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NEEDS AND SOURCES REPORT FOR THE PORT CHARLOTTE SERVICE AREA

Prepared for

GENERAL DEVELOPMENT UTILITIES, INC.

Port Charlotte, Florida

TM

Prepared by

CH2M HILL

October 1987

202923

NEEDS AND SOURCES REPORT
FOR THE PORT CHARLOTTE SERVICE AREA

Prepared for

GENERAL DEVELOPMENT UTILITIES, INC.
Unit 13, Promenades Mall
Port Charlotte, Florida 33952

Prepared by

CH2M HILL
7201 N.W. 11th Place
P.O. Box 1647
Gainesville, Florida 32602

CH2M HILL

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EXECUTIVE SUMMARY

General Development Utilities, Inc. (GDU), a wholly-owned subsidiary of General Development Corporation (GDC), supplies water to the communities within the Port Charlotte service area, which includes portions of southeast Sarasota, northeast Charlotte, and southwest DeSoto counties. Surface water withdrawals for water supply are governed by a Consumptive Use Permit (No. 202923) issued by the Southwest Florida Water Management District (SWFWMD). Under Condition 15 for permit renewal, GDU must submit a Needs and Sources Report addressing water supply needs for the Port Charlotte area through the year 2000. GDU retained CH2M HILL to prepare the report for submission to the SWFWMD by November 5, 1987. Consideration was given to the following possible service area growth scenarios:

1. Minimum: existing and obligated/committed areas
2. Most likely: existing, obligated/committed, and most probable areas to be added

WATER DEMAND PROJECTIONS

The two sources of supply for the Port Charlotte service area are the Myakkahatchee Creek and the Peace River. Current permitted withdrawals (as measured by plant effluent meters) are a combined annual average day volume of 8.2 million gallons per day (mgd), with a maximum withdrawal on any day of 16.4 mgd. The annual average day demand for the system in 1986 was 5.4 mgd, and reflects an increase of 18 percent since 1981. The historical maximum day pumpage was 8.3 mgd in April 1986, which is about 10 percent greater than the peak demand day in 1981. Connections increased by 33 percent from 1981 to 1986, but usage per connection dropped by 11 percent. Based on a 1986 population of 60,000, the average demand per capita was 90 gallons per day (gpd); the average day demand per equivalent residential connection (ERC) was 155 gpd.

Future demands for the Port Charlotte service area were estimated using GDU-projected water demands per ERC and projected growth in ERC values. Based on historical water usage data for the service area for 1981 through 1986, GDU projects an average water demand of 165 gpd per ERC. One ERC is equivalent to a single family residential connection; ERC values for commercial and institutional users are calculated by multiplying the estimated square footage of building spaces by historical usages in gpd per square foot for the specific classification, then dividing by the ERC value of 165.

The average day demand for the most likely growth scenario is expected to be 6.5 mgd in 1987, increasing to 14.5 mgd in 2000. The maximum day demand is estimated to increase from 10.3 mgd in 1987 to 23 mgd in 2000.

WATER SOURCE SUPPLY POTENTIAL

Existing streamflow and water quality records were statistically analyzed to evaluate the potential for continued or increased water supply from the Myakkahatchee Creek, which currently provides raw water to the 4.4-mgd rated North Port Water Treatment Plant (WTP). The evaluations, which began in late 1986, also included the tributary canals to the creek, the Snover and Cocoplum waterways. Although the mean annual flows from these sources indicate a potential for increased water supply development, monthly flows are highly variable and about 10 months of offline storage capacity in surface or aquifer storage recovery (ASR) facilities would be needed to ensure a reliable water supply. In addition, water from the Cocoplum Waterway would require treatment beyond existing capabilities to reduce concentrations of total dissolved solids (TDS) to acceptable limits. It must be noted that, since the completion of this initial evaluation, indications are that the Cocoplum Waterway may be eliminated (under certain conditions) as a potential water supply source as a result of the pine forest effluent spray system proceedings with the City of North Port.

The potential of the Peace River, the current raw water source for the 12-mgd Peace River WTP, for increased supply was analyzed using the PEACE computer model developed by CH2M HILL for GDU in 1985, updated with additional flow and water quality data. A new diversion formula was developed for SWFWMD approval based on data from Environmental Quality Laboratory, Inc., and considering overall environmental impacts such as possible changes to riparian vegetation and movement of the saltwater/freshwater interface in the estuary. The proposed formula is structured to allow withdrawal from the Peace River as a function of actual flow. Analytical results indicate that the Peace River can be used to meet all or part of projected demands for the Port Charlotte service area through the year 2000. Although withdrawals from the river would be limited daily by the diversion rule and low flows, the ultimate safe yield needed can be met by providing adequate capacity in raw water storage, treatment, and ASR facilities.

The availability of groundwater sources was evaluated based on a literature review on the water resources of the region. Development of the shallow aquifer by GDU is not recommended for economical and technical reasons. The potential for development of the shallow artesian aquifer is significantly southwestern

limited by low yield capacities and high TDS concentrations. The best sources of groundwater in the area appear to be the upper and lower Floridan aquifers in [redacted] DeSoto County. Both zones are highly productive and contain water with TDS and sulfate concentrations slightly above drinking water standards. The main constraint on the quantity of water available would be drawdown effects on adjoining users. In the rest of the service area, TDS content in both upper and lower Floridan water is well above drinking water standards; the most feasible method of developing a groundwater supply in these areas would probably be through desalination of brackish water from the upper Floridan aquifer zone.

WATER CONSERVATION AND WASTEWATER REUSE OPTIONS

Implementation of methods to reduce demand are most practical for the Port Charlotte service area, where water losses and unaccounted for water are not significant problems. GDU should continue with public education programs on water conservation, such as its "Slow the Flow" campaign, promote and provide water conservation devices, and investigate economic incentives for water conservation. The new GDU project, Residential Xeriscape, promotes water conservation by illustrating creative landscaping techniques to the public on a local demonstration site. A matching grant from the SWFWMD is helping to fund the project.

Land irrigation of treated wastewater will continue to be a favorable method for reuse in the service area. Currently, treated wastewater is applied to a dedicated spray irrigation site and the North Port Golf Course.

STORAGE OPTIONS AT NORTH PORT WTP

The PLANT model developed by CH2M HILL for a 1985 ASR study for GDU was used to evaluate storage facility configurations for the North Port WTP. Modeling results indicate that a 145-acre surface storage reservoir would be required to enable the plant to reliably produce a 2.75-mgd average daily flow. Surface reservoir storage, however, while a proven alternative, is costly and GDU began testing an ASR system at the Peace River WTP site in July 1984 in an attempt to reduce the cost of service to its customers. This ASR system was permitted by the SWFWMD in May 1986.

The knowledge gained from the Peace River ASR testing was applied to the North Port site to assess ASR potential there. It was found that use of ASR alone is limited by the low quality native groundwater. A combination surface storage/ASR facility with a 45-acre reservoir and 4.4-mgd plant and ASR capacities can, however, reliably meet a 2.2-mgd average daily production rate. Analyses indicate that other water supply options (e.g., Peace River and groundwater sources) are preferable for future facility expansion to meet projected water demands.

TREATMENT AND SUPPLY FACILITIES EVALUATION

Current firm capacity of the North Port WTP is 4.4 mgd. Average water production for 1986 was 1.2 mgd. Concerns about the quantity and quality of its raw water source, the Myakkahatchee Creek, make future long-term usage uncertain. The existing secondary drinking water standards for TDS and sulfates are not met at times, a circumstance that is magnified because existing treatment processes at the plant increase concentrations of those substances. Hardness removal (when the softening operational mode is used) is limited because caustic soda can be added only to an extent that does not exceed the current sodium primary drinking water standard. Taste and odor control is an ongoing aesthetic problem.

In addition, new federal and state drinking water standards are under consideration in response to the Amendments enacted by the U.S. Congress in 1986 to the 1974 Safe Drinking Water Act. Some existing standards are expected to be revised and new standards established for several unregulated contaminants. New trihalomethane (THM) and turbidity standards under consideration will probably be too strict for existing facilities at the North Port WTP to meet.

The Peace River WTP, which has a current firm capacity of 12.0 mgd, can be expected to continue as the major treated water source for the Port Charlotte service area. Average water production from the plant in 1986 was 4.2 mgd. With adequate storage capacity for both raw and finished waters, all current drinking water standards are typically met. Major concerns about water quality center on taste and odor control, and possibly stricter future THM and turbidity standards. GDU is committed to monitoring changes in the drinking water regulations and to taking appropriate action to ensure compliance.

Additionally, in an effort to anticipate and correct potential surface water reservoir management problems, GDU, in a separate project, contracted with the consultants Jones, Edmunds and Associates to develop both short- and long-term management scenarios for the Peace River WTP surface reservoir. The first draft of that report, entitled Peace River Reservoir Master Plan, was submitted to GDU in October 1987.

Future studies should focus on sludge production, handling, and disposal for both WTPs, both for the large costs associated with these items and to address environmental concerns.

ALTERNATIVES FOR FACILITY EXPANSION

To compare the cost-effectiveness of each of the potential raw water supplies (the Myakkahatchee Creek, the Peace River, and brackish groundwater), facility requirements for expansion were developed and an economic analysis made based on planning level cost estimates. The most favorable area for groundwater development (in south~~east~~west DeSoto County) is currently being developed as an ASR well field for the Peace River WTP. Therefore, two areas to the south and west of the plant were identified for potential brackish groundwater supply (well field A between the Peace River and North Port WTPs and well field B near the Gulf Cove Wastewater Treatment Plant).

Cost analysis indicates that the development of the Peace River source will cost less than half of that for the Myakkahatchee Creek (North Port WTP) for equal yields. The cost for developing well field A is higher than the development cost for the Peace River but less than for the Myakkahatchee Creek. Development of well field B is somewhat more expensive than expansion of the North Port facilities and is, therefore, the most costly alternative. The estimated maximum potential yields from well fields A and B are 10 and 6 mgd, respectively.

The existing Peace River system can supply an average daily demand of at least 6.5 mgd. An additional 8 mgd of capacity is needed to meet the most likely demand of 14.5 mgd for the year 2000. Total annual costs for a 14.5-mgd system can be compared for each of the four sources, as summarized below, to estimate the best expansion alternative based on cost:

System	Total Annual Cost (million $)
14.5 mgd Peace River	4.3
6.5 mgd Peace River + 8 mgd Myakkahatchee Creek	6.2
6.5 mgd Peace River + 8 mgd Well Field A	5.6
8.5 mgd Peace River + 6 mgd Well Field B	6.4

The comparison of costs for development of the raw water sources indicates that the Peace River alone is the best alternative for meeting demands through 2000. Under the most likely growth scenario, only one 6-mgd expansion of the

Peace River WTP and an additional 12 mgd of ASR capacity would need to be constructed through the year 2000. The North Port WTP would be gradually phased out as a treatment facility as GDU completes planned distribution system improvements. The North Port WTP may be converted to a storage and repumping distribution center as part of these improvements. The costs and feasibility of brackish groundwater development, along with an updated distribution system expansion and cost analysis, should be examined in the future, when average day demand is within 5 years of reaching 18 mgd.

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FIGURES (Continued)

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Introduction

Section 1
INTRODUCTION

General Development Utilities, Inc. (GDU), a wholly owned subsidiary of General Development Corporation (GDC), owns and operates water production, treatment, and distribution systems to meet the water supply requirements of existing and planned communities within the designated Port Charlotte service area. As shown in Figure 1-1, the service area encompasses portions of southeast Sarasota, northeast Charlotte, and southwest DeSoto counties on Florida's west coast, and includes the communities of Port Charlotte and North Port.

Raw water for the Port Charlotte service area is withdrawn from the Myakkahatchee Creek for treatment at the North Port Water Treatment Plant (WTP), which has a rated capacity of 4.4 million gallons per day (mgd), and from the Peace River for treatment at the 12-mgd Peace River WTP. The two plants, which are located as shown in Figure 1-1, provide all potable water to the Port Charlotte service area. The North Port WTP has no raw water storage facilities; the Peace River WTP has an 85-acre (625-million-gallon) offstream raw water reservoir. A nominal 1.5-mgd treated water aquifer storage and recovery (ASR) system is also provided at the Peace River WTP and is currently being expanded to 5 mgd.

Through Consumptive Use Permit (CUP) No. 202923, issued May 5, 1982, GDU is authorized by the Southwest Florida Water Management District (SWFWMD) to make average annual surface water withdrawals from both sources of 8.2 mgd, with a maximum withdrawal on any day of 16.4 mgd. Under Condition 15 for permit renewal, GDU is required to submit to the SWFWMD 6 months before the expiration date of May 5, 1988, a Needs and Sources Report addressing water supply needs for the Port Charlotte service area through the year 2000. GDU has retained CH2M HILL to prepare the report for submission to the SWFWMD by November 5, 1987.

The Needs and Sources Report presented herein includes water demand projections through the year 2000 for minimum (existing and obligated areas) and most likely (minimum plus most probable areas to be added) growth scenarios for the Port Charlotte service area. The report includes an evaluation of the supply potential of surface and groundwater sources, the feasibility of offstream storage at the North Port WTP, and water conservation and wastewater reuse options. Existing treatment and supply facilities are evaluated and alternatives presented for expansion of those facilities to meet the projected needs.

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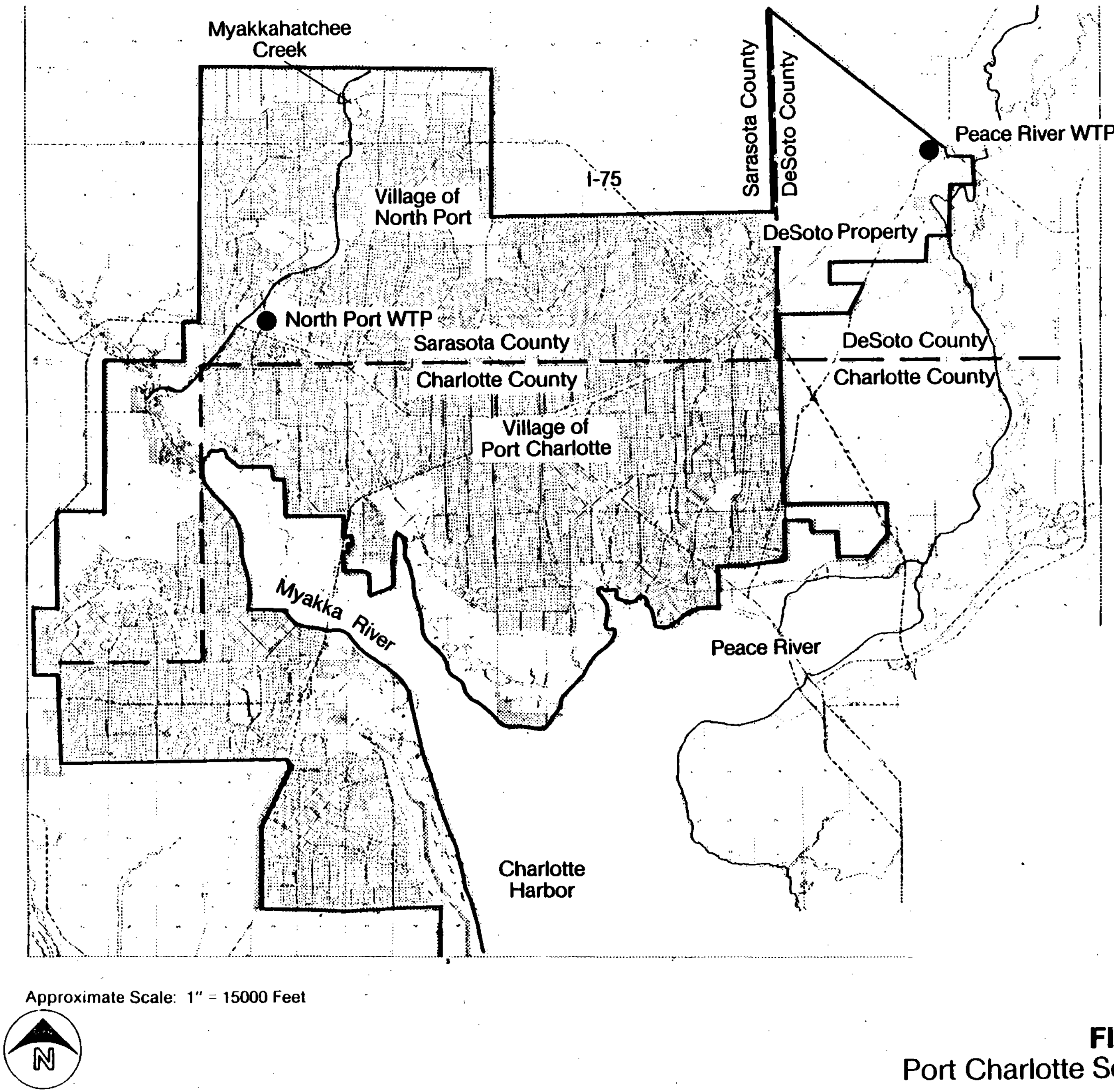

FIGURE 1-1.
Port Charlotte Service Area.

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SECTION 2
Water Demand Projections

Section 2
WATER DEMAND PROJECTIONS

Historical water demands for the Port Charlotte service area are summarized and future water demands projected through the year 2000 buildout. The demand projections are based on GDU estimates for the minimum and most likely scenarios for future growth of the service area, as shown in Figure 2-1.

HISTORICAL DEMANDS

Average day and maximum day finished water pumpages from the combined North Port and Peace River WTPs to the existing service area from 1981 through 1986 are listed by month in Table 2-1 and Figure 2-2. The combined pumpages are approximately equal to system demands. Water drawn from the Peace River WTP ASR system is metered with the plant's daily output and is included in these pumpage values.

The average and maximum day demands, which have generally followed the same patterns over the last 6 years, are increasing significantly. Figure 2-3, which shows annual average and maximum day pumpages, clearly indicates the service area demand trends from 1981 through 1986. Although there was an initial decrease in average system demands during the period, steady increases have occurred over the last 3 years. From 1981 to 1986, the annual average day demand increased 18 percent, from approximately 4.6 to 5.4 mgd. Of that production, approximately 4.2 mgd was supplied by the Peace River WTP and 1.2 by the North Port WTP. However, since 1983, the annual demands have increased approximately 10 percent per year (29 percent total). The historical maximum daily pumpage was approximately 8.3 mgd, which occurred on April 28, 1986, and was about 10 percent greater than the peak demand day in 1981.

The 18 percent increase in the annual average day water demands since 1981 occurred at the same time that the number of service area water customers (based on total metered connections) increased 33 percent. At the end of 1986, there were approximately 27,700 metered service connections (including master meters for multi-unit customers) in the service area.

Figure 2-4 presents the total number of metered connections and the average daily demand per connection from 1981 through 1986. The usage per connection dropped approximately 11 percent during this period, from 219 gallons per day (gpd) in 1981 to 195 gpd in 1986. This decline is attributed in part to the water rate increase in 1981.

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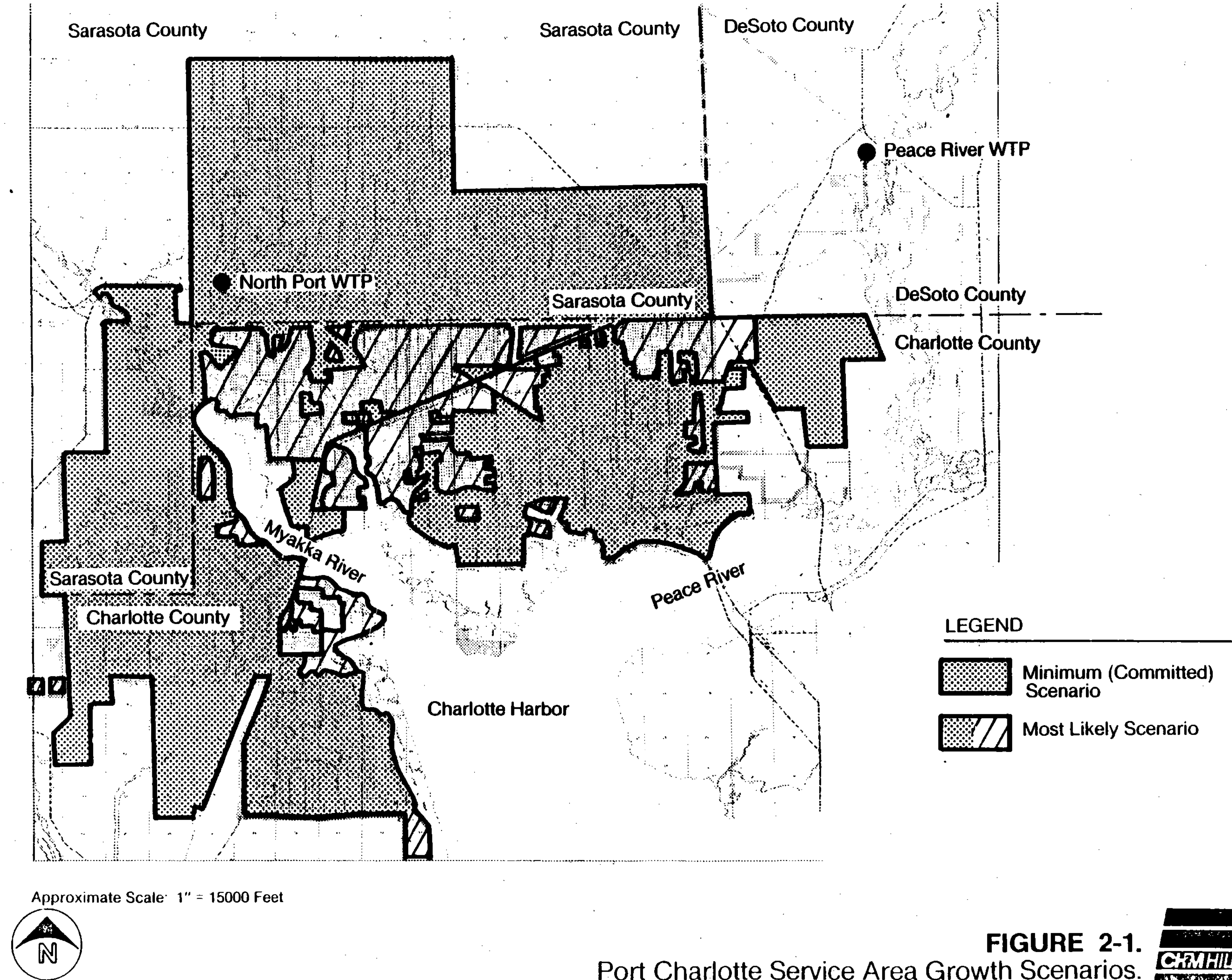


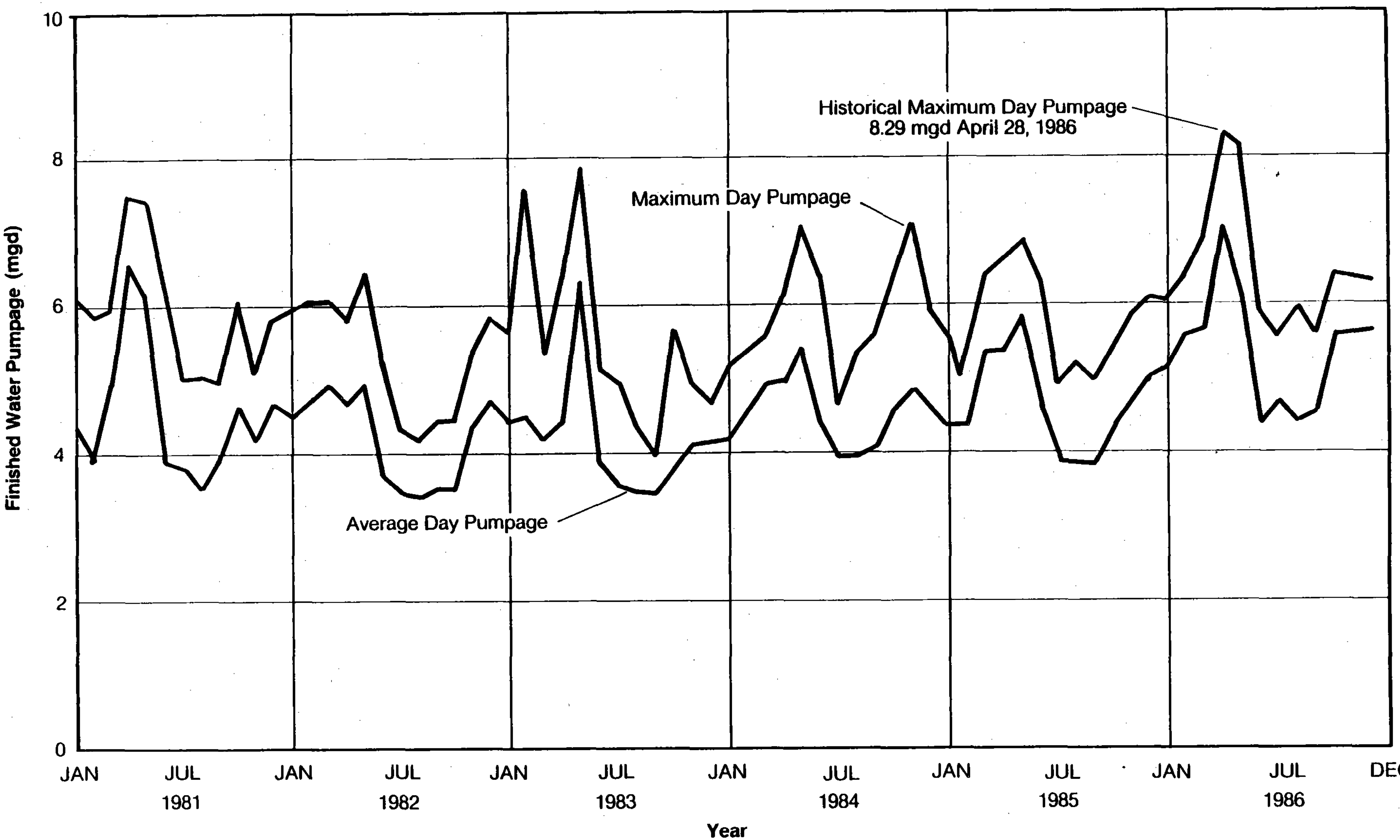
FIGURE 2-1.
Port Charlotte Service Area Growth Scenarios.

Table 2-1
MONTHLY MAXIMUM AND AVERAGE DAY PUMPAGES (1981-1986)

Date	Pumpage (x 1,000 gal) Maximum Day	Pumpage (x 1,000 gal) Average Day	Date	Pumpage (x 1,000 gal) Maximum Day	Pumpage (x 1,000 gal) Average Day
1981			1984		
January	6,250	4,452	January	5,237	4,247
February	5,875	3,918	February	5,392	4,532
March	6,024	4,884	March	5,591	4,865
April	7,508[a]	6,630	April	6,169	5,019
May	7,376	6,181	May	7,078	5,408
June	6,187	3,892	June	6,398	4,386
July	5,015	3,763	July	4,687	3,893
August	5,024	3,530	August	5,342	3,919
September	4,974	3,936	September	5,593	4,034
October	6,099	4,677	October	6,428	4,612
November	5,134	4,209	November	7,142[a]	4,876
December	5,796	4,705	December	5,875	4,602
Annual Average	--	4,569	Annual Average	--	4,532
1982			1985		
January	5,915	4,479	January	5,596	4,330
February	6,048	4,679	February	5,058	4,367
March	6,067	4,973	March	6,380	5,354
April	5,801	4,748	April	6,557	5,412
May	6,465[a]	4,985	May	6,851[a]	5,817
June	5,263	3,742	June	6,287	4,543
July	4,333	3,495	July	4,935	3,893
August	4,234	3,413	August	5,221	3,836
September	4,404	3,534	September	4,992	3,820
October	4,412	3,532	October	5,399	4,335
November	5,454	4,319	November	5,787	4,726
December	5,794	4,739	December	6,143	5,112
Annual Average	--	4,217	Annual Average	--	4,631
1983			1986		
January	5,675	4,443	January	6,045	5,245
February	7,551	4,474	February	6,428	5,650
March	5,298	4,220	March	6,914	5,700
April	6,362	4,427	April	8,291[a]	7,051
May	7,785[a]	6,335	May	8,153	6,146
June	5,109	3,835	June	5,891	4,410
July	4,935	3,634	July	5,608	4,657
August	4,338	3,455	August	5,963	4,450
September	3,966	3,396	September	5,570	4,546
October	5,667	3,782	October	6,441	5,597
November	4,959	4,107	November	6,399	5,592
December	4,762	4,220	December	6,341	5,658
Annual Average	--	4,194	Annual Average	--	5,390

Source: GDU Peace River WTP and North Port WTP operation reports. Data for the plants were combined.

[a]Maximum day for specified year.



Note: Combined Data from Peace River and North Port WTPs.

FIGURE 2-2.
Monthly Maximum and Average Day Finished Water Pumpages (1981-1986).

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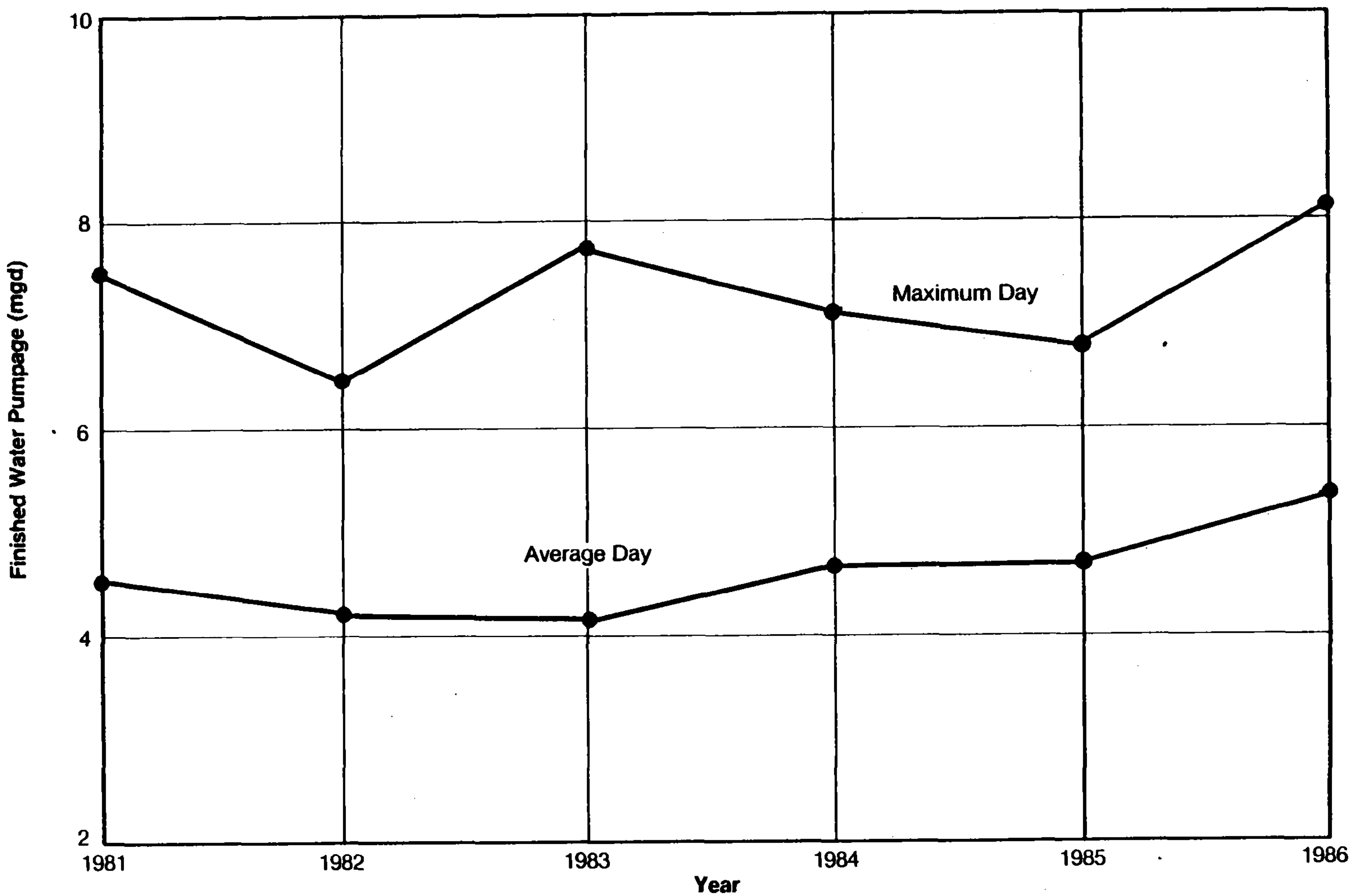

Note: Combined Data from Peace River and North Port WTPs.

FIGURE 2-3.
Annual Maximum and Average Day Finished Water Pumpages (1981-1986).

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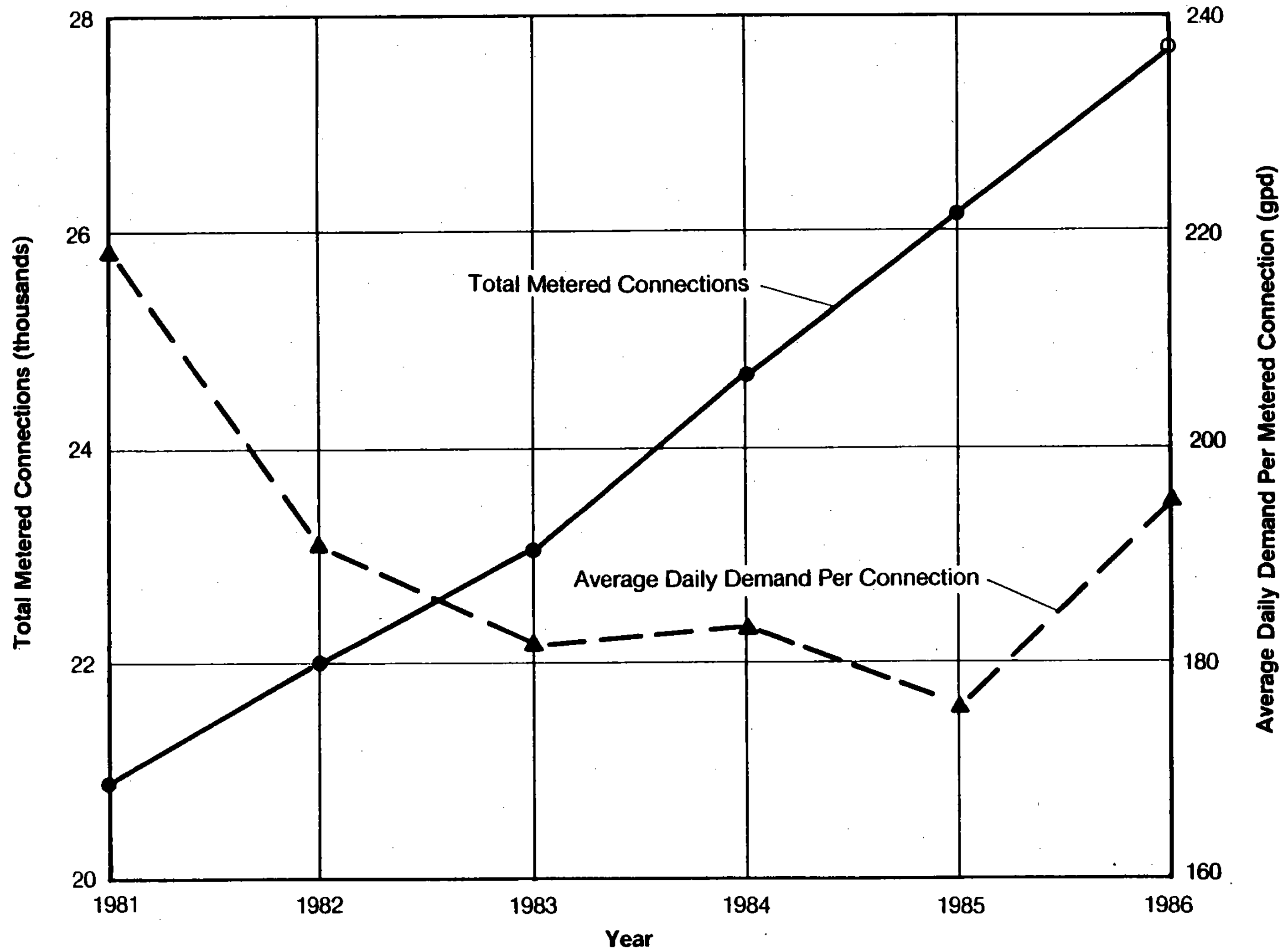

FIGURE 2-4.
Total Metered Connections and Average Daily Demand Per Connection (1981-1986).

Below-normal rainfall periods in 1981, 1984, and 1985, and the associated mandatory and voluntary water restrictions, also affected water usage. Rainfall data collected at the North Port WTP from 1981 through 1986 are presented in Table 2-2.

The estimated service area population, based on the Port Charlotte Area Growth Model (Paul G. Van Buskirk Associates, 1981), was approximately 45,000 in 1981, which equates to an average of 2.2 people per connection. At this population, the 1981 average demand was approximately 100 gallons per capita per day (gpcd). Assuming a 1986 projected population of about 60,000 from the growth model, the average 1986 demand dropped to about 90 gpcd.

Seasonal variations in demand are important in water supply, treatment, and storage facility planning. The ASR system at the Peace River WTP takes advantage of the significant Port Charlotte seasonal demand fluctuations. Figure 2-5 shows the maximum and average water demand flow range by month for the 1981 through 1986 period. Typical peak demands in the spring (March through May) are approximately 20 to 60 percent greater than the minimum demand periods that usually occur in the late summer months (July through September).

Projections of future average day demands are based on equivalent residential connections (ERCs). GDU has calculated ERC values for the past 6-year period by deducting the water usage of the 17 largest master-metered customers from the total water volume billed to all customers, and dividing by the resulting number of metered connections. As shown in Figure 2-6, the average day demand per ERC has declined from approximately 177 gpd in 1981 to about 147 gpd in 1985 and 155 gpd in 1986. GDU projects an average demand of 165 gpd per ERC in estimating future demands for the Port Charlotte service area.

Maximum day to average day demand ratios are used in projecting future peak demands. The maximum to average demand ratios calculated for the Port Charlotte service area from 1981 through 1986 are presented in Table 2-3 and Figure 2-7. The maximum day demand ranged from 148 to 186 percent greater than the annual average daily demand over the 6-year period, indicating a fairly constant pattern. The average maximum to average day demand ratio over the past 6-year period (1.60) is used for projecting maximum day demands.

Table 2-2
NORTH PORT WTP MONTHLY PRECIPITATION DATA[a]
1981-1986

	1981	1982	1983	1984	1985	1986	Normal[b]
January	.48	2.00	3.41	.74	1.59	1.58	2.55
February	4.35	1.77	9.02	3.44	1.11	1.36	3.08
March	.81	3.82	6.66	5.08	3.32	5.13	2.82
April	.02	3.71	2.20	5.42	2.61	.74	2.16
May	2.28	2.93	.77	2.86	1.03	1.71	3.84
June	9.36	9.94	9.75	3.24	4.34	10.26	8.33
July	7.14	15.25	9.50	11.29	9.51	6.14	8.43
August	18.25	8.38	10.65	8.21	8.69	7.00	9.35
September	5.86	8.08	11.75	4.98	5.53	3.18	8.59
October	1.22	6.48	7.54	1.66	4.68	4.47	3.37
November	.74	1.27	4.81	1.31	2.58	2.53	2.12
December	.43	.84	5.67	.27	.39	7.18	2.17
TOTAL	50.94	64.47	81.73	48.50	45.36	51.28	56.81

[a]Measured by GDU.

[b]Based on precipitation records from 1951 to 1980 at Myakka River State Park (approximately 10 miles north of North Port and Port Charlotte) (NOAA, 1987).

Note: "Water Shortage Declarations" by the Southwest Florida Management District from 1981 through 1986: May-September 1981, February-September 1985, and beginning December 1986.

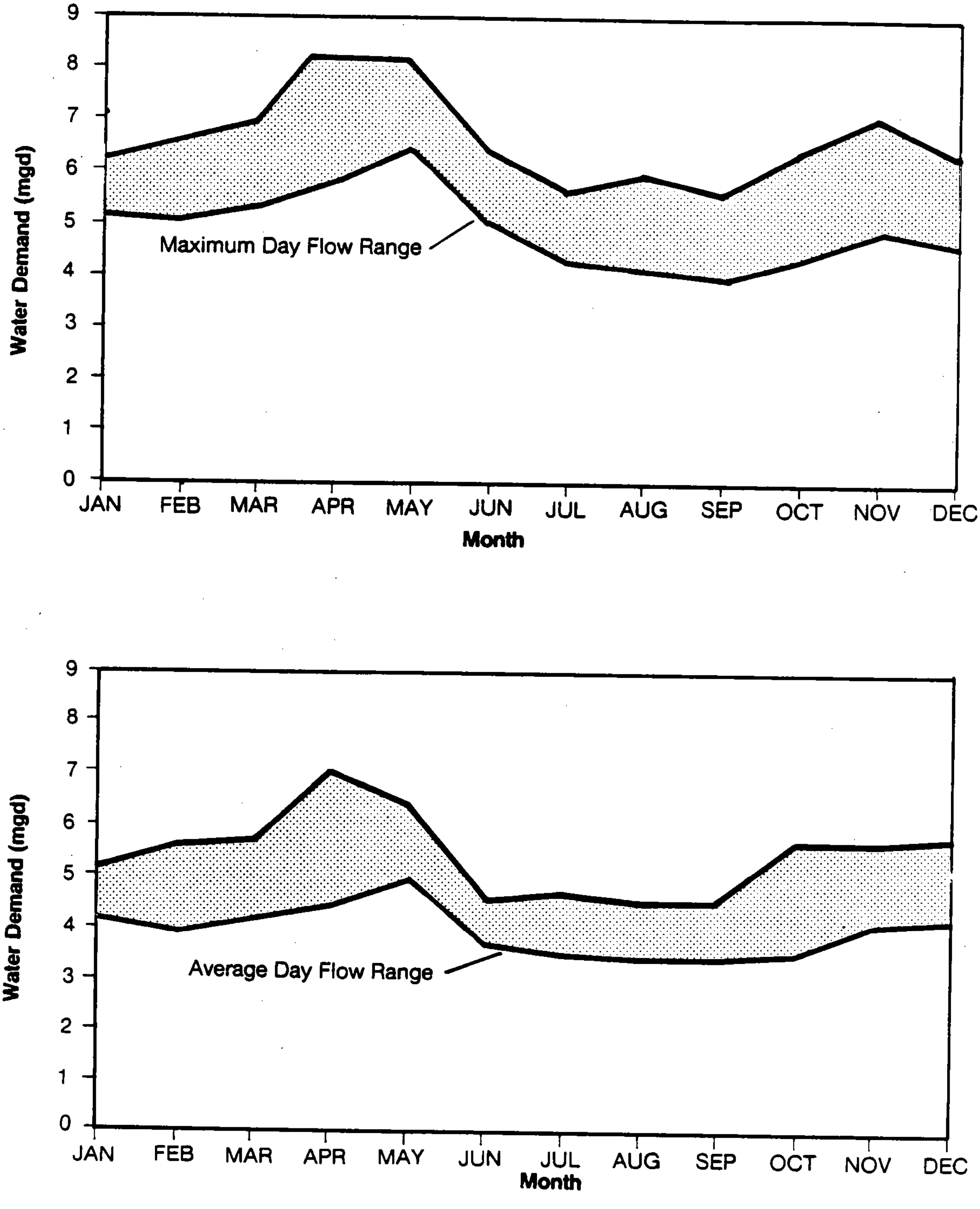

FIGURE 2-5.
Seasonal Water Demand Variations (1981-1986).

