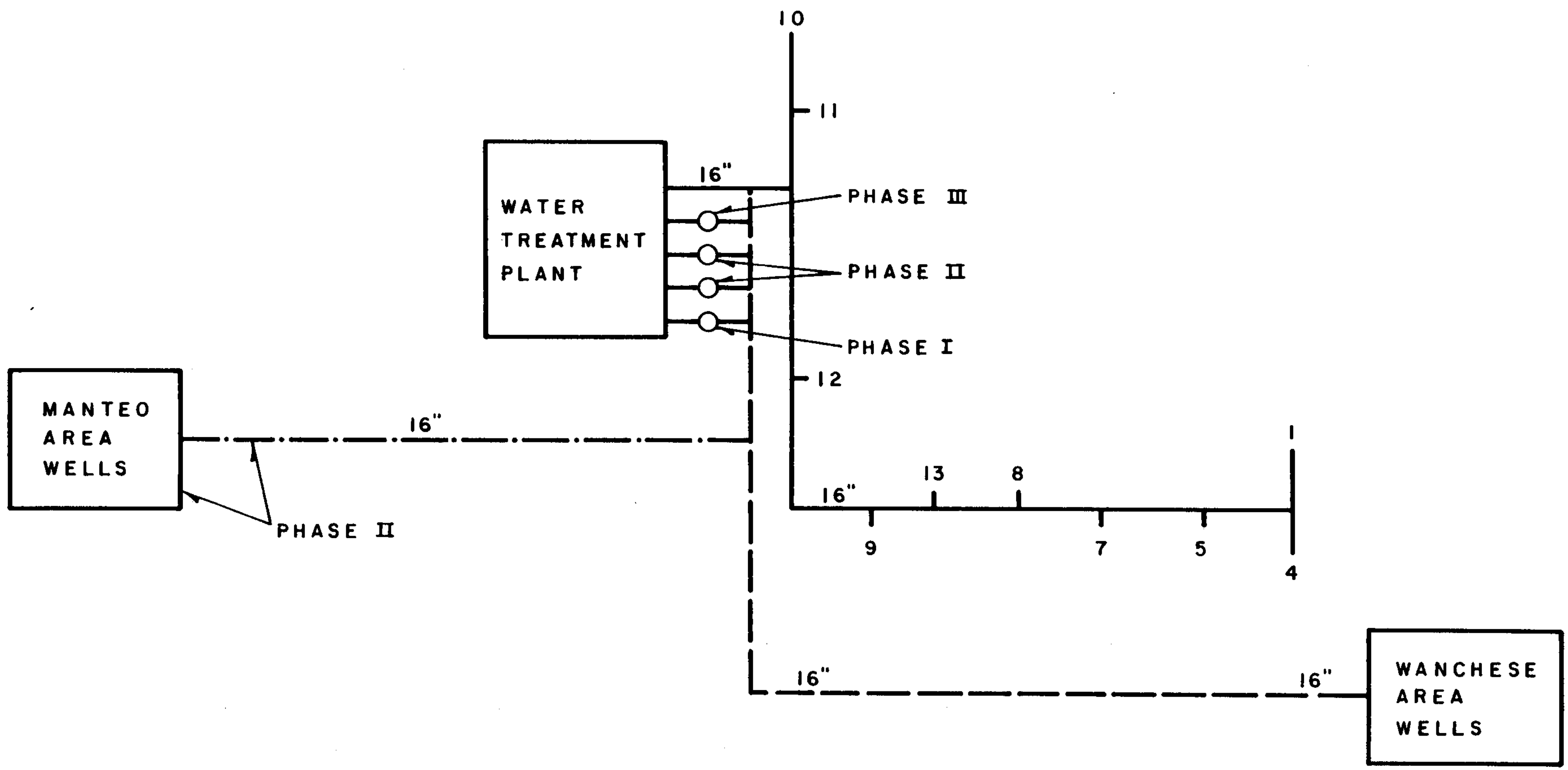


FIGURE 6.4-2 RAW WATER SYSTEM SCHEMATIC
ALTERNATIVE B

TABLE 6.4-3

LIFE CYCLE ANALYSIS OF PLANT CAPACITY IMPROVEMENTS

	Alt. A (Plant & Mains Modification)	Alt. B (Booster Pumping)
Capital Costs		
Phase I		
Treatment Plant	34,000	47,000
Raw Water	241,000	20,000
	275,000	67,000
Phase II		
Treatment Plant	1,115,000	1,179,000
Raw Water	995,000	995,000
	2,110,000	2,174,000
Phase III		
Treatment Plant	667,000	700,000
Raw Water	1,466,000	1,428,000
	2,133,000	2,128,000
Average Annual Excess Power Costs	---	7,200
Present Worth	2,277,749	2,167,715

is capable of a throughput of 200,000 gal. Operational experience has shown that lower throughputs than this are actually achieved, some times as low as 100,000 gal. This reduction in through out can be caused by resin damage, excessive hardness leakage, imcomplete 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 throughput.

Permanent resin damage can occur due to the presence of a number of substances in the water. Replacement of the resin is required to correct this deficiency. Excessive hardness leakage occurs when the 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 latter 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 this correct dosage rate.

The softeners are presently regenerated at a relatively low salt dosage, that is the most economical in terms of operating costs. However, at this low salt dosage, it is possible for regeneration to be incomplete, which inturn 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.

Due to the hydraulic problem mentioned in item 1 above, the softeners have never been run to capacity. However, an analysis of the softener system indicates that the existing automatic controls will effectively limit plant capacity to 3.0 mgd. The controls in question are those which regulate the regeneration process and the blending rate. This limitation has not been evident due to the hydraulic limitations discussed in item 1 and the extensive manual operation practiced due to the meter break down mentioned in item 2. Manual operation will alleviate this problem. It is recommended that the controls be changed to provide automatic operating capabilities that are consistent with treatment plant capacity.

The problem of reduced throughput can most likely be corrected by operating changes particularly in the regeneration process and by the changing of controls. Any additional improvements can only be determined after further experience is gained following the suggested changes.

4. Chlorination System. Difficulty in maintaining proper chlorine residuals is experienced in peak demand periods. This is due to the three different chlorine feed rates and residuals required.