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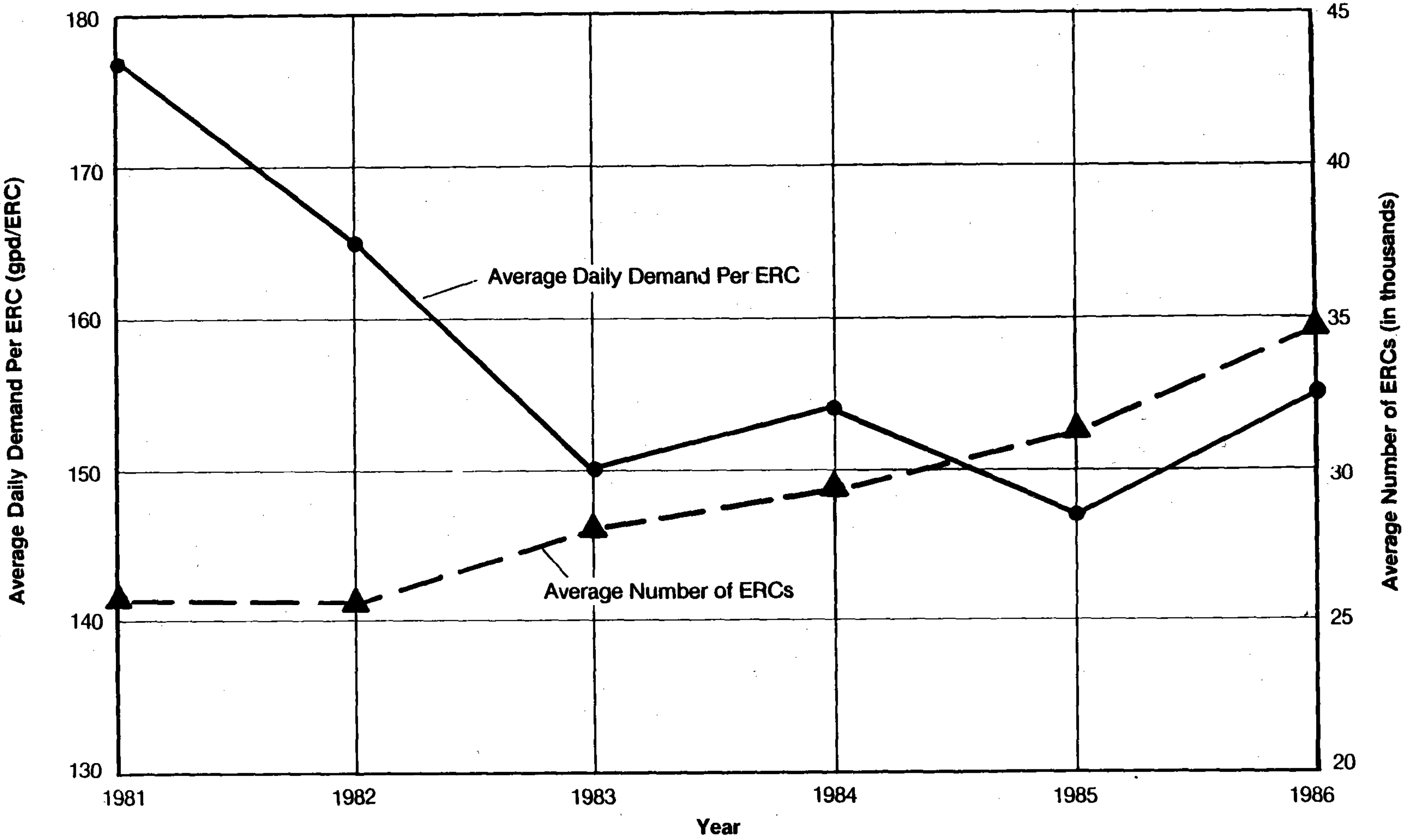

Source: GDU, Port Charlotte and North Port Service Area Water Forecast (1986).

FIGURE 2-6.
Equivalent Residential Connections (ERC) and Average Daily Demand Per ERC (1981-1986).

Table 2-3
MAXIMUM AND AVERAGE DAY DEMAND RATIOS

Year	Maximum Day[a] (mgd)	Average Day[b] (mgd)	Ratio: Maximum Day To Average Day
1981	7.508	4.569	1.64
1982	6.465	4.217	1.53
1983	7.785	4.194	1.86
1984	7.142	4.532	1.58
1985	6.851	4.631	1.48
1986	8.291	5.390	1.54
Average			1.60

[a]Maximum day flow is the maximum combined pumpage rate on a single day in the year shown for the Peace River and North Port WTPs.

[b]Average day flow is the average combined daily pumpage rate in the year shown for the Peace River and North Port WTPs.

Source: GDU, based on flowmeter readings at the Peace River and North Port WTPs.

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FIGURE 2-7.
Maximum to Average Day Demand Ratios (1981-1986).

PROJECTED DEMANDS

The projected average and maximum day water demands for the minimum and most likely Port Charlotte service area growth scenarios are presented in Tables 2-4 and 2-5, respectively. The projected flows shown are based on the GDU-projected ERCs and the following major assumptions:

- o The estimated average daily demand per ERC will remain constant at 165 gpd through buildout.
- o One single family connection equals one ERC.
- o Institutional and commercial user ERC values are derived from projected use, calculated by multiplying the estimated square footage of buildings by the historical gpd per square foot of the specific classification of user. ERC values are then calculated by dividing the projected usage by the ERC use factor (165 gpd per ERC).
- o Port Charlotte buildout is possible by 2050, with all service area buildout completed by 2070.

A complete listing of projection assumptions, including growth estimates for developers other than GDC, are presented in the GDU report entitled Port Charlotte and North Port Service Area Water Forecast (1986).

Average and maximum day projected water demands for the two growth scenarios are shown graphically in Figures 2-8 and 2-9. The average daily demand in the most likely scenario is estimated to increase from 6.5 mgd in 1987 to 14.5 mgd in the year 2000. The maximum daily demand is estimated to increase from 10.3 to 23 mgd during the same period.

FIRE FLOWS

The primary function of a water system is to supply potable water for residential, commercial, and industrial use. Typically, another function of a municipal water system is to provide water for fire protection. While the annual volume of water required for fire protection is small, the instantaneous water use rate required during fire events can be high. The following fire flows are required by Charlotte County Ordinance Number 85-9 for the Port Charlotte service area:

Table 2-4
AVERAGE AND MAXIMUM DAY WATER DEMAND PROJECTIONS
MINIMUM SCENARIO

Year	Total Projected ERCs[a]	Projected Average Day Water Demand[b] (mgd)	Projected Maximum Day Water Demand[c] (mgd)
1987	38,600	6.4	10.2
1988	41,200	6.8	10.9
1989	43,900	7.2	11.6
1990	46,700	7.7	12.3
1995	60,300	9.9	15.9
2000	73,000	12.0	19.3

[a]Source: GDU Port Charlotte and North Port Service Area Water Forecast (1986).

[b]Based on 165 gpd/ERC.

[c]Based on maximum day:average day demand ratio of 1.60.

Table 2-5
AVERAGE AND MAXIMUM DAY WATER DEMAND PROJECTIONS
MOST LIKELY SCENARIO

Year	Total Projected ERCs[a]	Projected Average Day Water Demand[b] (mgd)	Projected Maximum Day Water Demand[c] (mgd)
1987	39,200	6.5	10.3
1988	42,500	7.0	11.2
1989	45,900	7.6	12.1
1990	49,400	8.2	13.0
1995	69,000	11.4	18.2
2000	88,100	14.5	23.3

[a]Source: GDU Port Charlotte and North Port Service Area Water Forecast (1986).

[b]Based on 165 gpd/ERC.

[c]Based on maximum day:average day demand ratio of 1.60.

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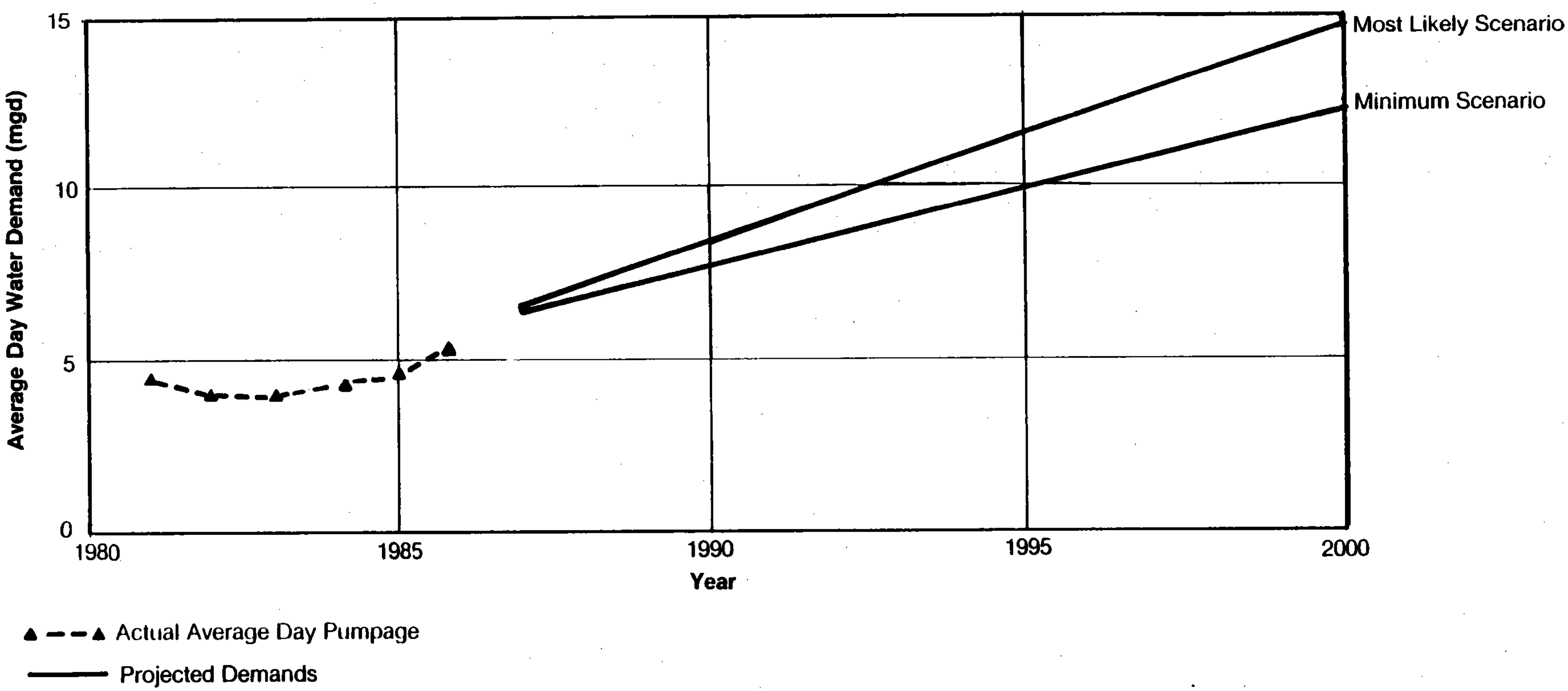

FIGURE 2-8.
Average Day Water Demand Projections.

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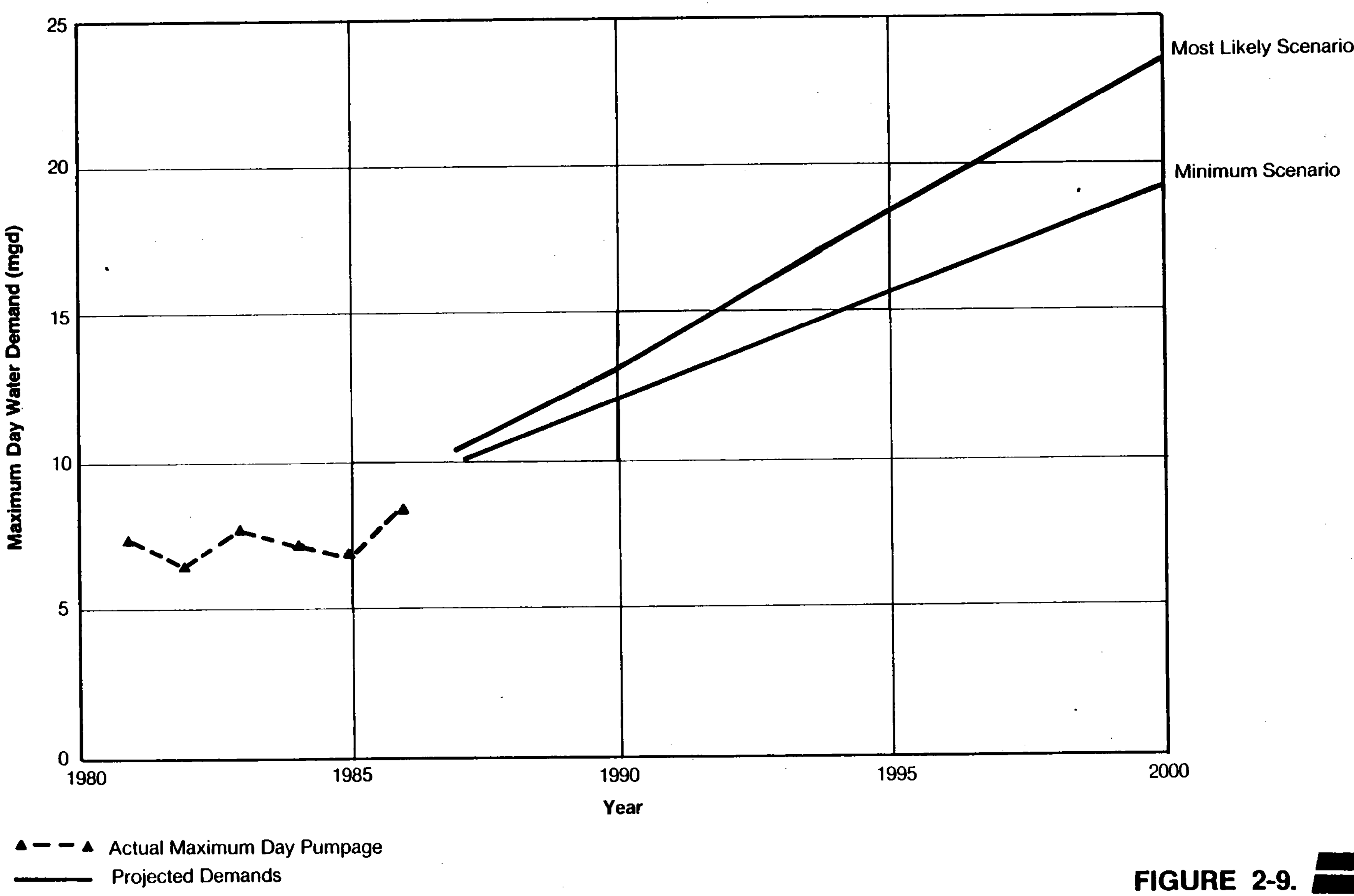

FIGURE 2-9.
Maximum Day Water Demand Projections.

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- Mobile home parks, mobile home subdivisions, and recreational vehicle parks require hydrants (spaced not more than 1,000 feet apart) that can deliver a minimum flow of 500 gpm at 20 pounds per square inch (psi) residual pressure for a minimum of 1 hour.

- Single, duplex, and triplex family units (except as defined above) require hydrants (spaced not more than 1,000 feet apart) that can deliver a minimum flow of 750 gpm at 20 psi residual pressure for a minimum of 1 hour.

- Industrial, commercial, apartment, and other high-value areas require hydrants (spaced not more than 600 feet apart) that can deliver a minimum flow of 1,250 gpm at 20 psi residual pressure for a minimum of 2 hours.

- Heavy manufacturing and heavy industrial areas require hydrants (spaced not more than 600 feet apart) that can deliver 1,250 gpm at 20 psi residual pressure for a minimum of 2 hours from each of two hydrants at the same time.

Fire flow requirements are generally better met by water system storage capacity than by increased treatment plant capacity. Water from system storage can be available more quickly than water from increased production. The treatment plant, however, should have adequate capacity to replenish within 72 hours water taken from storage for fire flows.

SECTION 3
Water Source Supply Potential

Section 3
WATER SOURCE SUPPLY POTENTIAL

MYAKKAHATCHEE CREEK AND THE NORTH PORT CANAL SYSTEM

A statistical analysis was conducted in late 1986 on existing streamflow and water quality records to evaluate the quantity, quality, and reliability of the available water supply from Myakkahatchee Creek and the tributary North Port Canal System. Myakkahatchee Creek, the existing source of water for the North Port WTP, has two main tributary canals: Snover Waterway and Cocoplum Waterway. Because Snover Waterway enters the creek upstream of the current WTP intake site, shown in Figure 3-1, it will not be considered separately from the creek. The Cocoplum Waterway enters Myakkahatchee Creek just downstream of the intake, passing near the WTP site boundary, and could be a potential future supply source if its quality and quantity characteristics are acceptable.

WATER QUALITY PARAMETERS

The two major parameters considered in the evaluation were flow and total dissolved solids (TDS) levels. The latter was found to be the limiting water quality parameter for the Myakkahatchee Creek and its tributaries, based on historical water quality data. As with many small waterways in Florida, flow in the creek varies widely by season, with corresponding variation in TDS levels. During low flow conditions, raw water TDS concentrations can increase to a level at which the WTP finished water exceeds the 500 milligrams per liter (mg/l) secondary drinking water standards. Limited plant operation data indicate that the coagulation/filtration treatment process currently in use at the North Port WTP increases TDS levels by approximately 100 mg/l. Thus, raw water TDS concentrations must either be below 400 mg/l or an additional treatment process, such as desalting, must be considered to achieve a 500 mg/l TDS level.

DATA ANALYSIS

Existing Data Base

Existing streamflow and water quality records were provided for the North Port Canal System by Environmental Quality Laboratory, Inc. (EQL), which has been monitoring the system for GDU since 1980 (EQL, 1987). Data for Myakkahatchee Creek were obtained from records for water control structure number 101 (CS No. 101), located on the creek downstream of

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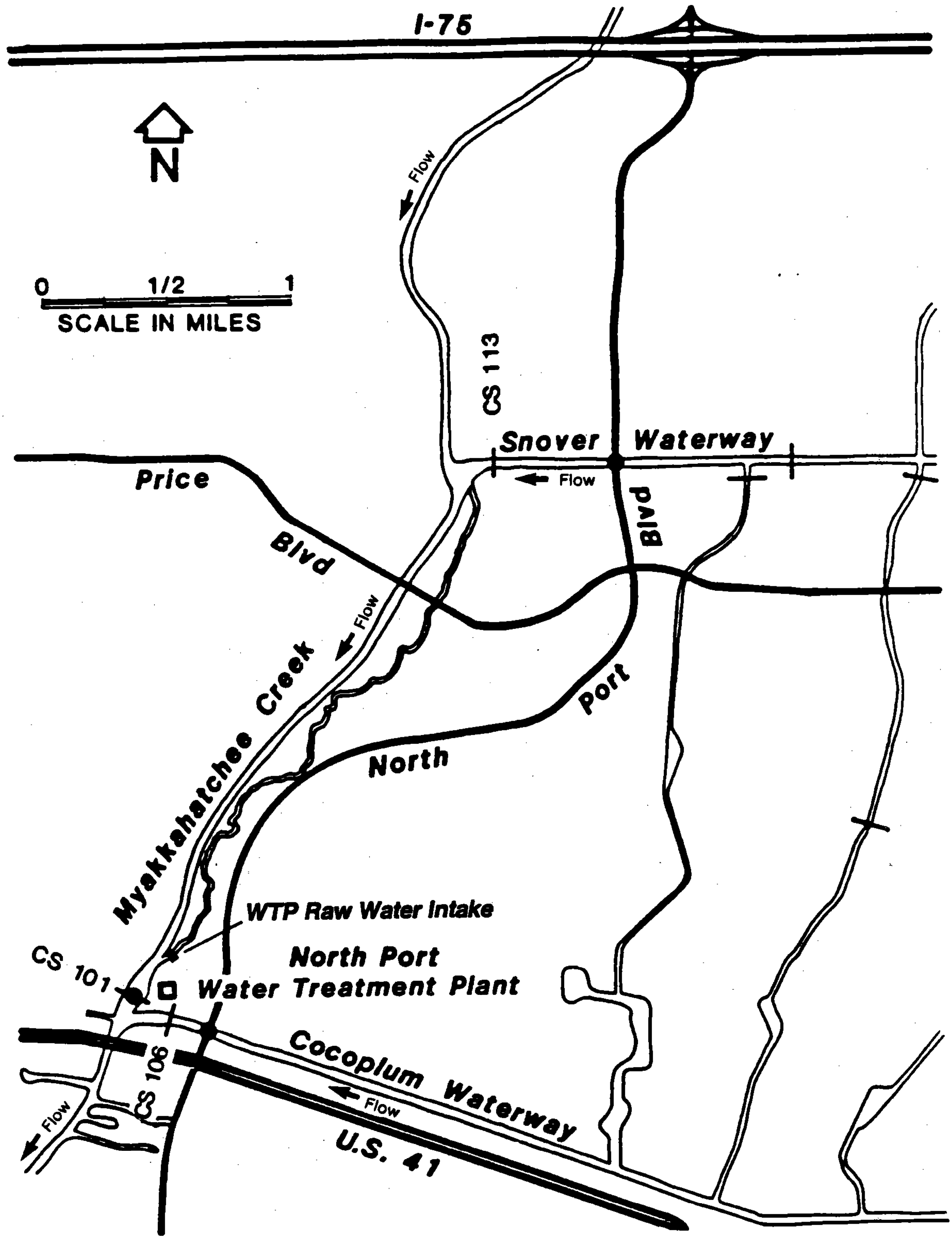

Note: Figure adapted from EQL-developed base map.

FIGURE 3-1.
Location of the North Port Water Treatment Plant.

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the North Port WTP intake (see Figure 3-1). Sixty-five observations of monthly flow were available from CS No. 101. For the Cocoplum Waterway, 64 observations of monthly flow were available from CS No. 106, located on the waterway just upstream of its confluence with Myakkahatchee Creek. Approximately 80 observations of selected water quality parameters were recorded for both stations. The available streamflow and water quality records are summarized in Tables 3-1 and 3-2 for Myakkahatchee Creek and the Cocoplum Waterway, respectively.

Flow Record Extension

The limited period of historic streamflow records available from the EQL monitoring network for Myakkahatchee Creek and Cocoplum Waterway was synthetically extended using long-term data from the hydrologically similar Horse Creek watershed to provide a broader base for analysis. A data set was developed for concurrent observations of monthly streamflow from Horse Creek, Myakkahatchee Creek, and Cocoplum Waterway. Statistical analysis indicated strong linear relationship between Horse Creek and Myakkahatchee Creek monthly flows, with a correlation coefficient of 0.93. The correlation coefficient is a measure of how well data pairs are linearly related. A correlation coefficient of 1.0 indicates a perfect linear relationship and strong linear relationships are indicated with correlation coefficients above 0.9. A moderately strong linear relationship was also exhibited between Horse Creek and Cocoplum Waterway monthly flows, with a correlation coefficient of 0.82. Based on these correlation results, the following linear regression models were developed:

- For Myakkahatchee Creek:

  $$MQm = 0.506\ MQhc - 4.5$$

  where

  MQm = Myakkahatchee Creek monthly flow, in cubic feet per second (cfs)
  MQhc = Horse Creek monthly flow, in cfs

- For Cocoplum Waterway:

  $$MQc = 0.253\ MQhc + 5.0$$

  where

  MQc = Cocoplum Waterway monthly flows, in cfs
  MQhc as defined above

Table 3-1
SUMMARY OF EQL WATER QUALITY AND FLOW DATA FOR MYAKKAHATCHEE CREEK
(1980-1986)

Variable	N	Mean	Std Dev	Minimum	Maximum
Total Dissolved Solids (mg/l)	80	364	205	48	975
Sulfate (mg/l)	26	106.8	124.9	12.3	500.0
Chloride (mg/l)	26	44.1	28.0	11.1	121.0
Fluoride (mg/l)	27	0.44	0.21	0.15	1.00
Hardness (mg/l as $CaCO_3$)	80	214	137	24	643
Color (co-pt)	80	156	100	32	380
Temperature (°C)	80	23.9	4.2	12.0	30.2
pH (pH Units)	80	6.93	0.53	5.40	9.00
Dissolved Oxygen (mg/l)	80	4.6	2.0	0.1	8.9
Turbidity (NTU)	80	4.06	5.16	0.49	32.00
Ortho-phosphate (mg/l)	80	0.236	0.144	0.025	0.998
Total Phosphate (mg/l)	12	0.20	0.08	0.05	0.29
Ammonia (mg/l)	26	0.065	0.125	0.001	0.648
Organic Nitrogen (mg/l)	26	0.05	0.06	0.00	0.24
Total Kjeldahl Nitrogen (mg/l)	13	1.43	2.00	0.31	8.00
Conductivity (μmhos)	82	513	299	70	1220
BOD_5 (mg/l)	36	1.36	1.21	0.35	8.02
Total Coliform (colonies per 100 ml)	78	603	1872	1	16000
Fecal Coliform (colonies per 100 ml)	78	111	334	1	2800
Daily Flow (cfs)	72	82	171	0	1112
Monthly Flow (cfs)	65	76	124	0	528

Table 3-2
SUMMARY OF EQL WATER QUALITY AND FLOW DATA FOR COCOPLUM WATERWAY
(1980-1986)

Variable	N	Mean	Std Dev	Minimum	Maximum
Total Dissolved Solids (mg/l)	80	533	146	111	801
Sulfate (mg/l)	26	140.7	49.3	56.5	245.0
Chloride (mg/l)	26	67.2	22.6	22.1	102.0
Fluoride (mg/l)	27	0.33	0.066	0.16	0.46
Hardness (mg/l as $CaCO_3$)	30	328	80	180	454
Color (co-pt)	9	67	38	27	130
Temperature (°C)	82	25.1	4.3	13.6	31.5
pH (pH units)	82	7.40	0.67	3.80	8.60
Dissolved Oxygen (mg/l)	82	6.5	1.7	1.6	10.2
Turbidity (NTU)	82	3.40	2.95	0.63	20.00
Ortho-phosphate (mg/l)	82	0.047	0.044	0.001	0.239
Total Phosphate (mg/l)	12	0.06	0.029	0.006	0.11
Ammonia (mg/l)	26	0.025	0.032	0.001	0.141
Organic Nitrogen (mg/l)	26	0.007	0.006	0.001	0.02
Total Kjeldahl Nitrogen (mg/l)	12	0.78	0.20	0.30	1.16
Conductivity (μmhos)	9	668	174	464	917
BOD_5 (mg/l)	35	2.40	1.30	0.58	5.50
Total Coliform (colonies per 100 ml)	9	377	583	20	1800
Fecal Coliform (colonies per 100 ml)	8	20	25	1	60
Daily Flow (cfs)	76	74	171	0	1400
Monthly Flow (cfs)	64	64	68	2	360

The R^2 values calculated for these linear flow models are 0.87 and 0.69, respectively. The R^2 value is a statistical indicator of the variation between the values predicted using the linear models and the actual observed values, and may be used as an indicator of the validity of the model. For example, the R^2 value of 0.87 for Myakkahatchee Creek indicates that 87 percent of the variation in monthly flow at Myakkahatchee Creek may be accounted for by the variation in monthly flow at Horse Creek. This indicates that these watersheds are hydrologically similar and are subject to similar monthly rainfalls.

Based on the R^2 values, these linear flow models will provide appropriate estimates of monthly flows for Myakkahatchee Creek and Cocoplum Waterway, and were used to generate flows for each month when observations were not available from the EQL data base. The resulting monthly flow array extends from 1951 through 1986, or 36 years. The extended streamflow record (from 64 and 65 months to 432 months) is given in Appendix A.

Water Quality Record Extension

TDS records were extended by developing TDS rating curves (TDS versus flow) for both monitoring stations from the EQL data base and applying these ratings to the extended streamflow record. Regression analyses of TDS as a function of the flow and the natural log (ln) of flow were used to develop the TDS rating curves. Estimated TDS concentrations were then developed for each month using the relationship:

$$TDS = e^{a}$$

where

a = A variable of flow
e = Base of the natural log

This resulted in the following equations for each waterway:

- For Myakkahatchee Creek:

$$TDSm = e^{[6.483 - 0.00167Qm - 0.217\ln(Qm)]}$$

where

TDSm = TDS concentration in mg/l for Myakkahatchee Creek corresponding to a given flow, Qm

Qm = Streamflow at Myakkahatchee Creek, in cfs o For Cocoplum Waterway:

$$TDSc = e^{[6.988 - 0.000355Qc - 0.193\ln(Qc)]}$$

where

TDSc = TDS concentration in mg/l for Cocoplum Waterway, corresponding to a given flow, Qc

Qc = Streamflow at Cocoplum Waterway, in cfs

The R^2 values between predicted and observed TDS values for these equations are 0.74 and 0.50, respectively. An effort was made to improve the correlation between flow and TDS for the Cocoplum Waterway by examining the following mathematical relationships:

$$TDSc = aQc$$
$$TDSc = aQc^b$$
$$TDSc = aQC^b + c$$

where TDSc and Qc are as defined earlier and a, b, and c are statistically determined constants.

A two-part model that applied each of these mathematical relationships to different flow ranges was also analyzed. None of the results produced an R^2 value equal to or greater than the 0.50 previously determined. A large percentage of base flow attributable to groundwater in the Cocoplum Waterway may be the cause of the relatively large unexplained variance.

The above equations provide the best available estimates of TDS concentration and may be used to extend the water quality record. The resulting TDS values for the entire 36 years of extended record are reported in Appendix A.

WATER SUPPLY CHARACTERISTICS

Streamflow Characteristics

Based on the extended flow data, the mean annual and minimum average monthly flow rates for the Myakkahatchee Creek are 91.4 cfs (59.1 mgd) and 11.0 cfs (7.1 mgd), respectively; for the Cocoplum Waterway, those values are 55.3 cfs (35.7 mgd) and 13.0 cfs (8.4 mgd). While the flow rates suggest a potential for increased development of these sources for water supply, monthly flows for both waterbodies vary widely, as can be seen in Table 3-3 and Figure 3-2. This fluctuation may greatly limit supply reliability. As shown in Figure 3-3, Myakkahatchee Creek can experience

Table 3-3
MONTHLY AVERAGE STREAMFLOWS FOR
MYAKKAHATCHEE CREEK AND
COCOPLUM WATERWAY

Month	Average Monthly Flow (cfs) Myakkahatchee Creek	Cocoplum Waterway	Total
OCT	140.0	83.0	223.0
NOV	33.0	26.0	59.0
DEC	24.0	21.0	45.0
JAN	42.0	31.0	73.0
FEB	54.0	35.0	89.0
MAR	72.0	43.0	115.0
APR	27.0	21.0	48.0
MAY	11.0	13.0	24.0
JUN	84.0	53.0	137.0
JUL	152.0	84.0	235.0
AUG	205.0	111.0	315.0
SEP	253.0	142.0	396.0
MEAN	91.4	55.3	146.6

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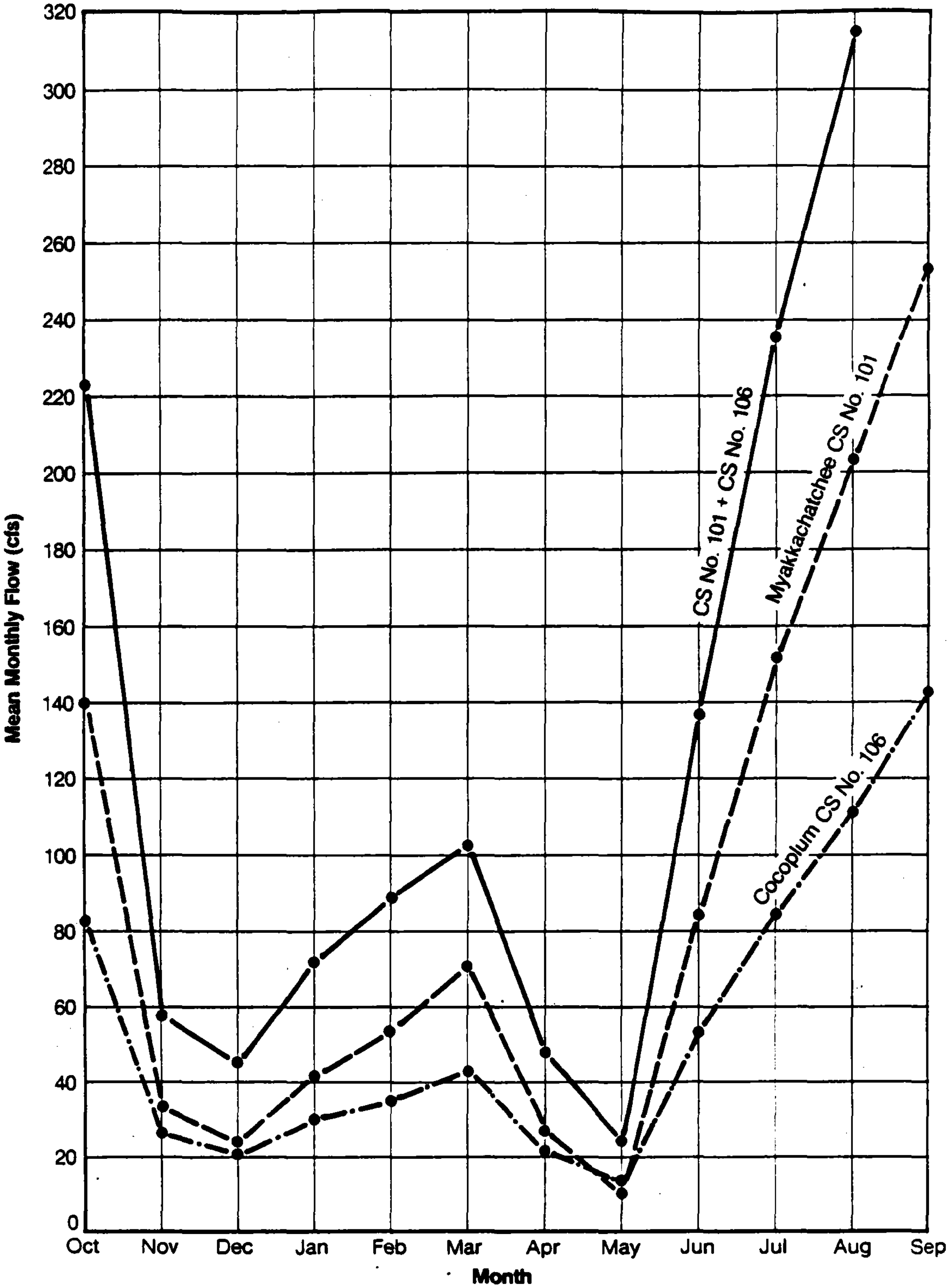

Note: Based on Extended Streamflow Record for the Years 1951 through 1986.

FIGURE 3-2.
Mean Monthly Flow for Myakkahatchee Creek and Cocoplum Waterway.

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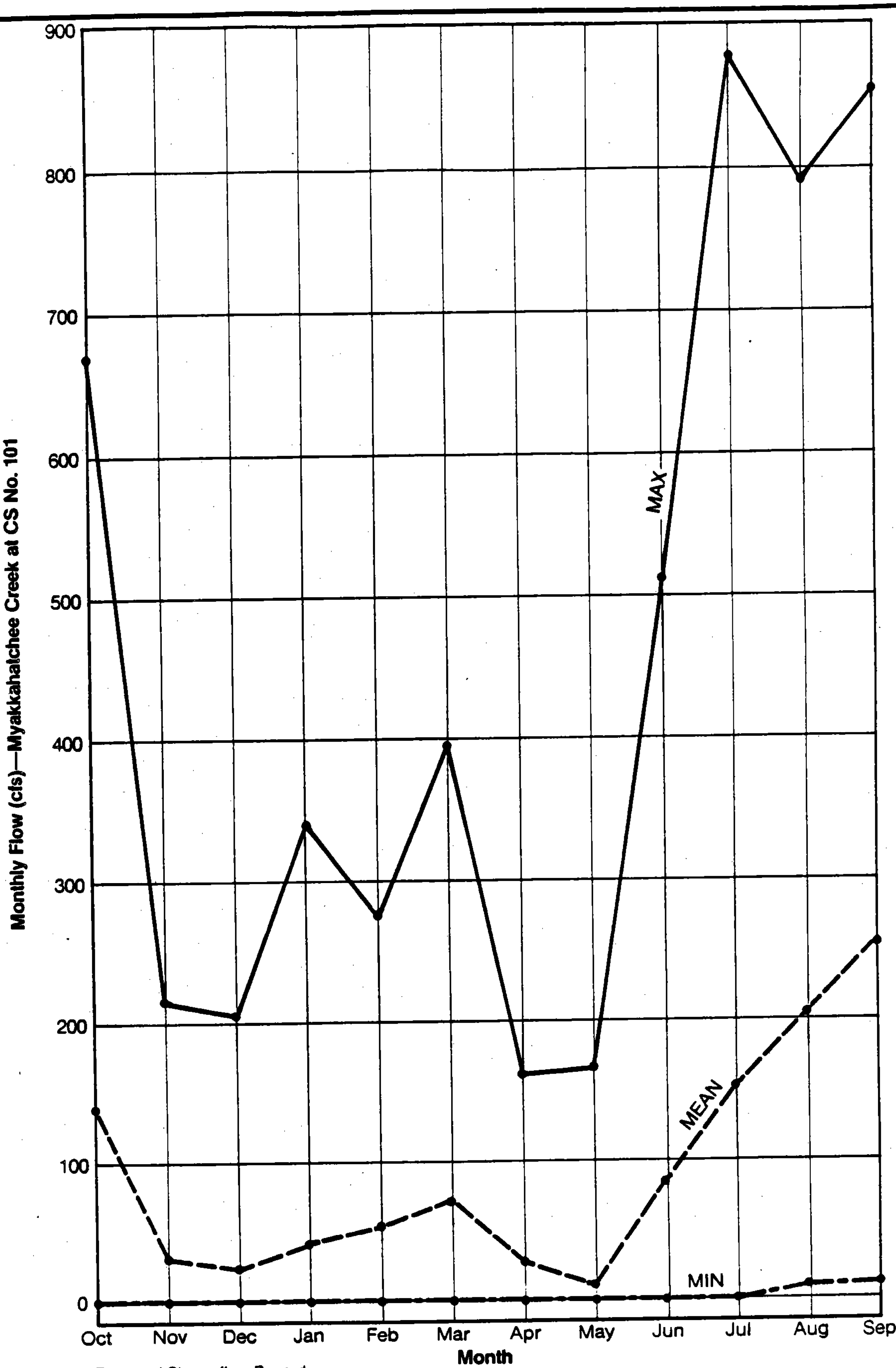

Note: Based on Extended Streamflow Record for the Years 1951 through 1986.

FIGURE 3-3.
Monthly Flow Variability of Myakkahatchee Creek.

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no-flow conditions for several months throughout the year, while the base flow in the Cocoplum Waterway, as shown in Figure 3-4, is approximately 5 cfs (3.2 mgd). As a result, significant storage capacity would be required for development of a reliable water supply using these sources.

Water Quality Characteristics

The monthly average and flow-weighted mean annual TDS values for Myakkahatchee Creek and Cocoplum Waterway are shown in Table 3-4. These values were obtained from the extended flow records and TDS rating curves described previously. For Myakkahatchee Creek, 6 out of 12 monthly averages exceed 400 mg/l; every monthly average exceeds 400 mg/l for Cocoplum Waterway. The flow-weighted average exceeds 400 mg/l for 8 out of 12 months. Based on the correlation between TDS and other water quality parameters reported in the EQL data base (Tables 3-1 and 3-2), TDS values will exceed 400 mg/l before any other reported water quality parameter exceeds its secondary drinking water standard.

As noted earlier, the current treatment process at the North Port WTP may be expected to add about 100 mg/l of TDS, which would produce a finished water that frequently exceeds the 500 mg/l secondary drinking water standard for TDS. The Florida drinking water standards, defined in Chapter 17-22 of the Florida Administrative Code (FAC), state that TDS levels exceeding 500 mg/l may be allowed if no other standard is exceeded. However, because the finished water TDS goal for this evaluation is 500 mg/l, TDS is considered the controlling water quality indicator.

Although the TDS standard is the most difficult to meet consistently, other parameters including sodium and sulfate should be monitored closely in the raw and/or finished water. The raw water also has high total hardness during the dry season, and softening at the North Port WTP is limited by the amount of sodium hydroxide that can be added without exceeding the sodium standard. This limitation is discussed in detail in Section 6.

The flow-weighted mean annual TDS values of 266 mg/l and 513 mg/l for Myakkahatchee Creek and Cocoplum Waterway, respectively, indicate that without treatment for TDS reduction, only Myakkahatchee Creek has realistic potential for increased development as a water supply source. GDU recently investigated the possibility of high TDS point sources of water entering the Cocoplum Waterway from artesian groundwater. No evidence of this was found, however, and investigations have been discontinued.

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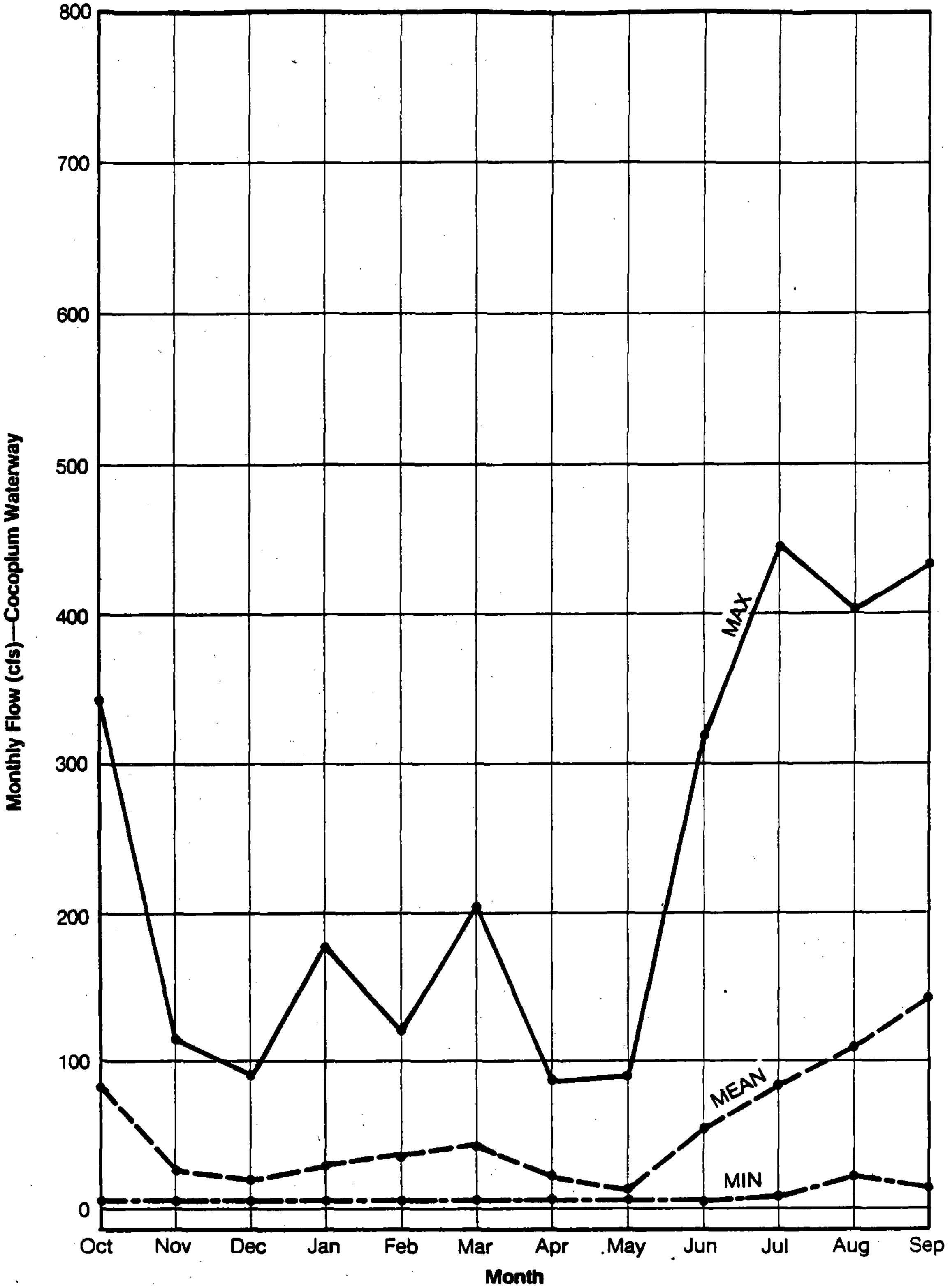

Note: Based on Extended Streamflow Record for the Years 1951 through 1986.

FIGURE 3-4.
Monthly Flow Variability of Cocoplum Waterway.

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Table 3-4
MONTHLY AVERAGE TDS VALUES FOR
MYAKKAHATCHEE CREEK AND COCOPLUM WATERWAY

Month	Average Monthly TDS (mg/l) Myakkahatchee Creek	Average Monthly TDS (mg/l) Cocoplum Waterway	Total Flow-Weighted Average TDS (mg/l)
OCT	240	485	343
NOV	405	613	513
DEC	467	653	566
JAN	450	628	539
FEB	393	610	492
MAR	409	608	501
APR	465	642	548
MAY	601	697	657
JUN	404	598	493
JUL	237	489	341
AUG	171	440	272
SEP	160	418	260
FLOW-WEIGHTED MEAN	266	513	370

To evaluate the reliability of the Myakkahatchee Creek as a water supply source, the cumulative frequency of consecutive months with no flow or consecutive months with TDS values of 400 mg/l or greater was developed as shown in Figure 3-5. The data indicate that a reliable water supply (with respect to water quality) will require up to 10 months of offstream raw water storage capacity.

CONCLUSIONS

The mean annual flows from Myakkahatchee Creek and Cocoplum Waterway of 91.4 cfs (59.1 mgd) and 55.3 cfs (35.7 mgd), respectively, indicate a potential for increased development of this water supply if adequate storage and treatment capacity are provided. The monthly flows from both sources are highly variable and no-flow conditions may be expected for several months throughout the year. As a result, a water supply developed from these sources would require significant offline surface storage or ASR capacity.

The flow-weighted mean annual TDS values are 266 mg/l and 513 mg/l for Myakkahatchee Creek and Cocoplum Waterway, respectively. Without additional treatment for TDS reduction or elimination of possible high salinity point sources, the Cocoplum Waterway will be an unsuitable water supply source. It must be noted that, since the completion of this evaluation, the Cocoplum Waterway has been eliminated as a potential water supply source as a result of the pine forest effluent spray system proceedings with the City of North Port.

Based on the extended flow and quality data, about 10 months of storage capacity will be required to supply the required volume of water from the Myakkahatchee Creek at a TDS level under 400 mg/l. The potential for providing this storage, and its impact on water cost, is discussed in Section 5.

PEACE RIVER

The potential of the Peace River as a water supply source for the Port Charlotte service area was evaluated with the PEACE model developed by CH2M HILL for a GDU study in 1985. Input to the model was updated with additional flow and water quality data and a new diversion formula was used as the basis for analysis.

PEACE MODEL

In 1985, CH2M HILL completed a report for GDU on the feasibility of using ASR at the Peace River WTP (CH2M HILL, April 1985a). As part of the 1985 analysis, two computer programs (PEACE and PLANT) were developed to simulate Peace River

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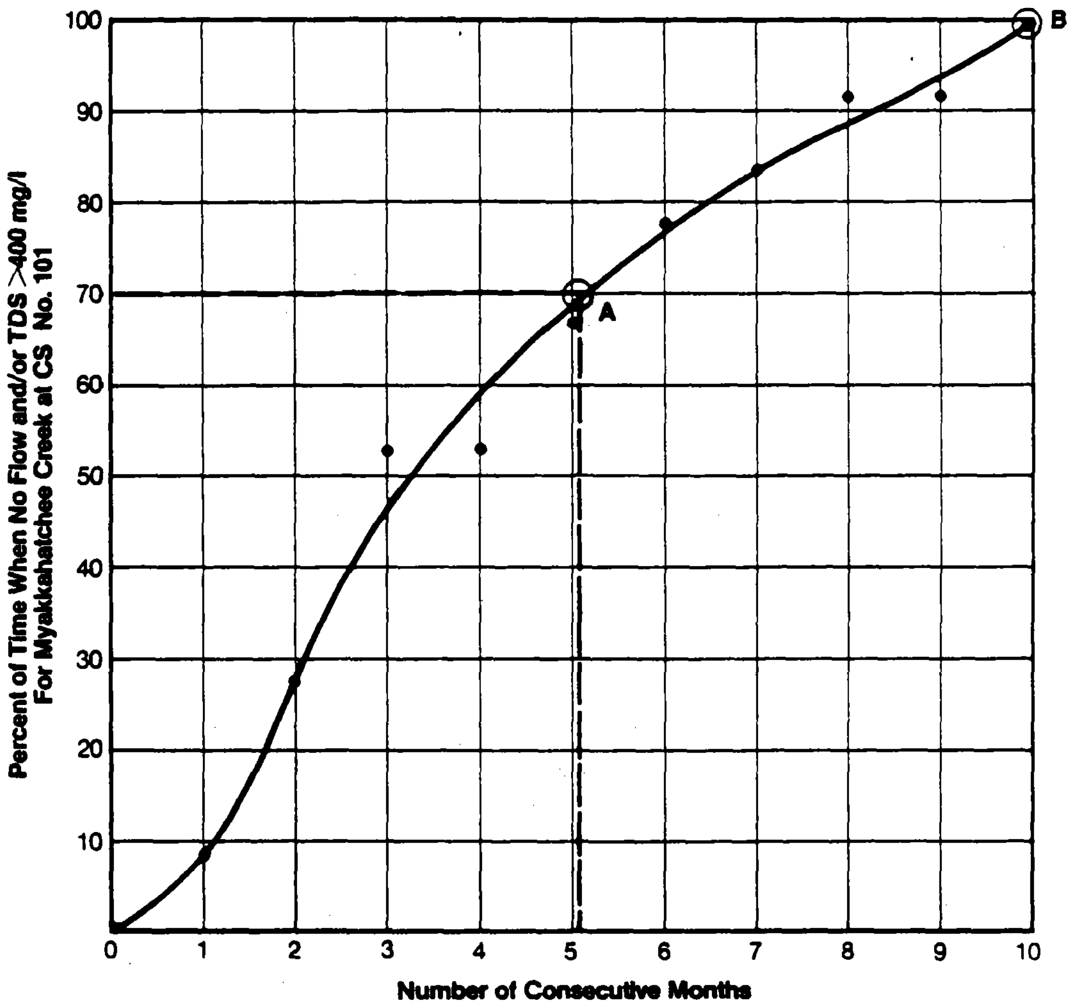

Example: Point A indicates that 5 or fewer consecutive months with no divertible flow will be experienced in 7 years out of 10 (70%). Conversely, 5 or more consecutive months with no divertible flow will be experienced in 3 years out of 10 (30%). Point B indicates that nearly 100% of the time 10 or fewer consecutive months will occur with no divertible flow.

FIGURE 3-5.
Cumulative Frequency of Consecutive Months with No Flow or with TDS Greater than or Equal to 400 mg/l at CS No. 101.

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flows and the operation of the Peace River WTP and associated facilities on a monthly basis. The programs were used to estimate the reliability of different combinations of plant facilities under various environmental and operational constraints. These programs were fully documented as part of the 1985 ASR feasibility project (CH2M HILL, April 1985b).

PEACE uses the historical record of Peace River flows and/or generates synthetic flow data based on historical flow patterns. The model considers the following factors in estimating the available raw water supply from the river:

- Historic or synthetic streamflow
- Water quality (TDS)
- River water algae content
- Regulatory diversion constraints of Peace River water for the Peace River WTP
- Capacity of raw water intake facilities

PEACE generates monthly flow and water quality information using available data for the lower Peace River basin. The locations of the monitoring stations used in the model are shown in Figure 3-6, which also identifies the location of the Peace River WTP. Output from PEACE includes monthly divertible flow and quality data that can be used as input to the PLANT model.

INPUT DATA FOR MODEL ANALYSIS

Streamflow Records

Streamflow records used in the 1985 ASR study covered the water years (October through September) 1932 through 1982; water years 1983 through 1986 have been added for this evaluation. The flow available at the intake structure (Station 18 in Figure 3-6) is calculated as the sum of flows at Arcadia (Station 23), Joshua Creek (Station JC), and Horse Creek (Station 21), increased by a factor to account for additional drainage area downstream of Horse Creek. In the 1985 study, this factor was estimated at 1.04. Table 3-5 lists the historical monthly average flows at the intake structure for each year, and Table 3-6 gives the statistics (mean, standard deviation, correlation coefficient, etc.) of those flows. Table 3-7 gives the same statistics for the natural logs (ln) of the flows in Table 3-5.

The PEACE program uses either the monthly streamflow statistical data, in the form of logarithms, or the historic streamflow records as input. This evaluation is based on analysis of the 55 years (1932 through 1986) of observed streamflow records using the PEACE program.

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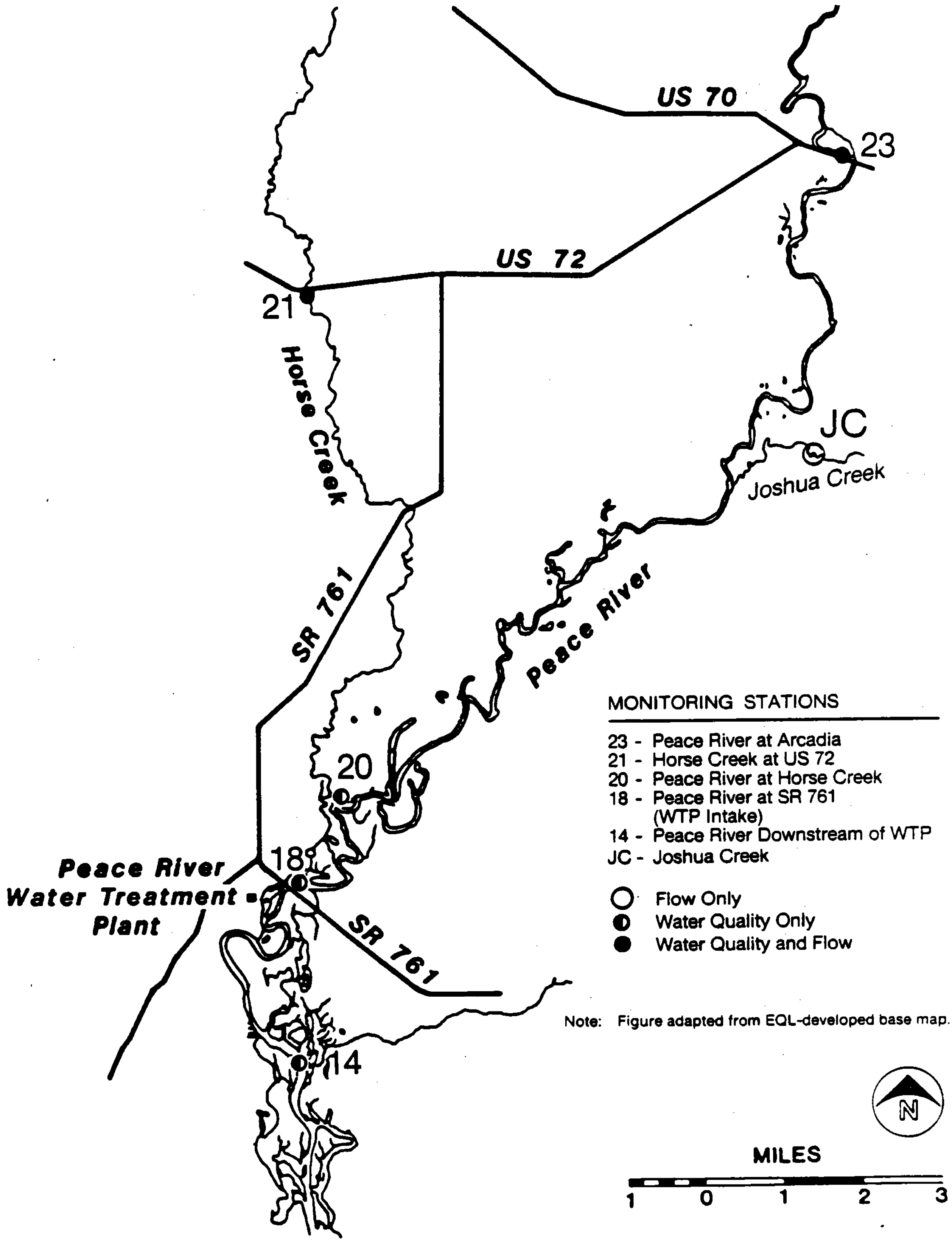

FIGURE 3-6.
Location of Peace River Water Treatment Plant and River Monitoring Stations.

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TABLE 3-5

1932-1986 PEACE RIVER HISTORICAL FLOWS AT WTP INTAKE (CFS)

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	AVERAGE
1932	763	144	95	111	98	114	76	169	685	237	709	3868	589
1933	456	172	126	122	150	128	252	124	677	2809	2050	17355	2035
1934	1387	500	283	221	244	374	511	1052	6398	3382	2895	3212	1705
1935	993	237	170	154	133	100	105	107	108	1135	1145	7219	967
1936	1578	357	371	1397	7269	2640	626	224	1086	1139	1121	2022	1653
1937	2152	490	292	221	926	615	2847	1096	542	1954	1511	1927	1214
1938	1279	696	1494	654	303	208	101	92	1009	2911	1510	475	894
1939	5054	1056	359	252	218	104	126	124	7073	7296	8996	8903	3297
1940	1975	578	266	669	1776	1148	897	178	326	2637	1963	3819	1353
1941	1349	163	230	879	980	634	2090	276	528	4694	1526	1874	1269
1942	536	770	765	2035	1797	3177	1052	534	4538	3847	745	1164	1747
1943	408	128	178	183	170	389	149	179	1158	7146	5736	4673	1708
1944	2467	364	253	229	171	248	152	124	322	702	2464	1457	746
1945	784	530	224	483	241	120	91	65	2006	10571	4922	5173	2101
1946	2977	764	524	519	379	610	146	164	260	2699	3680	2661	1282
1947	1352	490	248	201	530	2495	2159	542	6424	5803	7182	16069	3624
1948	4772	1571	1205	2122	1840	825	556	264	198	1215	5220	7896	2307
1949	8096	679	508	358	271	179	171	92	951	1761	9948	7968	2582
1950	5736	852	454	462	293	211	159	168	179	201	391	1844	912
1951	2483	619	504	392	464	243	2083	471	195	2528	3063	1856	1242
1952	3562	897	561	354	596	1040	710	268	323	890	1310	1457	997
1953	7301	2112	794	1245	1622	729	1194	225	3878	2995	5348	10066	3126
1954	9418	2763	3723	1851	852	664	692	953	4013	4043	1929	2702	2800
1955	1707	695	608	541	662	283	269	142	230	675	1340	3798	913
1956	538	225	208	213	235	134	86	285	145	274	824	2490	471
1957	2215	501	238	310	339	1717	1651	3266	1712	2346	3632	4791	1893
1958	3721	442	551	3755	1983	4497	3013	1647	719	1911	1143	1165	2046
1959	524	557	559	916	705	5007	2165	932	5917	6359	5399	8917	3163
1960	3736	1386	672	531	1797	3424	2003	699	805	3475	9771	11782	3340
1961	6202	1244	489	755	1105	708	550	284	444	1109	1105	1329	1277
1962	201	127	125	175	180	192	360	142	1985	1689	2181	7983	1278
1963	1302	575	303	368	1774	1944	255	317	1796	1531	1600	2126	1157
1964	678	985	694	1695	2838	1204	790	737	229	549	1199	3370	1247
1965	601	245	324	263	429	1257	259	102	751	4343	4001	1006	1132
1966	2523	517	322	1081	3152	1813	488	213	1824	1846	4152	2283	1685
1967	1554	271	208	195	353	233	137	114	743	1531	4560	2173	1006
1968	1667	291	340	274	260	244	131	196	4843	7827	1582	3117	1731
1969	1250	1495	390	739	535	3285	636	313	1618	1198	2887	2953	1442
1970	3439	1773	1879	2403	1109	3400	1462	365	1305	2197	1740	1721	1899
1971	777	277	220	219	481	260	158	164	289	665	1766	4057	778
1972	1758	633	697	290	1283	366	612	290	1374	772	1404	1065	879
1973	384	257	587	2042	1795	842	2091	245	263	1376	2423	3505	1318
1974	894	271	271	253	231	154	103	118	1447	7273	4254	1747	1418
1975	345	207	238	227	180	130	70	109	481	1102	1695	3046	653
1976	2020	1186	286	225	178	174	135	286	1328	2015	2118	1847	983
1977	812	289	336	428	351	254	91	91	217	886	797	2240	566
1978	560	316	1102	1007	1610	1695	217	490	898	3555	4562	736	1398
1979	356	187	213	1532	817	936	169	1329	716	1010	1785	6064	1260
1980	3884	386	476	496	876	722	1461	574	603	618	584	717	950
1981	202	178	230	172	707	202	87	74	307	121	854	2541	473
1982	280	122	120	142	245	392	461	606	9467	4543	2791	3519	1891
1983	2875	619	321	494	4159	4429	2528	347	781	1310	2245	3258	1947
1984	1147	461	1259	1422	898	1743	477	209	384	1664	1644	383	974
1985	163	134	112	103	111	108	110	76	158	381	1388	2359	434
1986	1001	403	169	254	416	924	238	85	642	1327	2295	1709	789

Table 3-6
1932-1986 PEACE RIVER HISTORICAL FLOWS AT WTP INTAKE
FLOW STATISTICS (cfs)

	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	AVERAGE
N	55	55	55	55	55	55	55	55	55	55	55	55	55
MEAN	2113	622	512	702	966	1085	731	406	1587	2547	2820	3917	1501
STD DEV	2091	534	567	734	1208	1266	818	520	2079	2277	2248	3611	782
COVAR	0.99	0.86	1.11	1.05	1.25	1.17	1.12	1.28	1.31	0.89	0.80	0.92	0.52
SKEW	1.75	1.98	3.89	2.03	3.22	1.67	1.39	3.70	2.16	1.59	1.71	2.10	1.07
CORR	0.497	0.743	0.728	0.516	0.453	0.516	0.626	0.489	0.180	0.571	0.500	0.563	
RCOEF	0.288	0.190	0.773	0.669	0.744	0.540	0.404	0.311	0.718	0.626	0.493	0.905	

NOTE: N = Number of observations
MEAN = Arithmetic average
STD DEV = Standard deviation
COVAR = Covariance = STD DEV / MEAN
SKEW = Skew coefficient
CORR = Correlation coefficient between each month and the previous month
RCOEF = Regression coefficient used in generating synthetic flows

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Table 3-7
1932-1986 PEACE RIVER HISTORICAL FLOWS AT WTP INTAKE
NATURAL LOG FLOW STATISTICS (cfs)

	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	AVERAGE
N	55	55	55	55	55	55	55	55	55	55	55	55	55
MEAN	7.20	6.13	5.92	6.13	6.35	6.34	5.96	5.56	6.71	7.44	7.68	7.94	6.61
STD DEV	1.00	0.78	0.74	0.91	1.01	1.17	1.16	0.88	1.13	0.98	0.73	0.82	0.56
COVAR	0.14	0.13	0.12	0.15	0.16	0.18	0.19	0.16	0.17	0.13	0.10	0.10	0.08
SKEW	-0.10	0.18	0.69	0.46	0.34	0.24	0.27	0.75	0.38	-0.49	0.15	0.05	-0.19
CORR	0.736	0.769	0.744	0.742	0.751	0.770	0.736	0.688	0.313	0.657	0.655	0.476	
RCOEF	0.893	0.597	0.709	0.909	0.836	0.893	0.729	0.526	0.402	0.565	0.493	0.535	

NOTE: N = Number of observations
MEAN = Arithmetic average
STD DEV = Standard deviation
COVAR = Covariance = STD DEV / MEAN
SKEW = Skew coefficient
CORR = Correlation coefficient between each month and the previous month
RCOEF = Regression coefficient used in generating synthetic flows

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TDS Rating Curve

Based on expected treated water quality, the usual limiting water quality parameter in the Peace River is TDS; that is, Peace River WTP finished water TDS would be expected to exceed the current drinking water standard before other constituents. Occasionally, however, the drinking water standard for fluoride may be exceeded before that of TDS.

The PEACE model requires estimates of TDS concentrations of river flow. These data are not available for the entire period analyzed and must be estimated from available TDS measurements. A relationship between flow and TDS in the river may be logically assumed: when flow is high, the effects of dilution with rain water and surface runoff produce lower TDS concentrations; when flow is low, TDS will be higher. A more precise definition of this correlation is necessary to develop a mathematical relationship between the amount of streamflow and its TDS level (water quality).

In the 1985 study, available TDS and flow data for the Arcadia gage (Station 23 in Figure 3-6) were used to define the relationship. For this study, CH2M HILL used 133 pairs of TDS and daily flow data collected by EQL at the SR 761 bridge (Station 18) between 1975 and 1986. These data are considered to be more representative of conditions at the Peace River WTP intake than those taken at Arcadia. Overall water quality conditions at the SR 761 bridge are summarized in Table 3-8, which was compiled by the EQL.

The mathematical relationship of TDS versus streamflow was developed from the collected data. It has the following general form:

$$TDS = a(FLOW)^{b}$$

where

a and b are constants
TDS is in mg/l
FLOW is in cfs

The constants a and b were determined by linear regression methods using the natural logs of TDS versus the natural logs of flow (Figure 3-7). The five TDS values shown in the upper left-hand corner of Figure 3-7 (out of 133 historical data pairs) were not used in the analysis because they were not representative of expected TDS values during periods of raw water diversion. The intent of the regression is to be able to estimate TDS at higher flows when diversion occurs; including the five low flow data points would skew estimates at larger flows on the high side. The general form of the linear equation is:

Table 3-8
SUMMARY OF EQL WATER QUALITY DATA FOR
THE PEACE RIVER AT SR 761 BRIDGE
(1976-1986)

Water Quality Parameter	N	Mean	Std Dev	Minimum	Maximum
Ammonia (mg/l)	97	0.059	0.052	0.001	0.376
Organic Nitrogen (mg/l)	97	1.11	0.47	0.34	3.34
Total Kjeldahl Nitrogen (mg/l)	97	1.17	0.46	0.41	3.35
Nitrate plus Nitrite (mg/l)	156	0.547	0.422	0.001	2.110
Total Nitrogen (mg/l)	156	1.74	0.69	0.07	4.99
Ortho-phosphate (mg/l)	156	1.697	0.908	0.590	4.680
Total Phosphate (mg/l)	97	1.75	0.98	0.57	4.79
Silica (mg/l)	157	2.52	1.17	0.30	5.87
Total Organic Carbon (mg/l)	154	28.7	10.5	4.2	59.9
Dissolved Organic Carbon (mg/l)	92	27.2	9.7	7.6	51.5
Inorganic Carbon (mg/l)	150	9.6	4.2	1.0	22.5
Color (co-pt units)	157	141	100	12	410
Turbidity (NTU)	157	4.49	4.48	0.21	37.00
Chlorophyll _a_ (mg/l)	112	14.7	25.4	0.1	156.0
pH (pH units)	130	7.24	0.48	5.75	9.01
TDS (mg/l)	143	281	245	99	2550
Sulfate (mg/l)	150	76.0	42.5	8.0	238.0
Chloride (mg/l)	149	41.8	116.5	3.5	1220.0
Fluoride (mg/l)	153	1.08	0.5	0.15	2.56
Iron (mg/l)	93	0.17	0.17	0.01	0.75
Calcium (mg/l)	151	33.00	12.90	6.64	71.6
Magnesium (mg/l)	151	14.10	10.10	1.77	97.60
Hardness(mg/l as $CaCO_3$)	138	140	68	25	581
Alkalinity (mg/l as $CaCO_3$)	148	53	17	14	90
Total Coliform (colonies per 100 ml)	74	494	576	5	3100
Fecal Coliform (colonies per 100 ml)	76	93	170	3	1300
Fecal Strep (colonies per 100 ml)	76	306	449	20	2900

N = Number of observations
Std Dev = Standard deviation

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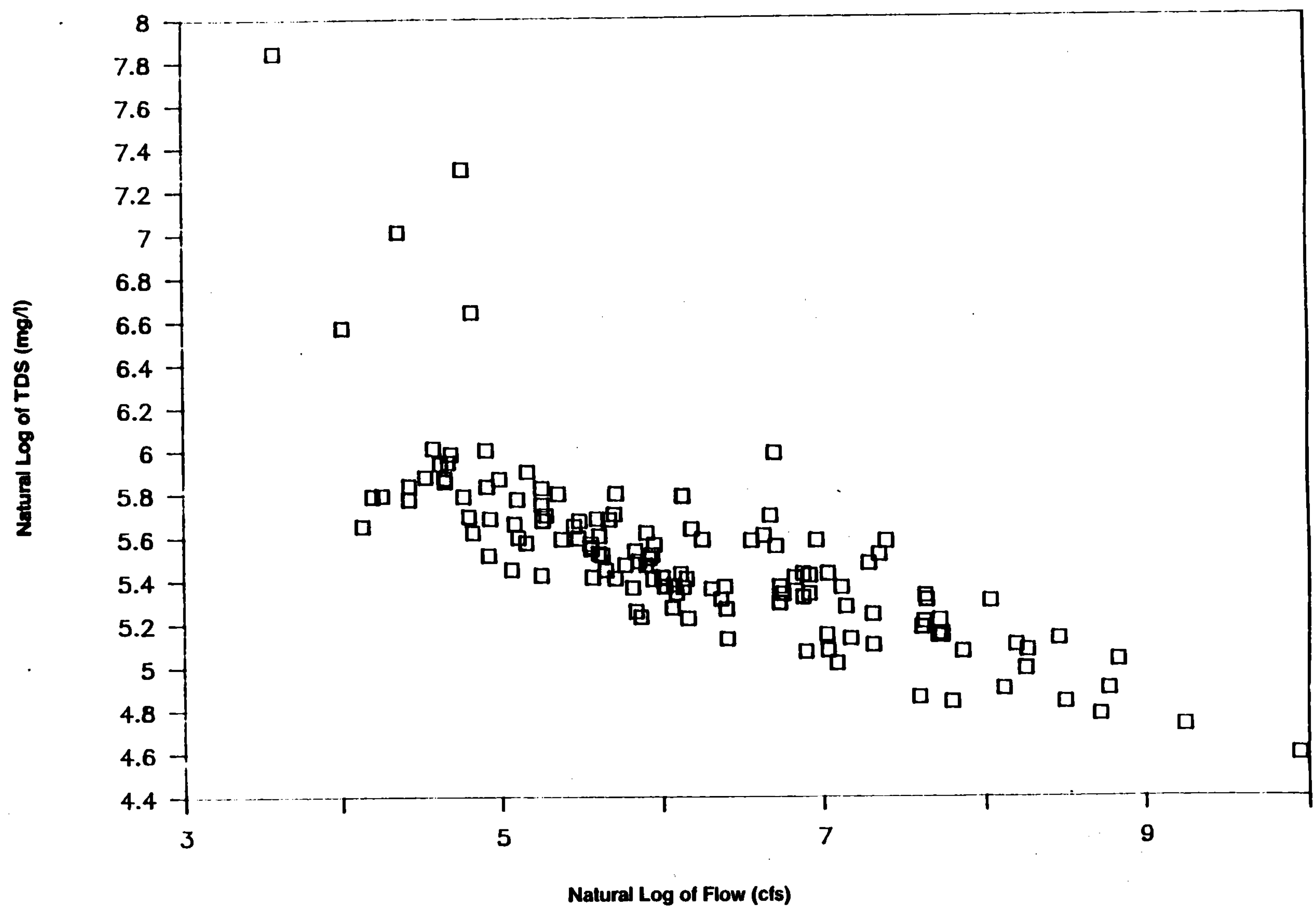

FIGURE 3-7.
Log TDS vs. Log Flow at Peace River Station 18 (SR 761 Bridge).

$$\ln (TDS) = \ln a + b \ln (FLOW)$$

The regression analysis results in estimates of ln a (the y-intercept of the straight line) and b (slope of the straight line). These values are 6.752 and -0.2077, respectively. The resulting non-linear TDS equation is:

$$TDS = 856\ (FLOW)^{-0.2077}$$

Figure 3-8 shows TDS versus flow, with the equation plotted as a solid line. The plot illustrates the correlation between the empirical equation and the actual data. Figure 3-9 presents TDS values predicted with the model versus observed TDS. The predicted relationship between TDS and streamflow is not totally accurate, because factors other than flow affect TDS at Station 18. For example, sustained winds in the upstream direction and spring high tides may cause high TDS concentrations even when flows are not low. Calculation of the R^2 value of the relationship provides a statistical measure of the applicability of the TDS versus flow equation. In this case, the R^2 value was 0.702, which means that approximately 70 percent of the variance in TDS values can be explained by the variance in flow.

Algae Bloom Probabilities

The monthly probability that diversion of water will be prevented by high algal content in the Peace River was determined in the 1985 report on ASR feasibility. The average probability was 11.4 percent, with the highest algal content reported from March through July. Because additional algae-related operational data were not available, these values remain unchanged and the same data, shown below, were used for the current PEACE model analysis.

Month		Probability of No Diversion*
January		0.000
February		0.063
March		0.218
April		0.333
May		0.218
June		0.200
July		0.204
August		0.129
September		0.000
October		0.000
November		0.000
December		0.000
	Average	0.114

*Analysis based on data from 1981-1984, Peace River WTP (CH2M HILL, April 1985a).

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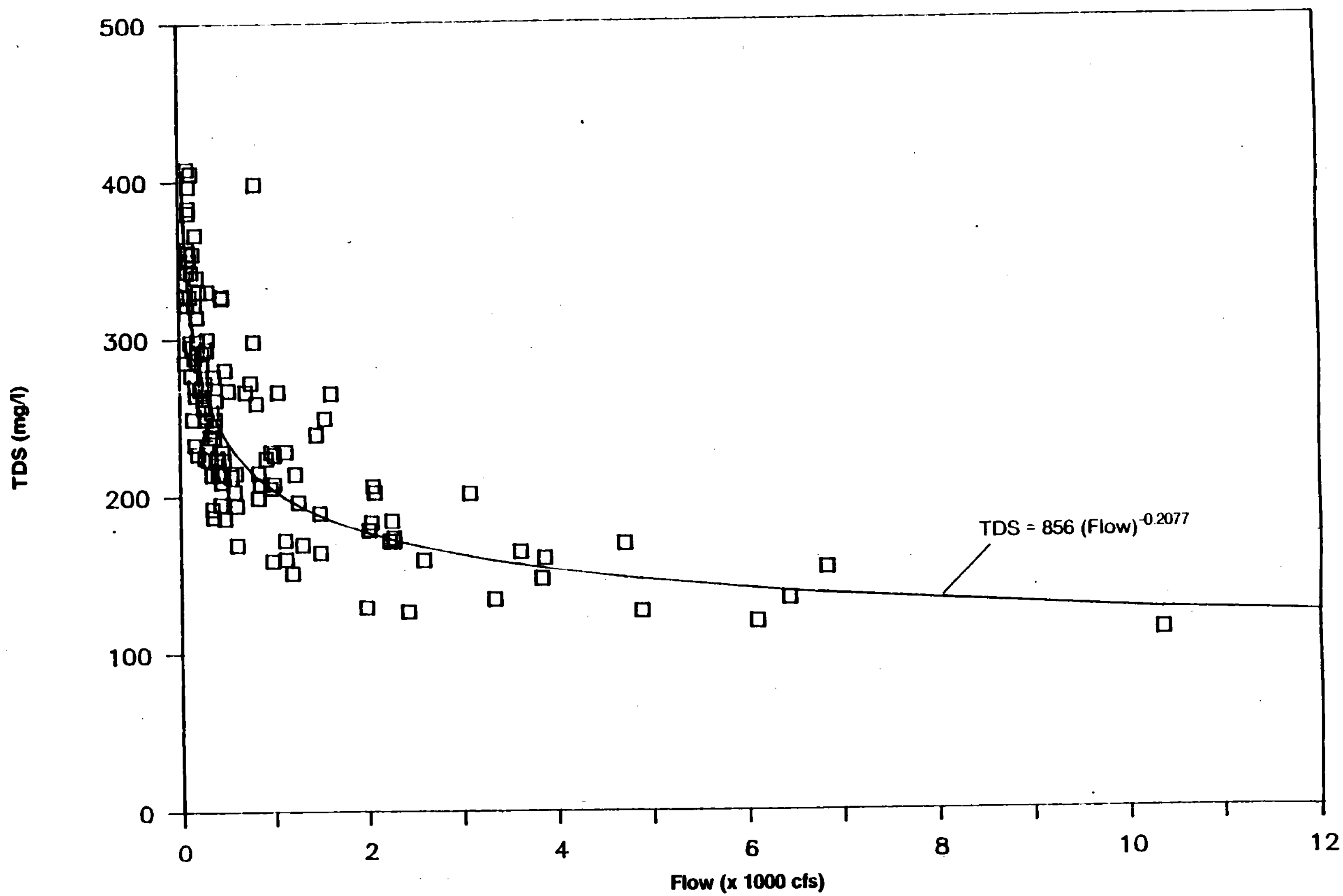

FIGURE 3-8.
TDS vs. Flow at Peace River Station 18 (SR 761 Bridge).

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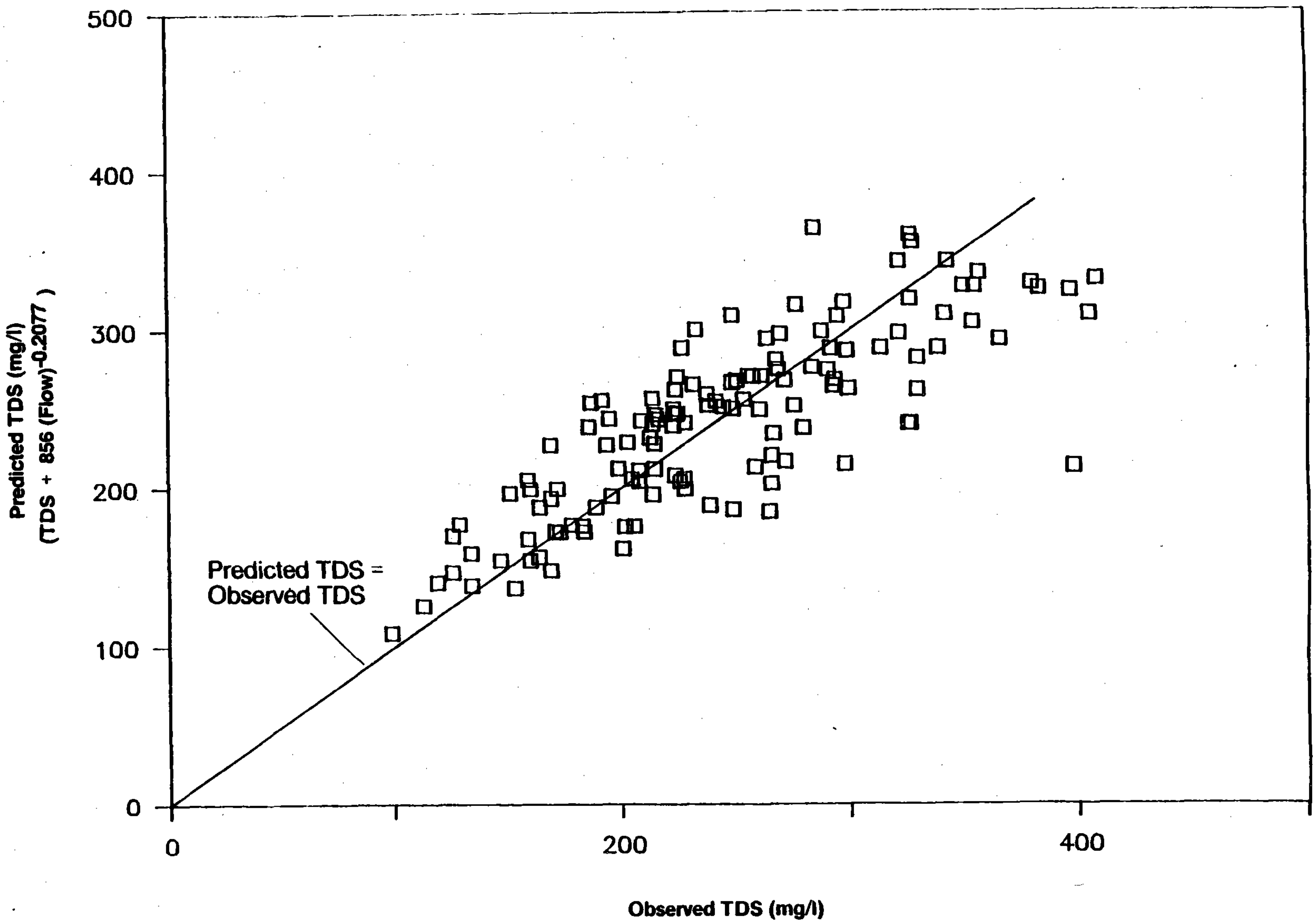

FIGURE 3-9.
Predicted vs. Observed TDS at Peace River Station 18 (SR 761 Bridge).

Diversion Rule

Currently, withdrawals from the Peace River for the Peace River WTP are allowed only when the flow at Arcadia is above the following minimum values set by the SWFWMD in CUP No. 202923:

October	278 cfs
November	158 cfs
December	149 cfs
January	149 cfs
February	153 cfs
March	123 cfs
April	100 cfs
May	100 cfs
June	178 cfs
July	356 cfs
August	520 cfs
September	664 cfs

Above these flows, water may be withdrawn up to 34.1 cfs (22 mgd), the current intake structure capacity. The minimum values are based on monthly average flows, and do not consider the specific environmental effects of withdrawals.

An analysis of water quality data from 1975 to 1986 by the EQL has shown that even at some of the high flows listed above, unlimited withdrawals from the river may be detrimental to the environment. On the other hand, EQL data analyses indicate diversion of some of the river flow to the Peace River WTP when flows are below the SWFWMD minimum river flow values could be allowed without measurable environmental impact. Therefore, an alternative diversion rule has been developed, based on the EQL technical memorandum dated February 1987, which allows withdrawal in proportion to the actual flow. This new diversion rule is submitted through this report for review and approval by the SWFWMD.

The alternative diversion rule is structured to allow withdrawal from the Peace River as a function of the actual flow, with stricter withdrawal limitations placed on extremely low flows. The following guidelines apply:

Peace River Flow at Arcadia	Allowed Withdrawal
Less than 100 cfs	None
100 to 250 cfs	10% of river flow
Greater than 250 cfs	15% of river flow

The low flow limit of 100 cfs was based on discussion presented in the EQL memorandum, which indicated that at flows less than or equal to 100 cfs, the saltwater interface would be within one mile of the intake, making the quality of water at these flow rates questionable. In addition, withdrawal during low flows (less than 100 cfs) could cause adverse environmental impacts. The 10 and 15 percent withdrawal rates used in the alternative diversion rule were also suggested in the text of the EQL memorandum as considering overall environmental impacts including possible changes to riparian vegetation and movement of the saltwater/freshwater interface in the estuary.

The alternative diversion rule defined above was used to evaluate the water supply potential of the Peace River. The river flow was referenced to the Arcadia gage (Station 23), because all of the statistical analyses developed by the EQL were based on streamflow measured at Arcadia and the current minimum river flow values in the regulations are referenced to the Arcadia gage. The average flows at the intake structure are slightly higher than those at Arcadia, because inflow from Horse Creek, Joshua Creek, and the ungaged area downstream of Horse Creek is not accounted for when Arcadia flows are used.

As previously discussed, the PEACE model was designed to generate monthly flows available for diversion at the Peace River WTP intake (Station 18). To obtain flows for Arcadia (Station 23), an equation was developed to relate the two flows so that one can be used to predict the other. This was done by statistically comparing the data for the Arcadia Station (Station 23) with the sum of the flow data from Arcadia, Horse Creek (Station 21), and Joshua Creek (Station JC), increased by a factor of 1.04 to account for the additional drainage area between stations. This relationship is expressed in the following regression equation:

$$\text{ARCADIA FLOW} = 1.261\ (\text{DIVERSION FLOW})^{0.9328}$$

The relative validity of the equation was examined statistically to determine its ability to predict one flow based on the other. In this case, the statistical R^2 value was 0.995, which means that 99.5 percent of the variance in the flow at Station 18 (DIVERSION FLOW) can be predicted by the flow at Arcadia (Station 23).

The PEACE model was modified with this equation to use the flows at Arcadia (Station 23) in its calculations.

POTENTIAL YIELD FROM THE PEACE RIVER

The PEACE model was run with the data modifications outlined in the previous sections to quantify the potential availability of divertible water from the Peace River. The results, when compared to projected water demands, indicate that the Peace River has the potential to supply all raw water needs projected through buildout. The ultimate yield from the water supply facility is governed and limited by its capacity to divert, store, and treat the water once it is withdrawn from the river.

The amount of water actually delivered to the plant site is limited by the capacity of the intake structure. The amount of divertible water available from the river may often be greater than the capacity of the intake. Figure 3-10 shows how intake structure capacity affects the amount of water that can be diverted from the Peace River over a period of time, based on the proposed diversion rule and using the past 55 years of flow data. The graph shows the maximum possible amount that can be diverted (based on a long-term average) to the WTP, assuming that the intake structure takes all the water it is capable of pumping when water is available for withdrawal. For example, the current plant intake, with a 22-mgd capacity, will allow approximately 18 mgd of average flow to be diverted during a typical water year. This acknowledges that during certain periods, no water will be withdrawn and the water supply must be furnished from storage.

Figure 3-11 is the cumulative distribution of the 133 TDS measurements taken by the EQL from 1975 through 1986. The distribution curve shows that TDS concentrations exceeded the 500 mg/l secondary standard only 3.5 percent of the time and have exceeded 400 mg/l only about 4 percent of the time. These infrequent high TDS occurrences are typically associated with low river flows, during which little or no withdrawal would be allowed based on the flow rate criteria. Thus, based on statistical analysis of the available data, withdrawals by GDU from the Peace River will generally not be affected by the TDS concentrations of the river.