    1. Chlorination of finished water before storage - flow = 2.0 to 5.5 mgd, residual = 0.5 mg/1.
    2. Pumping discharge to Outer Banks - flow = 5.8 mgd, residual 0.5 mg/1
    3. Pumping discharge to Roanoke Island - flow = 0.4 mgd, residual = 20. mg/1

Both 100 pound per day chlorinators are used during peak periods but the piping and feed regulating system do not allow for correct feeding to these different points. This problem can be corrected by piping and control improvments. However, the chlorine feed system will require expansion when treatment plant capacity is increased in the Phase II improvements. Any improvements made in Phase I may not be applicable later. It is recommended that improvements to the chlorine feed system be deferred until Phase II and that the operators cope with the present system to the best of their abilities.

5. Pumping Difficulties. During peak demand season, vortexing occurs in the storage reservoir when the water level drops to approximately one-half full. This vortexing results in excessive wear on the pumps. At present, the high service pumps supplying the Outer Banks are shut down when vortexing occurs until the reservoir can be refilled. Half of the storage reservoir volume cannot be used in emergencies or during fires.

Vortexing is normally caused by improper pump suction conditions and can be controlled by a redesign of the suction or by baffling in the suction well. Such baffling is only a corrective measure, and its design is based primarily on judgment rather than any established rules. It is recommended that baffling be installed in Phase I, and that the effectiveness of this baffling be reviewed and revised as necessary in Phase II.

6. Electrical. Irregular wiring practices have caused operational difficulties in the regular tripping of certain circuit breakers and the 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. Since the plant electrical system will require major revisions and expansion under the Phase II treatment plant expansion, it is recommended that these improvements to the existing system also be made in Phase II.

7. Telemetry. The existing telemetry system has never worked consistently to allow the overall water system to operate efficiently. The existing equipment also 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 Phase II improvements.

7.0 RECOMMENDED SYSTEM IMPROVEMENTS

Growth and development of Dare County has created the need for the County to expand its water system. The water system was completed in 1980, and after only three years the system needs major improvements to keep up with the growing water demands. Water projections indicate demands are expected to double within the next ten years. Also, immediate improvements are required to insure that an adequate supply will be available to meet the growing demands by 1985.

It is recommended that the County phase the improvements over a ten to fifteen year period, eliminating the large up front capital cost which prolong project construction. The proposed phases of improvements are summarized below.

7.1 PHASE I

The initial phase (1984-1985) of development insures that the system will meet current demands and carry the system until major improvements are made. These initial improvements consist of increasing the treatment capacity, providing additional storage at the northern beaches (Kitty Hawk area) and installing observation wells on Roanoke Island. In increasing the treatment capacity, the County is faced with two alternatives. Alternative I is to modify the existing treatment plant by changing some of the components to treat 7.2 mgd, which is the production capacity of the wells. Alternative II is restore and upgrade one of the Town's (Nags Head or Kill Devil Hills) treatment plants at the Fresh Pond, which will provide a supplemental supply of 1.5 mgd.

From an engineering standpoint, the most cost effective alternative would be utilizing the supplemental supply at the Fresh Pond; however, the actual decision will be based on negotiations between the County and the Towns.

The supplemental supply from the Fresh Pond would also enable the County to deliver an additional 1.5 mgd to the northern beaches where problems were experienced in delivering water during the peak season of 1983.

Modifications of the existing treatment plant initially would increase the capacity to 7.2 mgd enabling the County to deliver 7 mgd to the beaches. However, the County will still experience problems delivering water to the northern beaches. In addition to the treatment plant modification, the County would need to install a booster pump station along the transmission main in the first phase. Utilizing the Fresh Pond would delay the construction of the booster pump station until the second phase (1985-1990).

Both alternatives are designed to solve the immediate problems, which is to meet the demands for the next several years. However, the County may select both alternatives initially, which would boost the treatment capacity to approximately 8.7 mgd. This would supply the demands until approximately 1988.

It is important that the County maintain adequate storage on the northern beaches (Kitty Hawk area). Due to the distance away from the water supply, this area would have less flow than any other of the take out points. During periods of peak demand, the flow to the area (Kitty Hawk) may be less than the required demand which may create a water shortage if adequate storage is not available to supplement the peak demand periods. Currently, 1.0 million gallons of storage is available. Water projections indicate that the demands will reach 1.9 mgd in 1985. It is recommended that an additional 1.0 million gallons of ground storage be provided initially.

As outlined in the water supply section of this report, in order to properly monitor the affects of large scale pumping of the groundwater supply on Roanoke Island, it is recommended that the County install additional observation wells that will be used to detect adverse migration of saltwater to the well field. These wells are needed now in order to insure high quality water.

Table 7.1-1 is an outline of the proposed phase improvements. Figure 6.1-1 illustrates the improvements.

TABLE 7.1-1

PHASE I

Water System Improvements

1. Modify existing water treatment plant.
2. Restore and upgrade water treatment plant at Fresh Pond.
3. Install 1.0 million gallon ground storage tank at Kitty Hawk.
4. Install 6 observation wells.
5. Install booster pump station on transmission main, depending on item 2 above.

7.2 PHASE II

Water projections indicate that water demands are expected to be approaching 9.5 mgd in 1990. This will require a major expansion of the treatment plant.

The second phase (1985-1990) of impoundments consists of expanding the existing water treatment plant to 10 mgd, expanding the distribution system on Roanoke Island to include the new service areas, and installing a booster pump station along the transmission main if it is not installed in the initial phase.

7.3 RAW WATER SYSTEM

As indicated in Section 6.1 above, the existing raw water systm can produce 7.2 mgd if modifications are made to reduce the TDH. This is best accomplished by adding a booster pump ahead of the water softeners in the treatment plant. This item should be included in the Phase I development scheme so that, with modifications to other parts of the water system, demands can be met for at least a few years before major additions to the entire system is required. Costs for these improvements are included in the treatment plant cost estimates for Phase I (Table 7.1-2).

In order to properly monitor the affects of long range pumping on the aquifer system, groundwater monitoring wells should be installed. These wells should be multicluster wells with screens in the upper aquifer, the aquitard, and the principal aquifer. These wells will allow monitoring of lateral movement of nonpotable water from the sounds toward the well field so that pumping schemes can be modified to prevent saltwater encroachment. A continuous water level recorder should be installed at each well cluster to monitor water levels in the principal aquifer.