SUMMARY

The Peace River appears to be a reliable source of raw water for the Peace River WTP and can be used to meet all or part of GDU's projected demands for the Port Charlotte service area through the year 2000. However, the ability to develop it to its full potential is dependent on the water supply facilities, including the water treatment plant, the off-stream raw water storage reservoir, and the ASR system.

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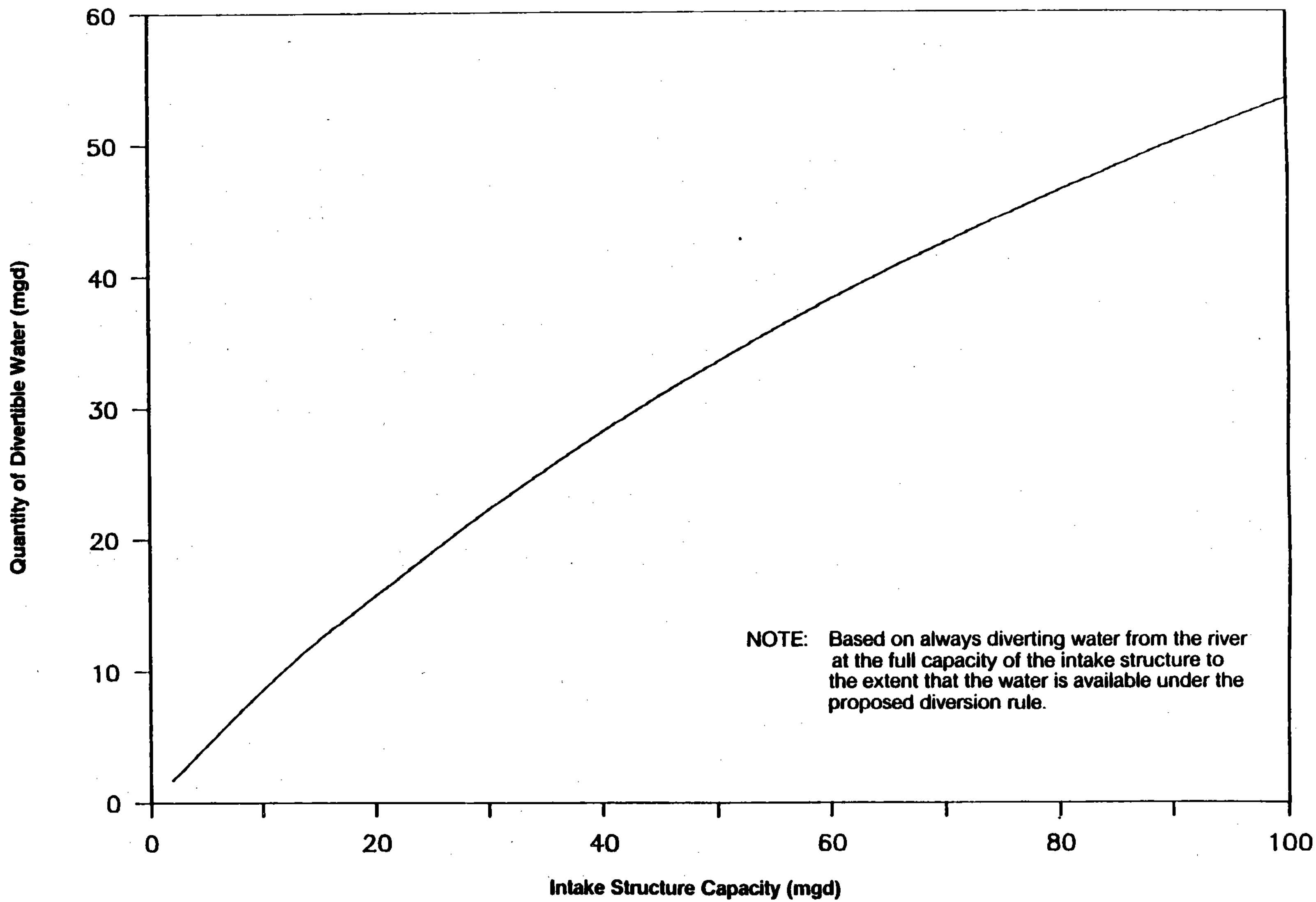

FIGURE 3-10.
Maximum Divertible Peace River Flow by Intake Structure Capacity.

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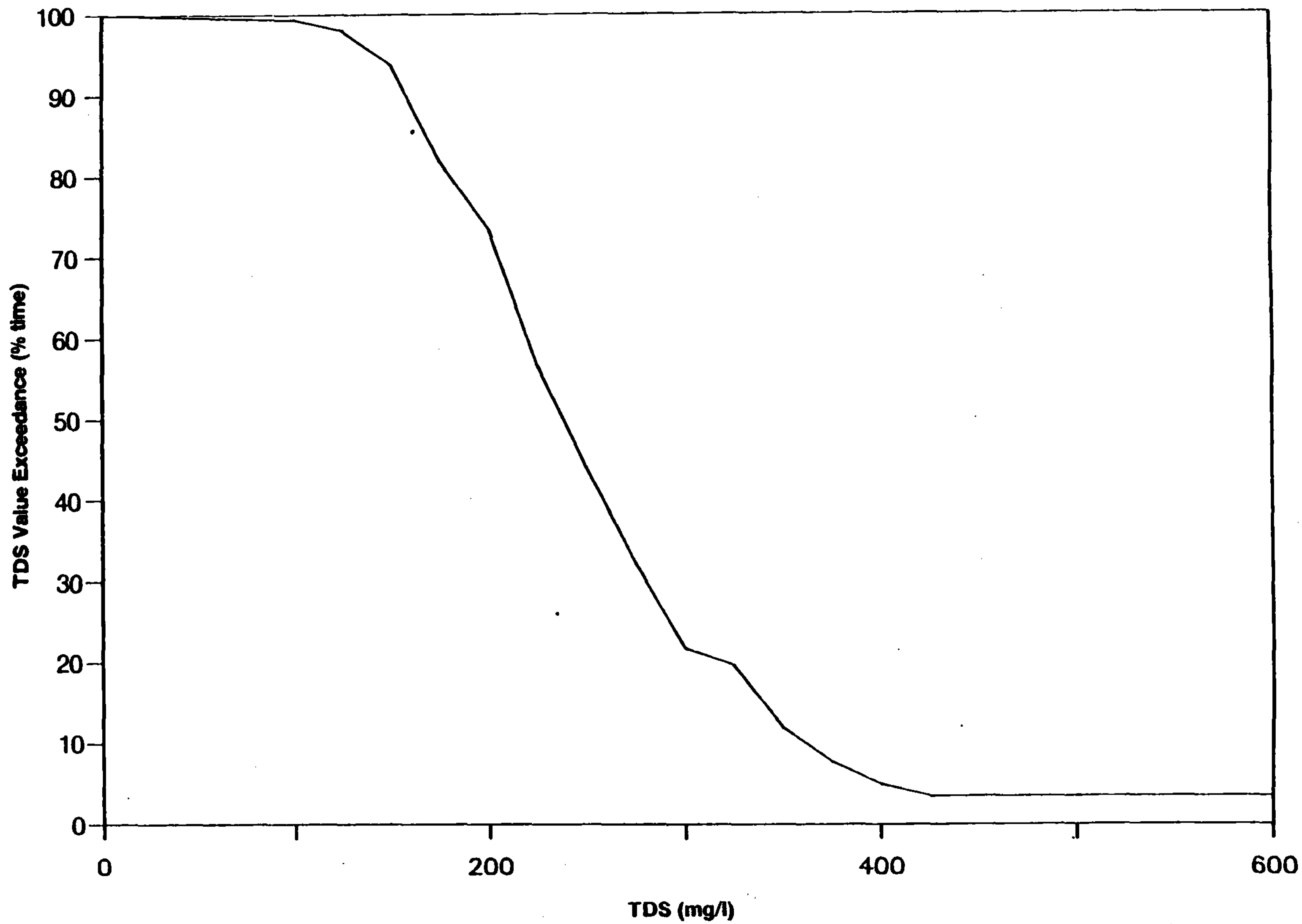

FIGURE 3-11.
Percent of Time Peace River TDS Concentrations Exceed Specified Values at Station 18 (SR 761 Bridge).

Without optimization of these facilities, the net potential yield from the Peace River will not be realized. The Peace River WTP facilities required to meet projected demands, along with economic analyses, are addressed in Section 7.

GROUNDWATER

DATA SOURCES

The availability of groundwater resources in the Port Charlotte service area to meet existing and projected uses was evaluated based on a review of available pertinent literature on the water resources of the region. Sources of information on the geology and groundwater resources of the service area are listed in the references. Reports on the following subjects were published by the U.S. Geological Survey (USGS), Florida Bureau of Geology, Florida Division of Water Resources, and the SWFWMD:

- Geology and groundwater resources of Sarasota County (Stringfield, 1933)
- Surficial deposits in DeSoto County (Bergendahl, 1956)
- One-year reconnaissance of groundwater data collection in Hardee and DeSoto Counties (Woodard, 1964)
- Continuance of data collection in Hardee, DeSoto, and Charlotte Counties resulting in a map series (Kaufman and Dion, 1967) and report (Kaufman and Dion, 1968)
- Drilling of 21 test wells in the Myakka River basin of Charlotte and Sarasota Counties (Sutcliffe and Joyner, 1968)
- Water resources and water supply problems of Charlotte County (Sutcliffe, 1975)
- Water resources in the Myakka River basin (Joyner and Sutcliffe, 1976)
- Water resources of DeSoto and Hardee Counties (Wilson, 1977)
- Feasibility of surficial aquifer water supply development in Charlotte County (Wolansky, 1978)
- Hydrogeology of Sarasota and Charlotte Counties (Wolansky, 1983)

- Use of groundwater in the western coastal area of Charlotte and Sarasota Counties (Sutcliffe and Thompson, 1983)

Consultant investigations include those by Geraghty and Miller (1973, 1976, 1982) for GDU in the service area and by CH2M HILL (April 1985a) on ASR at the Peace River WTP. Additional investigations, both by state agencies and private consultants, make reference to the service area. Those reports pertinent to this investigation are given in the references.

SITE DESCRIPTION

The Port Charlotte service area includes northwestern Charlotte County, southeastern Sarasota County, and a small section of southwestern DeSoto County (see Figure 1-1). Topographically, the area is a nearly flat low-lying plain, sloping gently toward the southwest. Elevations range from a high of about 40 feet in the northeastern part of the area to near sea level in the southern part.

Parts of Charlotte Harbor, the Peace and Myakka rivers, and their tributaries occupy the service area. Flow from the Peace and Myakka river basins empties into Charlotte Harbor before entering the Gulf of Mexico. Tidal influences are apparent adjacent to Charlotte Harbor.

HYDROGEOLOGIC SETTING

The service area is underlain by water-bearing sediments, ranging from Holocene to Eocene age and extending to more than 1,500 feet below land surface. The sediments are divided into three major water-bearing units, termed the surficial, intermediate, and Floridan aquifer systems. Geologic formations constituting these aquifer systems are, from the youngest: the undifferentiated Holocene/Pleistocene deposits, Tamiami Formation, Hawthorn Formation, Tampa Limestone, Suwannee Limestone, Ocala Limestone, and Avon Park Limestone.

The water-bearing sediments or deposits consist of quartz sand, shell, clay, limestone, and dolomite of varying proportions. Permeabilities of the deposits vary with composition and depositional characteristics. Low permeability beds consisting primarily of clays and fine silts separate the surficial, intermediate, and Floridan aquifer systems from one another. Higher permeability beds consisting of sand, shell, limestone, and dolomite constitute the water-bearing units of the aquifer systems.

The three major water-bearing units have been further subdivided into a number of aquifer zones and subzones by authors of several of the reports listed in the references. For this assessment, the aquifer zones are grouped into the following four units on the basis of their water supply potential and probable modes of development: the shallow aquifer, the shallow artesian aquifer, and the upper Floridan and lower Floridan aquifers. Physical and water-bearing properties of these units are summarized in Table 3-9. The relationship of the four units to the regional system of aquifer nomenclature proposed by Sutcliffe and Thompson (1983) is shown in Figure 3-12.

The development of potable groundwater supplies in western Charlotte and Sarasota Counties has been a problem almost since the earliest development of the area. In most of the area, especially the more heavily populated coastal sections, the only potable water for conventional treatment occurs in shallow aquifers, which tend to be limited in extent and subject to saltwater intrusion. Wells completed in the shallow aquifers generally have low yields. The deeper, more productive aquifers are too highly mineralized for direct use in potable water production.

It is unlikely that significant quantities of water to satisfy long-term needs can be obtained by further development of the shallow aquifers. The deeper aquifers have the potential to yield large quantities of water that would, however, need to be desalted or blended with fresh surface water to reduce mineral content. These deep aquifers can also be used in some areas for storage and recovery of freshwater from other sources. For example, an ASR well in the upper Floridan aquifer has been in operation at the Peace River WTP since 1984; expansion of this system to 5-mgd capacity is underway.

GROUNDWATER DEVELOPMENT SOURCES

The potential for development of groundwater supplies in the area is influenced by interacting economic, environmental, and hydrologic factors. Therefore, except in qualitative terms, assessing groundwater development potential is seldom possible for other than site-specific or use-specific purposes. For this report, subjective judgments were made on the basis of the most dominant influences affecting development potential of each aquifer in the service area.

The diversity of conditions within the service area make it necessary to assess development potential of each of the aquifer systems in several parts of the area. The two general aspects of groundwater development considered in

Table 3-9
SUMMARY OF AQUIFER CHARACTERISTICS

Aquifer Unit	Lithology	Well Construction	Well Yield	Water Quality
Shallow Aquifer	Mainly sand and shell in varying proportions.	Screened wells 2" to 12" in diameter. Generally less than 100 feet deep, commonly 50 feet or less.	Usually 20 to 50 gpm. Rarely more than 100 gpm.	Moderately hard, mainly carbonate hardness. Usually high color and iron. TDS generally less than 1,000 mg/l, except where intruded by saltwater from artesian wells or salty surface water.
Shallow Artesian Aquifer	Cemented shell and sandy limestone.	Generally completed without screens, 4" to 12" in diameter, 100 to 250 feet deep.	Usually less than 100 gpm. Up to 500 gpm for large-diameter wells in favorable locations.	Hard, with TDS greater than 1,000 mg/l over all but NE 1/3 of project area. Where TDS is in potable range, hardness is mainly carbonate-type. Sometimes high in fluoride.
Upper Floridan Aquifer	Limestone and dolomitic limestone.	Open hole, 4" to 16" in diameter. Depths 500 to 1,000 feet, depending on locations; depth to the aquifer is greatest in southern part of the project area.	500 to 1,000 gpm for large-diameter wells in favorable locations.	Non-potable (TDS greater than 1,000 mg/l) over all but extreme NE part of project area. TDS increases with depth and toward the south and west.
Lower Floridan Aquifer	Limestone and dolomite.	Open hole, 6" to 16" in diameter. Depth 1,100 to 1,600 feet, depending on location; depth to aquifer is greatest in southern part of project area.	1,000+ gpm for large-diameter wells in favorable locations.	Non-potable (TDS greater than 1,000 mg/l) over all but extreme NE part of project area. TDS increases with depth and toward the south and west. TDS exceeds 10,000 mg/l west of Myakka River and in the Port Charlotte area.

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<table>
<tr><th>SYSTEM</th><th colspan="2">AQUIFER ZONES[a]</th><th>FORMATION</th><th>AQUIFER ZONES (This Report)</th><th>APPROXIMATE DEPTH (Feet)</th></tr>
<tr><td>Quaternary</td><td colspan="2">Surficial Aquifer</td><td>Undifferentiated Deposits</td><td rowspan="3">Shallow Aquifer</td><td rowspan="3">0
~80</td></tr>
<tr><td rowspan="12">Tertiary</td><td rowspan="7">Intermediate Aquifers and Confining Beds</td><td>Confining Bed</td><td rowspan="4">Tamiami Formation</td></tr>
<tr><td>Zone 1</td></tr>
<tr><td>Confining Bed</td><td rowspan="4">Shallow Artesian Aquifer</td><td rowspan="4">~100
~250</td></tr>
<tr><td>Zone 2</td></tr>
<tr><td>Confining Bed</td><td rowspan="2">Hawthorn Formation</td></tr>
<tr><td>Zone 3</td></tr>
<tr><td>Confining Bed</td><td rowspan="2">Tampa Limestone</td><td rowspan="3">Upper Floridan Aquifer</td><td rowspan="3">~500
~1000</td></tr>
<tr><td rowspan="5">Floridan Aquifer</td><td>Zone 4</td></tr>
<tr><td>Semi-Confining Bed</td><td>Suwannee Limestone</td></tr>
<tr><td>Zone 4</td><td rowspan="2">Ocala Limestone</td><td rowspan="3">Lower Floridan Aquifer</td><td rowspan="3">~1100
~1600</td></tr>
<tr><td>Semi-Confining Bed</td></tr>
<tr><td>Zone 5</td><td>Avon Park Limestone</td></tr>
</table>

[a]Adapted from Sutcliffe and Thompson, 1983.

FIGURE 3-12.
Aquifers in the Service Area.

CH2M HILL

this evaluation are potable groundwater supply and nonpotable groundwater supply for desalination. The following criteria were considered in assessing the potential of each aquifer for these types of groundwater development:

- Usable thickness of aquifer

  The depth of potable water is a constraint on well depth and, consequently, on well yield. For the Floridan aquifer system, including the upper and lower zones, the thickness of an aquifer is considered to be the interval between the top of the aquifer and the depth at which the aquifer contains water with more than 10,000 mg/l TDS. For shallower aquifers, the usable thickness is the interval consisting of relatively permeable strata capable of yielding a reasonable amount of water. The usable depth of shallow aquifers near the coast is generally less than the total thickness of permeable material because of saltwater intrusion.

- Estimated well yield

  This is a largely subjective estimate of the highest practicable well yield for the area and aquifer. It involves consideration of aquifer thickness and transmissivity, available drawdown, saltwater intrusion potential, and, when available, production records of existing high capacity wells in the area.

  Well yields are controlled by the hydraulic characteristics of the aquifers, which are highly variable. In theory, obtaining high yield from a well in the state of Florida is simply a matter of drilling a hole large enough and deep enough to obtain the desired yield. In practice, this approach would result in the production of salty or brackish water in much of the service area.

  This factor is primarily economic: how many wells of what size must be constructed to produce the required amount of water of appropriate quality?

- Water quality

  One of the criteria used to delineate hydrologically similar areas is water quality. The range of TDS and the chemical type of water present in each component of the aquifer system varies within the service area. In general, water quality is best toward the north and east, and deteriorates toward the south and west. Water quality is also highly variable with depth.

Ordinarily, public water supply imposes the most restrictive quality and cost constraints upon the source. However, these water quality constraints are complex. State of Florida drinking water standards, defined in Chapter 17-22, FAC, are used to evaluate aquifer water quality characteristics where data are available. Desalination processes can make almost any water usable for public supply if the need is great enough to justify the cost. Because cost is largely related to the TDS content of the feedwater, it is generally desirable to provide the lowest TDS water that is feasibly available for desalination. The economic feasibility of a desalination process may, however, be strongly influenced by the concentration of specific substances in the water. Therefore, more detailed evaluations of site- specific feedwaters are needed before the costs and performance of desalination processes at specific locations can be properly estimated.

Shallow Aquifer

Description. The shallow aquifer extends throughout the service area and ranges in thickness from 10 to 80 feet, with the greatest aquifer thickness occurring in the southern and eastern parts of the area. Lithologic units present within the shallow aquifer consist of quartz sand, shell, sandy clay, and minor limestone. The most permeable units are those containing coarser grained sand and shell. These permeable units occur scattered throughout the area and vertically throughout the surficial aquifer system; none appear to be extensive.

Water levels in the shallow aquifer vary from land surface to nearly 15 feet below the surface. The aquifer is recharged directly by rainfall and responds principally to changes in drainage and pumpage within the aquifer. The regional groundwater gradient is toward the south-southwest.

Aquifer Characteristics. Water quality in the shallow aquifer is variable across the service area. In areas along the coast subject to storm inundations, the shallow aquifer can contain high TDS concentrations, particularly in the form of high chlorides. The aquifer also contains saltwater (chloride >10,000 mg/l) in areas adjacent to saltwater canals and along Charlotte Harbor. In the central part of the service area, control structures have been installed on canals adjacent to U.S. 41 to prevent further inland migration of saltwater.

In places not affected by these sources of saltwater contamination, potable water is available from the shallow aquifer. TDS concentrations are within potable limits ranging from less than 200 mg/l to about 600 mg/l. Color and dissolved iron are usually higher than the Florida drinking water secondary standards (primarily related to aesthetic characteristics of the water), but can be treated by conventional methods (i.e., without the need for desalination).

Aquifer test data for the shallow aquifer are available from the Englewood, Venice, and Gasparilla Island well fields. These tests indicate the aquifer has a transmissivity ranging from 1,200 to about 3,000 ft^2/day. The wells yield from about 20 to about 120 gpm. The Rotunda development also obtains water from a shallow aquifer at its well field south of the study area. The well capacity here is typically less than 50 gpm to prevent salt- water intrusion.

Development Potential. Groundwater supply for the now-abandoned Port Charlotte WTP No. 1 was obtained from the shallow aquifer beginning in the 1950's and early 1960's. Currently, the aquifer is used by homeowners and other individuals for lawn maintenance and similar purposes. Little potential exists for significant further development of the shallow aquifer for long-term water needs. No suitable locations for well field development of the shallow aquifer are known in the service area, and much of the potentially suitable area has already been developed for housing. Also, experiences in adjacent parts of the southern Gulf Coast indicate that individual sites underlain by highly permeable and productive strata are mainly less than one square mile in extent, and are limited to 1 to 2 mgd of developable capacity.

Because well yields are low, a relatively large number of wells would be required to develop water supplies from this source. Construction and operating costs for several small well fields with numerous wells spread over a large area appear to eliminate the shallow aquifer as an economically viable source of additional water supply.

Shallow Artesian Aquifer

Sutcliffe and Thompson (1983) subdivided the intermediate artesian aquifer in Charlotte and Sarasota Counties into three zones, as shown in Figure 3-12. For the purpose of this assessment, Zone 1 is combined with the surficial aquifer, and Zones 2 and 3 represent permeable zones within the intermediate (or shallow artesian) aquifer system.

Zone 2. Artesian Zone 2 occurs in the upper part of the Hawthorn Formation and consists of limestones, clays, and sandy clays. The thickness of the zone is approximately 100 to 200 feet. Zone 2 is separated from the zone above by a bed of gray clay, locally termed the Venice Clay (Sutcliffe and Thompson, 1983), in the western part of the service area. Depth to the top of Zone 2 ranges from as shallow as 80 feet in the northern and eastern areas, where Zone 1 is thin or absent, to over 150 feet in the southwest. The top of Zone 2 in the Port Charlotte service area is approximately 120 feet below land surface.

Zone 3. Artesian Zone 3 occurs in the lower part of the Hawthorn Formation and upper part of the Tampa Limestone and consists of a thick sequence of consolidated phosphatic limestone, highly fossiliferous and sandy in areas. It is separated from Zone 2 by a thick sequence of clays and sandy clays in the middle part of the Hawthorn Formation. Zone 3 ranges in thickness from about 100 feet in the northern part to over 300 feet in the southern part of the service area. The top of the zone ranges from about 200 feet below land surface in the northwestern part of the area to over 300 feet in the southern and extreme northern parts of the area.

Zone 3 is probably more closely related hydraulically and in water quality to the underlying upper Floridan aquifer than it is to Zone 2. Because of this relationship, it will be considered as part of the Floridan aquifer in evaluating development potential. The following discussions of aquifer characteristics and development potential are therefore applicable primarily to the upper part (Zone 2) of the shallow artesian aquifer.

Aquifer Characteristics. Water quality in the shallow artesian aquifer is generally poorer than the surficial aquifer, although it meets drinking water standards in some local areas. Typical water quality of this aquifer along with selected drinking water standards are shown in Table 3-10. In general, TDS concentrations are expected to be greater than drinking water standards over much of the service area and can range to nearly 2,000 mg/l in the southwestern section (Sutcliffe and Thompson, 1983). As with the surficial aquifer, water quality is best in the northeastern sections. Sulfate concentrations are greater than 250 mg/l along the coastal area and in the southern Port Charlotte area adjacent to Charlotte Harbor. Sulfate concentrations are high, but within potable limits in the remaining service area.

Table 3-10

WATER QUALITY, SHALLOW ARTESIAN AND FLORIDAN AQUIFERS

Constituents	Concentration (mg/l)[a]						FAC 17-22 Drinking Water Standard[d]
	1	2[b]	3	4[b]	5[c]	6	
Total dissolved solids	600[e]	460	650-800	2,000-3,000	820	6,860	500
Calcium, Ca	50	100	80-115	120-180	88	280	-
Magnesium, Mg	40	18	30-55	100-150	4	5,240	-
Sodium, Na	70[e]	42	100-130	400-600	-	1,950	160[f]
Bicarbonate, HCO_3	188	322	100-150	220-280	185	175	-
Sulfate, SO_4	86	13	215-225	400-500	205	770	250
Chloride, Cl	145	100	160-205	800-1,000	156	3,530	250
Total Hardness, as $CaCO_3$	290	330	370-480	1,200-1,600	404	1,690	-

Aquifer Source

1. Shallow Artesian Aquifer, DeSoto area (Geraghty & Miller, 1982).
2. Shallow Artesian Aquifer, near Port Charlotte (Sutcliffe, 1975).
3. Upper Floridan Aquifer, DeSoto Property (CH2M HILL, 1985).
4. Upper Floridan Aquifer, North Port/Port Charlotte (Composite, multiple sources).
5. Lower Floridan Aquifer, DeSoto Property (Geraghty & Miller, 1982).
6. Lower Floridan Aquifer, North Port area (Oil test file, Florida Bureau of Geology).

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[a] Ranges given when data were obtained from a well or wells producing from multiple zones.

[b] Composite from multiple wells.

[c] May include water from upper Floridan.

[d] Selected Florida drinking water standards, as defined in FAC 17-22.

[e] Estimated using ion summation and ion balance calculations.

[f] Indicates primary standard; other entries are secondary standards.

Aquifer tests conducted in the northeastern part of the service area (Geraghty and Miller, 1973) and in adjoining parts of Charlotte and Sarasota Counties give transmissivity values of 4,000 to about 9,000 ft^2/day. The higher values reported appear to be for wells that penetrate both Zones 2 and 3. Estimated transmissivity of Zone 2 alone ranges from 500 to 3,000 ft^2/day.

Development Potential. The water supply development potential for the shallow artesian aquifer is unknown for most of the service area. An assessment by Geraghty and Miller (1982) and results of previous investigations (Geraghty and Miller, 1973) indicated that potable water from this source could be obtained in the northeastern part of the service area (DeSoto County). This earlier assessment is supported by tests conducted in connection with ASR studies at the Peace River WTP.

Analysis 1 in Table 3-10 is probably representative of the best quality water obtainable from a shallow artesian aquifer well field located in the northeastern part of the area. In a practical well field development, it would probably be necessary to complete wells in deeper zones with poorer water quality to obtain adequate well yields. The blended water from these zones would result in greater sulfate concentrations because sulfate levels are characteristically higher in the deeper zones. Total dissolved solids would be in the 700 to 800 mg/l range, chlorides between 100 and 150 mg/l, and sulfates between 90 and 200 mg/l. The following constraints would be the main impediment to development of water supply from this source:

- The low aquifer transmissivity, which would require an uneconomically large number of wells to produce a significant amount of water.

- Interference with existing uses of water from this zone.

In the early 1960's, nine shallow artesian wells were installed at the Port Charlotte Country Club in the southeastern part of the service area. Water from these wells was intended to be used to supplement water from the surficial aquifer wells at WTP No. 1. The wells proved to be unsuitable for this purpose, primarily because of poor yield, and were subsequently abandoned. These wells were reported to range from 130 to 325 feet deep.

Analysis 2 in Table 3-10, a composite analysis of water from several of these wells, shows that the water is very hard, but treatable to drinking water quality by conventional means. It is not known if the presence of relatively fresh water at this site is a local occurrence, or if the producing stratum is extensive enough for additional significant development. Published data and experience in other parts of the south Gulf Coast suggest that, even if the freshwater producing stratum is extensive, its development potential is limited by the presence of brackish water in the underlying artesian aquifers. Extensive development of this source would probably result in a rapid degradation of water quality in the fresher zone.

In our opinion, development potential for the shallow artesian aquifer as a major water supply source is generally slight, limited by yield capacity and the probable need for desalination treatment. However, the shallow artesian aquifer may be developed to a limited extent for supplemental water supplies to meet seasonal peak demands. It is probable that water obtained from this source would need to be blended with water from other sources with lower TDS concentrations. Additional exploration and testing would be required to confirm the location, quality, and availability of water from this source.

A potentially favorable area for water supply, based on geologic conditions, is shown in Figure 3-13. It is probable that both the shallow and shallow artesian aquifers reach their greatest thickness in the area delineated as potentially favorable. This would indicate higher productivity. In addition, the thicker section of the overlying shallow aquifer is a potential source of recharge to the artesian aquifer.

Floridan Aquifer

Description. The Floridan aquifer system is the regional artesian aquifer. In the service area, it is represented by two zones, the upper Floridan and the lower Floridan. Geological formations constituting the aquifer include all or parts of the Tampa, Suwannee, Ocala, and Avon Park Limestones. The lower Hawthorn/Tampa producing zone of the intermediate aquifer system is also sometimes included in the upper Floridan aquifer. In this study, Zone 3 is considered a distinct unit, but is combined with the upper Floridan aquifer in evaluating water supply development potential.

In the service area, the upper Floridan includes parts of the Tampa, Suwannee, and Ocala Limestones. In the northeast part of the service area (southwest DeSoto County), the upper Floridan lies at a depth of approximately 500 to 700 feet, with the lower Floridan generally below 1,000 feet. In the southern part of the area, the upper Floridan occurs at 700 to 1,000 feet in depth, and the lower Floridan begins at

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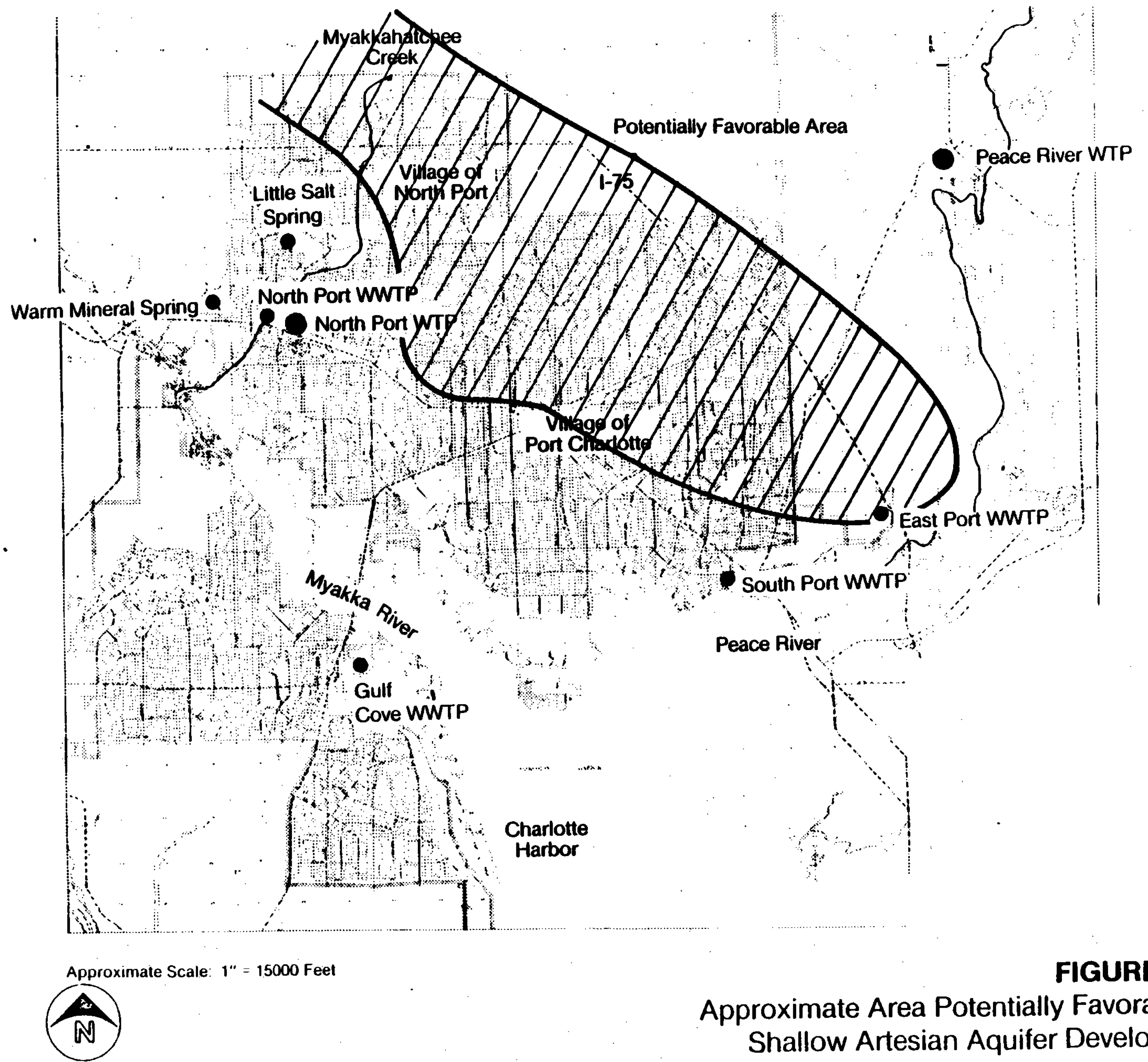

FIGURE 3-13.
Approximate Area Potentially Favorable for Shallow Artesian Aquifer Development.

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1,200 to 1,400 feet. The upper and lower Floridan are separated by low-permeability limestone in the Ocala group.

The potentiometric surface of both the upper and lower Floridan zones ranges from 50 feet above sea level in the northeast to less than 30 feet in the western part of the service area. The hydraulic gradient is to the west. Recharge to the zone occurs laterally from areas north-northeast of the service area.

Aquifer Characteristics. Figures 3-14 and 3-15 show the approximate distribution of TDS in the upper and lower Floridan in the area. Water quality is generally non-potable in both zones, except in the extreme northeast part of the area in southwest DeSoto County. In this location, chloride concentrations are less than 250 mg/l, and TDS ranges from 500 to 1,000 mg/l. Analyses 3, 4, 5 and 6 in Table 3-10 are typical for Floridan aquifer wells in the service area.

In general, chloride concentrations increase toward the south and west and with increasing depth in the aquifer. Analyses 4 and 6 in Table 3-10 are typical for Floridan aquifer wells in the Port Charlotte/North Port area. Chloride concentrations range up to 1,000 mg/l for well depths up to 800 feet (upper Floridan, analysis 4), and over 3,000 mg/l for wells into the lower Floridan (analysis 6). Chloride concentrations increase toward the south and west to over 2,000 mg/l in the upper Floridan and over 6,000 mg/l in the lower Floridan.

Analyses 3 and 5 are typical for upper and lower Floridan waters, respectively, in the DeSoto County area. Comparison of these analyses shows the similarity of water quality in both aquifer zones in this area.

Transmissivities ranging from 13,000 ft^2/day to 30,000 ft^2/day have been calculated for the upper Floridan in the northern part of the service area (CH2M HILL, 1985; Geraghty and Miller, 1976). The transmissivity of the lower Floridan aquifer is estimated to be about 150,000 ft^2/day within the brackish-water part of the aquifer. More highly transmissive zones may be present near the base of the aquifer in the saltwater zone. These values are probably also representative of the lower Floridan throughout the service area.

Development Potential of the Upper Floridan. The potential for development of the upper Floridan for use as a raw water source for a desalination treatment plant is considered to be good. Large quantities of water with moderate TDS content can be withdrawn from a reasonable number of wells.

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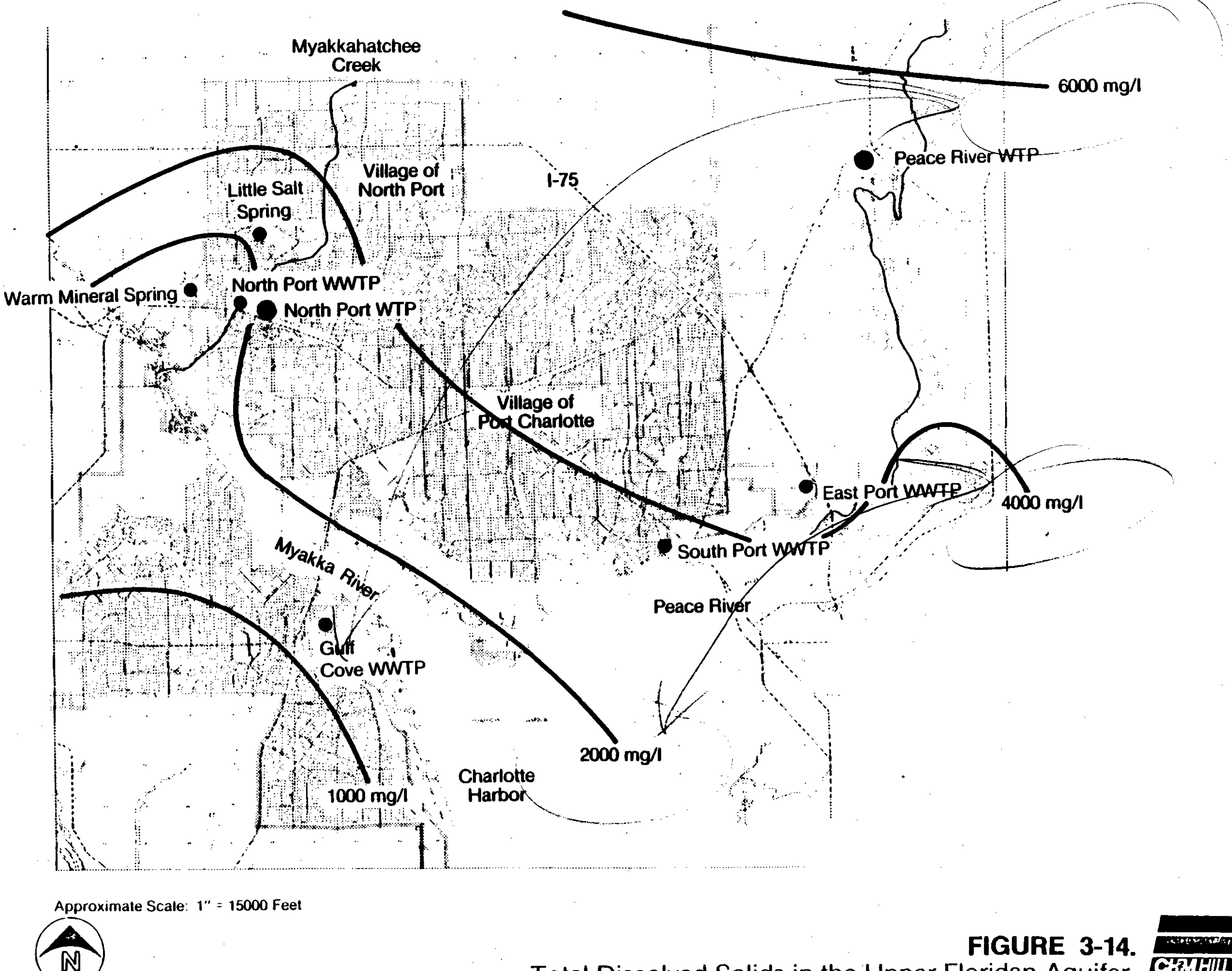

FIGURE 3-14.
Total Dissolved Solids in the Upper Floridan Aquifer.

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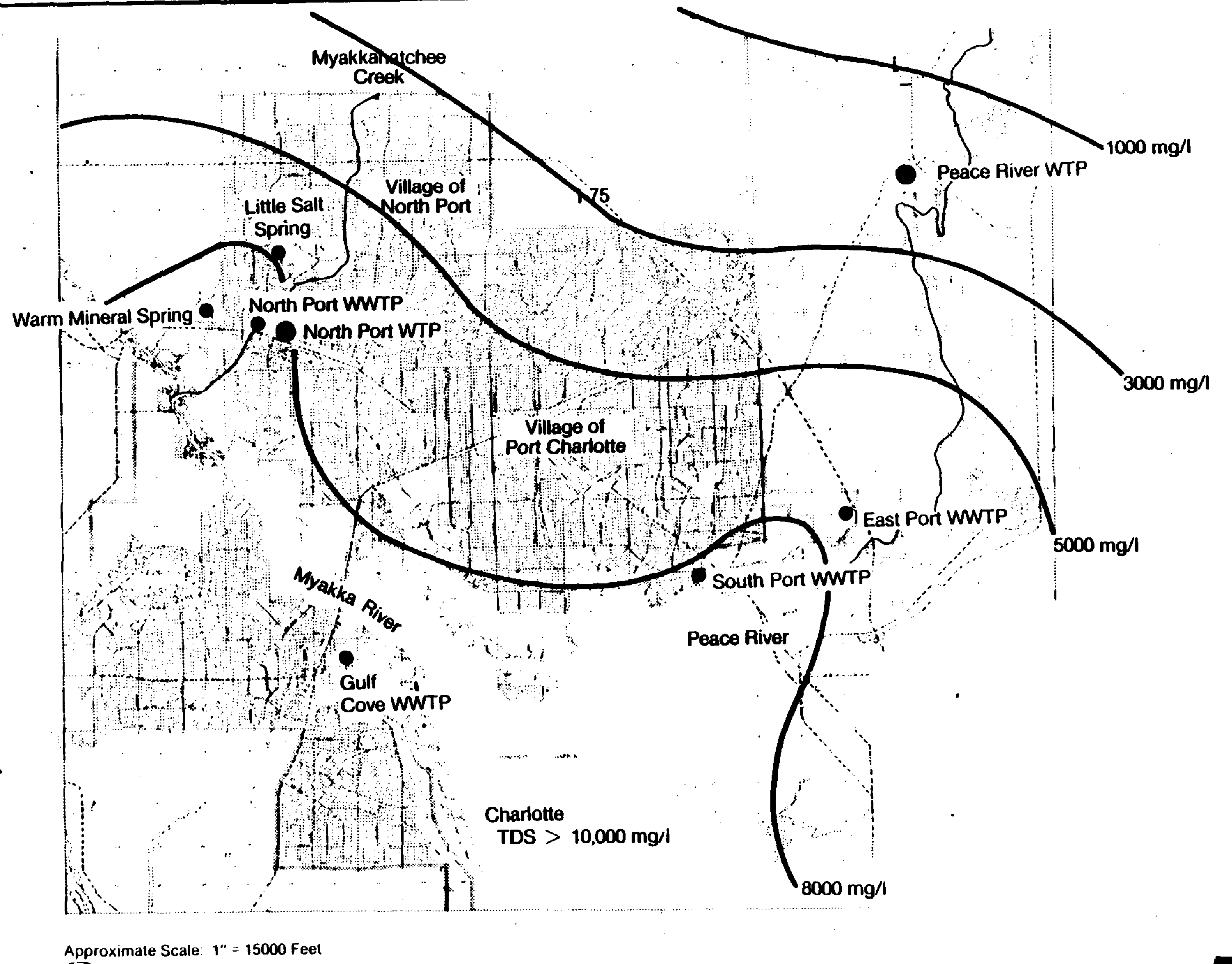

FIGURE 3-15.
Total Dissolved Solids in the Lower Floridan Aquifer.

Wells can be expected to yield 500 to 1,000 gpm. The zone is not much used in the Charlotte County section of the service area, but is the principal source of irrigation water in the adjoining parts of DeSoto County. Water quality appears suitable for treatment by brackish water reverse osmosis (RO) or electrodialysis (ED) systems, except in the extreme southwestern section where the water may approach seawater quality.

The groundwater source with the largest potential for future development in this area is a combination of the lower zone (Zone 3) of the intermediate artesian system and the upper Floridan between the depths of about 200 and 800 feet. Brackish water from this source would probably be suitable for treatment by desalting almost anywhere in the service area. The most probable area (based on limited TDS data) for development of groundwater supplies is shown in Figure 3-16. The development potential is constrained by competition with existing users in the northern (DeSoto County) part of the area, but the source is almost unused in the southern Sarasota and Charlotte County portions.

Hydraulically, well fields could be located almost anywhere in the subject area, with the following known constraints:

1. The cone of depression produced by pumping should not extend to and intercept flow from Warm Mineral Springs.

2. The pumping should not induce upward leakage to the extent that the source becomes economically unusable for desalination.

In general, the TDS content of water in the aquifer increases from north to south and east to west.

Based on water quality considerations, the best location for a well field would be in the northeast part of the area, in southwestern DeSoto County. At present, however, this area is considered best used as a well field for expanded ASR operations for the Peace River WTP. Water quality is generally poorer to the southwest across the service area, but TDS content is still in the reasonably usable range for desalination throughout the central part of the area, as shown in Figure 3-16. In the North Port area, chloride concentrations are less than 250 mg/l and only TDS and sulfate are above the Florida state drinking water standards. The water would require either treatment by desalination or blending with low TDS water from another source.

Previously cited aquifer data for the upper Floridan in the DeSoto County area and other test well data (Wells, 1969) indicate that a potentially large quantity of groundwater

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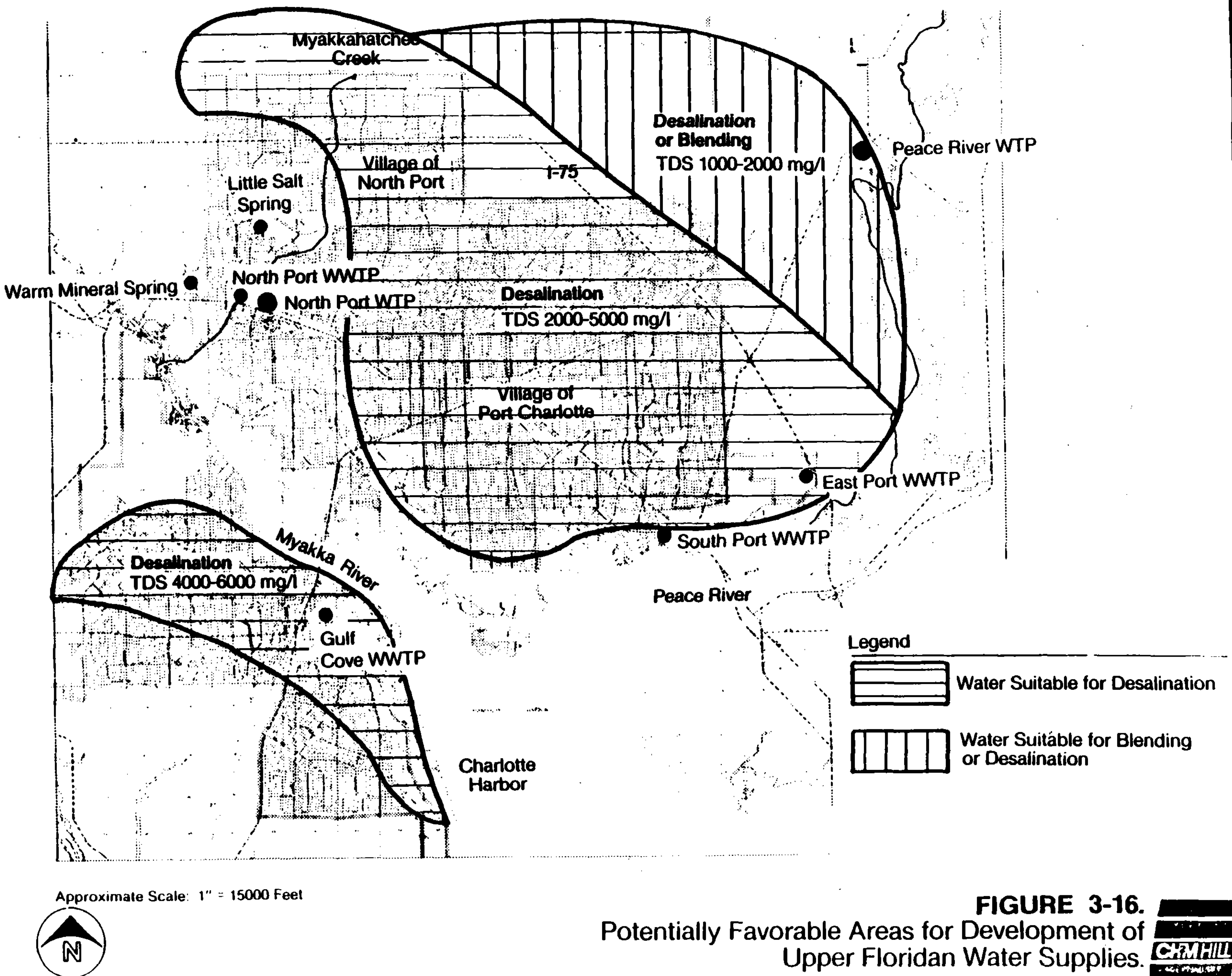

FIGURE 3-16.
Potentially Favorable Areas for Development of
Upper Floridan Water Supplies.

could be withdrawn from this aquifer. Use of the lower part of the shallow artesian aquifer and both the upper and lower Floridan would be possible in the DeSoto County area.

To simulate the hydraulic effects, a hypothetical 40-mgd well field near the Peace River WTP, consisting of 40 wells spaced 1,000 feet apart, each pumping at 1 mgd, was laid out in the DeSoto County property. The Trescott-Larson model (1976) was used for simulation of aquifer response. Hydraulic characteristics of the upper Floridan used in the model were: transmissivity = 30,000 ft^2/day and storage coefficient = 0.0002. It was assumed that the upper Floridan would be recharged by leakage from both the overlying shallow artesian aquifer and the underlying lower Floridan. The simulation indicated that pumping water levels would essentially stabilize after about 60 days of pumping at 40 mgd. Figure 3-17 shows the layout of the hypothetical well field, and the drawdowns in the upper Floridan aquifer.

The study indicated that aquifer drawdowns of about 10 feet could occur at a radial distance of 10,000 feet from the edge of the well field and would represent the main constraint on usage in this area. Drawdowns would be approximately one-half as great if both the upper and lower Floridan zones were utilized, but some increase in TDS would result.

The well field simulation is generally applicable to the Floridan aquifer in the North Port and Port Charlotte areas, where water quality considerations would probably limit development to the upper Floridan (including the lower part of the shallow artesian aquifer). Estimated TDS of the water produced would be 2,000 to 3,000 mg/l in the North Port area, and 4,000 to 5,000 mg/l at Port Charlotte.

Water with higher TDS concentrations, but still likely suitable for desalination, could be obtained from the Upper Floridan aquifer in the Gulf Cove area, west of the Myakka River. TDS concentrations would be in the range of 4,000 to 6,000 mg/l. Proximity to more highly saline water (TDS >10,000 mg/l) would limit feasible maximum withdrawals in the less saline zones. The safe yield for the potentially favorable area west of the Myakka River in Figure 3-16 is estimated to be 4 to 6 mgd; practicable well yields are estimated to be 200 to 400 gpm.

Development Potential of the Lower Floridan. The lower Floridan is believed to be reasonably developable as a water source only in the northern part of the service area. In the DeSoto County property, water quality in the lower Floridan is similar to that of the upper Floridan and the

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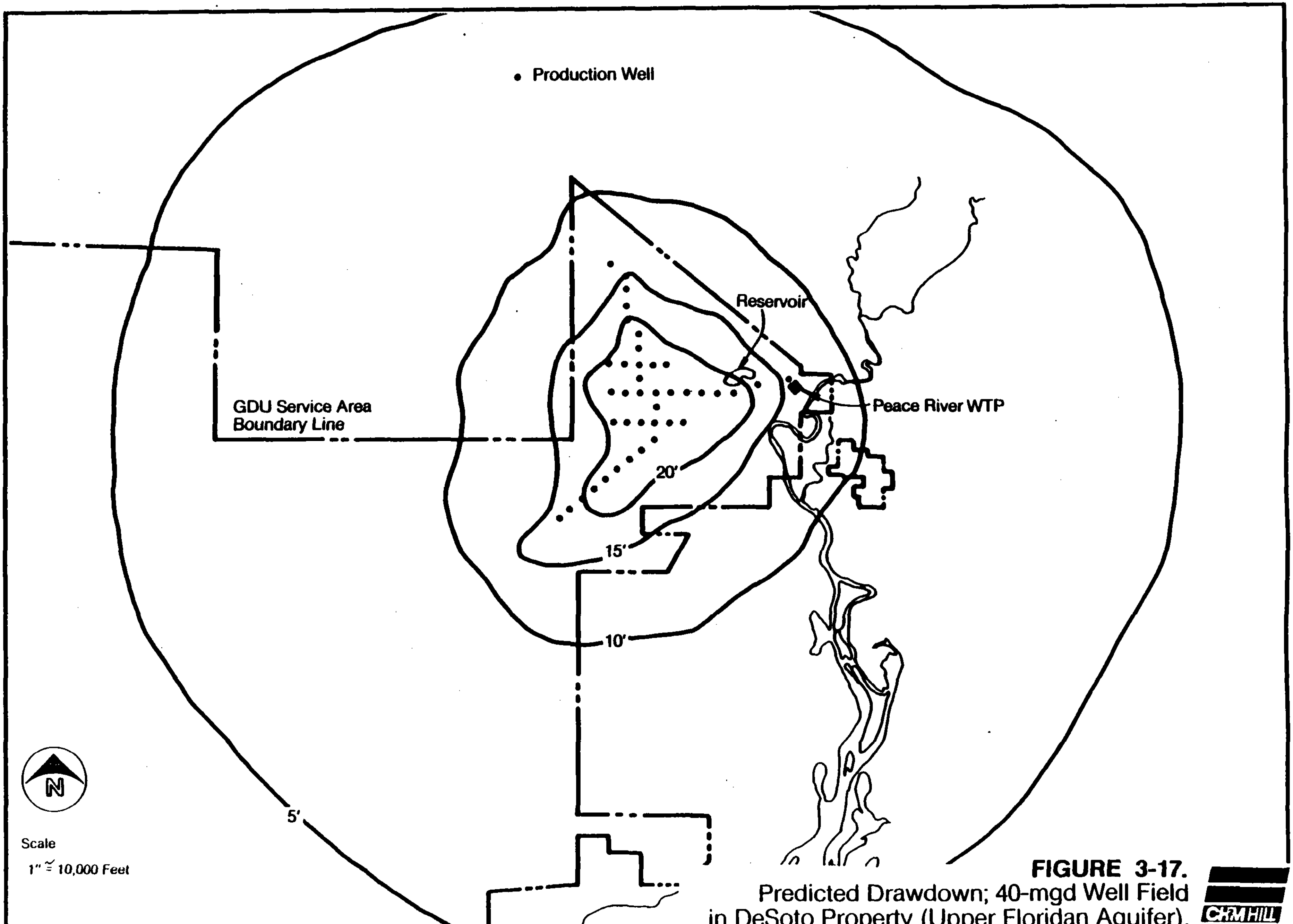

FIGURE 3-17.
Predicted Drawdown; 40-mgd Well Field
in DeSoto Property (Upper Floridan Aquifer).

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potential for developing high-yield wells (1 to 3 mgd) indicates a viable source of water for blending or desalination.

TDS content of the water increases sharply toward the south. From about North Port southward, the aquifer is probably too saline to be suitable as a water source. Alternative sources of water for desalination are more readily available.

CONCLUSIONS AND RECOMMENDATIONS

The best sources of groundwater within the service area are the upper and lower Floridan aquifers in southwestern DeSoto County. Both zones are highly productive here and contain water that is just slightly above drinking water standards for sulfates and TDS. This mineral content would need to be reduced before usage. Possible means of developing this groundwater source are:

- Storage of fresh treated or untreated surface water in the aquifer, followed by withdrawal of a mixture of native and stored water for blending with fresh surface water treated by conventional processes at the Peace River WTP.

- Withdrawal of the native groundwater for blending with fresh surface water, followed by treatment by conventional processes.

- Withdrawal of the native groundwater for treatment by desalination, followed by either direct use or use after blending with fresh (treated) surface water.

An estimated 20 to 40 mgd of groundwater can be obtained from the aquifer to supplement fresh surface water sources. The main constraint on the quantity of water available is the lowering of water levels in the source aquifer from the withdrawal, and the effects of this lowering upon adjoining users of the aquifer. Simulations of a 40-mgd well field in the Peace River WTP area predict drawdowns of 10 feet or more extending 1 to 2 miles beyond GDC property boundaries. The effect of drawdowns of this magnitude on the movement of the saline water in other parts of the aquifer would also need to be addressed.

In the rest of the service area, TDS content in both the upper and lower Floridan is for the most part well above drinking water limits. The most feasible method of developing a groundwater supply in these areas would probably be desalination of upper Floridan aquifer water.

While the potential for developing relatively small water supplies from the shallow artesian aquifer in the central part of the service area cannot be reliably estimated from limited available data, it should be considered. Test wells should be constructed at selected locations within the potentially favorable area to evaluate the potential of this source. The most probable mode of development would be as a supplementary source for meeting peak demands.

The surficial aquifer appears to have no significant potential as a developable water source, although it does provide recharge to the underlying shallow artesian aquifer.

No apparent major sources of groundwater suitable for conventional treatment occur within the service area. The only nearby groundwater sources of this type are located in northern Sarasota County and southwestern DeSoto County, several miles outside existing GDC holdings.

SECTION 4
Water Conservation and Wastewater Reuse Options

Section 4
WATER CONSERVATION AND WASTEWATER REUSE OPTIONS

Current water conservation and wastewater reuse practices in the Port Charlotte service area are summarized in this section. Other options for conservation and reuse are evaluated and a proposed water conservation and wastewater reuse plan presented. The four wastewater treatment plants (WWTPs) currently operating in the service area are South Port, North Port, East Port (previously known as Bionitrogen or Flag Area), and Gulf Cove.

CURRENT PRACTICES

WATER CONSERVATION

As a private utility with no legal authority to enact or enforce legislation governing water conservation, GDU has nevertheless been active in support of rules and regulations that promote and encourage conservation measures. These regulations generally call for either the use of water-conserving devices (e.g., low volume water closets and low flow fixtures for showers and faucets) or restrictions on major consumption to hours when evaporation loss and peaking conditions are minimized.

GDU and its parent corporation, GDC, have consistently supported the use of mandatory water-saving devices, which typically conserve up to 10 percent of potable water usage. The housing division of GDC has been installing such devices in all new construction for the past four years. This is in accordance with Subsection 553.14 of the Florida Statutes, known as the "Water Conservation Act," which requires the use of water-conserving fixtures in new buildings constructed subsequent to September 1, 1983, and in certain substantial renovations.

GDU also supports the passage of resolutions by communities in the Port Charlotte service area that recommend practices for lawn irrigation and other physical and operational actions that promote water conservation by restricting usage to lower-demand time periods.

GDU water rates are composed of a base facility charge, covering the availability of service, and an actual use charge, metered and billed on a per gallon basis. A similar procedure is used for billing sewer service. This type of billing structure helps to promote water conservation, because it establishes a direct relationship between reductions in water use and cost savings.

GDU is actively involved in promoting public awareness of water conservation through its "Slow the Flow" program (see Appendix B) and the "Residential Xeriscape" project. The multi-dimensional "Slow the Flow" public information program is especially tailored to GDU customers, including young people, new residents, and the business community. Activities of the program include insertion of informational brochures in customer bills, plant tours for children, and a "career shadowing" presentation for high school students interested in utility work as a career. In addition, slide shows and public meetings are held with interested local organizations to acquaint as many people as possible with water conservation measures and goals. The "Residential Xeriscape" project is a local demonstration site where creative landscaping is used to promote water conservation methods. The project is partially funded by a SWFWMD matching grant to GDU.

Finally, the effectiveness of water supply, treatment, and distribution facilities in the Port Charlotte service area has a major impact on the use and conservation of water resources. Leaks in pressurized raw and finished water transmission lines can lead to significant water losses. These are known as "unaccounted for losses", because they appear as discrepancies in the amounts pumped from the supply wells, treated in the plant, and billed to the customers. These losses are caused by leaks in the system, unmetered flows generated from fire fighting and flushing fire hydrants, meter inaccuracies, and stolen water. The "unaccounted for loss" category represents a small percent of total water use in the Port Charlotte system, and is well within industry standards.

WASTEWATER REUSE

Treated wastewater can augment or replace a water supply with limited expansion capacity if required criteria for water quality can be met satisfactorily. Existing reuse sites in the Port Charlotte service area are shown in Figure 4-1. Treated effluent from the South Port WWTP is currently pumped and applied to the spray irrigation site at the East Port WWTP. Treated effluent from the North Port WWTP is currently applied to either a land application site or to the North Port golf course for irrigation. Treated effluent from the East Port WWTP is applied to an onsite irrigation area. Percolation ponds are the sole means of effluent disposal at the Gulf Cove WWTP.

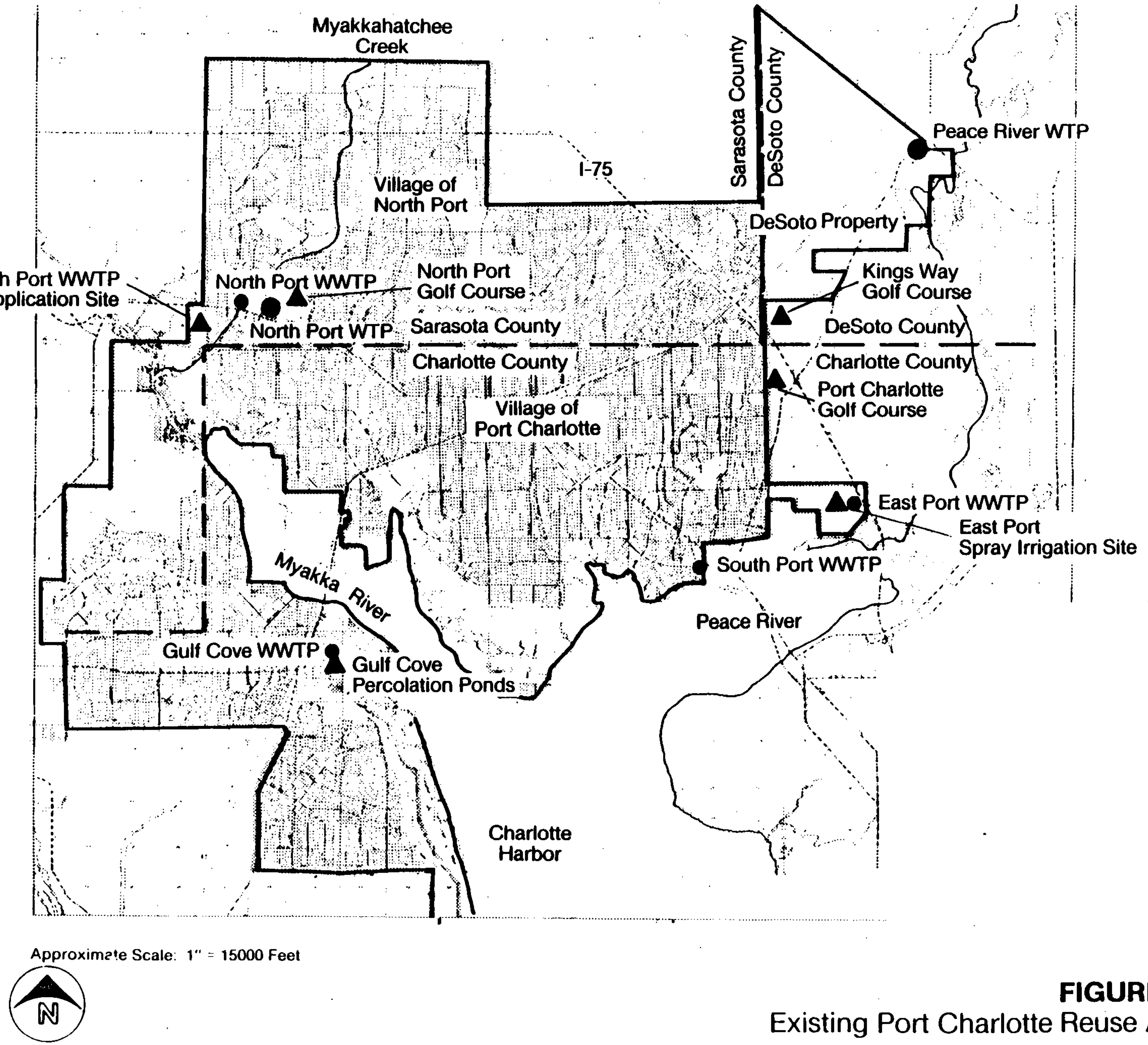

FIGURE 4-1.
Existing Port Charlotte Reuse Areas.

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OPTIONS

WATER CONSERVATION

Viable conservation options can be classed as either demand-side options or supply-side options. Demand-side options are aimed at conserving water through reduced demand, while supply-side options target supply enhancement.

Demand-Side Options

Consumer cooperation and acceptance are the key to realizing effective reductions when initiating any of the demand-side options, which are generally easy to implement, flexible, and inexpensive. Demand-side conservation options include: residential water conservation, agricultural water conservation, industrial and commercial conservation, water rate structure alteration, public education, legislative regulation, government trend setting, and economic incentives.

Residential Water Conservation. Residential water usage occurs both indoors and outdoors, with approximately 50 to 75 percent of residential consumption resulting from indoor bathroom and kitchen activities. Indoor residential water conservation can be accomplished through the installation of various water-saving and flow-reducing devices, while outdoor residential water conservation involves modifying consumer methods and procedures related to lawn watering and general household and yard maintenance. Besides significantly reducing water usage, water conservation measures usually result in lower energy requirements, reduced wastewater flows, and overall monetary savings to consumers.

Agricultural Water Conservation. Alternative irrigation systems to traditional ditch and overhead irrigation practices have been demonstrated to save significant quantities of water and energy. The alternative systems include: semi-closed (seep) sub-irrigation, subsurface tile sub-irrigation, water recovery and recycle systems, and trickle irrigation. Agriculture in the Port Charlotte service area is limited to orange groves, sod farms, and pasture lands. No significant water conservation can be undertaken in this category.

Industrial and Commercial Conservation. Approaches to water conservation for industrial and commercial establishments may include installing residential-type water saving devices, landscaping conservation measures and devices, and process modifications for water recovery and reuse. Although Port Charlotte has no industry, commercial establishments can be advised in the same fashion as in residential water conservation.

Water Rate Structures. Reductions in water consumption may be possible through changes in water rate structures. A summary of water rate structures and their influence on water conservation is presented in Table 4-1.

Public Education. Public education not only serves as an independent option in a water conservation program, but also is instrumental in supporting other possible programs before and during their implementation. A summary of public education techniques is presented in Table 4-2. GDU began a public awareness program on water conservation in February of 1982. The "Slow the Flow" program (see Appendix B) addressed such topics as lawn irrigation, planting and landscaping with native flora, watering with soaking hoses, how to find water leaks, the use of water displacement devices in toilets, and other such information--all of which stress the reduction of domestic water usage.

Legislative Regulation. Water consumption can be reduced through the regulatory powers of state, county, and city governments, as well as through regional agencies and water management districts. As a private utility with no legal authority to enact or enforce legislation governing water conservation, GDU has nevertheless been active in support of rules and regulations that promote and encourage conservation measures.

Government Trend Setting. Other than direct regulation, governmental bodies can promote water conservation through funding assistance programs and general purchases and expenditures. Grant programs providing matching funds for water conservation measures, and state and federal moneys for research and construction of water use and conservation projects are currently available in Florida.

Economic Incentives. Water conservation can be promoted through means other than increasing prices to reduce consumption. Incentives for installing water-saving devices in homes can include providing free devices and installation, or as a result of such installation, implementation of guaranteed reduced water and sewer fees for that particular residence.

Supply-Side Options

Rather than relying on reduced water consumption (as demand-side options do) for water conservation, supply-side options are aimed at improving the efficiency with which the water is handled by the supplying utility. Prior to adopting a program to minimize system water losses, the extent and sources of unaccounted for water must be determined. Unaccounted for water, the difference between the water put into the distribution system and the sum of the total

Table 4-1
SUMMARY OF WATER RATE STRUCTURES

Type of Structure	Description	Potential for Water Conservation
Average Cost Pricing	Prices for water are based on the average cost of supplying water to the consumer	None; no incentive to conserve water
Set Price	Consumer pays a set price for any amount of water used, with no quantity limits	None; price of water not matched to quantity of water consumed
Decreasing Block Rate	Price per unit of water decreases in a stepwise fashion. Consumer pays one price for a certain quantity of water and a lower price for water used beyond this quantity	None; no incentive to conserve water and may encourage large users to use more water
Uniform Rate (Single Block Rate)	Price per unit of water is constant, regardless of quantity consumed	Some; total cost of water matched to quantity consumed
Increasing Block Rate	Price per unit of water increases in a stepwise fashion (just opposite of Decreasing Block Rate)	Some; increased cost of water matched to increased consumption
Peak Demand Rates	Rates structured to stabilize demand for water (variable season rates)	Some; increasing cost of water seasonally matched to increased consumption
Lifeline Rates	Minimum basic quantity of water sustaining a minimum standard of living is assigned a low fixed rate. Quantities above this minimum are subject to much higher rates	Limited; incentive to conserve only for those who consume above the minimum basic quantity

Note: Adapted from Water Conservation Options Inventory, Planning and Performance Evaluation Section, Southwest Florida Water Management District. June 1984.

Table 4-2
SUMMARY OF PUBLIC INFORMATION PROGRAMS

Type of Programs	Advantages	Disadvantages
Direct Mail--Utility Bill Inserts	o Postage-free way of conveying tips and announcements	o Reaches only water customers and not all service area water users
Direct Mail--Newsletter	o Increased amount of information sent independent of billings o Information reaches non-customer water users	o Additional mailing costs
News Media	o Wide range of exposure possible	o May be expensive o Professional outside help may be required
Personal Contact	o Immediate feedback o Increased consumer response to material	o Time consuming

Note: Adapted from Water Conservation Options Inventory, Planning and Performance Evaluation Section, Southwest Florida Water Management District. June 1984.

metered sales and the estimated unmetered water usage, may be a result of any or all of the following: faulty or slow registering meters, significant unmetered usage, and underground leaks. Establishing a meter change-out and master meter calibration program, tracking unmetered water usages, and detecting and repairing underground leaks are all methods to reduce unaccounted for water.

All GDU Port Charlotte area customers are metered for water usage. Currently, approximately 10 percent of the total water flow is unaccounted for, with approximately half of this water consumed in unmetered uses such as fire fighting and main flushing. In addition, GDU maintains a program for changing-out and calibrating meters within the water distribution system.

WASTEWATER REUSE

Although reuse definitions vary, the use of secondary or tertiary wastewater effluent in an application that replaces the use of potable water constitutes a "reuse" option. Reuse systems may also recycle to natural environmental settings such as riverine surface waters and wetlands, enhancing the natural hydrology of these important ecosystems. The feasibility of a particular wastewater reuse method depends on regulatory, public health, technical, and economic aspects of the method, as well as overall public opinion and acceptability.

Public and Private Landscape Irrigation

Public landscape irrigation systems apply treated wastewater to parks, golf courses, cemeteries, playgrounds, and other unrestricted public access areas. This method of wastewater reuse is practiced in Florida as permitted by the Florida Department of Environmental Regulation (FDER). Figure 4-1 shows primary areas in which irrigation of public land is or may be feasible.

Private landscape irrigation consists of the application of treated wastewater to residential lawns. The technical requirements are similar to those for public landscape irrigation. As a private utility, however, GDU cannot implement a private landscape irrigation system without exposing itself to potential liabilities.

Agricultural Irrigation

The citrus and sugarcane industries in Florida have had encouraging success with agricultural irrigation using secondary treated effluent. As of now, specific water quality standards have not been developed by regulatory agencies and there are many unresolved issues on the short-and long-term effects of this type of irrigation. This method may apply to Port Charlotte as there are orange groves, sod-producing fields, and cattle grazing lands in this service area.

Industrial Reuse

Industry can use reclaimed wastewater in once-through cooling systems and as makeup cooling water for closed cycle systems. Disadvantages of this usage include the need for additional treatment to reduce or prevent scaling, solids deposition, corrosion of piping systems, and excessive total dissolved solids. This method is considered essentially inapplicable in the Port Charlotte area, because there are no major industrial operations there.

Recycle to Surface Waters

Maximum withdrawal rates for potable water supply for up to 22.0 mgd of surface water from the Peace River are allowed by CUP No. 202923. The annual average permitted withdrawal rate is 6.12 mgd. Although extensive analysis has indicated that this withdrawal has not had significant impact on downstream components of the Peace River ecosystem or flora and fauna in Charlotte Harbor, a plan to recycle fresh water of equal or better water quality to the Peace River downstream of the Peace River WTP could provide environmental benefits. Treated municipal wastewater from the DeSoto or East Port WWTP might be available for such a reuse option, subject to certain environmental and regulatory constraints.

The three important environmental considerations for recycling treated wastewater to the Peace River are (1) water quality, (2) water quantity, and (3) timing. In general, if the water quality of treated wastewater meets or exceeds the water quality of the Peace River and the discharge of recycled water is timed to coincide with potable water withdrawals upstream, there are not likely to be any negative environmental consequences. How closely water quality and quantity must equal the ambient Peace River conditions to avoid a negative impact would be a matter for discussion and additional study.

Ambient water quality in the Peace River has been studied by the EQL and others since 1975. The typical range of water quality parameter values measured at the SR 761 Station are presented in Table 4-3, along with comparable Class III criteria. To meet or exceed average water quality conditions in the Peace River, the concentrations of total nitrogen and total phosphorus in treatment plant effluent must be further reduced through a higher level of pretreatment prior to recycle. Class III parameters that appear to be naturally violated in the Peace River at SR 761 under

Table 4-3
AMBIENT SURFACE WATER QUALITY IN THE PEACE RIVER AT SR 761 BRIDGE AND CLASS III WATER QUALITY CRITERIA

Parameter	Units	Peace River Mean	Peace River Range	FDER Class III
Alkalinity	mg/l as $CaCO_3$	53.1	14.4-90	>20
Ammonia (total)	mgN/l	0.086	0.001-0.560	--[a]
Ammonia (unionized)	mgN/l	--	--	0.02
Dissolved Oxygen	mg/l	6.9	2.8-13.0	5
Iron	mg/l	0.16	0.01-0.75	1.0
Nitrate + Nitrite	mgN/l	0.547	0.001-2.11	--[a]
Organic -N	mgN/l	1.11	0.341-3.34	--
Total -N	mgN/l	1.78	0.585-3.35	--[a]
Orthophosphorus	mgP/l	1.70	0.59-4.68	--[a]
Total -P	mgP/l	1.92	0.57-4.79	--[a]
pH	units	7.23	5.75-9.01	±1 unit or 6.0-8.5
Turbidity	NTU	4.49	0.20-37	+29
Fecal Coliforms	col/100 ml	93	3-1,300	200 (monthly avg) 400 (10% samples) 800 (any sample)
Total Coliforms	col/100 ml	494	5-3,100	1,000 (monthly avg) 1,000 (20% samples) 2,400 (any sample)

[a] Class III criterion for nutrients: "in no case shall nutrient concentrations of a body of water be altered so as to cause an imbalance in natural populations of aquatic flora and fauna".

some conditions are alkalinity, dissolved oxygen, fecal and total coliforms, and nutrients. Advanced wastewater treatment would be required to meet the average Peace River concentrations for nitrogen and phosphorus listed in Table 4-3. Such treatment would be expensive and may not be cost-effective.

A method for matching actual quantity and frequency of flows would be technically feasible as long as an alternative effluent disposal option is available. Discharge of the treated wastewater effluent would equal the previous day's withdrawal as measured at the Peace River WTP. A simpler management alternative would be to discharge all of the DeSoto WWTP treated effluent to the river. This discharge would approximately equal freshwater withdrawals from the river at the Peace River WTP, but with a one- to two-day lag.

Regulatory constraints to treated wastewater recycling are closely linked to the environmental restrictions. Minimum surface water quality criteria (Section 17-3.051, FAC) established by the FDER are designed to prevent serious degradation of surface waters by the discharge of substances that would create a "nuisance" or toxic conditions. General surface water quality criteria (Section 17-3.061, FAC) specify levels of parameters such as BOD, dissolved oxygen, pH, suspended solids, nutrients, and metals that, depending on the classification of the waters, cannot be significantly altered. Classification of waters of the State is based on intended usage, and ranges from Class I waters representing potable water supplies to Class V waters designated primarily for navigation and/or industrial use (Section 17-3.081, FAC). The Peace River at the likely discharge point for treated wastewater (about one-half mile downstream of the Peace River WTP) is designated as Class III surface waters by the FDER (Figure 4-2).

Tidally influenced wetlands and waters in the mouths of the Myakka and Peace rivers and adjacent bays north are classified as Class II waters (Section 17-3.161(2)(c)36, FAC), and are designated for shellfish propagation and harvesting. These waters are protected by the criteria described in Section 17-3.111, FAC. Section 17-6.080(1)d, FAC, states that "...outfalls shall not discharge effluent into Class II waters." Facilities which would discharge to water tributary to or contiguous to Class II waters shall be required to conform to additional standards as set by Section 17-6.080(1)(e), FAC, if the travel time of effluent (the elapsed time from the point of final disinfection monitoring to arrival at conditionally approved or approved shellfish harvesting areas during maximum expected surface water velocities) is less than or equal to 72 hours.

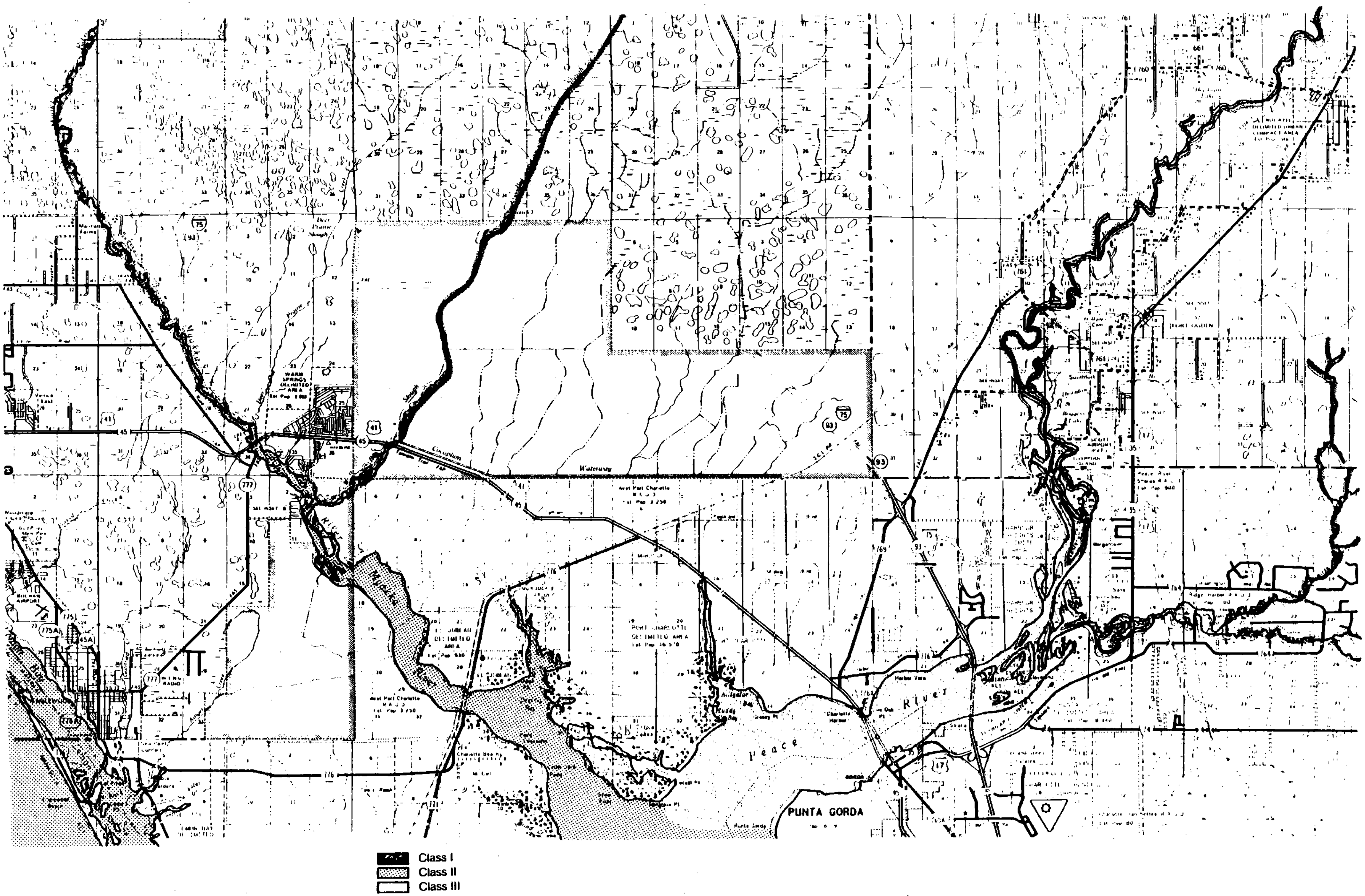

FIGURE 4-2.
State of Florida Water Quality Classifications
in Port Charlotte Area.

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All other connected canals and natural inland wetlands, including the flood plain forest along the Peace River, are Class III waters (Section 17-3.161(1), FAC) with a designated use for recreation and propagation and maintenance of a healthy, balanced population of fish and wildlife. Specific criteria applicable to this classification are designed to maintain the minimum conditions necessary for the stated use (Section 17-3.121, FAC). Secondary treatment as defined in Section 17-6 is required as a minimum before discharge to these waters.

All of Charlotte Harbor south of the U.S. 41 bridge and Gasparilla Sound have been designated Outstanding Florida Waters (OFW) (Section 17-3.042(r)h, FAC) and receive the highest water quality protection by the FDER as stated in Section 17-4.242, FAC. In particular "... the existing ambient water quality within an OFW will not be lowered as a result of proposed activity or discharge..." (Section 17-4.242(2)6, FAC). In addition, man-induced nutrient enrichment (total nitrogen or total phosphorus) shall be considered degradation in relation to the provisions of Section 17-3.041, FAC.

The current high nutrient levels in the Peace River indicate that additional assimilative capacity may not be available. Therefore, under current regulations, it is realistic to assume that pretreatment to ambient water quality conditions would be a prerequisite for discharge of treated wastewater effluent.

Other Options

Options used for reuse of treated wastewater can also include recreational impoundments, direct groundwater recharge, salinity intrusion control, and discharge to wetland systems. None of these options is considered feasible in the Port Charlotte area.

SELECTED CONSERVATION AND REUSE PLAN

GDU has a well-earned reputation as an environmentally aware private utility. An "open door" policy at all of their facilities fosters pride and excellence in system operators. It also encourages public interest in the business of supplying water and treating wastewater. This helps make the public more aware of their water resources every time they turn on the tap.

GDU believes that workable conservation measures and the protection of the water resources of Florida are in the best interest of the general public. The development of effective conservation methods, whether conservation of potable water or reuse of treated wastewater, is strongly supported by GDU. The most important concept that GDU has recognized is that the goal of all potable water conservation and wastewater reuse programs is exactly the same: water resources conservation. Effective implementation of any conservation/reuse program is contingent upon the recognition of the benefits of such a program by the people who must bear the costs--the consumers.

Water conservation in the Port Charlotte service area will be successful through the implementation of several demand-side options. Because water losses and unaccounted for water are not a significant problem in the Port Charlotte area (and other supply-side options are not applicable), no supply-side options are recommended. The following demand-side options should be implemented by GDU to promote water conservation and reduce water consumption:

1. Initiate a residential water conservation program by promoting and providing water conservation devices.

2. Continue with public education programs, such as the "Slow the Flow" literature distribution campaign, the "Residential Xeriscape" project, and other public awareness programs in the Port Charlotte service area.

3. Investigate economic incentives for water conservation.

Land application will continue to be the primary method for wastewater reuse in the service area, because agricultural irrigation and industrial reuse are not appropriate for this area.

The following specific options are included in the selected plan:

1. Treated effluent from the South Port and East Port WWTPs will continue to be applied to the East Port spray irrigation site. This system is currently being expanded.

2. Treated effluent from the North Port WWTP will continue to be applied to the North Port golf course.

3. Treated effluent from the Gulf Cove WWTP will continue to be applied to percolation ponds.

4. Although prohibited or rendered economically infeasible by current regulations, surface discharge alternatives should continue to be explored. If environmentally-sound and cost-effective means can be developed and permitted, this option could provide for reuse by returning high quality treated water to the environment.

SECTION 5
Storage Options at North Port WTP

Section 5
STORAGE OPTIONS AT THE NORTH PORT WTP

The evaluation in Section 3 concluded that the Myakkahatchee Creek would not be a reliable raw water source for the North Port WTP without significant offline surface storage or ASR capacity. To assess the potential for using offstream surface storage, ASR, or a combination of the two with treatment facilities at the North Port WTP to increase the reliability of Myakkahatchee Creek as a water source, conceptual storage and treatment facility alternatives were evaluated with the PLANT computer model developed for the 1985 Peace River ASR feasibility study (CH2M HILL, April 1985b). The computer simulations were used to estimate expected reliability for each alternative.

For each simulation, the North Port WTP was considered as an independent source supplying the demand of a portion of the service area. Maximum day demands were based on the maximum day to average day flow ratio of 1.60 developed in Section 2 and it was assumed that plant capacity requirements equal maximum day demands. For example, the existing WTP with a 4.4-mgd rated capacity can meet average and maximum day demands of 2.7 mgd and 4.4 mgd, respectively, for a given portion of the GDU service area. The WTP capacity requirements can be smaller to meet specified demands if ASR facilities are available to meet peak demands. It was also assumed that plant capacities must be at least as large as average day demands, even with ASR facilities.

PLANT SIMULATION MODEL

The PLANT computer model consists of a main program and three subroutines that perform the required calculations for monthly flow distribution, water quality, storage quantity, and time series statistics for a given water supply facility. A detailed description of the model and its use is presented in CH2M HILL's 1985 report on the ASR feasibility study (CH2M HILL, April 1985a).

The PLANT model requires two input files for each simulation. One file contains the monthly divertible streamflow quantity and quality data developed in Section 3; the other contains the water supply system parameters. The system parameters are grouped into four general categories: (1) demand data, (2) plant data, (3) surface reservoir data, and (4) ASR data. The key variables used in the model are listed in Table 5-1.