Long range development will include drilling additional wells as the water demand increases. New well development should be based on peak demand plus two wells. This means that enough wells will be in the ground to meet the peak demand with two wells as standby. This will allow for cycling of wells so that no well has to operate continuously for several days. It will also provide for reserve wells that could be used in an emergency or when other wells are being serviced. A program of adding two wells every two years during the period 1985-1990 will add the required six wells needed to meet the 1990 projected demand of +10 mgd. By the end of the planning period, a total of 22 wells will be required to produce the 15 mgd peak. The groundwater development program should be based on updated water demand information so that the raw water system progressively tracks the growing water demand.

As the well field is expanded south or north, additional monitoring well clusters will be required. These installations should be located as the well field influences growth.

Cost opinions for the Raw Water System Improvements are shown in Table 7.1-2.

7.4 TRANSMISSION SYSTEM

Based upon the analysis of Section 6.3, the following are recommendations for transmission system improvements.

Phase I

No improvements are required

Phase II

Install a booster pumping station at Whalebone Junction

TABLE 7.1-2

RAW WATER SYSTEM IMPROVEMENTS

Preliminary Opinion of Probable Project Cost

PHASE I (Immediate)

Booster Pump ahead of Water Softeners	1	LS	22,700	$ 22,700
Observation Wells	2	LS	10,000	20,000
				$ 44,700

PHASE II (1985 - 1990)- Manteo

Item	Quantity	Unit	Unit Cost	Total Cost
16" Water Main	5,600	lin ft	32.00	$179,200
12" Water Main	3,600	lin ft	24.00	86,400
10" Water Main	3,200	lin ft	20.00	64,000
8" Water Main	4,600	lin ft	16.00	73,600
6" Water Main	3,200	lin ft	12.00	38,400
Wells & Pumphouses	6	LS	60,000	360,000
Construction Cost				$ 801,600
Legal & Administrative				8,000
Engineering Design				55,000
Construction Observation				30,000
Contingencies				80,400
Project Cost				$ 975,000

PHASE III (1990 - 1995)

Item	Quantity	Unit	Unit Cost	Total Cost
12" Water Main	3,600	lin ft	24.00	$ 86,400
10" Water Main	3,000	lin ft	20.00	60,000
8" Water Main	11,800	lin ft	16.00	188,800
Wells & Pumphouses	6	LS	60,000	360,000
20" Water Main	10,960	LS	40.00	438,400
Construction Cost				$1,133,600
Legal & Administrative				11,000
Engineering Design				70,000
Construction Observation				39,000
Contingencies				113,400
Project Cost				$1,367,000

Phase III

1. Install a booster pumping station on Roanoke Island.
2. Expand the booster pumping station at Whalebone Junction.
3. Install a booster pumping station at Kill Devil Hills.

These recommended improvements are discussed in detail in the sections which follow.

7.4.1 Phase I

No improvements are anticipated to meet immediate needs. This statement is based on two assumptions.

1. It is assumed that the Fresh Pond water treatment plant is improved and brought on-line before the 1985 peak season. As previously discussed, this must be done in order for treatment capacity to equal projected water demands.

2. It is assumed that Nags Head will receive the majority of their water at the storage reservoir now under construction.

It either of these assumptions are not met, it will be necessary to construct the Whalebone Junction booster pumping station, intended for Phase II, during Phase I. The station would need to be on-line before the 1985 peak season, and may actually be needed during the 1984 peak season.

7.4.2 Phase II Improvements

A booster pump station should be constructed in the Whalebone Junction vicinity. The purpose of this pumping station is to increase pressure in the 24 inch main, allowing more water to be delivered. The station will be utilized during maximum day conditions. Average day conditions will require no booster pumping. The number of pumps in the station and their size should be carefully matched to treatment plant capacity, water demands, and future expansion needs. It is recommended that three pumps be installed now with room provided for a pump to be added during Phase III. These pumps should be 3,350 gpm at 170 ft tdh. During the station design, it is possible that two larger pumps in lieu of the three recommended above will be installed. This may reduce costs, but may also reduce operating flexibility.

7.4.3 Phase III Improvements

1. A booster pumping station should be installed on Roanoke Island, on an adequate site as close to Roanoke Sound as possible. Installing this station during Phase III will allow lower head high service to be installed during Phase II, thereby reducing pressures in the main under Roanoke Sound. The pumping capacity should be 15 mgd (the water treatment plant capacity) at 60 ft tdh. The number of pumps will be determined during booster station design and should be consistent with other elements of the pumping system.

2. The booster pumping station at Whalebone Junction should be expanded (add one pump) to meet the larger water demands. The station should require no other modifications.

3. A booster pumping station should be constructed at Kill Devil Hills just after the point where the reservoirs receive water from the transmission main. This station will serve Kitty Hawk and points north only. The station should contain two pumps, each capable of pumping 2,900 gpm (4.2 mgd) at 190 ft tdh.

Table 7.4-1 is a cost opinion for Transmission System Improvements.

7.5 WATER TREATMENT SYSTEM

Based upon the analysis of Section 6.4, the following recommendations for treatment plant expansion are made. Unless specifically stated otherwise, the improvements apply to the Roanoke Island plant.

Phase I.

1. Provide a booster pump upstream of the water treatment plant, and modify plant controls.
2. Construct vortex inhibiting baffles in the storage reservoirs.
3. Bring Fresh Pond treatment plant on-line.

TABLE 7.4-1

TRANSMISSION SYSTEM IMPROVEMENTS

Phase I - Under Alternate I -- none required		
Phase II -	Booster Pump Station (Whalebone Junction)	$180,000
Phase III -	Booster Pump Station (Whalebone Junction)	40,000
	Booster Pump Station (Roanoke Island)	190,000
Total		$410,000

Phase II.

1. Expand plant capacity to 10.0 mgd.
2. Provide additional booster pump capacity.
3. Provide additional high service pump capacity.
4. Replace faulty equipment in existing plant.

Phase III.

1. Expand plant capacity to 15 mgd.
2. Provide additional booster pump capacity.
3. Provide additional high service pump capacity.

These recommended improvements are discussed in detail in the sections which follow.

7.5.1 Treatment Process Options.

The existing Roanoke Island treatment plant is an ion-exchange softening plant. Such plants are relatively common throughout the USA in sizes below 3.0 mgd. The existing plant is a relatively large ion-exchange plant. A 15 mgd ion-exchange plant would be almost without precedent. The reason for this is that life cycle costs generally favor other processes for softening at higher flow rates. Ion-exchange softening is a proven process and, when properly designed, specified, and constructed, can satisfactorily treat any flow rate.

The alternative to ion-exchange softening is lime softening. Lime softening is very common at large sizes, and the design parameters are well known and easily applied to most waters. The equipment and operating techniques for lime softening are also well proven, and the process is generally insensitive to substances in water that would adversely affect ion-exhange (such as chlorine and trivalent iron).

The factors that affect the cost effectiveness of lime versus ion-exchange softening are numerous. Each water plant has to be evaluated based on its specific conditons. For the Roanoke Island plant, these conditions intuitively favor ion-exchange. The major reasons for this are as follows.

1. The existing softening capacity can be best utilized with addition of similar equipment.
2. New softening units can be installed in the existing building by building a new pump room. This new pump room is desirable for reasons discussed later.
3. Brine wastes can be discharged without treatment, whereas lime sludge will require treatment.
4. Running a 10 mgd lime plant next to a 5 mgd ion-exhange plant will present great operational difficulties, especially in shared components such as the raw water system and the storage reservoir. Low service pumping would be required for the lime plant but not for ion-exchange. Spare parts requirements would be multiplied.
5. The relatively low treatment level (175 ppm to 80 ppm) allows significant blending which favors ion-exchange.
6. Operators are already familiar with the ion-exchange equipment.
7. The treatment level can be easily altered with the ion-exchange process, allowing greater flexibility in meeting peak demands.
8. Land requirements are less for ion-exchange.

All of these items point to ion-exchange softening as the more desirable treatment process. The following disadvantages of ion-exchange are mentioned to provide a more complete description of the process comparison.

1. Ion-exchange replaces hardness in raw water with sodium. Sodium presents an increased risk of heart disease when consumed in sufficient amounts.
2. Power costs will be greater with ion-exchange due to the pressure required to operate the softeners.

It is recommended that ion-exchange be the process of the expanded plant. It is also recommended that a more detailed economic analysis based upon actual raw water quality and process design be made prior to the design effort to quantitatively confirm the above recommendation. This analysis should include the ion-exchange design parameters study mentioned in Section 6.4.3.

7.5.2 Phase I - Improvements

Immediate improvements are required to allow the plant to function at the capacity recommended in Section 6.3.3. A booster pump with associated piping is required to increase the pressure available to the softeners. Piping should be arranged to bypass the booster pump during average day conditions, and should also allow future booster pumps to be installed in parallel. The pumps will be vertical turbine can pumps, installed outdoors, that will provide a 30 ft boost in pressure to the influent raw water.

The softener automatic regeneration controls should be replaced to allow immediate operation of a softener upon completion of regeneration. This will consist of changing out the control panel. Also, to achieve a 60:40 blending ratio, the automatic blending system should be abandoned, and blending should be done manually.

Baffles should be installed in the 2.0 mil gal storage reservoir to eliminate vortexing. These will be either vertical or horizontal baffles, or some combination, placed near the suction pipe. The exact location of these baffles will be determined during design. This will be an inexpensive improvement.

These improvements are all relatively inexpensive, but will allow the plants to operate at a higher than design capacity for a few years.

7.5.3 Phase II - Improvements

The plant capacity should be increased to 10.0 mgd by 1988. The following improvements are recommended to accomplish this.

1. High Service Pumping. Due to the problems with pump suction and the difficulty of expanding pumping capacity in the existing pump room, it is recommended that a new high service pumping room be constructed in a more suitable location. Since the larger plant will require more facilities for operators, it is recommended that the building housing the new pumps also include office space, lavatories and showers, and chlorine feed and storage areas. It is estimated that the new building will be 2,100 sq ft. The pump floor will be below grade to improve suction characteristics. Pump room size, suction piping, and discharge piping, will be sized to allow for a maximum pumping rate of 18,750 gpm.

Initially, three pumps at 3,750 gpm each will be installed to provide a firm pumping capacity of 10.8 mgd. Future pumping units will be installed in the next plant expansion or if the storage reservoir capacity is required to satisfy a peak demand. By sizing the initial facility, maximum flexibility for future expansion is assured.

2. Chlorine Facilities. The new pump building should include expanded chlorination facilities. The exact size of the feeders and storage area will be determined during design. Maximum use of the existing chlorine facility will be evaluated. However, based upon present dosage rates required to maintain residuals, the existing chlorine feeders will be inadequate to meet future demand for chlorination of the softeners effluent above. It may be necessary for an entirely new chlorine feed system to be provided.

3. Softening Capacity Expansion. The existing pump room will be converted to space for new softening units. The suitability of this area for all future softening units depends upon the selected design parameter. If the volumetric loading rate is used, the area will be sufficient to house the units for both the Phase II and Phase III expansions. If the more conservative surface loading rate is used, the area will be sufficient for Phase II expansion only. An additional 1,400 sq ft of building area will be required for Phase II if the surface loading limitations apply (see Section 6.4.3 for a discussion of these different loading rates). For purposes of cost projections, it is assumed that the needed treatment units will fit in the existing area.

Expansion of the plant under Phase II will require a new salt regeneration system equal in size and capacity to the present system. A new discharge line to the storage reservoir will also be required, as will interconnecting, regeneration and waste piping. Expansion of the chlorination facility has been discussed previously.

4. Booster Pumping. It is recommended that two additional booster pumps be added in Phase II. This will provide a firm capacity of 10.1 mgd. The need for booster pumping is actually tied to the softener surface loading rate. If this rate remains as originally designed, then additional booster pumping will be required. If the surface loading rate is reduced for process considerations, the pressure required to operate the plant will be reduced and booster pumping may not be needed. This recommendation needs to be carefully reviewed and revised as required when the plant expansion is being designed.

5. Plant Electrical System. The plant electrical system was not designed to provide for future expansion. Therefore, it will have to be expanded or mostly replaced in the Phase II expansion. New primary transformers will be required, as will new main feeder lines, switches, motor control center, secondary panels, etc. The electrical system installed in Phase II should include sufficient expansion capacity and provide 20% spare capacity beyond this. Also included in Phase II expansion is correction of wiring deficiencies in the existing plant.

6. Telemetry. The decision on replacing the telemtry system should be based on cost effectiveness and reliability. It is anticipated that studies presently underway will confirm the viability of replacing the telemetry system. The cost of this is included in the Phase II improvements.

7.5.4 Phase III Improvement

The maximum day demand will exceed the treatment plant capacity by 1999. Therefore, expansion of the treatment capacity to 15.0 mgd by 1998 is required. The following improvements are recommended to accomplish this.