Table 5-1
VARIABLES USED IN THE PLANT MODEL

Monthly demand flows (mgd)

Quality standard (TDS) (mg/l)

Monthly TDS added by treatment plant (mg/l)

Treatment plant capacity (mgd)

Minimum treatment rate (mgd)

Initial volume in surface reservoir (ac-ft)

Initial surface reservoir quality (TDS, mg/l)

Initial surface area of reservoir (acres)

Minimum allowable surface reservoir volume (ac-ft)

Maximum allowable surface reservoir volume (ac-ft)

Monthly rainfall input to surface reservoir (inches)

Monthly evaporation from surface reservoir (inches)

Volume levels of surface reservoir (ac-ft)

Surface area levels of surface reservoir (acres)

Elevation levels of surface reservoir (ft)

Native groundwater quality (TDS, mg/l)

Initial volume of injected water in aquifer (MG)

Initial injected water quality (TDS, mg/l)

Aquifer water quality parameter, ALFA (dimensionless)

Aquifer water quality parameter, BETA (dimensionless)

Maximum ground reservoir injection rate (mgd)

Maximum ground reservoir withdrawal rate (mgd)

The PLANT model provides an estimate of the expected reliability of the simulated water supply facility by counting the number of months during which both quantity and quality requirements are satisfied. This total is divided by the total number of months in the simulation period and multiplied by 100 to obtain percent reliability. Because the PLANT model simulates monthly operation, no information is provided for shorter periods (e.g., one day or one hour). Therefore, it is possible that even if a given month has an average TDS concentration greater than 500 mg/l and is considered a failure, acceptable water may still have been produced for a portion of that month.

CONCEPTUAL STORAGE AND TREATMENT FACILITY ALTERNATIVES

The conceptual diagram of a possible water supply facility for the North Port WTP shown in Figure 5-1 illustrates the relationship between offstream surface storage, ASR, and treatment plant facilities that may be used to improve water supply reliability. With the PLANT model, varying capacities of each of these components can be evaluated for their impact on the quantity and quality of finished water produced by the facility.

For the Port Charlotte service area, the following three basic cases were evaluated:

Case 1: Offstream surface storage only

Case 2: ASR only

Case 3: Combined surface and aquifer storage

Use of ASR for raw water storage was not addressed.

CASE 1: OFFSTREAM SURFACE STORAGE ONLY

Figure 3-5 in Section 3 showed the expected cumulative frequency of consecutive months with no divertible flow from Myakkahatchee Creek. Based on the relationship described, it is apparent that about 10 months of storage capacity will be required for development of a reliable water supply from Myakkahatchee Creek.

Table 5-2 presents a summary of the approximate storage volume and land area requirements for yields ranging from 1 to 12 mgd and a 10-month storage volume. Land area requirements for an offstream storage reservoir at the North Port WTP site would be rather extensive, even for relatively

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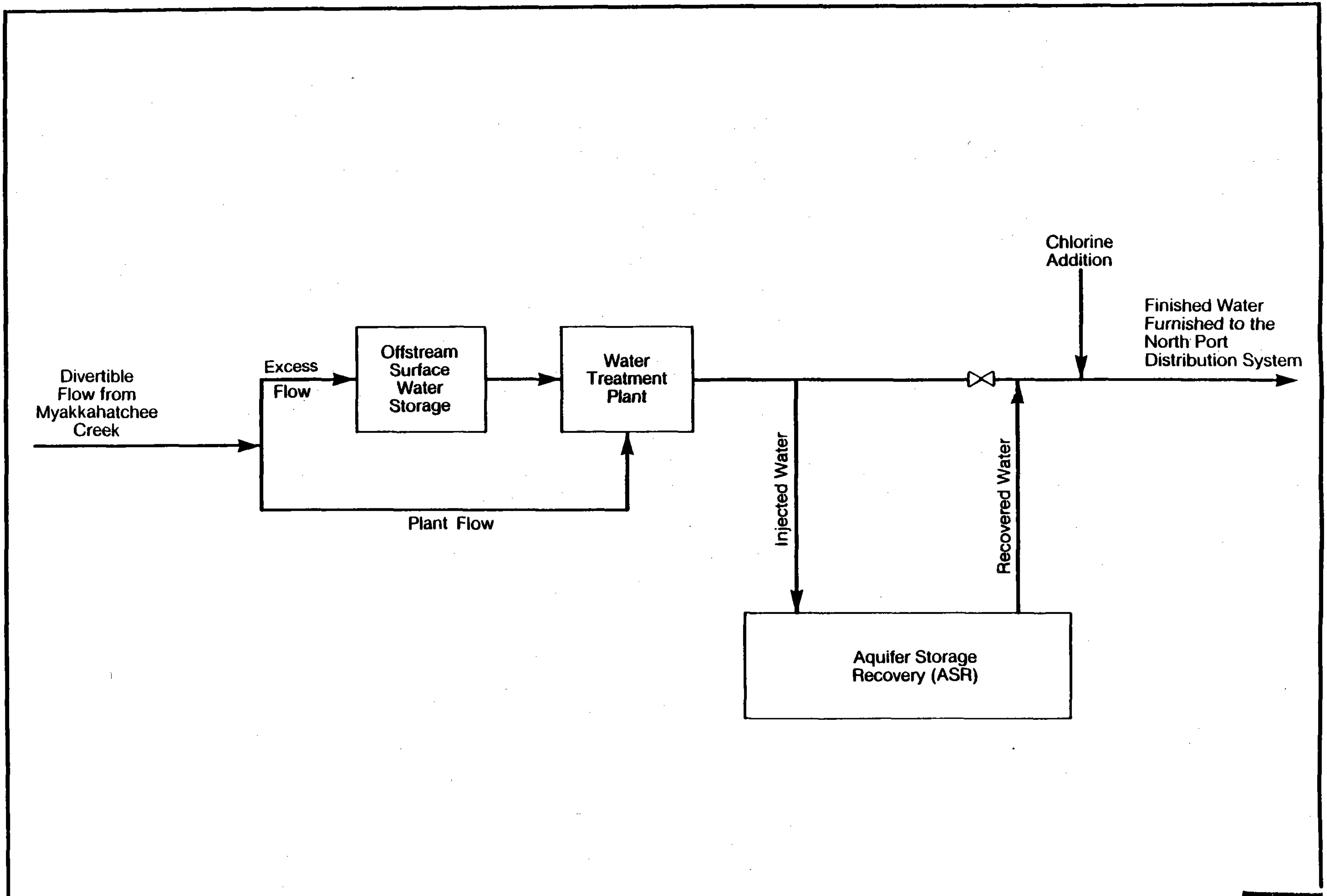

FIGURE 5-1.
Conceptual Diagram for the North Port Water Supply Facility.

Table 5-2
SURFACE STORAGE REQUIREMENTS FOR NORTH PORT WTP

Desired Yield[a] (mgd)	Storage Requirements[b] (ac-ft)	Approximate Area Requirements[c] (ac)
1	930	60
2	1,870	115
4	3,730	220
6	5,600	320
8	7,460	425
12	11,200	620

Notes:

a. Desired yield = Average daily flow (ADF).

b. Based on 10 months storage volume.

c. Area based on a square reservoir excavated 10 feet below land surface and bermed 15 feet above land surface with a 20-foot working depth. Side slopes are 3 horizontal to 1 vertical. Top of berm width is 15 feet. A 15-foot wide maintenance easement is assumed on the perimeter.

small yields. As shown in Table 5-2, the total land area required to reliably supply average daily flows of 4 and 12 mgd would be approximately 220 and 620 acres, respectively. Treatment plant capacities required to meet 4-mgd and 12-mgd average yields would be 6.4 mgd and 19.2 mgd (1.6 times average day flow rate), respectively.

The existing North Port WTP (4.4-mgd rated capacity) would require about 2,500 acre-feet of surface storage to reliably meet an average daily flow of 2.7 mgd with a finished water TDS concentration not exceeding 500 mg/l (interpolated from Table 5-2). Land area required to provide this storage would be about 150 acres.

Existing and projected development has limited the available land areas suitable for offstream surface storage. As a result, surface storage alone does not appear to be a feasible alternative.

CASE 2: ASR ONLY

In areas where aquifer conditions are favorable, ASR has been shown to be a feasible storage alternative. The potential for ASR may be limited by the availability of a suitable aquifer storage zone, and the background water quality, including TDS, in the specific aquifer used.

To simulate WTP operation with ASR facilities with the PLANT model, the following simulation parameters and corresponding values were used:

1. Native groundwater quality = 2,700 mg/l TDS

2. Initial volume of injected water in aquifer

   Alternative 1: 3 months of storage
   Alternative 2: 30 months of storage

3. Initial injected water quality = 340 mg/l TDS

4. Aquifer mixing parameter ALFA = 0.9 and 1.0

5. Aquifer mixing parameter BETA = 0.9 and 1.0

Native groundwater quality was estimated from the previous CH2M HILL report (April 1985a). The groundwater quality is representative of the Suwannee limestone zone of the upper Floridan aquifer. Two alternatives for initial injected water volume were simulated to determine the sensitivity of the system to this parameter. The initial injected water quality was estimated as the annual average TDS value for Myakkahatchee Creek plus TDS added during treatment (estimated to be 100 mg/l). The ALFA and BETA parameters were estimated using data collected from ASR operations at the Peace River WTP and are defined below.

When treated water is injected into the aquifer, it will displace the native waters near the injection wells. The maximum volume available for storage of injected waters is unlimited. However, the amount in storage at any given time will be the sum of all waters previously injected, less the sum of all injected waters previously recovered. The quality of the injected waters is assumed to be the composite quality of all injected waters in storage at any given time.

The blend of injected and native waters withdrawn from the aquifer during a recovery period is computed by application of an exponential relationship. This empirical relationship does not simulate the mixing process itself, but only defines the results (i.e., response) of the aquifer mixing. The relationship and its parameters are given below:

$$FI = ALFA\ e^{-\ BETA\ (VRT/VIB)}$$

where:

FI	=	Instantaneous fraction of injected water contained in recovered water mixture
VRT	=	Total volume recovered since beginning of recovery cycle
VIB	=	Volume of injected water in aquifer storage at beginning of recovery cycle
ALFA and BETA	=	Aquifer mixing parameters

ALFA and BETA should be determined by analysis of onsite ASR testing data. Since there are no data available at the North Port WTP site, data collected from ASR operations at the Peace River WTP were used to define these values. Physically, an ALFA value of 0.9 means that the initial blend of recovered water will be 90 percent injected water and 10 percent native water. An ALFA value of 1.0 indicates a perfect aquifer storage system, in which the initial blend of recovered water will be 100 percent injected water, i.e., initial mixing with native groundwater does not occur. The portion of native water in the recovered mix will increase during the recovery period. A BETA value equal to the ALFA value indicates that all injected water will be available for withdrawal and that none will be lost because of overall groundwater movement.

Analysis of the Peace River ASR data indicates ALFA and BETA values of approximately 0.9, which are assumed to be the expected values at North Port. ALFA and BETA values of 1.0 are also considered in this analysis, to establish a theoretical upper limit on potential ASR performance at North Port.

Simulation runs were conducted for the ASR facilities shown in Table 5-3. Treatment plant capacities considered ranged from 2 to 24 mgd; desired yield simulated ranged from 1 to 12 mgd. In each case, the ASR capacity (RATE) was equal to the treatment plant capacity or the maximum daily pumpage (equal to 1.60 times average day flow), whichever was larger, to ensure a 100 percent hydraulic reliability. Initial volume of treated water in aquifer storage (VGR) was set at 3 months and 30 months.

In general, the PLANT model tracks both hydraulic and quality related failures (quality failures occur when finished water exceeds 500 mg/l TDS). For these simulations, the maximum injection and recovery rates were equal to or greater than the demand. Because native water may be pumped to meet these demands, only quality-related failures are expected to occur.

The expected reliabilities for the facilities described in Table 5-3 are summarized in Part A of Table 5-4 for both the 3- and 30-month initial storage cases. The simulation results indicate that ASR alone (i.e., no surface storage) cannot be used to develop a reliable water supply at the North Port WTP, if aquifer storage characteristics are the same as those observed at the Peace River WTP. The best reliability reported is only 64 percent, which means that the simulated system will meet the 500 mg/l TDS standard in fewer than 2/3 of the months. In addition, the plant capacity required to achieve this relatively low reliability is twice the desired yields and the ASR capacity required is 1.6 to 2.0 times the desired yields, expressed as average daily flows.

Increasing the initial storage volume from 3 to 30 months did not significantly increase system reliability. Increasing the initial storage volume did, however, have an impact on the magnitude of the quality failures. Recovered water TDS values of up to 1,000 mg/l were obtained when simulations used 3 months of initial storage; recovered water TDS concentrations greater than 700 mg/l seldom occurred, however, when 30 months of initial storage were provided. Therefore, initial storage volume had an impact on the magnitude of the TDS failures, but not on the frequency of failure.

Table 5-3
INITIAL ASR STORAGE VOLUMES AND MAXIMUM ASR FLOW RATES VS
YIELD AT NORTH PORT WTP

	VGR and RATES (as a Function of Plant Capacity in mgd)									
	2.0		4.0		8.0		12.0		24.0	
Desired Yield (mgd)	VGR (MG)	RATE (mgd)	VGR (MG)	RATE (mgd)	VGR (MG)	RATE (mgd)	VGR (MG)	RATE (mgd)	VGR (MG)	RATE (mgd)
1.0	90	2.0	90	4.0	90	8.0	90	12.0	90	24.0
2.0	180	3.2	180	4.0	180	8.0	180	12.0	180	24.0
4.0	--	--	360	6.4	360	8.0	360	12.0	360	24.0
8.0	--	--	--	--	720	12.8	720	12.8	720	24.0
12.0	--	--	--	--	--	--	1,080	19.2	1,080	24.0

Notes:

1. Desired Yield = Average daily flow (ADF).
2. VGR = Initial volume in ground reservoir. Listed for 3 months of storage only. 30-month storage values are 10 times the values listed.
3. RATE = Maximum injection and recovery rates.

Table 5-4
EXPECTED WATER SUPPLY SYSTEM RELIABILITY FOR ASR FACILITY

A) ALFA = BETA = 0.9 (expected conditions)

Desired Yield (mgd)	% Reliability (as a Function of Plant Capacity in mgd)									
	2.0 mgd Initial ASR Storage		4.0 mgd Initial ASR Storage		8.0 mgd Initial ASR Storage		12.0 mgd Initial ASR Storage		24.0 mgd Initial ASR Storage	
	3 Month	30 Month	3 Month	30 Month	3 Month	30 Month	3 Month	30 Month	3 Month	30 Month
1.0	64.1	64.1	64.1	64.1	64.1	64.1	64.1	64.1	64.1	64.1
2.0	56.2	59.3	64.1	64.1	64.1	64.1	64.1	64.1	64.1	64.1
4.0	--	--	56.2	59.3	63.7	63.7	63.9	63.9	63.9	63.9
8.0	--	--	--	--	54.9	57.6	62.3	62.3	62.3	62.3
12.0	--	--	--	--	--	--	52.3	55.1	61.6	61.6

B) ALFA = BETA = 1.0 ("perfect" aquifer storage system)

Desired Yield (mgd)	% Reliability (as a Function of Plant Capacity in mgd)									
	2.0 mgd Initial ASR Storage		4.0 mgd Initial ASR Storage		8.0 mgd Initial ASR Storage		12.0 mgd Initial ASR Storage		24.0 mgd Initial ASR Storage	
	3 Month	30 Month	3 Month	30 Month	3 Month	30 Month	3 Month	30 Month	3 Month	30 Month
1.0	95.8	98.6	98.8	100	99.5	100	99.8	100	100	100
2.0	56.2	61.3	96.3	99.1	98.8	100	99.3	100	99.8	100
4.0	--	--	62.7	67.1	95.8	98.6	97.7	99.5	99.3	100
8.0	--	--	--	--	55.1	59.9	75.2	84.5	97.2	99.3
12.0	--	--	--	--	--	--	52.6	57.4	91.4	97.7

Notes:

1. Desired Yield = Average daily flow (ADF).
2. Plant Capacity = Maximum treatment rate.
3. Reliability is estimated by summing the number of months without quantity or quality related failures (a failure occurs when the demand or quality standard are not met), dividing this sum by the total number of months in the simulation period, and multiplying by 100 percent.

Part B of Table 5-4 presents the expected reliabilities if perfect aquifer storage characteristics are assumed. In this case, reliabilities of up to 100 percent were simulated. This means that ASR alone may work if there is no initial mixing of injected and native waters, an unlikely circumstance. Increasing the initial storage volume from 3 to 30 months did not have a significant effect on overall system reliability.

Based on the results of these simulations and expected aquifer storage characteristics, it is concluded that finished water of the desired quality (i.e., TDS less than or equal to 500 mg/l) cannot be produced on a reliable basis by conventional water treatment supplemented by ASR alone. This is due in part to the marginal quality of the Myakkahatchee Creek water supply and in part to the poor quality of the native groundwater in the storage aquifer.

CASE 3: COMBINED SURFACE AND AQUIFER STORAGE

The potential for using surface storage alone is limited by the large storage and associated land requirements, while ASR reliability is limited by poor background water quality in the aquifer. A combined facility, however, may be used to reduce surface storage requirements and continue to meet demand and quality goals.

Simulation runs were conducted for several combined-facility alternatives using the PLANT model. The facilities tested were identical to those shown in Table 5-3, with the exception that 3 months of surface storage capacity was included for each simulation.

The expected reliability for the alternative facilities described above is summarized in Part A of Table 5-5 for both 3-month and 30-month initial aquifer storage. The surface reservoir storage requirements in acre-feet and approximate land area required for each desired yield are also shown in Table 5-5. Reliability is reported in percent of monthly failures. Because ASR is included, no hydraulic failures occurred. As shown in Part A of Table 5-5, a combined facility using 3 months of surface storage with plant and ASR capacities at about twice the demand is expected to be nearly 98 percent reliable on a monthly basis. This reliability is relatively unchanged by the initial ground reservoir storage volume.

Part B of Table 5-5 presents the expected reliabilities of the combined surface storage/ASR system if perfect aquifer storage characteristics are assumed. In this case, overall system reliability is not particularly sensitive to the initial mixing characteristics of the storage aquifer,

Table 5-5

EXPECTED WATER SUPPLY SYSTEM RELIABILITY FOR COMBINED SURFACE STORAGE-ASR FACILITY

A) ALFA = BETA = 0.9 (expected conditions)

Desired Yield (mgd)	% Reliability (as a Function of Plant Capacity in mgd) 2.0 mgd Initial ASR Storage 3 Month	2.0 mgd 30 Month	4.0 mgd 3 Month	4.0 mgd 30 Month	8.0 mgd 3 Month	8.0 mgd 30 Month	12.0 mgd 3 Month	12.0 mgd 30 Month	24.0 mgd 3 Month	24.0 mgd 30 Month
1.0	98.2	98.6	99.8	99.8	--	100	99.8	100	100	100
2.0	74.1	81.2	98.4	98.6	99.5	99.8	99.8	100	99.8	100
4.0	--	--	74.8	82.6	98.2	99.1	--	99.8	99.8	100
8.0	--	--	--	--	73.4	82.4	95.6	96.1	--	99.8
12.0	--	--	--	--	--	--	74.3	81.0	98.4	99.1

B) ALFA = BETA = 1.0 ("perfect" aquifer storage system)

Desired Yield (mgd)	% Reliability (as a Function of Plant Capacity in mgd) 2.0 mgd Initial ASR Storage 3 Month	2.0 mgd 30 Month	4.0 mgd 3 Month	4.0 mgd 30 Month	8.0 mgd 3 Month	8.0 mgd 30 Month	12.0 mgd 3 Month	12.0 mgd 30 Month	24.0 mgd 3 Month	24.0 mgd 30 Month
1.0	100	100	100	100	100	100	100	100	100	100
2.0	76.6	81.5	99.8	100	100	100	100	100	100	100
4.0	--	--	84.3	86.6	99.8	100	100	100	100	100
8.0	--	--	--	--	74.8	81.7	--	99.5	98.4	100
12.0	--	--	--	--	--	--	75.2	80.3	99.1	100

Notes:

1. Desired Yield = Average daily flow (ADF).

2. Plant Capacity = Maximum treatment rate.

3. All simulations include 3 months of surface storage capacity, based on ADF.

Desired Yield (mgd)	Storage Requirement (ac-ft)	Approximate Area Requirement (ac)
1	280	20
2	560	40
4	1,120	75
8	2,240	140
12	3,360	200

4. Reliability is estimated by summing the number of months without quantity or quality related failures (a failure occurs when the demand or quality standard are not met), dividing this sum by the total number of months in the simulation period, and multiplying by 100 percent.

because there is more than one storage unit available for blending. Increasing the initial ASR storage volume from 3 to 30 months did not have a significant effect on overall system reliability.

CONCLUSIONS

If the Myakkahatchee Creek is to be relied upon to meet future water supply demands at the North Port WTP, significant storage must be provided. If this storage is provided only in the form of surface storage, approximately 10 months of storage volume is necessary. The potential for providing large volumes of offstream surface storage is limited by land availability and costs. For example, to fully use the existing 4.4-mgd North Port WTP to reliably produce an average daily flow of 2.75 mgd, a total of 2,560 acre-feet of raw water storage at a 145-acre reservoir site would be required.

Use of ASR alone (i.e., without surface storage) is limited by the low quality native groundwater (approximately 2,700 mg/l TDS). Even with very large initial stored water volumes, ASR alone is not a statistically reliable alternative for the North Port WTP.

A combined surface storage-ASR facility can reduce surface storage land requirements while meeting demand and quality requirements. A facility with 3 months of surface storage and plant capacity (including diversion and injection/recovery capacity) of about twice the demand is expected to be 100 percent reliable for quantity and nearly 98 percent reliable for quality. For example, a facility having 3-month surface storage (requiring a 45-acre reservoir site), combined with plant and ASR capacities of 4.4 mgd each, can reliably meet a 2.2-mgd average daily production rate.

Both surface water (only) storage and combination surface water-ASR storage facilities are technically feasible alternatives at the North Port WTP. However, they require substantial land area (which may not be available), and may be economically unfeasible. Of these two alternatives, the combination system is estimated to be the most feasible. Costs of various expansion alternatives will be addressed in Section 7.

Land area and building size requirements and O&M costs were developed based on the same assumptions and unit costs presented for well field A.

SECTION 6
Treatment and Supply Facilities Evaluation

Section 6
TREATMENT AND SUPPLY FACILITIES EVALUATION

This section summarizes and updates the findings of the North Port Water Treatment Plant Evaluation (CH2M HILL, January 1985), and presents the findings of a limited facilities evaluation of the Peace River WTP conducted in January and February 1987. Each plant is evaluated for compliance with generally accepted design standards, capacity, physical condition, and performance. In addition, the ability of the overall treatment process of each plant to meet current and expected water quality regulations is addressed. This consideration may be especially important in light of the 1986 Amendments to the 1974 Safe Drinking Water Act. As a result of the U.S. Congress' actions, some existing federal and state drinking water standards are expected to be revised and new standards established for several currently unregulated contaminants. Finally, sludge handling and disposal needs, as well as improved treatment for taste and odor control, have been considered.

Information on existing facilities and operations is based on one-day site visits to each WTP, record drawings, plant operation reports, and interviews with Sam Stone, Jerry Tindell, and Bob Sacilowski (North Port WTP only) of GDU. Sludge handling/disposal options and discussions of taste and odor treatment are developed from literature reviews and CH2M HILL experience rather than site-specific studies, and are general in nature.

NORTH PORT WTP

GENERAL DESCRIPTION

The North Port WTP has two nearly-identical treatment trains rated at 2.2 mgd each, for a total rated capacity of 4.4 mgd. Average water production for 1986 was 1.2 mgd. The plant was originally constructed in 1962 and expanded to its present capacity in 1974. Several process modifications have been made since then. A site plan and schematic flow diagram of the present configuration are shown in Figures 6-1 and 6-2, respectively.

The source of supply for the North Port WTP is the Myakkahatchee Creek, a low-turbidity, highly-colored surface water. The normal treatment mode for color removal includes conventional alum coagulation/flocculation, clarification, filtration, and disinfection. Limited softening capability is available when needed. Figure 6-3 shows the monthly average flow and hardness and color concentrations in the Myakkahatchee Creek. In dry months when flows are low,

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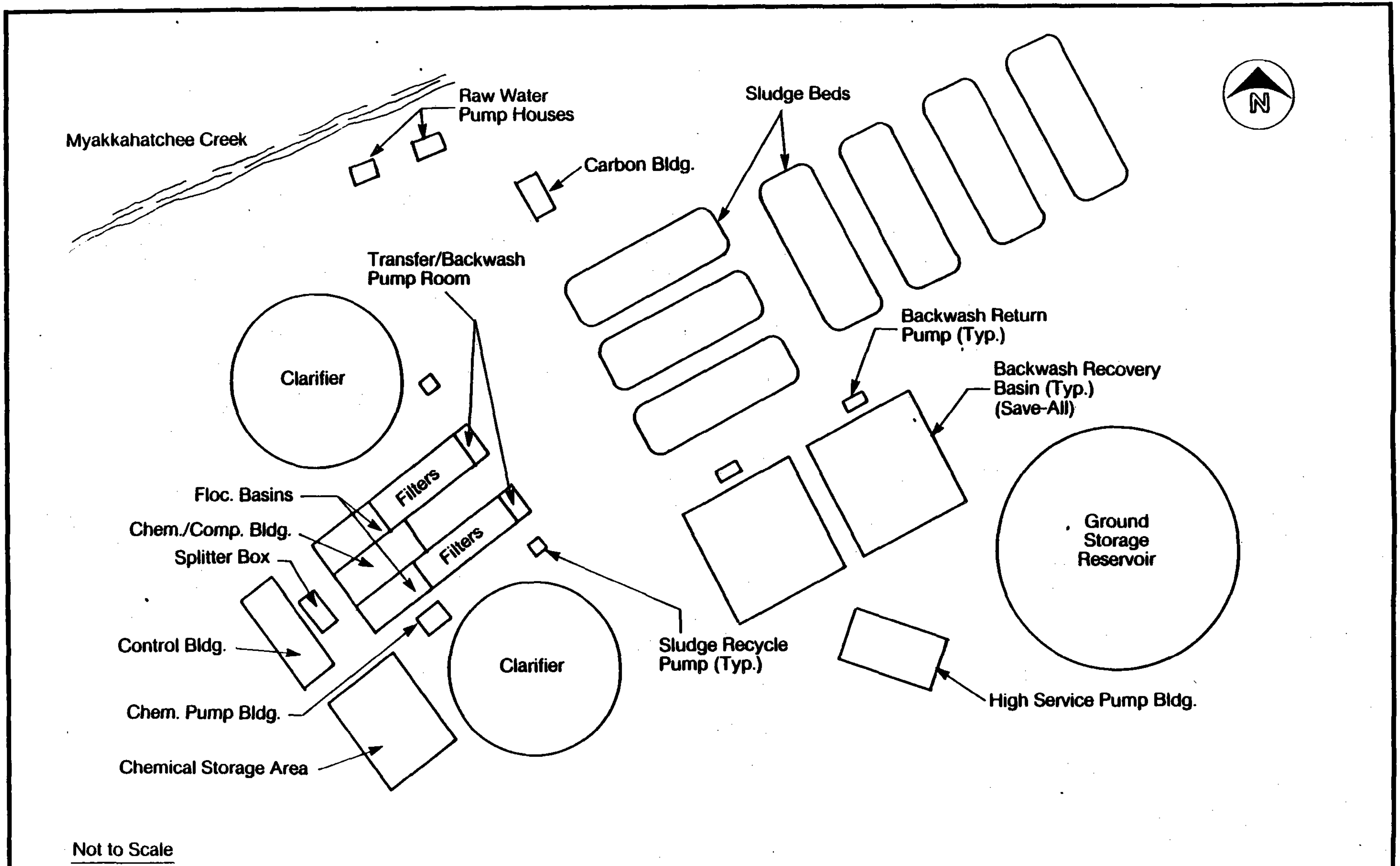

FIGURE 6-1.
Site Plan, North Port WTP.

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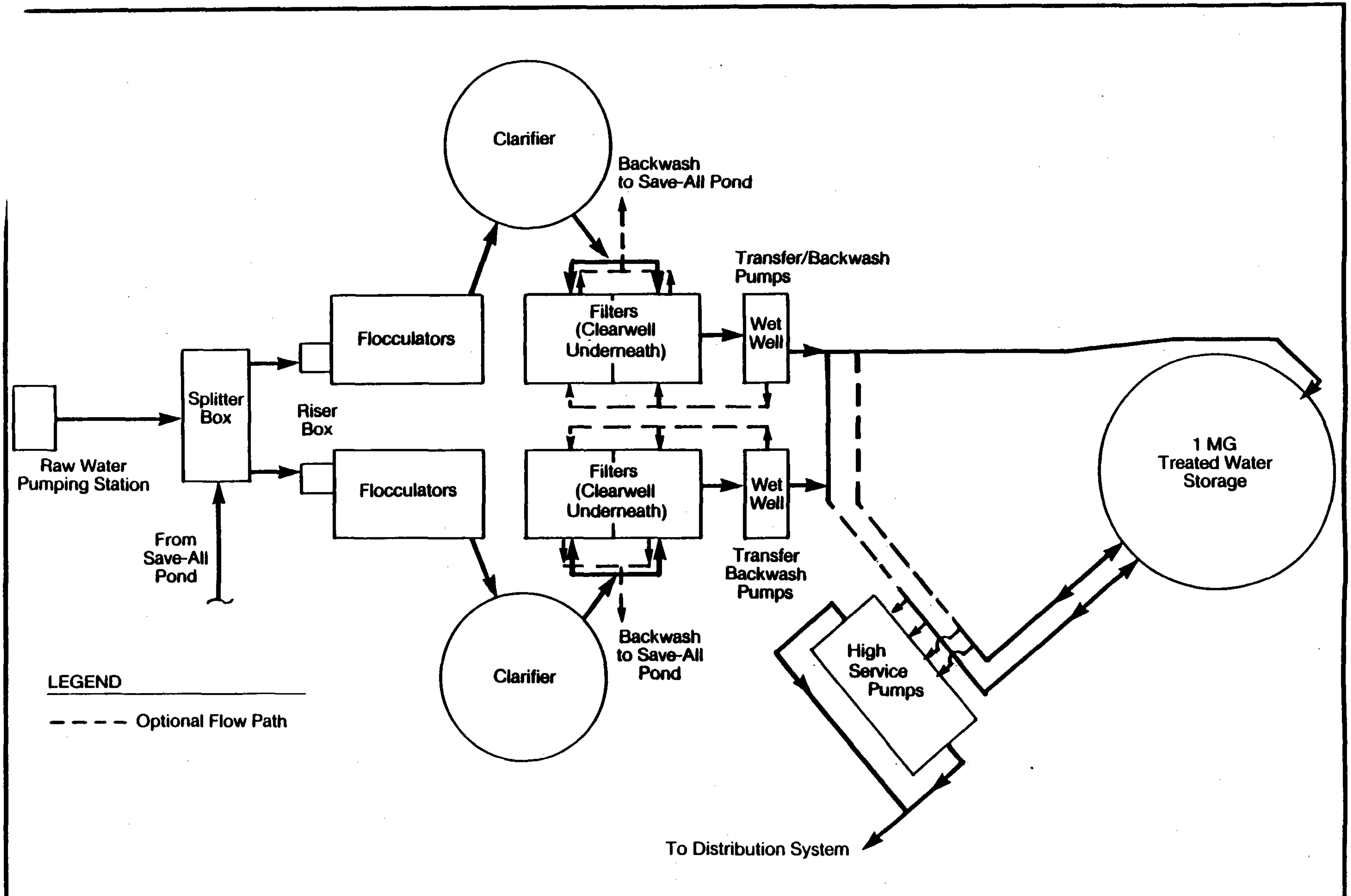
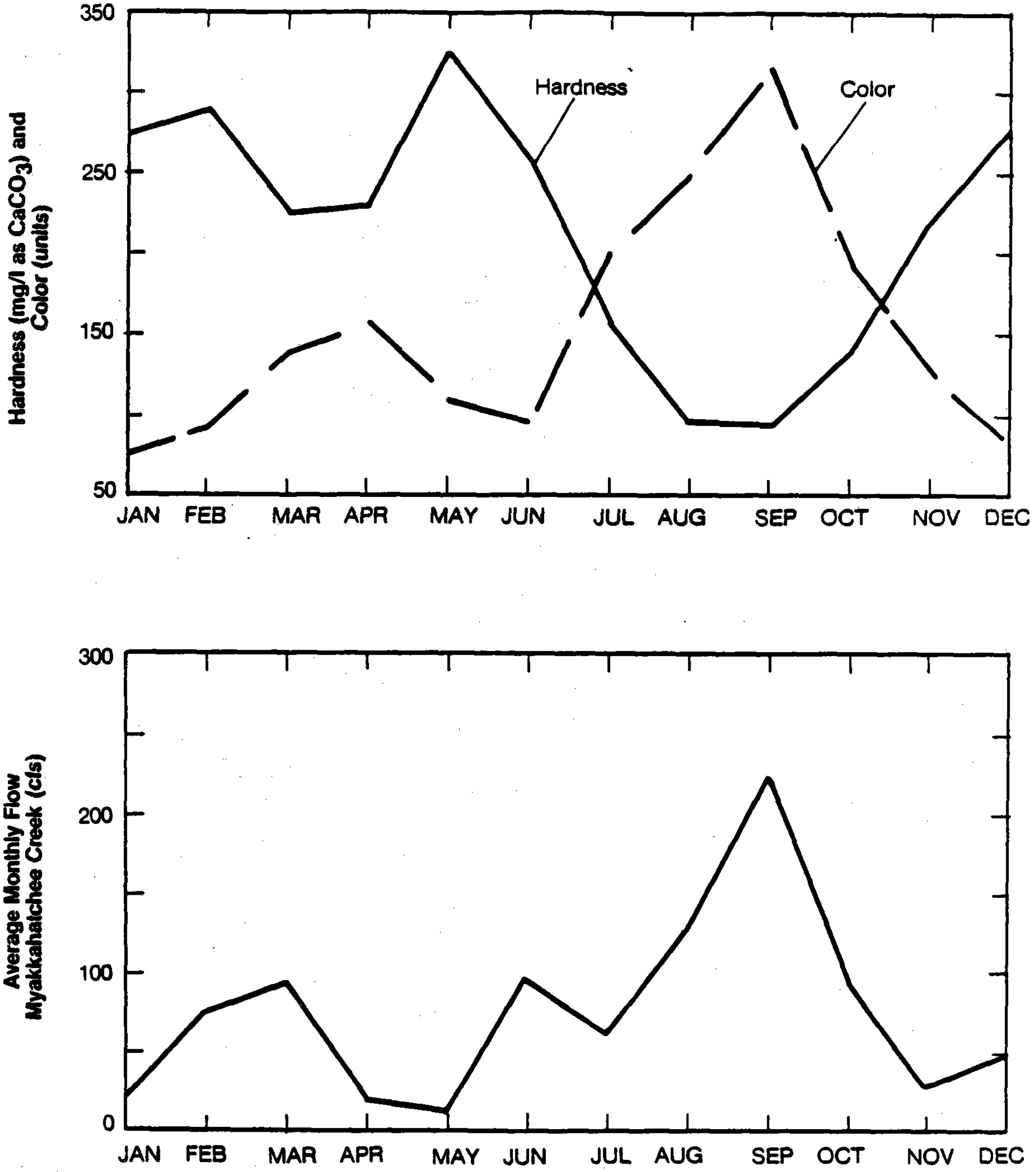


FIGURE 6-2.
General Flow Schematic, North Port WTP.



Data Base: 1980-1986
Data Source: EQL (1987).

FIGURE 6-3.
Myakkahatchee Creek Average Monthly Flow at North Port WTP vs. Hardness and Color Content.

color levels are relatively low and hardness concentrations are relatively high. Conversely, in the wet months when flows are higher, the water becomes more colored and the hardness drops. Plant operations staff vary treatment processes depending on the raw water quality.

Other important quality characteristics of the Myakkahatchee Creek that affect its usefulness as a raw water source to the North Port WTP are TDS and sulfate concentrations. As shown in Figure 6-4, the creek's monthly average TDS concentrations are high during low flow periods. Sulfate levels are also higher during drier months when creek flows are low. The plant's treatment processes add TDS and sulfate to the water (discussed later in this section). Thus, there are periods when the finished water does not meet secondary drinking water standards for TDS and sulfate.

Raw water is pumped from the Myakkahatchee Creek to a flow-splitting structure that directs it to either or both of the parallel treatment trains. Treated water is pumped to a 1.0-million-gallon (MG) ground storage reservoir and delivered to the distribution system by high service pumps. The plant currently operates from 4 to 16 hours a day, depending on demand, which rarely requires the two trains to be operated simultaneously. Typically, there are no operators on duty at night when the plant is not operating.

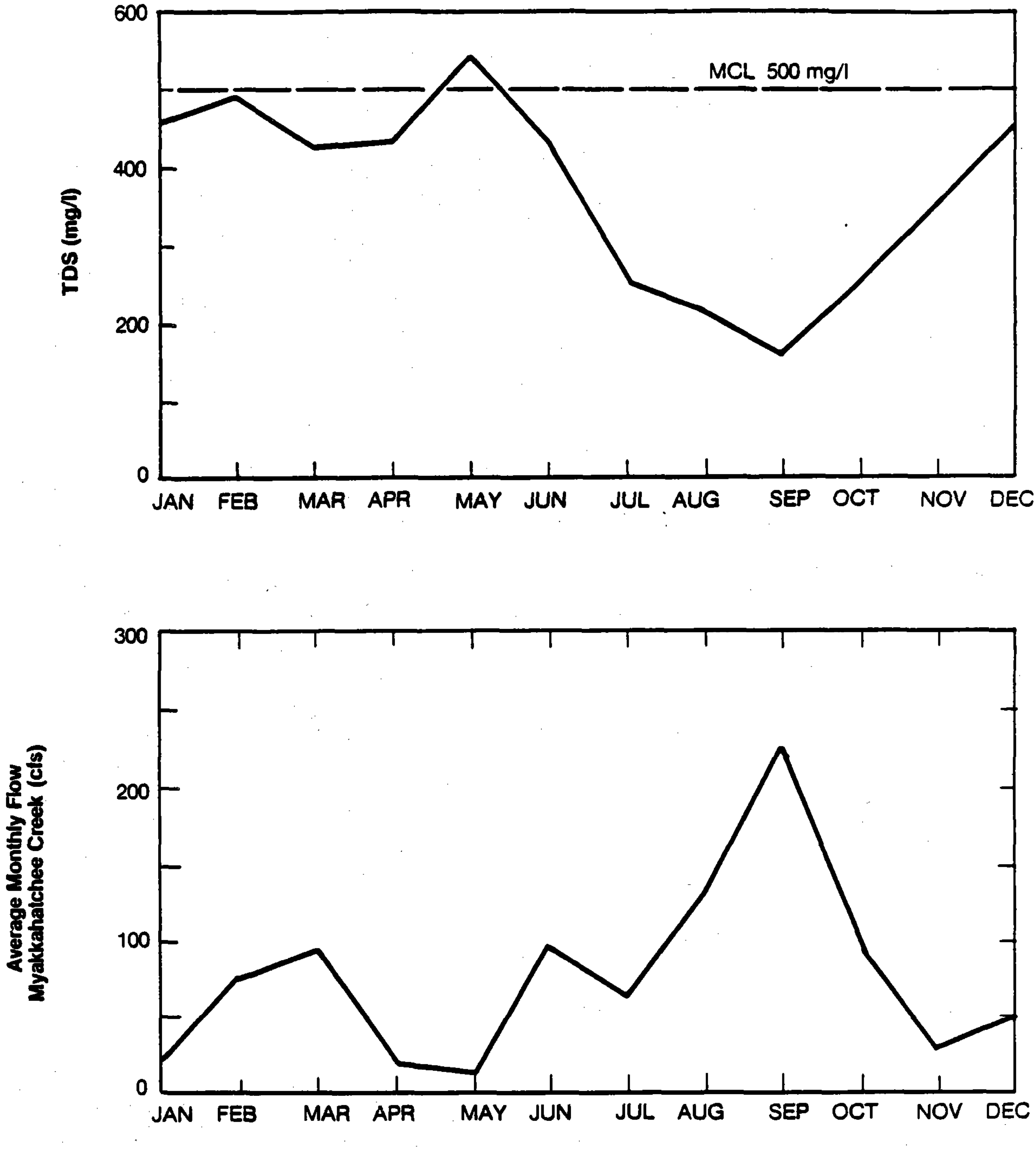
PROCESS EVALUATION

Color Removal Mode

A process schematic for the color removal treatment mode is shown in Figure 6-5. Powdered activated carbon (PAC) is added downstream of the raw water pumps as required for taste and odor control. Sulfuric acid is added as needed at the splitter box to bring pH into the range of optimum coagulation (5.5 to 6.5). Sodium hydroxide (caustic soda) may be added as needed at the splitter box to increase alkalinity for alum coagulation.

Alum is added at the influent riser box ahead of the flocculation basins. Energy for mixing and dispersion of the alum solution is derived primarily from head loss; no mechanical mixer is provided in the riser box. Polymer is added in the first flocculation basin to enhance flocculation and subsequent clarification. Energy imparted to the water by slow turbine mixers is decreased successively through each of the three flocculators to promote optimum flocculation and to avoid floc shear.

Chlorine is added to the filter influent at dosages sufficient to provide a small free chlorine residual in the filter effluent. Sodium hydroxide is applied as needed to



Data Base: 1980-1986
Data Source: EQL (1987).

FIGURE 6-4.
Myakkahatchee Creek Average Monthly Flow
at North Port WTP vs. TDS Content.

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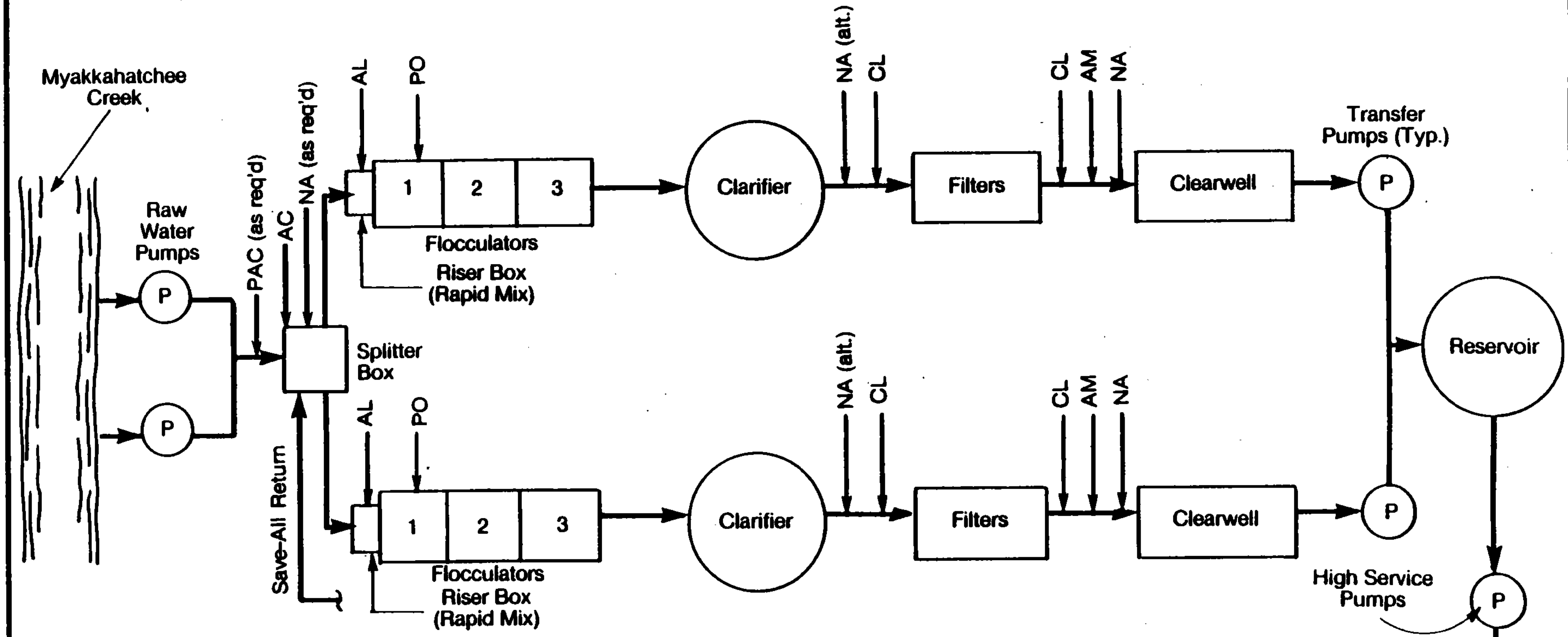

LEGEND

AC	Acid
AL	Alum
AM	Ammonia
CL	Chlorine
NA	Caustic
PAC	Carbon
PO	Polymer

FIGURE 6-5.
Process Schematic of Color Removal Mode, North Port WTP.