1. High Service Pumping. One additional 3,750 gpm pump should be added to provide a firm pumping capacity at least equal to plant capacity. This will be added to the pump room. At this flow rate, the capacity

of the transmission line from Roanoke Island to the Outer Banks will be exceeded. It will be necessary to add another booster station on the existing main or to add a parallel transmission main. This is discussed in Section 7.1.

2. Chlorine Facilities. Additional chlorine feed capacity will be required for the increased plant flow. It should be necessary only to add a chlorine feeder and some additional piping.

3. Softening Capacity Expansion. Additional softeners will be required to bring the plant capacity up to 15.0 mgd. As in the Phase II expansion, the required treatment units will fit in existing building space if the volumetric design parameter is used for design. For the purpose of cost projections, it is assumed that the required additional treatment units can be installed in the existing area.

   Additional regeneration system capacity, interconnecting piping, a 3,500 gpm booster pump, electrical system expansion, and telemetry system expansion will also be required. The need for booster pumping must be confirmed, as stated in Section 7.4.2.

4. Finished Water Storage. The original plant layout included room for adding two 2-mil gal finished water reservoirs. Based on projected water demands, it appears that these reservoirs will not be needed through the study period. During the Phase III design, the storage needs should be reviewed and this recommendation either confirmed or adjusted as required.

Cost opinions of Treatment System Improvements are given in Table 7.5-1.

7.6 DISTRIBUTION SYSTEM

Improvements to the distribution system include adding service areas in Manteo and Wanchese plus 2.5 million gallons of storage at Kitty Hawk.

TABLE 7.5-1

TREATMENT SYSTEM IMPROVEMENTS

PHASE I	
Restore & Upgrade Fresh Pond WTP	$ 500,000
Modify Roanoke Island WTP	47,000
	$ 547,000
PHASE II	
Expand Existing WTP to 10 mgd	1,179,000
PHASE III	
Expand WTP to 15 mgd	700,000
Total	$2,426,000

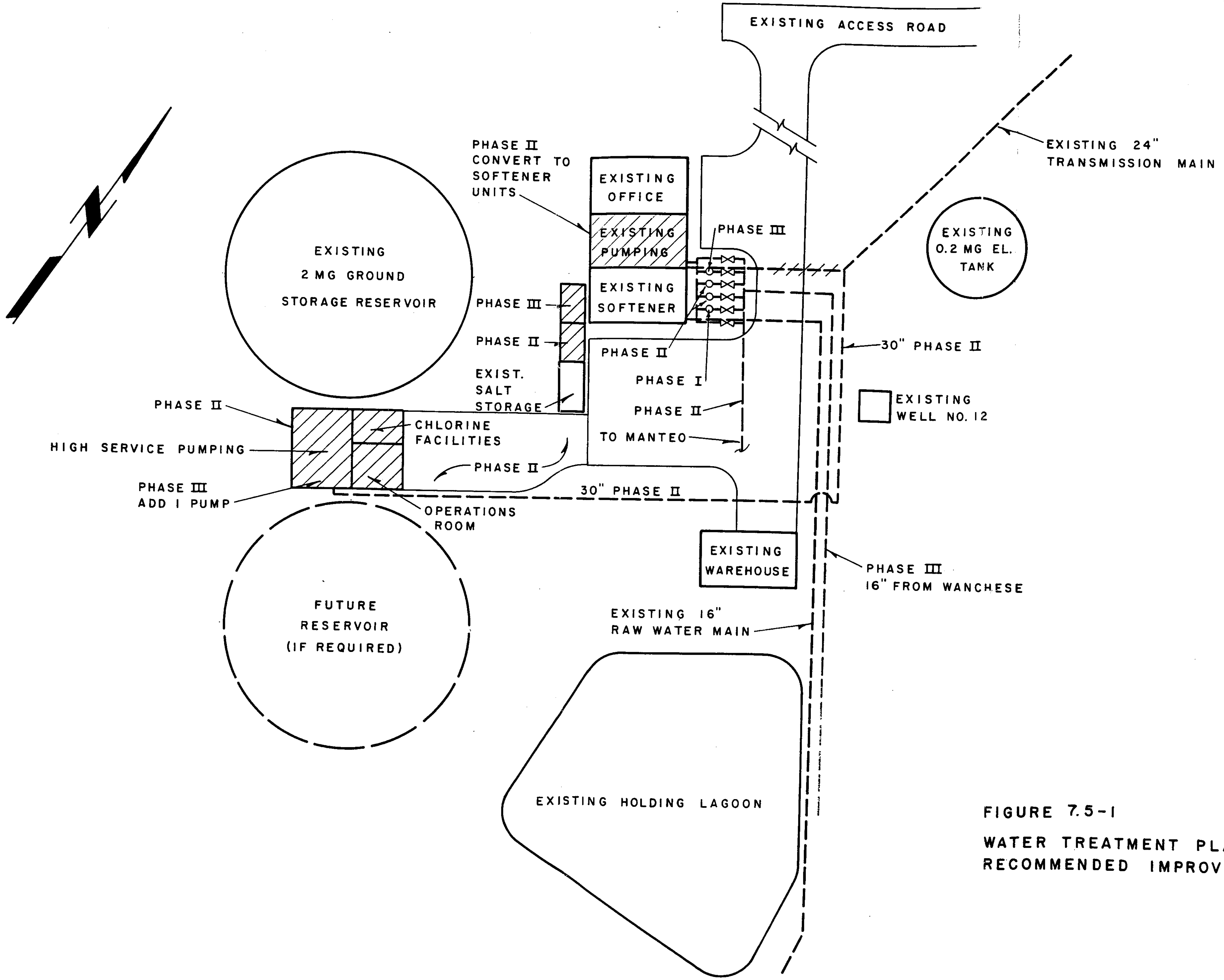


FIGURE 7.5-1

WATER TREATMENT PLANT
RECOMMENDED IMPROVEMENTS

By adding the service areas at Manteo and Wanchese, adverse effects on domestic wells from pumping the County well field will be eliminated. This will add additional customers to the system on Roanoke Island near the water plant, which currently is providing water to the area with minimum transmission costs. Wanchese would be served during Phase I and Manteo during Phase II.

The additional 2.5 million gallon storage at Kitty Hawk will provide for one day storage in the area. This will help eliminate peak production demand problems and provide an emergency supply should a break in the transmission line across Roanoke Sound occur. The increased storage capacity will be added incrementally to reduce costs. Initially in Phase I, a 1 million gallon ground storage tank will be added. A .5 million gallon elevated storage tank will be constructed in Phase II and another 1 million gallon ground storage tank will be built in Phase III.

Cost opinions for the Distribution System Improvements are provided in Table 7.6-1.

7.7 MASTER DEVELOPMENT PLAN

A Master Development Plan for the total system improvements is shown in Table 7.7-1. This table provides a description of the improvements, their development phase and a cost opinion for development. This master plan should serve as a valuable planning tool for the County in developing its water system over the next 20 years.

TABLE 7.6-1

ROANOKE ISLAND (WANCHESE)

Preliminary Opinion of Probable Project Cost

PHASE I

Item	Quantity	Unit	Unit Cost	Total Cost
8" Water Main	25,000	lin ft	$ 7.00	$175,000
6" Water Distribution Line	36,000	lin ft	4.50	162,000
4" Water Distribution Line	15,000	lin ft	3.50	52,500
2" Water Distribution Line	8,000	lin ft	3.00	24,000
Service Connections	540		225.00	121,500
8" Valves & Boxes	12	each	400.00	5,000
6" Valves & Boxes	40	each	300.00	12,000
4" Valves & Boxes	20	each	250.00	5,000
2" Valves & Boxes	15	each	200.00	3,000
Fire Hydrants	60	each	800.00	48,000
Fittings, Concrete Encasement, Pavement Repair, Stone		LS	LS	50,000
Contingency		LS	LS	66,000
Net Construction Cost				$ 724,000
Engineering Design Fee, Surveying and Construction Observation				86,000
Legal & Administrative				10,000
Land & Rights-of-way				10,000
Total Probable Project Cost				$830,000

TABLE 7.6-1
(Continued)

ROANOKE ISLAND (MANTEO AREA)

Preliminary Opinion of Probable Project Cost

PHASE II

Item	Quantity	Unit	Unit Cost	Total Cost
8" Water Main	13,000	lin ft	$ 7.00	$ 91,000
6" Water Main	29,200	lin ft	4.50	131,400
4" Water Main	9,200	lin ft	3.50	32,000
2" Water Main	7,400	lin ft	3.00	22,200
8" Valves & Boxes	6	each	400.00	2,400
6" Valves & Boxes	42	each	300.00	12,600
4" Valves & Boxes	6	each	250.00	1,500
2" Valves & Boxes	8	each	200.00	1,600
Fire Hydrants	30	each	600.00	18,000
Service Connection	440	each	225.00	99,000
Fittings, Concrete Encasement,		LS	LS	25,000
0.24 mil gal Elevated Tank				225,000
Booster Pump Station		LS	LS	25,000
Subtotal				$ 686,700
Congingency				68,100
Legal & Administrative				7,000
Engineering Design				46,000
Construction Observation				25,000
Land & Rights-of-way				10,000
Total Probable Project Cost				$ 842,800

TABLE 7.6-1
(Continued)

WATER DISTRIBUTION SYSTEM
DARE BEACHES (KITTY HAWK)

Preliminary Opinion of Probable Project Costs

PHASE I:

Item	Quantity	Unit	Unit Cost	Total Cost
1 mil gal Ground Storage Tank	1	LS	--	$225,000
Tank Foundation	1	LS	--	25,000
Subtotal				$250,000
Contingencies				25,000
Construction Cost				$275,000
Legal & Administrative				2,000
Engineering Design				20,000
Construction Observation				15,000
Total Probable Project Cost				$312,000

PHASE II:

Item	Quantity	Unit	Unit Cost	Total Cost
0.5 mil gal Elevated Tank	1	LS	--	$325,000
Tank Foundation	1	LS		50,000
Booster Pump Station		LS		50,000
Contingencies				25,000
				$450,000
Legal & Administrative				4,000
Engineering Design				33,000
Construction Observation				20,000
Total Probable Project Cost				$507,000

PHASE III

Same as I above.

TABLE 7.7-1

WATER SYSTEM IMPROVEMENTS
MASTER DEVELOPMENT PLAN

Preliminary Opinion of Probable Project Cost

PHASE I (1984-1985)

Improvements	Alternative I	Alternative II
A. Distribution System:		
1. Ground Storage Tank (Kitty Hawk)	$ 312,000	$ 312,000
2. Water Main Extension (Wanchese)	810,000	810,000
Subtotal	$1,122,000	$1,122,000
B. Transmission System:		
1. Booster Pump Station (Whalebone Junction)	---	$ 180,000
C. Water Treatment System:		
1. Restore & Upgrade WTP (Fresh Pond)	$ 500,000	---
2. Modification of Existing WTP	47,000	47,000
Subtotal	$ 547,000	$ 47,000
D. Raw Water System:		
1. Observation Wells(2)	$ 20,000	$ 20,000
Phase I Probable Project Cost	$1,689,000	$1,369,000

TABLE 7.7-1
(continued)

Preliminary Opinion of Probable Project Cost

PHASE II (1985-1990)

Improvements	Alternative I	Alternative II
A. Distribution System:		
1. Elevated Tank & Booster Pump (Duck)	$ 505,000	$ 505,000
2. Water Main Extension (Manteo Area)	843,000	843,000
Subtotal	$1,348,000	$1,348,000
B. Transmission System:		
1. Booster Pump Station (Whalebone Junction)	$ 180,000	---
C. Water Treatment System:		
1. Existing WTP Expansion (10 mgd)	$1,179,000	$1,179,000
2. Upgrade WTP (Fresh Pond)		500,000
Subtotal	$1,179,000	$1,679,000
D. Raw Water System:		
1. Observation Wells (2)	$ 20,000	$ 20,000
2. Wells & Pumphouses & Mains	975,000	975,000
Total Probable Project Cost	$3,702,000	$4,022,000

TABLE 7.7-1
(Continued)

Preliminary Opinion of Probable Project Cost

PHASE III (1990-1995)

Improvements

A. Distribution System:	
1. Ground Storage Tank (Duck)	$ 312,000
2. Booster Pump Station (Kill Devil Hills)	80,000
Subtotal	$ 392,000
B. Transmission System:	
1. Booster Pump Station (Whalebone Junction)	$ 40,000
2. Booster Station (Roanoke Island)	190,000
Subtotal	$ 230,000
C. Water Treatment System:	
1. Existing WTP Expansion (15 mgd)	$ 700,000
D. Raw Water System:	
1. Observation Wells(2)	$ 20,000
2. Wells & Pumphouses & Mains	$1,408,000
Subtotal	$1,428,000
Total Probable Project Cost	$2,750,000

8.0 WATER RATE STRUCTURE

The water rate structure presented in this section is intended to illustrate a fair methodology for allocating capital, operation, and maintenance expenditures to wholesale and retail customers of the County water system. Capital expenditures are recommended to be recovered using lump sum or periodic fixed payments based on peak water consumption (retail customers) or an amount of reserved capacity in the water treatment facilities (wholesale customers). Operation and maintenance expenditures are recommended to be recovered using a flat charge based on actual water consumption.