CH2M HILL

the filter effluent to raise pH, thereby producing a more stable (less corrosive) finished product.

Caustic along with chlorine and ammonia are added to the filter effluent to provide a combined chlorine residual in the finished water. Combined chlorine, as monochloramine, provides the required disinfection in the distribution system without producing objectionable levels of chlorine byproducts such as trihalomethanes (THM). By operating the plant at flow rates greater than 1,700 gpm, the free chlorine contact time through the filters is limited to approximately 20 minutes maximum. This helps to control THM formation. Holding the pH in the 5.5 to 6.5 range until after ammoniation occurs further retards the THM formation rate.

Softening Mode

Softening is often required for short periods (1 to 3 months) during the dry season when raw water hardness increases. In general, softening is practiced if raw water hardness reaches 250 to 300 mg/l and color is low enough (generally about 100 units) to be effeotively removed concurrently with hardness. Color removal takes priority over softening at the plant.

Treatment in the softening mode follows basically the same process as for color removal, except that sodium hydroxide (caustic) is added for softening at the flocculation basins (see Figure 6-6). With the necessary dosage and application point adjustments, other chemicals are generally used as described for color removal.

The extent of softening with caustic is limited by the level of sodium ion permitted in the finished waters. Sodium ion is currently limited to 160 mg/l. Existing operational policy limits the caustic dosage to 165 mg/l, which maintains sodium values in the range of 100 to 125 mg/l. This level of treatment normally yields a finished water with a hardness value of about 150 mg/l as calcium carbonate, but hardness values up to 500 mg/l as calcium carbonate have occurred.

Overall Water Quality

Public water supplies must be free from disease-causing organisms and from substances that produce adverse physiological effects. In addition, these waters should be pleasing to the eye and palate: free from apparent turbidity, color, taste, odor, and other objectionable characteristics.

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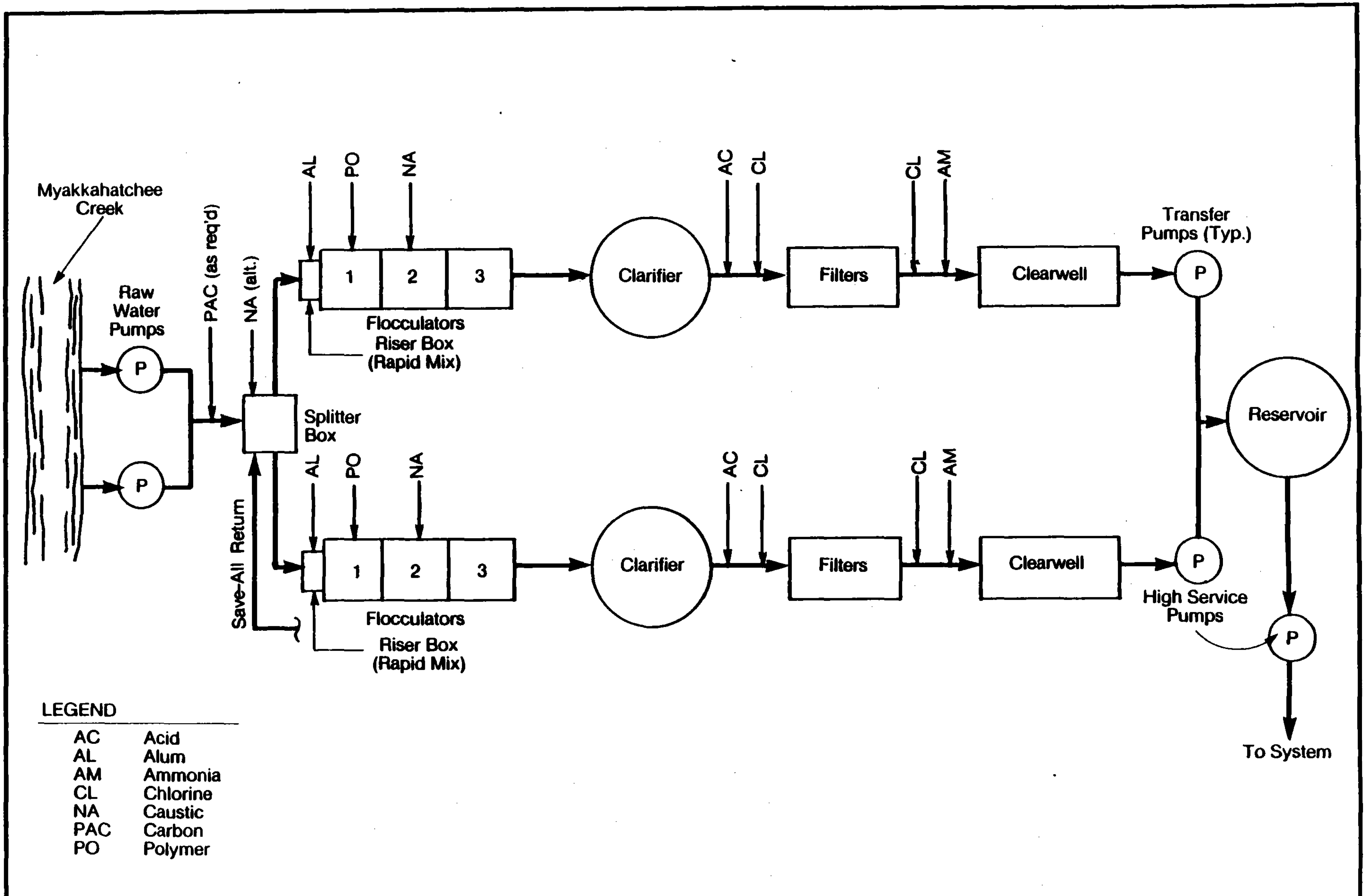

FIGURE 6-6.
Process Schematic of Softening Mode, North Port WTP.

CH2M HILL

Plant operation reports for 1981-1986 and water quality reports of selected distribution system samples (1983-1986) were reviewed. Although current monitoring, reporting, and data management practices hamper comprehensive analysis of water quality, it appears that the North Port WTP is generally producing water that meets or exceeds state and federal drinking water standards. Current Florida water quality standards are compared to distribution system samples in Table 6-1.

The THM control program has been effective in meeting the maximum contaminant level (MCL) of 0.1 mg/l for total trihalomethanes (TTHMs). Whereas distribution system samples taken in 1982 contained 0.404 mg/l of TTHMs, yearly averages (based on four samples per year) for 1983 through 1986 (Table 6-2) were 0.083, 0.075, 0.077, and 0.078 mg/l, respectively. However, the U.S. Environmental Protection Agency (EPA) is expected to announce more stringent standards for THMs as part of a new overall disinfection rule in the near future. Maximum contaminant level goals (MCLGs) of zero and MCLs as low as 0.005 mg/l are being considered for THMs. The new regulations will likely place additional limitations on other disinfection byproducts of chlorination, as well as byproducts of alternative disinfectants such as chloramines and chlorine dioxide. The new regulations may also restrict the allowable disinfectant residuals. The North Port WTP could probably not meet the new regulations without major process modifications. At a minimum, the existing chlorination processes would have to be modified or replaced by an alternative disinfectant such as ozone.

A comparison of selected raw and finished water quality parameters from 1981 through 1986 is shown in Table 6-3. The average finished water turbidity at the North Port WTP was 0.4 NTU, with high values reaching nearly 1 NTU. State and federal regulatory agencies are considering new turbidity requirements, with the new standard for finished water turbidity expected to be 0.5 NTU (for 95 percent of the time). Based on the finished water turbidities reported for 1986, the North Port WTP should be able to meet this level of treatment. However, if a lower turbidity standard is adopted, the North Port WTP finished water turbidities are not likely to be acceptable. Process and/or facility modifications will be required.

As indicated in Table 6-3, the total hardness averages about 175 mg/l of calcium carbonate, but has exceeded 400 mg/l (during periods of low rainfall). While hard water is not harmful to health and may even reduce the risk of cardiovascular disease, excessive hardness can reduce the efficiency of appliances such as hot water heaters, coffee makers, ice

Table 6-1
SUMMARY OF FLORIDA DRINKING WATER STANDARDS (FAC 17-22)
WITH COMPARISON TO NORTH PORT WTP FINISHED AND DISTRIBUTED WATER QUALITY

Group/Parameter	Maximum Contaminant Level (MCL)[a]	Date and Location of Sample				
		01/17/83 339 Jordan	03/26/84 339 Jordan	08/27/85 339 Jordan	02/13/86 399 Jordan	08/13/86 399 Jordan
I. Primary Standards						
Arsenic	0.05	<0.01	<0.01	<0.002	<0.003	<0.007
Barium	1.0	<0.1	<0.1	<0.03	<0.03	<0.15
Cadmium	0.010	<0.005	<0.005	<0.003	<0.003	<0.005
Chromium	0.05	<0.01	<0.01	<0.014	<0.03	<0.037
Lead	0.05	<0.01	<0.01	<0.027	<0.024	<0.05
Mercury	0.002	<0.0005	<0.0005	<0.0001	<0.0001	<0.0002
Nitrate (as N)	10.0	0.08	0.10			0.33
Selenium	0.01	<0.005	<0.005	<0.002	<0.003	<0.003
Silver	0.05	<0.01	<0.01	<0.006	<0.006	<0.008
Sodium	160.0	73	50	40	98	48
Fluoride[b]	1.4	0.93	0.19	0.24	0.50	0.26
Turbidity	1.0[c]	0.80	0.10	0.12	0.09	0.62
Endrine	0.002	ND	ND	ND	ND	ND
Lindane	0.004	ND	ND	ND	ND	ND
Methoxychlor	0.1	ND	ND	ND	ND	ND
Toxaphene	0.005	ND	ND	ND	ND	ND
2, 4-D	0.1	0.03	ND	ND	ND	ND
2,4,5-TP (Silvex)	0.01	ND	ND	ND	ND	ND
Coliform Bacteria[d] (colonies per 100 mL)	1					
Radionuclides						
Combined Radium 266 & 228[e].	5 pCi/L					0.7 ± 0.2
Gross Alpha	15 pCi/L		3.8 ± 3.6		3.9 ± 3.5	<1.5 5.5
Beta and Photon Radioactivity (man-made Radionuclides)	4 millirem/year		NOT REPORTED			

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Table 6-1
(Continued)

	Group/Parameter	Maximum Contaminant Level (MCL)[a]	Date and Location of Sample 05/03/82 399 Jordan	01/17/83 339 Jordan	03/26/84 339 Jordan	08/27/85 339 Jordan	02/13/86 339 Jordan	08/13/86 399 Jordan
I.	Primary Standards (continued)							
	Trihalomethanes (THM) (Sum of Bromodichloromethane, Dibromochloromethane, Tribromomethane, Trichloromethane)	0.10	REPORTED IN TABLE 6-2					
	Volatile Organics (VOC)							
	Trichloroethylene	3 µg/L	ND	ND	ND	ND	ND	ND
	Tetrachloroethylene	3 µg/L	ND	ND	ND	ND	ND	ND
	Carbon Tetrachloride	3 µg/L	ND	ND	ND	ND	ND	ND
	Vinyl Chloride	1 µg/L	ND	ND	ND	ND	ND	ND
	1,1,1-Trichloroethane	200 µg/L	ND	ND	ND	ND	ND	ND
	1,2-Dichloroethane	3 µg/L	ND	ND	ND	ND	ND	ND
	Benzene	1 µg/L	ND	ND	ND	ND	ND	ND
	Ethylene Dibromide	0.02 µg/L	ND	ND	ND	ND	ND	
II.	Secondary Standards							
	Chloride	250	58	30	37	68	28	
	Color	15 A.P.H.A units	0	0	1.0	4.0	9	
	Copper	1	<0.01	<0.01	0.03	<0.008	<0.01	
	Corrosivity	-0.2 to +0.2 Langelier index[f]	-0.2	-0.2	-0.1	-0.3	0.0	
	Foaming Agents	0.5	<0.01	<0.01	<0.1	<0.1	<0.1	
	Iron	0.3	<0.05	<0.1	0.03	<.012	<0.015	
	Manganese	0.05	<0.05	<0.05	0.007	<0.005	<0.008	
	Odor	3 (threshold odor number)	NOT REPORTED					
	pH (at collection point)	≥ 6.5	7.6[g]	8.2[g]	7.89[g]	8.17[g]	8.3[g]	

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Table 6-1
(Continued)

	Group/Parameter	Maximum Contaminant Level (MCL)[a]	Date and Location of Sample 01/17/83 339 Jordan	03/26/84 399 Jordan	08/27/85 399 Jordan	02/13/86 399 Jordan	08/13/86 399 Jordan
II.	Secondary Standards (continued)						
	Sulfate	250	235	98.9	136	303	111
	TDS	500 (or greater)[b]	612	226	314	641	282
	Zinc	5	<0.01	<0.01	0.003	0.015	<0.015
III.	Synthetic Organic Contaminants (SOC)	NA	ND	ND	ND	ND	

The following synthetic organic contaminants shall be analyzed once every 3 years on all community water systems. No MCLs have been established for these compounds (except for the eight VOCs shown previously).

1. PURGEABLES

Acrolein
Acrylonitrile
Bromodichloromethane[1]
Bromoform[1]
Bromomethane
Chlorobenzene
Chloroethane
2-Chloroethylvinyl ether
Chloroform[1]
Chloromethane
Dibromochloromethane[1]
Dichlorodifluoromethane
1,1-Dichloroethane
1,1-Dichloroethene
trans-1,3-Dichloropropene
1,2-Dichloroethene
1,2-Dichloropropane
cis-1,3-Dichloropropene
Ethylbenzene
Methylene chloride
1,1,2-Trichloroethane
Trichlorofluoromethane
Toluene
Xylene
Styrene
Dichlorobenzene
1,2-Dibro-3-Chloropropane
1,1,2,2-Tetrachloroethane

3. BASE NEUTRAL EXTRACTABLES (continued)

Benzo(b)fluoranthene
Benzo(k)fluoranthene
Benzo(g,h,i)perylene
Benzidine
Bis(2-chloroethyl)ether
Bix(2-chloroethoxy)methane
Bix(2-ethylhexyl)phthalate
Bis(2-choroiosopropyl)ether
4-Bromophenyl phenyl ether
Butyl benzyl phthalate
2-Chloronophthalene
4-Chlorophenyl phenyl ether
Chrysene
Dibenzo(a,h)anthracene
Di-n-butylphthalate
Dioctylphthalate
1,2-Diphenylhydrazine
Fluorene
Nexachlorobenzene
Hexachlorobutadiene
Hexachloroethane
Hexachlorocyclopentadiene
Ideno(1,2,3-cd)pyrene
Isophorone
Naphthalene
Nitrobenzene
N-Nitrosodimethylamine
N-Nitrosodi-n-propylamine
N-Nitrosodiphenylamine
Phenanthrene

Table 6-1
(Continued)

2. PESTICIDES

Aldrin
α-BHC
β-BHC
λ-BHC
Φ-BHC
Chlordane
4,4'-DDD
4,4'-DDE
4,4'-DDT
Dieldrin
Endosulfan I
Endosulfan II
Endrin Aldehyde
Heptachlor
Heptachlor Epoxide
Toxaphene
PCB-1016
PCB-1221
PCB-1232
PCB-1242
PCB-1248
PCB-1254
PCB-1260
Aldicarb (non-extractable)

3. BASE NEUTRAL EXTRACTABLES

Acenaphthene
Acenaphthylene
Anthracene
Benzo(a)anthracene
Diethylphthalate
Dimethylphthalate
2,4-Dinitrotoluene
2,6-Dinitrotoluene
1,3-Dichlorobenzene
1,4-Dichlorobenzene
1,2-Dichlorobenzene
3,3 Dichlorobenzidine
Pyrene
2,3,7,8-Tetrachlorodibenzo-p-dioxin (Dioxin)
1,2,4-Trichlorobenzene

4. ACID EXTRACTABLES

2-Chlorophenol
2,4-Dichlorophenol
2,4-Dimethylphenol
2,4-Dinitrophenol
2-Methyl-4,6-Dinitrophenol
4-Nitrophenol
Pentachlorophenol
Phenol
2,4,6-Trichlorophenol

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[a] MCL reported in mg/l except where noted otherwise.
[b] Maximum annual average of local daily air temperature determines the MCL shown for fluoride.
[c] MCL is for average of 2 consecutive days or longer if proven to not reduce water quality.
[d] For membrane filter technique one colony/100 ml = monthly average; four/100 ml in more than one sample if 20 samples or less are analyzed monthly; four/100 ml in more than 5% of samples if 20 samples or more are analyzed monthly.
[e] Gross Alpha may be substituted for Radium 226 & 228 provided the alpha particle activity is $\leq$ 5 pCi/L at a 95% confidence interval (1.65 SIGMA).
[f] May be different if stabilizers are used.
[g] Laboratory values.
[h] More than 500 mg/l TDS is acceptable if no other MCL is exceeded.
[i] Regulated as trihalomethanes
Note: ND = None detected.

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Table 6-2
NORTH PORT WTP DISTRIBUTED WATER QUALITY
TOTAL TRIHALOMETHANE BY QUARTERS

Year	Quarter	Total Trihalomethane[a] (mg/l)
1982	4th	0.404
1983	1st	0.086
	2nd	0.100
	3rd	0.070
	4th	0.076
	Avg	0.083
1984	1st	0.082
	2nd	0.072
	3rd	0.077
	4th	0.070
	Avg	0.075
1985	1st	0.084
	2nd	0.064
	3rd	0.067
	4th	0.091
	Avg	0.077
1986	1st	0.064
	2nd	0.075
	3rd	0.083
	4th	0.089
	Avg	0.078

[a]TTHM MCL is 0.10 mg/l.

Table 6-3
NORTH PORT WTP AVERAGE MONTHLY WATER QUALITY[a]

Parameter	Raw Min	Raw Max	Raw Avg	Finished Min	Finished Max	Finished Avg	FAC 17-22 Drinking Water Standard[b]
pH	6.6	7.8	7.2	7.3	8.7	8.0	6.5 (min)
Alkalinity, mg $CaCO_3$/l	31	229	104	33	165	81	--
Total Hardness, mg $CaCO_3$/l	58	672	228	41	439	174	--
Calcium, mg $CaCO_3$/l	29	433	178	42	277	125	--
Magnesium, mg $CaCO_3$/l	15	243	57	12	167	50	--
Sodium, mg/l	6	89	25	12	160	50	160[c]
Chloride, mg/l	27	88	50	33	95	57	250
Sulfate, mg/l	4.8	482	100	57	507	169	250
Total Dissolved Solids, mg/l	69	808	297	178	877	396	500[d]
Color, units	30	360	158	0	2	0	15
Turbidity, NTU	--Not Reported--			0.2	0.9	0.4	1.0[c,e]

[a]Based on GDU monthly plant operating reports for 1981-1986 and other WTP laboratory analyses records (unpublished).

[b]Selected Florida drinking water standards.

[c]Primary standards, others are secondary standards.

[d]TDS may exceed 500 mg/l, if no other MCL is exceeded.

[e]Monthly average primary standard for surface water systems, except that five or fewer turbidity units may be allowed if certain specified criteria are met.

machines, and dishwashers. Laundering, bathing, and household cleaning are also adversely affected by hardness minerals.

Figure 6-3 showed the monthly variations of streamflow, color, and water hardness. During the 6 years of data analyzed, water hardness was at its highest average concentrations during the month of May and lowest average levels in September. Table 6-4 shows the average raw and finished water hardness data, as well as TDS and color concentrations, during May and September of 1981 through 1986. During May, while primarily in the softening mode of operation, only about 32 percent of the water hardness was removed. At the North Port WTP, softening is limited by the amount of sodium hydroxide that can be applied without exceeding the sodium standard (160 mg/l MCL).

TDS and sulfate levels occasionally exceed desirable levels during periods of low rainfall, when the mineral content of the raw water supply is highest. Table 6-3 showed that from 1981 through 1986, TDS and sulfate concentrations increased an average of about 100 mg/l and 70 mg/l, respectively, during the treatment process. These increases are a result of the addition of caustic soda (sodium hydroxide) and alum (aluminum sulfate) and are unavoidable under the present treatment regime.

As shown in Tables 6-3 and 6-4, the North Port WTP is doing an excellent job of removing color in either the softening or color removal mode.

No quantitative or qualitative data are available on taste and odor, but control is an ongoing concern. Since taste and odor problems are common to both the North Port and Peace River WTPs, control measures are discussed in a later subsection.

In summary, the North Port WTP meets or exceeds most current criteria for drinking water. At times, TDS and sulfate concentrations are above desirable levels and alternatives for their control should be investigated. A means of improving taste and odor control, and meeting future (lower) THM and turbidity standards should also be investigated. In addition, a computerized data management system for water quality and operational data is strongly recommended.

Chemical Consumption

Chemical use data for 1986 are presented in Table 6-5. During 1986, the plant was generally operated in the softening mode during the first 6 months of the year and in the color removal mode thereafter.

Table 6-4
NORTH PORT WTP WATER QUALITY: SOFTENING VS. COLOR REMOVAL TREATMENT MODES[a]

	Softening Mode May (1981-1986)			Color Removal Mode September (1981-1986)		
	Raw	Finished	Removed	Raw	Finished	Removed
Total Hardness (mg/l as $CaCO_3$)						
Average	372	253	119	109	103	6
Maximum	672	439		181	170	
Color (units)						
Average	93	0	93	257	0	257
Maximum	221	0		334	0	
TDS (mg/l)						
Average	497	587	-90	137	245	-108
Maximum	808	877		234	308	

[a]Based on GDU monthly plant operating reports for 1981-1986.

Table 6-5
1986 CHEMICAL DOSAGE/USAGE[a]
NORTH PORT WTP

Chemical	Dosage (mg/l) Avg.	Dosage (mg/l) Approx. Range	Usage (lb/day)[b] Avg.	Usage (lb/day)[b] Approx. Range
Powdered Activated Carbon, as PAC	8	0-20	86	13-134
Acid, as 93% H_2SO_4	20	0-82[e]	235	0-558[e]
Alum, as 49% $Al_2(SO_4)_3$	117	23-219	1,224	328-1,902
Caustic, as 50% NaOH	46	21-231	1,098	268-3,163
Polymer[c], as neat polymer	0.64	0-2	7	0-15
Chlorine[d], as Cl_2	9	7-13	96	58-138
Ammonia, as NH_3	1.8	1.2-2.3	20	12-31

[a]Based on GDU monthly plant operating reports for January-December 1986. Plant operated in the softening mode January-June and in the color removal mode for July-December.

[b]Average day plant throughput for 1986 was about 1.2 MG.

[c]Whispro floc 20 (nonionic starch).

[d]Total for both pre-filter and post-filter application.

[e]Acid fed approximately 75% of the year.

Table 6-6 summarizes chemical feed data for the months of May and September (1981 through 1986) when the plant was operated primarily in the softening and color removal modes, respectively. During the month of May (softening mode), caustic soda and alum were fed at approximately 150 mg/l and 85 mg/l, respectively. During September, when operating in the color removal mode during the month with highest average raw water color levels, caustic soda and alum were fed at approximately 75 mg/l and 170 mg/l, respectively.

FACILITIES EVALUATION

The North Port WTP has been evaluated by unit process. Nominal capacities of each major component are given in Table 6-7. A comparison of plant facilities to certain FDER water treatment plant design guidelines is shown in Table 6-8.

OVERALL CAPACITY EVALUATION

The plant was evaluated for its overall estimated capacity to treat water, hydraulically and in process terms, and possible capacity limitations identified. The evaluation is based on typical design criteria, observations at the plant, and general considerations; no onsite testing was done.

Hydraulics

Hydraulically, the raw water pumping and transfer pumping capacities are most important. Plant storage and high service pumping capacities relate to water demands rather than treatment rates and are not considered here. Based on pump nameplate ratings, the firm capacity (assumes largest single unit out of service) of both raw water and transfer pumping is about 4,500 gpm (6.5 mgd). However, the raw water firm pumping capacity is currently about 3,000 gpm (4.3 mgd).

The physical and hydraulic characteristics of the splitter box, flocculation basins, clarifiers, and filters are also significant. Hydraulic controls on water levels are provided successively upstream by the slide gates/stub walls at the splitter box, the clarifier effluent weir, and the filter effluent control valves. The limiting factor appears to be the top-of-wall elevation at the flocculation basin. According to plant record drawings, only 1 foot of elevation difference exists between the clarifier weir and the top of the flocculation basin walls. Plant personnel report that the flocculation basin influent riser box would approach overflow at high flow rates before metal extensions were

Table 6-6
CHEMICAL ADDITION AT NORTH PORT WTP:
SOFTENING VS. COLOR REMOVAL TREATMENT MODES[a]

Chemical	Softening Mode May (1981-1986) Averages		Color Removal Mode September (1981-1986) Averages	
	Dose (mg/l)	Usage (lb/day)	Dose (mg/l)	Usage (lb/day)
Powdered Activated Carbon, as PAC	8.1	87	9.3	61
Acid, as 93% H_2SO_4	50	541	18.4	121
Alum, as 49% $Al_2(SO_4)_3$	85	913	169	1,109
Caustic, as 50% NaOH	147	1,570	76	499
Polymer[b], as neat polymer	1.2	13	0.9	6
Chlorine, as Cl_2	12.3	132	14.2	93
Ammonia, as NH_3	2.0	21	1.7	11

[a]Based on GDU monthly plant operating reports for 1981-1986.

[b]Whispro floc 20 (nonionic starch).

Table 6-7
EXISTING FACILITIES/EQUIPMENT
NORTH PORT WTP

Item	Type	Size/Nameplate Capacity
Raw Water Intakes	CMP supported on wood piles	24-inch diameter
Raw Water Pumps (4)	Vert. turbine	25 hp, 1,500 gpm @ 40 ft
Flash Mix Box	------------------------	Vol. approx. 150 ft^3
Flocculation Basins (2)	Rectangular, 3-compartment	11 ft SWD, 6,800 ft^3 ea.
Flocculation Mixers (6)	Axial-flow, propeller	3 hp, gear reducer, variable speed
Clarifiers (2)	Circular, cntr. feed,	12 ft SWD, 70 ft dia. peripheral weir
Sludge Recycle Pumps (2)	Self-priming centrifugal	3 hp, capacity NA
Sludge Beds (5)	Earthen	Various
Filters (4)	Gravity, dual-media	384 ft^2 surface area ea., rate-of-flow control
Clearwells (2)	Below filters	Eff. Vol. 40,000 gal. ea.
Transfer/Backwash Pumps (3)	Vert. turbine	50 hp, 3,000 gpm @ 45 ft 50 hp, 2,750 gpm @ 49 ft 30 hp, 2,880 gpm @ NA
Backwash Recovery Basins (2)	Concrete, below grade	Vol. approx. 70,000 gal. ea.
Backwash Return Pumps (2)	Self-priming centrifugal	5 hp, 75 to 400 gpm
Plant Water Pump	Vert. turbine	15 hp, 180 gpm @ 165 ft
Reservoir		Pre-stressed conc. 1.0 MG
High Service Pumps (6)	Vert. turbine, can type	#1 - 10 hp, cs, 200 gpm @ 115 ft #2 - 15 hp, cs, 375 gpm @ 115 ft #3 - 30 hp, cs, 680 gpm @ 115 ft #4 - 50 hp, cs, 1,100 gpm @ 150 ft #5 - 125 hp, cs, 2,500 gpm @ 150 ft #6 - 125 hp, vs, 2,500 gpm @ 150 ft
Finished Water Flow Meter	Venturi, diff. pressure	----------------------
Carbon Feed System	Slurry	Day tank nom. capacity 1,100 gal. 1.5 hp mixer PD met. pump, 80 gph
Acid Feed System	Liquid	PD met. pump, max. cap. 500 gpd
Alum Feed System	Liquid	2 storage tanks @ 7,500 gal. ea. 1 gear-type met. pump, 3.2 - 9.7 gpm 1 PD met. pump, max. 1,500 gpd
Caustic Feed System	Liquid	2 storage tanks @ 7,500 gal. ea. 1 gear-type met. pump, 3.2 - 9.7 gpm 1 PD met. pump, max. 1,500 gpd
Polymer Feed System	Dry, liquid feed	2 - 150 gal. day tank w/mixer 2 - PD met. pump, max. 500 gpd
Chlorination System	TC supply, solution feed	1 chlorinator @ 1,000 ppd 1 chlorinator @ 2,000 ppd
Ammoniation System	Gas feed	2 ammoniators @ 135 ppd (fitted for 50 ppd)

NA = Not available.

Table 6-8
COMPARISON TO FDER GUIDELINES[a]
NORTH PORT WTP

FDER WTP Design Guidelines[b]	North Port WTP	Status
Plant design Q = max. day demand	Yes (assumed)	OK
Plant design Q + finished water storage = 4 hr of max. hr demand	4.4 + 1.0 vs. 0.17 x 8.8 5.4 MG is greater than 1.5 MG	OK
Chlorination capacity to provide 0.6 mg/L combined residual	Yes	OK
Standby chlorinator	Yes	OK
Auto chlorine supply switchover if unmanned while operating chlorination	Always manned when operating	OK
Chlorine facilities in separate room, above grade, cross ventilation, weighing devices, safety equipment	Yes	OK
Coagulant aid on approved list	Yes	OK
Chemical additional points should be separated to avoid potential interaction to chemicals	Yes	OK
Flash mix t_d = 5 - 10 sec	Undetermined	?
Flocculation t_d = 10 - 15 min	35 min	OK
Flash mix + flocculation + settling t_d = 4 hr	4.1 hr	OK
Max. filtration rate 2-3 gpm/ft^2[c]	2.65 gpm/ft^2 (max) 2.0 gpm/ft^2 (avg)	OK OK
Min. filter backwash rate = 15 gpm/ft^2	15 gpm/ft^2 (min)	OK
High service pumping capacity = max. hr demand	10.6 mgd is greater than 8.8 mgd	OK
Finished water metering required	Yes	OK
Auxiliary power (with auto startup) to provide at least one hour of max. day demand	Yes	OK
Protective fencing around plant	Yes	OK

[a]Based on assumptions that plant nominal rated capacity of 4.4 mgd is equal to maximum day demand, and that maximum hour demand = 2 x maximum day = 8.8 mgd.

[b]Only guidelines applicable to North Port WTP are listed. This is not a complete listing of applicable FDER regulations.

[c]Up to 6 gpm/ft^2 (with one filter out of service) may be allowed with acceptable performance test results.

installed. A flow in the range of 2,500 to 3,000 gpm (at one side) might cause overflow of the flocculation basin walls.

The present firm hydraulic capacity of the plant is estimated at 4,500 gpm (6.5 mgd), with raw water and transfer pumping capacities the limiting factor.

Process

The capacity of flocculation, at a minimum detention time of 25 minutes, is about 3,500 to 4,000 gpm. Clarifier capacity is estimated at 4,200 gpm, based on a maximum surface overflow rate of 800 gpd/ft^2. Using a maximum 3 gpm/ft^2 loading rate criterion and assuming three of four filters in service, the maximum firm filtration capacity is about 3,450 gpm.

The firm process capacity of the plant is thus estimated at approximately 3,450 gpm (5.0 mgd). The limiting factor is the filtration capacity.

Conclusion

Based on this evaluation, it is apparent that the maximum firm plant capacity, on a continuous service basis, is about 1,725 gpm for each treatment train or 3,450 gpm total plant capacity (2.5 mgd and 5.0 mgd, respectively). Plant personnel report that the plant has been successfully operated at these rates. Based on the limited information available, it appears that ancillary systems such as chemical feeds, backwash water recovery, etc., would be capable of meeting plant needs at these rates, although in several instances significant upgrade is recommended. Therefore, the original rated capacity of the plant of 4.4 mgd (2.2 mgd each train) is considered conservative.

PEACE RIVER WTP

GENERAL DESCRIPTION

The Peace River WTP was originally constructed in 1979 as a 6-mgd facility with provision for expansion to 30 mgd. The plant has been recently upgraded to 12 mgd with the addition of a second 6-mgd solids contact unit and miscellaneous modifications. Average water production for 1986 was 4.2 mgd.

The source of supply for the Peace River WTP is the Peace River, a highly colored surface water. Raw water is pumped to a unique five-sided flow distribution structure called "the pentagon." During periods of the year when supply exceeds demand, river water is also pumped to an 85-acre retention pond. When demand exceeds the permitted river withdrawal rates, the stored waters augment the supply. The plant is manned and operated 24 hours per day, 7 days per week. An aerial view of the supply, treatment, and storage facilities is shown in Figure 6-7. Figure 6-8 is the site plan of the Peace River WTP treatment facilities, with locations of major future tanks and basins indicated. A process flow diagram for the plant is provided in Figure 6-9.

PROCESS EVALUATION

The Peace River WTP was originally designed to operate either as a coagulation/filtration plant for removal of color and turbidity or a softening plant for reduction of carbonate hardness. Figure 6-10 shows the monthly average flow in the Peace River, as well as the hardness and color concentrations in the Peace River near the plant intake. Although the raw water hardness can exceed 200 mg/l as $CaCO_3$ during periods of low rainfall, the Peace River WTP is not normally operated in the softening mode. Some equipment integral to the softening process is not operational.

Figure 6-11 presents monthly average flow and TDS content of the Peace River. The monthly average TDS concentrations (from 1976 through 1986) were at their highest levels in May and June. The lowest monthly average river flow (from 1981 through 1986) occurred in May.

Color Removal Mode

PAC is added to the raw water as needed for taste and odor control, but is not always effective at reasonable dosages. On occasion, sodium hydroxide must be added at the pentagon to provide additional alkalinity for the alum coagulation process. Alum is also added at the pentagon. The turbulent zone just downstream of the influent weirs is used to rapidly mix alum with the raw water. This is an effective, low cost method to instantly disperse the alum into the raw water stream.

Downstream of the pentagon are two 85-foot diameter, upflow solids contact reactor/clarifiers that provide both flocculation and clarification. Polymer is added at the rapid mix zone of the reactor/clarifiers to enhance flocculation and subsequent clarification. The nonionic polymer starch currently used (Whispro floc 20) is often not the best suited for use with alum; more effective flocculation aids are available and are currently being tested for performance characteristics.

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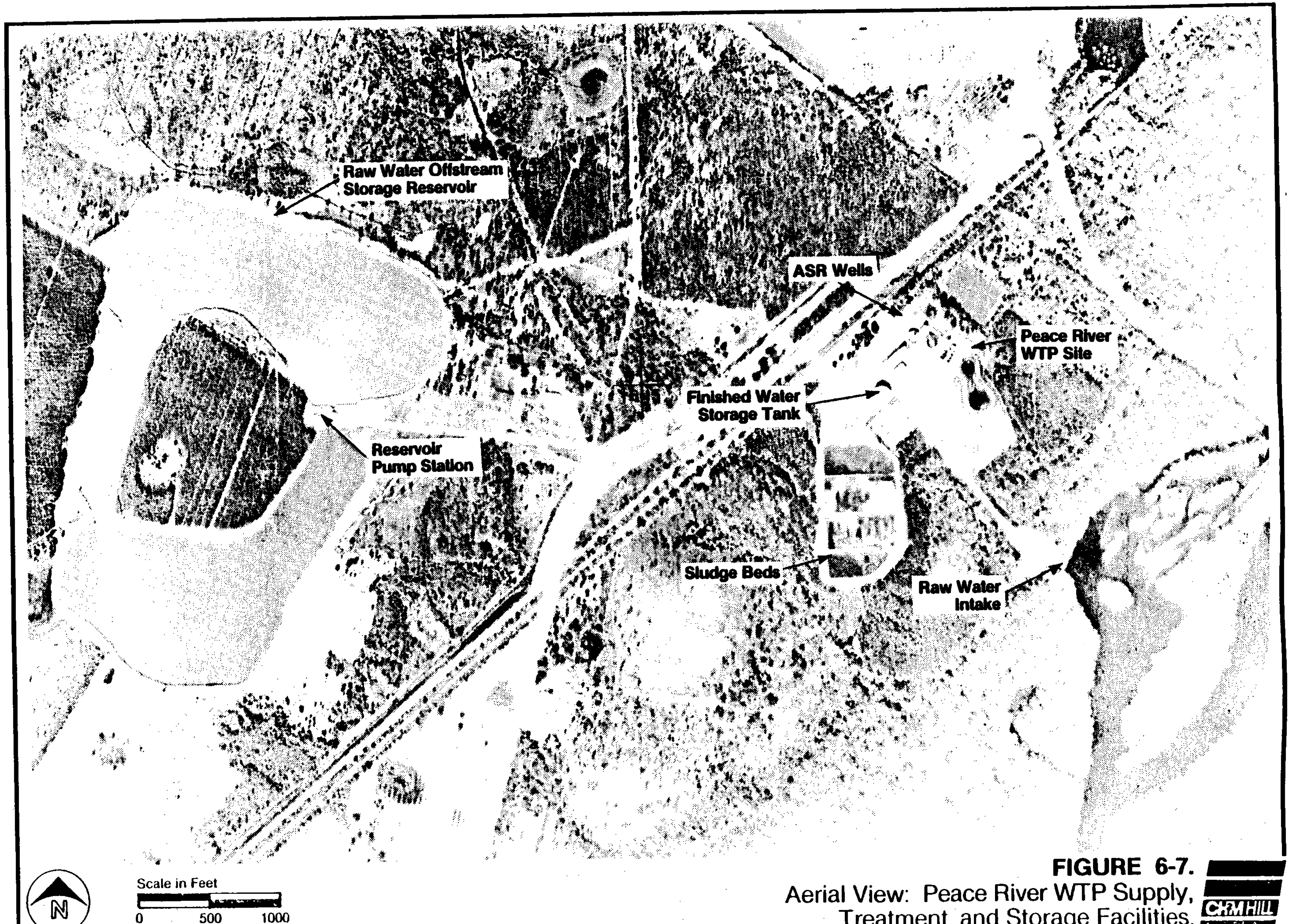

FIGURE 6-7.
Aerial View: Peace River WTP Supply, Treatment and Storage Facilities.

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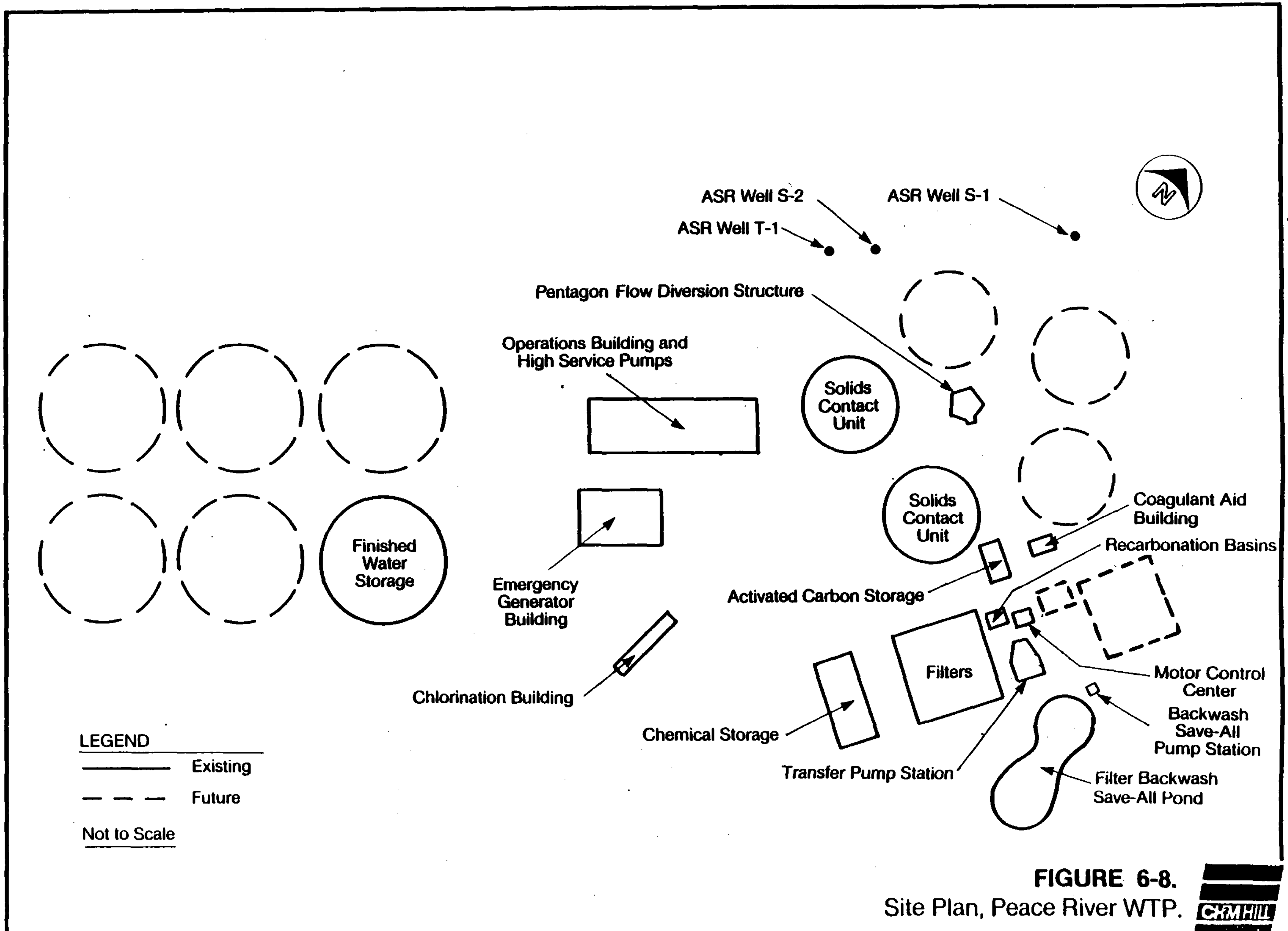

FIGURE 6-8.
Site Plan, Peace River WTP.