8.1 OPERATION AND MAINTENANCE EXPENDITURES

Rates for recovering operation and maintenance expenditures are estimated for the near future only. The methodology should be used to evaluate rates at least annually.

8.1.1 Wholesale Customers

Wholesale customer (Nags Head, Kill Devil Hills, Manteo) rates are based on the operation and maintenance expenditures for the raw water system, water treatment facilities, and finished water transmission lines. Expenditures for these items in fiscal year 1983-84, projected using actual expenditures for the first seven and one-half months, are estimated at $400,000. The estimated water production during the fiscal year is 750,000,000 gallons. Based on this and the fact that production cost at the Fresh Pond WTP will be the same or slightly higher, the current wholesale rate of $0.54/1,000 gallons is considered fair and equitable. Due to projected water demands, it also is likely to be adequate (except for increases due to inflation) for the foreseeable future.

8.1.2 Retail Customers

Retail customers are expected to pay for the remaining operation and maintenance expenditures for the treatment facilities and distribution system. The rate is determined in the following manner.

Revenue from wholesale customers is estimated using water sold in 1983 and is subtracted from total expenditures to obtain the costs to be allocated to retail customers.

Nags Head (226,000,000 gallons/year)	$122,040
Kill Devil Hills (258,000,000 gallons/year)	139,320
Manteo (42,000,000 gallons/year)	22,680
Estimated Revenue from Wholesale Customers	$284,040
County Share of O&M costs for Raw Water System, Water Treatment Plant, Transmission Lines ($400,000 - $252,700)	$115,968
County Distribution System O&M Costs	160,000
O&M to be Recovered from Retail Customers	$275,968

With an estimated amount of water sold to retail customers of 200,000,000 (line losses have been considered) gallons per year, $1.38/1,000 gallons is necessary to recover the $275,968. Again, not counting inflation, this rate is adequate for the foreseeable future.

8.2 CAPITAL EXPENDITURES

Capital expenditures are to be recovered using fixed payments either in the form of contractual amounts from wholesale customers or quarterly minimum payments from retail customers. The methodology should be reviewed at least every year to assure that the rates are adequate to recover debt service payments, capital outlay, and to maintain a capital reserve for future improvements.

8.2.1 Wholesale Customers

It is anticipated that wholesale customers will make lump sum or annual payments for their share of capital expenditures on the raw water system, treatment facilities, and transmission line and booster pumps. Manteo is not making annual payments at this time for capital expenditures. Since Manteo's reserve capacity in the plant is not projected to exceed

0.2 mgd, and since the allowable capital expenditures are for improvements to the plant and the transmission system to the beaches, no annual payments are projected for the Town of Manteo.

Table 8.2-1 is used to estimate the future payments for Nags Head and Kill Devil Hills for capital expenditures. The figures in the table are based on projected required capacities for each Town in the water treatment plants. Allocated costs in Table 8.2-2 through 8.2-4 are based, in part, on the projected required capacities. For example, the cost of upgrading the Roanoke Island WTP in Phase II is allocated based on the additional capacity needed by each customer. The projected need for Nags Head is 1.9 mgd of the total expansion of 5 mgd, or 38%.

Costs for restoring the Fresh Pond WTP are allocated one-third each to Nags Head, Kill Devil Hills, and the County, based on each obtaining one-third of the WTP capacity. Costs for observation wells are considered long-term investments and are allocated based on the ultimate reserve capacity in the Roanoke Island WTP. For example, the future capacity for Nags Head is 6 mgd of the total of 15 mgd, or 40%.

8.2.2 Retail Customers

Minimum payments for retail customers are based on peak water consumption. The method for determining rates is illustrated in Table 8.2-5. The amount to be collected from retail customers is estimated using projected capital expenditures, existing and projected debt service, anticipated grants, etc. as indicated below:

Existing Debt Service (Overall)	$335,000/year
Debt Service - Phase I Improvements (assumes obtaining $200,000 CWB Grant and financing $700,000 @ $95/$1,000)[1]	$ 66,000/year
Revenue from Kill Devil Hills and Nags Head for Existing Debt Service	($152,000/year)

1 Phase I costs for the County are estimated to be $1,304,000. The CWB Grant is 25% of the Wanchese distribution system costs. It is anticipated that the County will also have approximately $400,000 collected from assessments by early 1985 when the Phase I improvements are projected to be done.

Capital Reserve for Future Improvements[2]	$444,000/year
Capital Expenditures to be Recovered from Retail Customers	$693,000/year

2 This is based on the requirement of collecting the Phase II costs of $2,232,500 between 1985 and 1987. Anticipated revenue from residential unit impact assessments will account for approximately $900,000 of the Phase II costs with the remaining coming from retail customers.

TABLE 8.2-1

WATER TREATMENT FACILITIES ALLOCATED CAPACITY

	PHASE I			PHASE II			PHASE III		
User	Roanoke Island WTP	Fresh Pond WTP	Total	Roanoke Island WTP	Fresh Pond WTP	Total	Roanoke Island WTP	Fresh Pond WTP	Total
Nags Head	2.0	0.5	2.5	3.9	0.5	4.4	6.0	0.5	6.5
Kill Devil Hills	2.0	0.5	2.5	3.2	0.5	3.7	4.5	0.5	5.0
Dare County	0.8	0.5	1.3	2.7	0.5	3.2	4.3	0.5	4.8
Manteo	0.2	—	0.2	0.2	—	0.2	0.2	—	0.2
	5.0	1.5	6.5	10.0	1.5	11.5	15.0	1.5	16.5

NOTE: This table illustrates the allocation of estimated capital costs based on water requirements through Phase III to each of the entities in the water service area. It is not intended to be exclusively used to establish contractual agreements, and the figures should be reviewed from time to time to determine accuracy and adjust cost allocations.

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TABLE 8.2-2

PHASE I (ALTERNATIVE 1) COST ALLOCATION

Item	Cost	Nags Head % Alloc.	Nags Head Alloc. Cost	Kill Devil Hills % Alloc.	Kill Devil Hills Alloc. Cost	Dare County % Alloc.	Dare County Alloc. Cost
Distribution System	$1,122,000	--	--	--	--	100	$1,122,000
Restore Fresh Pond WTP	$ 500,000	33.33	166,700	33.33	166,700	33.33	166,600
Modifications to[1] Roanoke Island WTP	$ 47,000	40	18,800	40	18,800	20	9,400
Observation Wells[2]	$ 20,000	40	8,000	30	6,000	30	6,000
TOTAL	$1,689,000		193,500		193,500		$1,304,000

1 Based on current allocation of Roanoke Island WTP capacity

2 Allocated based on long-term reserve capacity in Roanoke Island WTP

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TABLE 8.2-3

PHASE II (ALTERNATIVE 1) COST ALLOCATION

Item	Cost	Nags Head % Alloc.	Nags Head Alloc. Cost	Kill Devil Hills % Alloc.	Kill Devil Hills Alloc. Cost	Dare County % Alloc.	Dare County Alloc. Cost
Distribution System	$1,348,000	--	--	--	--	100	$1,348,000
Whalebone Junction[1] Booster Pump Station	$ 180,000	33.33	60,000	33.33	60,000	33.33	60,000
Expand Roanoke[2] Island WTP	$1,179,000	38	448,000	24	283,000	38	448,000
Raw Water System[2]	$ 975,000	38	370,500	24	234,000	38	370,500
Observation Wells[3]	$ 20,000	40	8,000	30	6,000	30	6,000
TOTAL	$3,702,000		886,500		583,000		$2,232,500

1 Allocated one-third to each entity because pump station is needed to maintain adequate pressure and flow in transmission line supplying water to the entire beach area.

2 Allocated based on additional required capacity in WTP.

3 Allocated based on long-term reserve capacity in Roanoke Island WTP.

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TABLE 8.2-4

PHASE III (ALTERNATIVE 1) COST ALLOCATION

Item	Cost	Nags Head % Alloc.	Nags Head Alloc. Cost	Kill Devil Hills % Alloc.	Kill Devil Hills Alloc. Cost	Dare County % Alloc.	Dare County Alloc. Cost
Distribution System	$ 392,000	--	--	--	--	100	$ 392,000
Expand Whalebone Junction Booster Pump Station[1]	$ 40,000	33.33	13,300	33.33	13,300	33.33	13,400
Roanoke Island Booster Pump Station[1]	$ 190,000	33.33	63,300	33.33	63,300	33.33	63,400
Expand Roanoke Island WTP[2]	$ 700,000	42	294,000	26	182,000	32	224,000
Raw Water System[2]	$1,408,000	42	591,400	26	366,100	33	450,500
Observation Wells[3]	$ 20,000	40	8,000	30	6,000	30	6,000
TOTAL	$2,750,000		$970,000		$ 630,700		$1,149,300

1 Allocated one-third to each entity because pump stations are needed to maintain adequate pressure and flow in transmission line supplying water to the entire beach area.

2 Allocated based on additional required capacity in WTP.

3 Allocated based on long term reserve capacity in Roanoke Island WTP.

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TABLE 8.2-5

RETAIL CUSTOMERS PERIODIC FIXED CHARGES

Customer	Peak Quarterly Consumption in Previous Calendar Year	Number of Customers[2]	Residential Customers Equivalency	Equivalent Residential Customers	Allocated Debt Service	Quarterly Fixed Charge[1]
Residential	---	3,000	1	3,000	416,216	34.68
Commercial	0 - 30,000	60	1	60	8,324	34.68
	30,000 - 60,000	35	2	70	9,712	69.36
	60,000 - 120,000	15	4	60	8,324	138.72
	120,000 - 250,000	25	8	200	27,748	277.44
	250,000 - 500,000	20	16	320	44,396	554.88
	500,000 - 750,000	15	25	375	52,027	867.00
	750,000 - 1,000,000	10	33	330	45,784	1,144.44
	1,000,000 - 1,500,000	5	50	250	34,685	1,734.00
	1,500,000 - 2,000,000	5	66	330	45,784	2,288.88
		3,190		4,995	693,000	

1 Total customer billing would include quarterly fixed charge plus $1.38 per 1,000 gallons for all water used.

2 Projected number of customers (distribution of commercial customers is for illustrative purposes) after Phase I improvements.

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PROJECT TEAM

Joseph E. Hardee, P.E. - Joseph Hardee, President of Moore, Gardner & Associates, Inc., served as project consultant for the Dare County Water Study. Joe has over twenty years' experience in development and design of municipal and county water systems in Eastern North Carolina, particularly in the Albemarle Region. He has assisted clients throughout North Carolina in the designs of wastewater treatment systems.

Y. Ellis Spake, P.E. - Ellis Spake, Project Manager for the Dare County Water Study Project, was responsible for the overall project coordination and management. Ellis has been employed with Moore, Gardner & Associates, Inc. since 1969. He has developed county-wide water and wastewater study projects for clients throughout North Carolina.

A. Barry Nelson, Project Geologist - A certified professional geologist, Barry Nelson's responsibilities on the Dare County Water Study Project included groundwater and surface water evaluations, and non-conventional water supply evaluation. Barry has extensive experience in groundwater and surface water hydrology and has participated in numerous water supply evaluations in the NC Coastal Plain Region.

Fredrick Waters, Civil Engineer - Fred Waters' involvement in the Dare County Water Study project included demographic evaluations, water demand projections and computer modeling of the distribution and transmission systems. Fred has participated in numerous water supply studies for municipalities and county governments throughout North Carolina.

David Todd, P.E. - David Todd's primary areas of expertise is in civil and structural engineering. As such, he was responsible for the water treatment plant evaluation and hydraulic evaluation of the distribution and transmission systems. He prepared the plan for upgrading and modifying the water treatment plant and transmission system. Dave has ten years' experience in water systems design in Mid-Western US and the Middle East.

Larry W. Pearce, P.E. - Larry Pearce, a registered engineer in North Carolina, was responsible for preparing the rate study and developing a new rate structure and financial mechanisms for financing the capital improvements and operation and maintenance costs for the long range development plan. Larry has prepared water and wastewater rate studies for several NC cities and counties.

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