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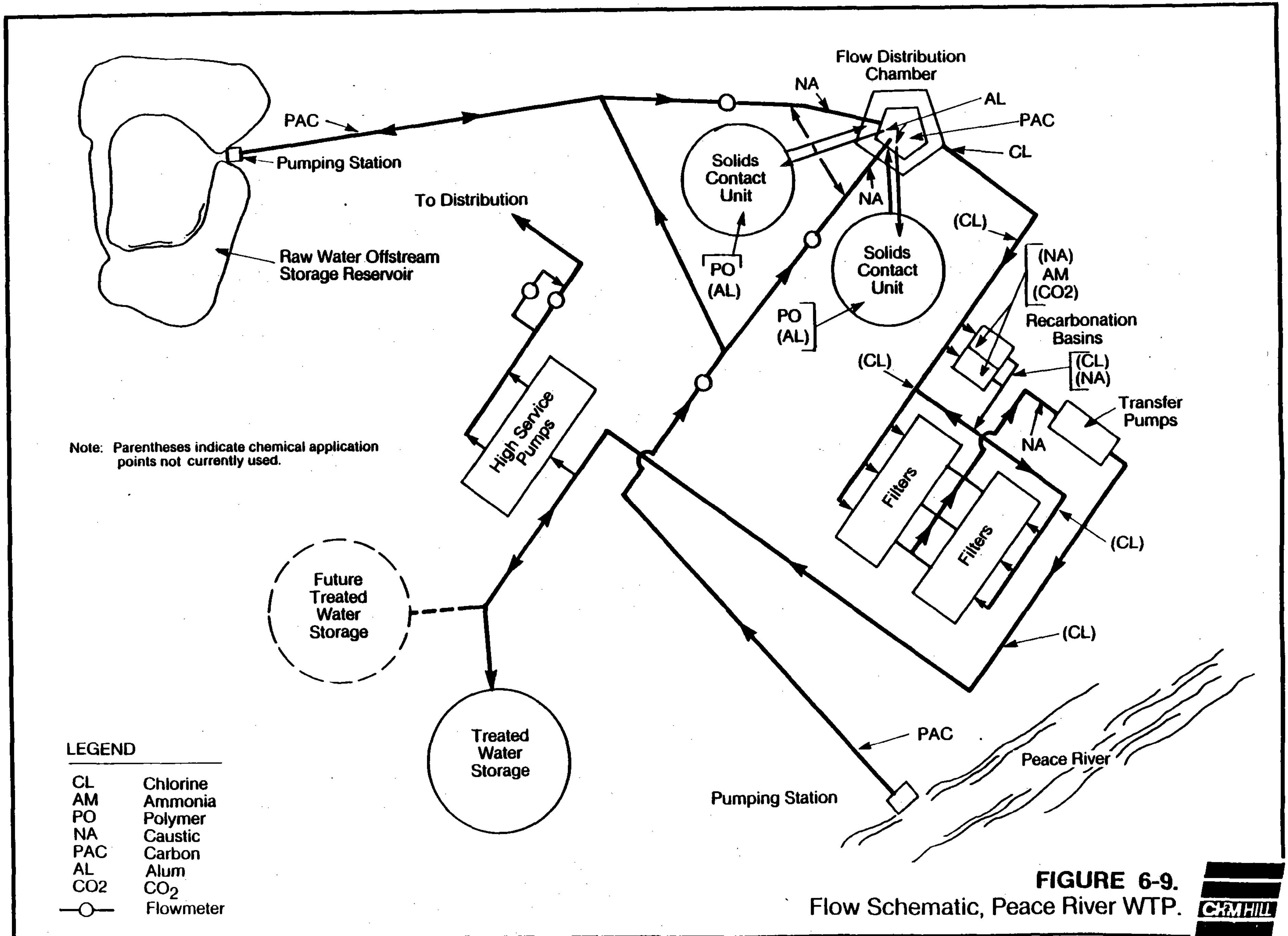

FIGURE 6-9.
Flow Schematic, Peace River WTP.

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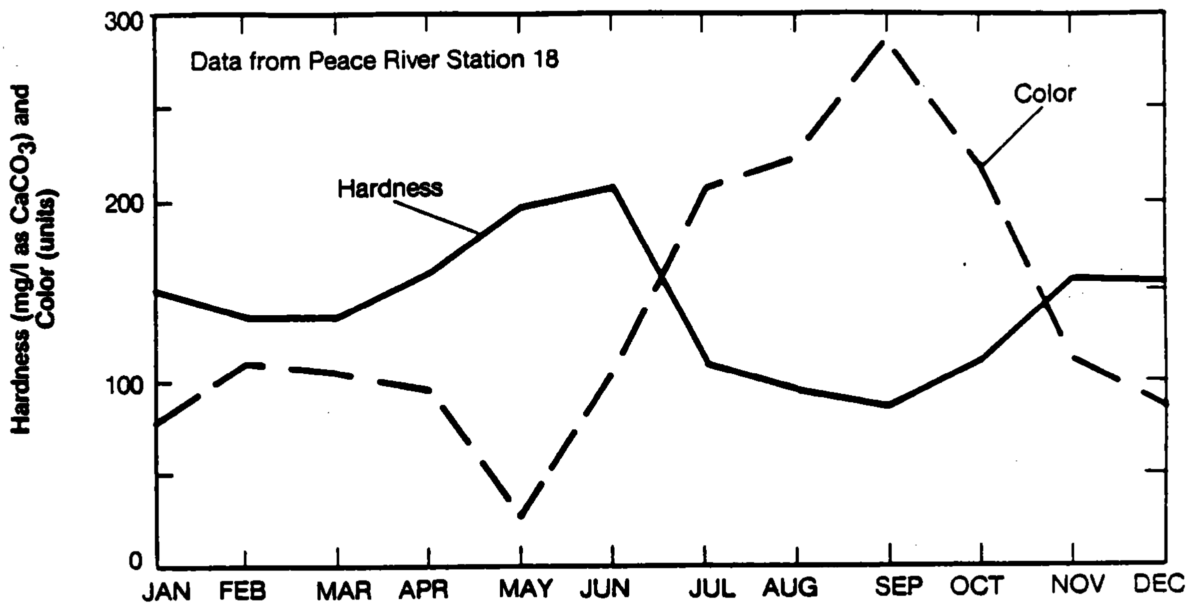

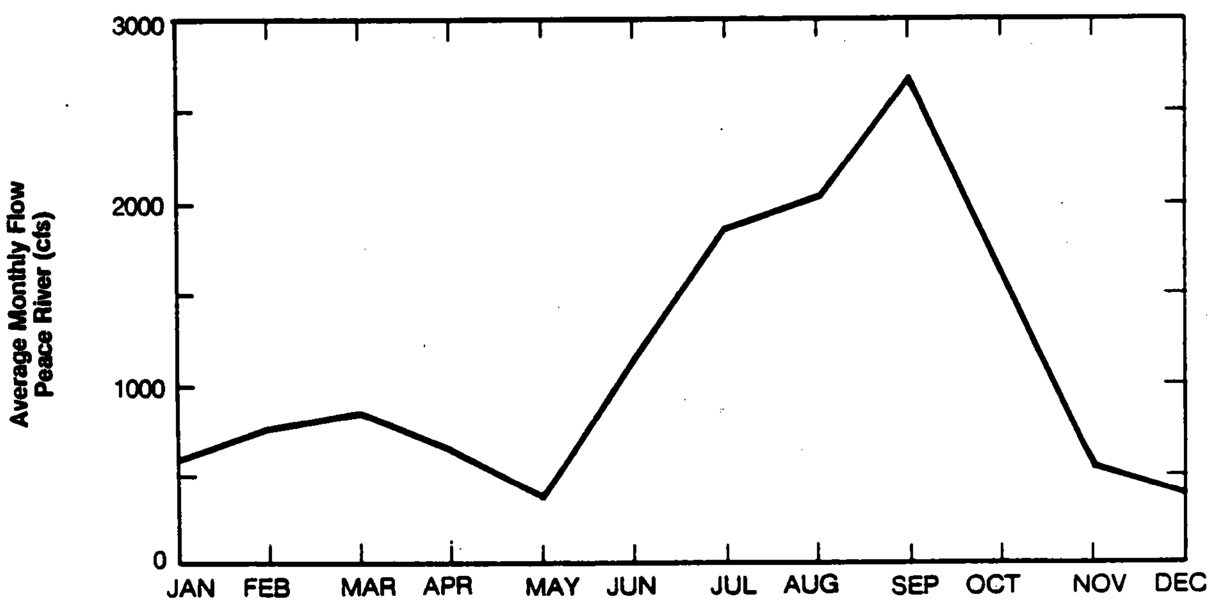

Data Base: 1931-1986 (Flow)
1976-1986 (Quality)
Data Source: EQL (1987).

FIGURE 6-10.
Peace River Average Monthly Flow at Arcadia vs. Hardness and Color Content at Peace River WTP.

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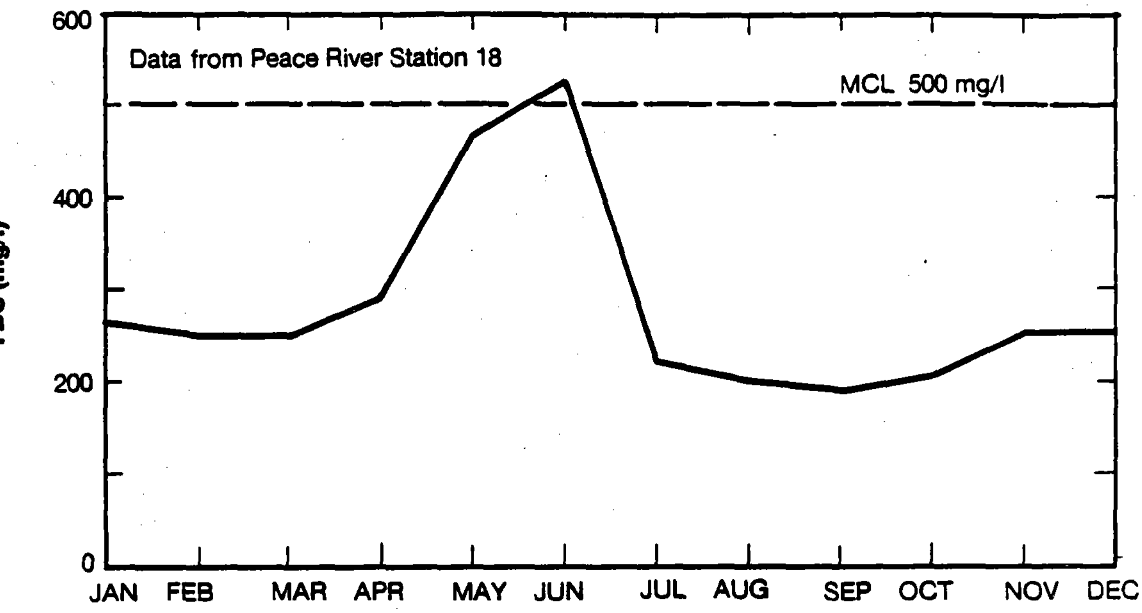

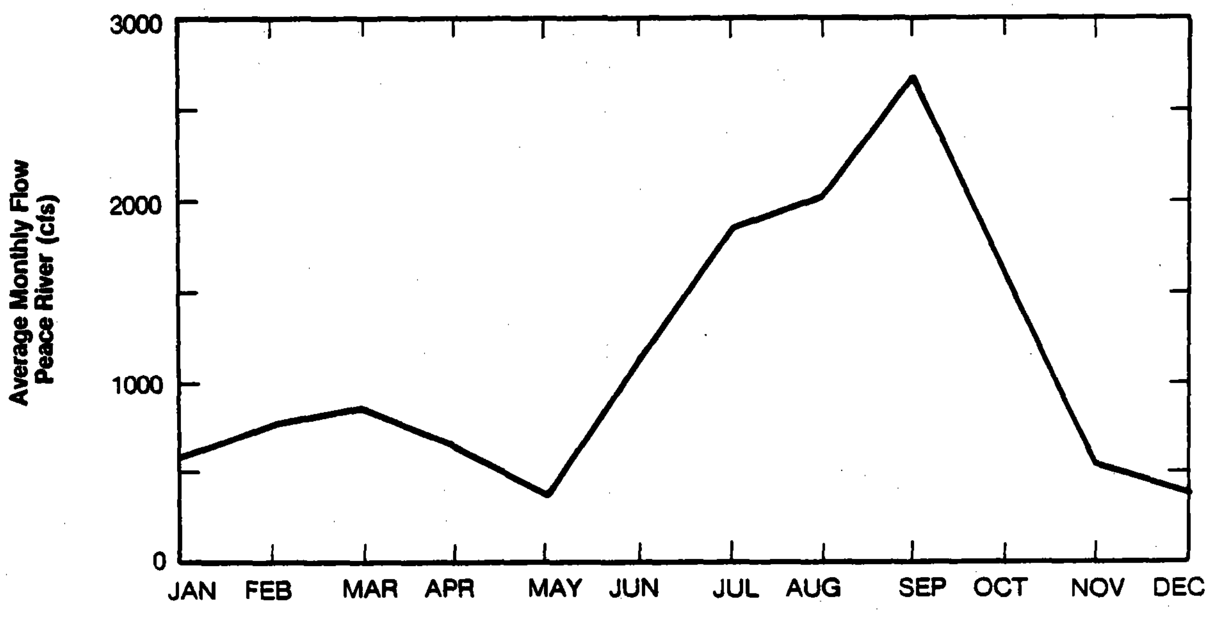

Data Base: 1931-1986 (Flow)
1976-1986 (TDS)
Data Source: EQL (1987).

FIGURE 6-11.
Peace River Average Monthly Flow at Arcadia vs.
TDS Content at Peace River WTP.

CH2M HILL

Settled waters from the reactor/clarifiers are combined in the outer ring of the pentagon and directed to the recarbonation basin. Chlorammoniation is used to provide adequate disinfection and to control THM formation. Chlorine is applied to the settled water as it leaves the pentagon distribution chamber. Ammonia is applied at the effluent end of the recarbonation basin. At current maximum day flows (6 mgd) at the Peace River WTP, this arrangement provides about 10 minutes of free chlorine contact time. After ammoniation, monochloramine provides the residual disinfectant for the finished waters. Monochloramine will not produce THMs. The system performs well, with no reported bacteriological quality problems and THM levels below current standards.

The six gravity filters are designed to operate in the variable declining rate mode, with the water level essentially the same in all operating filters at any time. By providing a relatively large influent header pipe to serve all the filters and a relatively large influent valve to each individual filter, head losses are small and do not restrict the flow to each filter. The header and influent valve are able to deliver the flow each individual filter is capable of taking at a given time. A flow-restricting orifice in the effluent pipe prevents excessively high filtration rates when the filter is clean.

At any point in time, each filter accepts a proportion of the total flow, dependent on its head loss and the common water level above all filters. As filtration continues, the flow through the dirtiest filter tends to decrease the most rapidly, causing an automatic redistribution of the flow to the cleaner filters. The water level rises concurrently to provide the additional head needed by the cleaner filters to handle the flow bypassing the dirtier filters. The cleanest filter accepts the greatest flow increase in this redistribution. As the water level rises, it offsets the decreased flow through the dirtier filters and, as a result, the overall flow rate remains the same.

This method of operation causes a gradually declining rate throughout a filter run. In general, filter effluent quality is affected adversely by abrupt increases in the rate of flow, but here the rate increases occur slowly in the cleaner filters, where they have the least adverse effect on the filter effluent quality. Rate changes occur gradually and smoothly throughout the day in all of the filters in reaction to varying total plant flows, without any automatic control equipment.

For a short period of time at the beginning of a filter run, the turbidity or suspended solids level in the effluent may be higher than average. This normal phenomenon is typically resolved with a filter-to-waste period when filters are first placed in service after backwashing. At the Peace River WTP, the filter-to-waste cycle is not used and valve operators for the process have been removed. The filter-to-waste system should be restored and used routinely after every backwash.

Filtered waters flow by gravity to the clearwell and are pumped by the filter effluent (clearwell) transfer pumps to the high service pump station or to ground storage.

Overall Water Quality

Plant operation reports for 1981-1986 and water quality reports of selected distribution system samples (1982-1986) were reviewed. Current FDER water quality standards are compared to distribution system samples in Table 6-9. Selected raw and finished water quality parameters are compared in Table 6-10. From these data, it is apparent that the Peace River WTP is consistently producing a high quality product that meets or exceeds all major criteria, except for taste and odor. Although quantitative and qualitative data are not available, taste and odor is a serious, on-going concern at Peace River WTP. Because the problem also exists at the North Port WTP, taste and odor removal processes are discussed in a later section.

Since THM control measures (chlorammoniation) were introduced in 1982, the Peace River WTP has consistently met or exceeded current MCLs for THM (0.10 mg/l), as shown in Table 6-11. However, much lower MCLs are anticipated in the future (as low as 0.005 mg/l are being considered). Existing treatment processes would probably not be able to consistently meet the new regulations when adopted. Because of the high THM potential of the raw waters and the comparatively low standard to be met, pilot testing involving alternative disinfectants such as ozone will be required.

A new turbidity standard of 0.1 NTU is under consideration. Current average finished water turbidities are 0.17 NTU, with highs of nearly 0.5 reported. New standards can probably be met by using a more effective polymer or relocating the polymer addition point in the process train.

In summary, with the exception of taste and odor, the Peace River WTP meets or exceeds criteria for drinking water. However, means of improving taste and odor control, and meeting future (lower) THM and turbidity standards should be investigated. In addition, a computerized data management system for water quality and operational data is strongly recommended.

Table 6-9
SUMMARY OF FLORIDA DRINKING WATER STANDARDS (FAC 17-22)
WITH COMPARISON TO PEACE RIVER WTP DISTRIBUTED WATER QUALITY

Group/Parameter	Maximum Contaminant Level (MCL)[a]	Date and Location of Sample						
		6-17-80 316 Augusta	5-3-82 138 Harbor Blvd.	1-17-83 160 Midway	3-26-84 432 Olean Blvd.	8-27-85 432 Olean Blvd.	2-13-86 432 Olean Blvd.	8-13-86 432 Olean Blvd.
I. Primary Standards								
Arsenic	0.05	<0.01	<0.01	<0.01	<0.01	<0.002	<0.003	<0.007
Barium	1.0	<0.1	<0.1	<0.1	<0.1	<0.03	<0.03	<0.15
Cadmium	0.010	<0.005	<0.005	<0.005	<0.005	<0.003	<0.003	<0.005
Chromium	0.05	<0.01	<0.01	<0.01	<0.01	<0.014	<0.03	<0.037
Lead	0.05	<0.01	<0.01	<0.01	<0.01	<0.027	<0.024	<0.05
Mercury	0.002	<0.0005	<0.0005	<0.0005	<0.0005	<0.0001	<0.0001	<0.0002
Nitrate (as N)	10.0	0.25	0.12	0.12	0.27	--	--	0.379
Selenium	0.01	<0.005	<0.005	<0.005	<0.005	<0.002	<0.003	<0.003
Silver	0.05	<0.01	<0.01	<0.01	<0.01	<0.006	<0.006	<0.008
Sodium	160.0	--	48	42	25	34	53	42
Fluoride[b]	1.4	0.99	0.89	0.90	0.41	0.42	0.48	0.36
Turbidity	1.0[c]	0.08	0.15	0.10	0.0	0.11	0.20	0.92
Endrine	0.002	ND	ND	ND	ND	ND	ND	ND
Lindane	0.004	ND	ND	ND	ND	ND	ND	ND
Methoxychlor	0.1	ND	ND	ND	ND	ND	ND	ND
Toxaphene	0.005	ND	ND	ND	ND	ND	ND	ND
2,4-D	0.1	ND	ND	ND	ND	ND	ND	ND
2,4,5-TP (Silvex)	0.01	ND	ND	ND	ND	ND	ND	ND
Coliform Bacteria[d] (colonies per 100 mL)	1					<1		
Radionuclides								
Combined Radium 266 & 288[e]	5 pCi/L						0.7±0.4	
Gross Alpha	15 pCi/L		1.9 ±3.3	1.9 ±2.0		4.3	0.5 ±0.4	4.9
Beta and Photon Radioactivity (man-made Radionuclides)	4 Millirem/year							

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Table 6-9
(Continued)

Group/Parameter	Maximum Contaminant Level (MCL)[a]	Date and Location of Sample 6-17-80 316 Augusta	5-3-82 138 Harbor Blvd.	1-17-83 160 Midway	3-26-84 432 Olean Blvd.	8-27-85 432 Olean Blvd.	2-13-86 432 Olean Blvd.	8-13-86 432 Olean Blvd.
I. Primary Standards (continued)								
Trihalomethanes (THM)	0.1	REPORTED IN TABLE 6-11						
(Sum of Bromodichloromethane, Dibromochloromethane, Tribromomethane, Trichloromethane)								
Volatile Organics (VOC)								
Trichloroethylene	3 μg/L		ND	ND	ND	ND	ND	ND
Tetrachloroethylene	3 μg/L		ND	ND	ND	ND	ND	ND
Carbon Tetrachloride	3 μg/L		ND	ND	ND	ND	ND	ND
Vinyl Chloride	1 μg/L		ND	ND	ND	ND	ND	ND
1,1,1-Trichloroethane	200 μg/L		ND	ND	ND	ND	ND	ND
1,2-Dichloroethane	3 μg/L		ND	ND	ND	ND	ND	ND
Benzene	1 μg/L		ND	ND	ND	ND	ND	ND
Ethylene Dibromide	0.02 μg/L		--	--	ND	ND	ND	ND
II. Secondary Standards								
Chloride	250	--	50	37	27	35	71	23
Color	15 A.P.H.A. units	--	0	0	0	2.0	2.0	12
Copper	1	--	<0.01	<0.01	<0.01	<0.02	<0.008	<0.01
Corrosivity	-0.2 to +0.2 Langelier index	--	0.0	+0.4	-0.23	-0.1	+0.18	+0.20
Foaming Agents (MBAS)	0.5	--	<0.01	<0.01	<0.1	<0.1	<0.1	<0.1
Iron	0.3	--	<0.05	<0.05	<0.1	<0.03	<0.012	<0.015
Manganese	0.05	--	<0.05	<0.05	<0.01	<0.005	<0.005	<0.008
Odor	3 (threshold odor number)	NOT REPORTED						

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Table 6-9
(Continued)

Group/Parameter	Maximum Contaminant Level (MCL)[a]	Date and Location of Sample						
		6-17-80 316 Augusta	5-3-82 138 Harbor Blvd.	1-17-83 160 Midway	3-26-84 432 Olean Blvd.	8-27-85 432 Olean Blvd.	2-13-86 432 Olean Blvd.	8-13-86 432 Olean Blvd.
II. Secondary Standards (continued)								
pH (at collection point	$\geq$ 6.5	--	8.2[g]	8.5[g]	8.2[g]	8.33[g]	8.16[g]	8.75[g]
Sulfate	250	--	104	115	97	97	150	86
TDS	500 (or greater)[g]	--	268	428	214	256	427	190
Zinc	5	--	0.15	<0.01	<0.01	<0.002	<0.006	<0.015
III. Synthetic Organic Contaminants (SOC) (listed separately; see Table 6-1)	NA	--	--	--	ND	--	--	--

[a] MCL reported in mg/l except where noted otherwise.
[b] Maximum annual average of local daily air temperature determines the MCL shown for fluoride.
[c] MCL is for average of two consecutive days or longer if proven to not reduce water quality.
[d] For membrane filter technique one colony/100 ml = monthly average; four/100 ml in more than one sample if 20 samples or less are analyzed monthly; four/100 ml in more than 5% of samples if 20 samples or more are analyzed monthly.
[e] Gross alpha may be substituted for Radium 226 & 288 provided the alpha particle activity is $\leq$ 5 pCi/L at a 95% confidence interval (1.65 SIGMA).
[f] May be different if stabilizers are used.
[g] Laboratory values.
[h] More than 500 mg/l TDS is acceptable if no other MCL is exceeded.
Note: ND = None detected.

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Table 6-10
PEACE RIVER WTP AVERAGE MONTHLY WATER QUALITY[a]

Parameter	Raw Min	Raw Max	Raw Avg	Finished Min	Finished Max	Finished Avg	FAC 17-22 Drinking Water Standard[b]
pH	6.4	7.8	7.2	8.0	8.6	8.3	6.5 (min)
Alkalinity, mg $CaCO_3$/l	33	84	54	34	72	50	--
Total Hardness, mg $CaCO_3$/l	67	211	136	66	240	137	--
Calcium, mg $CaCO_3$/l	42	150	95	38	140	87	--
Magnesium, mg $CaCO_3$/l	18	79	43	28	103	50	--
Sodium, mg/l	4	257	37	36	78	53	160[c]
Chloride, mg/l	30	403	69	38	123	65	250
Sulfate, mg/l	11	108	61	21	166	108	250
Total Dissolved Solids, mg/l	70	1,112	243	145	406	227	500[d]
Color, units	26	799	116	0	3	0	15
Turbidity, NTU	NOT REPORTED			0.04	0.46	0.17	1.0[c,e]

[a]Based on GDU monthly plant operating reports for 1981-1986 and other WTP laboratory analyses records (unpublished).

[b]Selected Florida drinking water standards.

[c]Primary standards, others are secondary standards.

[d]TDS may exceed 500 mg/l, if no other MCL is exceeded.

[e]Monthly average primary standard for surface water systems, except that five or fewer turbidity units may be allowed if certain specified criteria are met.

Table 6-11
PEACE RIVER WTP DISTRIBUTED WATER QUALITY
TOTAL TRIHALOMETHANE BY QUARTERS

Year	Quarter	Total Trihalomethane[a] (mg/l)
1982	4th	0.389
1983	1st	0.065
	2nd	0.064
	3rd	0.078
	4th	0.077
	Avg	0.071
1984	1st	0.050
	2nd	0.072
	3rd	0.057
	4th	0.074
	Avg	0.063
1985	1st	0.087
	2nd	0.046
	3rd	0.095
	4th	0.047
	Avg	0.069
1986	1st	0.036
	2nd	0.048
	3rd	0.062
	4th	0.033
	Avg	0.045

[a] TTHM MCL is 0.10 mg/l.

Chemical Consumption

Chemical use data for 1986 are summarized in Table 6-12. As mentioned previously, the Peace River WTP is normally operated in the color removal mode. The 1986 average alum dosage was 125 mg/l, just slightly higher than the 117 mg/l average feed rate at the North Port WTP during the year (the North Port WTP was operated in the softening treatment mode several months during 1986).

FACILITIES EVALUATION

The Peace River WTP facility has been evaluated by unit process. Nominal capacities of each major component are given in Table 6-13. Comparison of facilities to certain FDER water treatment plant design guidelines is shown in Table 6-14.

OVERALL CAPACITY EVALUATION

The plant was evaluated for its overall estimated capacity to treat water, hydraulically and in process terms. The evaluation is based on typical design criteria; no onsite testing or detailed hydraulic analysis has been done.

Hydraulics

Hydraulically, the raw water pumping and transfer pumping capacities are most important. Plant storage and high service pumping capacities relate to water demands rather than treatment rates and are not considered here. Based on pump nameplate ratings, the firm capacity of the raw water and transfer pumping is about 13 and 12 mgd, respectively.

Based on the above, the present firm hydraulic capacity of the plant is about 12 mgd, limited by transfer pumping capacity.

Process

The nominal capacity of the solids contact units is 12 mgd. Based on typical turbidity removal rise rate criteria (1.0 gpm/ft^2), these units may be capable of treating 14 mgd. The maximum firm filtration capacity is about 13 mgd, using a 3.0 gpm/ft^2 loading rate criterion and assuming 5 of 6 filters in service. The design capacity of the recarbonation basins is reported to be 15 mgd.

The firm process capacity of the plant is estimated at 12 to 14 mgd, limited by the solids contact process capacity.

Table 6-12
1986 CHEMICAL DOSAGE/USAGE[a]
PEACE RIVER WTP

Chemical	Dosage (mg/l) Avg.	Dosage (mg/l) Approx. Range	Usage (lb/day)[b] Avg.	Usage (lb/day)[b] Approx. Range
Powdered Activated Carbon, as PAC	20	8-45	766	570-1,045
Alum, as 49% $Al_2(SO_4)_3$	125	90-188	4,801	3,279-6,245
Caustic, as 50% NaOH	37	16-74	1,467	796-2,087
Polymer[c], as neat polymer	1	0.6-2.0	52	29-171
Chlorine, as Cl_2	9	6-17	394	333-466
Ammonia, as NH_3	1.3	.67-1.8	54	46-60

[a]Based on GDU monthly plant operating reports for January-December 1986.

[b]Average day plant throughput for 1986 was about 4.2 MG.

[c]Whispro floc 20 (nonionic starch).

Table 6-13
PEACE RIVER WTP
EXISTING UNIT PROCESS RATED CAPACITIES

RAW WATER PUMPS	
Peace River Intake Structure	
Number	3
Type	Vertical Turbine
Capacity (each) gpm	2 @ 4,600 1 @ 8,320
Raw Water Storage Reservoir Pump Station	
Number	3
Type	Vertical Turbine
Capacity (each) gpm	2 @ 4,200 1 @ 8,400
SOLIDS CONTACT UNIT	
Number	2
Type	General Filter Type C
Diameter, ft	85
Volume (each) gal	594,000
Clarification area (each) ft^2	4,920
Capacity (each) mgd	6
GRAVITY FILTERS	
Number of Bays	6
Type	Declining Rate
Media	Dual
Anthracite	18 inches
Sand	12 inches
Gradated gravel	13 inches
Filter area (each) ft^2	600
Capacity, gpd	12,960,000
Loading (max.), gpm/ft^2	3.0
Loading (avg.), gpm/ft^2	2.5
CLEARWELL	
Number	1
Type	Concrete
Capacity	66,800 gal
TRANSFER PUMPS	
Number	3
Type	Vertical Turbine
Capacity (each) gpm	2 @ 4,200 1 @ 8,400
HIGH SERVICE PUMPS	
Number	6
Type	Horizontal Splitcase
Capacity (each) gpm	1 @ 800 1 @ 1,150 1 @ 2,300 1 @ 3,250 2 @ 5,500
FINISHED WATER STORAGE RESERVOIR	
Number	1
Type	Prestressed Concrete
Diameter, ft	110
Capacity, MG	2
AQUIFER STORAGE RECOVERY SYSTEM (ASR)	
Recovery capacity, mgd	1.5 (nominal) (Currently being expanded to 5.0 mgd)

Table 6-13
(continued)

CHLORINATION	
Number	3
Type	W&T V800 Series
Capacity (each) ppd	2 @ 2,000 1 @ 500
ALUM FEED PUMPS	
Number	2
Type	Diaphragm
Capacity (each) gph	182
CAUSTIC FEED PUMPS	
Number	2
Type	Diaphragm
Capacity (each) gpm	30
CARBON METERING PUMPS	
Number	3
Type	Plunger
Capacity (each) gph	2 @ 115 1 @ 80
CARBON FEED PUMPS	
Number	2
Type	NA
Capacity	NA
COAGULANT AID (POLYMER) FEED PUMPS	
Number	1
Type	Diaphragm
Capacity (each) gph	205
ALUM STORAGE TANKS	
Number	2
Type	Steel Horizontal
Capacity (each) gal	15,000
CAUSTIC SODA STORAGE TANKS	
Number	2
Type	Steel Horizontal
Capacity (each) gal	15,000
CARBON STORAGE TANKS	
Number	2
Type	Concrete
Capacity (total) gal	59,600
CARBON DAY TANKS	
Number	2
Type	Steel
Capacity (each) gal	NA
CARBON DIOXIDE SYSTEM	Not Operational
AMMONIATION	
Number	2
Type	Direct Feed
Capacity (each) ppd	500
SAVE-ALL RETURN PUMPS	
Number	2
Type	NA
Capacity (each) gpm	600

NA = Not available.

Table 6-13
(continued)

SAVE-ALL SLUDGE PUMPS	
Number	2
Type	NA
Capacity (each) gpm	1,400
RAW WATER FLOWMETER	
Number	3
Type	Differential Pressure
Size	1 30-inch 2 42-inch
Range	NA
FINISHED WATER FLOWMETER	
Number	2
Type	Magnetic
Size	1 8-inch 1 24-inch
Range	NA
AUXILIARY GENERATOR	
Number	1
Type	Diesel
Size, kW	1,010
SLUDGE DRYING BEDS (with underdrains)	
Bed Number, approximate size, acres	
1	3
2	2
3	2
4	1.5

NA = Not available.

Table 6-14
COMPARISON TO FDER GUIDELINES[a]
PEACE RIVER WTP

FDER WTP Design Guidelines[b]	Peace River WTP	Status
Plant design Q = max. day demand	Q exceeds max. day	OK
Plant design Q + Finished water storage = 4 hr of max. hr demand	Yes	OK
Chlorination capacity to provide 0.6 mg/L combined residual	Yes	OK
Standby chlorinator	Yes	OK
Auto chlorine supply switchover if unmanned while operating chlorination	Yes, but always manned when operating	OK
Chlorine facilities in separate room, above grade, cross ventilation, weighing devices, safety equipment	Yes	OK
Coagulant aid on approved list	Yes	OK
Chemical additional points should be separated to avoid potential interaction to chemicals	Yes	OK
Flash mix t_d = 5 - 10 sec	Unknown	Works effectively
Upflow Solids Contact		
Detention time 4 hr	2.4 hr	--[c]
Weir loading 10 gal/ft of weir	5.1 gal/ft	OK
Upflow rate 1.0 gpm/ft²	0.85 gpm/ft²	OK
Flash mix + flocculation + settling t_d = 4 hr	2.4 hr at design Q	OK at present Q
Max. filtration rate 2-3 gpm/ft²[d]	2.3 gpm/ft² (avg) 2.8 gpm/ft² (max)	OK OK
Min. filter backwash rate = 15 gpm/ft²	Unknown	OK
High service pumping capacity = max. hr demand	26 mgd is greater than 12 mgd	OK
Finished water metering required	Yes	OK
Auxiliary power to provide at least one half of max. day demand	Yes	OK
Protective fencing around plant	Yes	OK

[a]Based on assumptions that plant nominal rated capacity is 12 mgd, maximum day demand is equal to 6 mgd, and maximum hour demand = 2 x maximum day = 12 mgd.

[b]Only guidelines applicable to Peace River WTP are listed. This is not a complete listing of applicable FDER regulations.

[c]Deviation is permissible for large units that operate continuously.

[d]Up to 6 gpm/ft² (with one filter out of service) may be allowed with acceptable performance test results.

Conclusions

The maximum firm plant capacity, on a continuous service basis, is estimated at 12 mgd. A comprehensive hydraulic analysis is beyond the scope of this project. Plant personnel report that plant individual unit processes have been successfully operated at or above their nominal ratings. Based on the limited information available, it appears that ancillary systems such as chemical feeds, backwash water recovery, etc., would be capable of meeting plant needs at these rates.

SLUDGE MANAGEMENT

As described previously, current sludge handling practices at both the North Port WTP and the Peace River WTP are nearly identical. Four major aspects of sludge treatment and disposal at the plants must be considered:

1. Quantity and quality of sludge
2. Sludge transport
3. Sludge dewatering
4. Ultimate disposal

The best overall sludge treatment process will minimize the quantity of sludge to be handled and optimize sludge dewatering and transport, to minimize both the amount and costs of ultimate disposal. Several options are available to meet these goals. While a comprehensive analysis is beyond the scope of this project, a summary of some possible options follows. These options must be verified by in-plant testing.

REDUCED SLUDGE PRODUCTION

Reducing the sludge produced without sacrificing finished water quality should be a major operational goal for both the short and long term. Some possible means to achieve this goal include (1) reduced alum addition through process optimization and close control of alum feed, (2) the use of alternative primary and secondary flocculants, (3) modification of sludge blowdown practices to increase solids concentration and minimize overall sludge volume, and (4) use of gravity thickening with or without chemical addition. Use of pre-oxidants such as ozone have been reported to reduce alum dosages and subsequently the volume of sludge produced.

SLUDGE TRANSPORT

Common sludge transport options include hauling in a dump truck or a tank truck and pipeline.

SLUDGE DEWATERING

Dewatering technology for alum sludge does not yet allow precise prediction of process performance. Alum sludge resulting from treatment of a raw water with a low solids content and high color, as is the case at the Peace River and North Port WTPs, is particularly noted as being the most difficult to dewater. Dewatering alternatives traditionally considered for lime sludges or sludges from wastewater treatment plants will not necessarily be applicable at these WTPs. Consequently, a screening process must be performed to identify the most promising option(s). An unprioritized list of possible alum sludge handling options is provided in Table 6-15.

ULTIMATE SLUDGE DISPOSAL

Any dewatering methods for alum sludge will result in both a concentrated sludge for ultimate disposal and supernatants or filtrates that must be either disposed or recycled. Additionally, if an alum recovery process is employed, an aqueous waste sidestream that must be disposed or recycled is produced.

These final steps in the sludge handling process can be the most difficult. As a result, all sludge treatment processes seek to minimize the final waste product and the associated disposal cost. Currently, dewatered sludge is stockpiled on site at both the Peace River and North Port WTPs. Although this practice is relatively inexpensive, it is inefficient and will likely have to be ended in the future. Finding an alternative site will become increasingly difficult and expensive as the pressures of urbanization make the land available for waste disposal scarce. In addition, because most Florida soils are sandy and have a low pH, the leaching of aluminum ion from sludge into ground or surface waters is a possibility. The aluminum ion concentration of the receiving waters could then exceed current criteria for agricultural use or future criteria for human consumption. Ultimate disposal of waste products will thus likely become a significant factor for any sludge treatment process. Table 6-16 is a list of possible ultimate sludge disposal options that should be considered.

TASTE AND ODOR CONTROL

Periodic taste and odor problems are experienced at both the Peace River and North Port WTPs. The repeated presence of objectionable tastes and odors in a water supply suggests to the public that potentially toxic chemicals could be present in the unsatisfactorily treated water. This perception may

Table 6-15
POSSIBLE SLUDGE DEWATERING OPTIONS

Description
Drying Techniques
(a) Sludge drying beds
(b) Vacuum assisted drying beds
(c) Wedge wire drying beds
(d) Lagoons/ponds
Mechanical Dewatering With or Without Chemicals
(a) Filter press
(i) Plate and frame
(ii) Diaphragm
(b) Centrifugation
(i) Scroll
(ii) Basket
(c) Belt press
(i) Low pressure
(ii) High pressure
(d) Vacuum filter
(i) Cloth
(ii) Coil
Alum Recovery
(a) Liquid-ion exchange
(b) Acid processes
Sewer Discharge
Use of existing sludge drying beds with overflow to sanitary sewer collection system (North Port WTP only)

Table 6-16
POSSIBLE ULTIMATE SLUDGE DISPOSAL OPTIONS

Description
Land Disposal
(a) Buried in a landfill
(b) Mixed with other materials to provide a suitable cover material for a landfill
Recovery and Reuse
(a) Industrial
(b) Commercial
(c) Agricultural
(d) Municipal
Direct Discharge
(a) Sanitary sewer system (North Port WTP only)
(b) Gulf of Mexico
(c) Phosphate slime ponds
(d) GDU-owned lagoons offsite
(e) GDU-owned land by land spreading

cause the public to turn to other sources that may not be as safe for its health.

Taste and odor problems are commonly caused in surface waters by algae and other microorganisms. Agricultural run-off and industrial contaminants are other contributors. The most cost-effective means for controlling taste and odors is at the source. Where this is not possible or practical, removal may be accomplished in the treatment plant. This discussion is limited to removal methods at the plant.

TREATMENT PROCESSES

Because of the various combinations of inorganic and organic compounds that cause tastes and odors in water supplies, a wide variety of treatments are employed. Many common water treatment processes, including coagulation, flocculation, sedimentation, and filtration, aid in the removal of odorous substances from water. Depending on the physical and chemical conditions of the water at the time of treatment, each process has varying degrees of efficiency. However, because most known taste and odor compounds are in a reduced state, some form of oxidation is usually required to provide effective treatment. Generally, no simple treatment process is cost-effective for all taste and odors that may develop, and a site-specific analysis is required. Processes most commonly employed for removal of tastes and odors via oxidation are aeration, chlorination, ozonation, and treatment with permanganate or chlorine dioxide.

Aeration

Aeration, principally used to oxidize soluble iron and liberate hydrogen sulfide and carbon dioxide, successfully removes highly volatile organic substances from water. While seldom, if ever, useful for taste and odor control by itself, the aeration process can decrease the amount of materials required to remove remaining taste and odors.

Chlorination

Chlorination is generally very effective with low-level inorganic odors, such as hydrogen sulfide, organic sulfides, disulfides, and mercaptans. Chlorination often increases problems when used for odors of industrial or certain algal products such as methylisoborneal (MIB) or geosmin. In such cases, superchlorination to a free residual followed by partial dechlorination is often necessary. The regulation on chlorinated organics and the cost of the process make this alternative less attractive today than it has been in the past.

Ozonation

Ozone is widely known for its ability to oxidize tastes and odors. Ozone is the strongest oxidizing agent available for water treatment and the only effective oxidant that does not increase TDS. Ozone must be generated onsite to meet demand and soon dissipates when introduced into solution. Generally, ozone doses of 0.5 to 5 mg/l have been required for effective odor control. However, ozone has not been universally successful at low levels. As a preoxidant, ozone has been reported to enhance coagulation of colloids, often permitting lower alum dosages. Ozone also appears to form much smaller amounts of undesirable byproducts than does either chlorine or chlorine dioxide.

Permanganate

Unlike chlorination, potassium permanganate ($KMnO_4$) treatment has been especially effective for certain industrial and algal odors. Permanganate is more effective in alkaline rather than neutral or acid waters and generally requires doses of 1 to 3 mg/l and a contact time of at least 1 to 2 hours. If excessive permanganate is used in the oxidation process, it will pass through the filters and enter the distribution system, where it forms manganese dioxide (MnO_2). Manganese dioxide will blacken the water. In addition, manganese concentrations in the final treated water will increase and may exceed the levels prescribed in the secondary regulations. The requirement for alkaline (high pH) conditions and relatively long reaction times probably explain why previous attempts to control odors with $KMnO_4$ at the Peace River WTP were not successful.

Chlorine Dioxide

Although not as widely used as chlorine, permanganate, or ozone, chlorine dioxide is also a strong oxidizing agent. Like ozone it must be generated onsite. A stable, aqueous form of chlorine dioxide is available that eliminates the need for onsite generation, but is currently too costly for routine use. Chlorine dioxide is especially effective for odors resulting from phenols and chlorinated phenols. Although chlorine dioxide can be used for control of algal odors, its cost is usually prohibitive. Chlorine dioxide does not form trihalomethanes, but is capable of chlorinating many organic compounds with which it comes in contact. Concern over the health effects of these potential reaction products as well as the reaction end products (chlorite and chlorate ion) has led regulatory agencies to limit the applied dosages of chlorine dioxide. In organic-rich waters such as the Peace River and Myakkahatchee Creek, the limited dosages are not effective.

Activated Carbon Adsorption

Activated carbon has long been viewed as a most reliable last recourse for removal of taste and odors. Activated carbon can be used in either of two forms, powdered activated carbon (PAC) or granular activated carbon (GAC). Currently, PAC is most widely used in normal taste and odor applications, but public concern for higher quality water is requiring increased use of GAC.

At the Peace River and North Port WTPs, PAC in slurry form is added to the raw water before coagulation. In this co-current mode, PAC is very inefficient because the most saturated (spent) carbon is in contact with the cleanest water. Experience has shown that a given amount of PAC is more effective when deposited in the filter (Cox, 1964). In this mode, care must be exercised to prevent PAC from passing through the filter and is not recommended for dosages of PAC greater than about 10 mg/l.

In general, the capacity of either PAC or GAC to remove humic substances is rather limited. However, research indicates that GAC is highly selective for some of the most troublesome odor compounds, MIB and geosmin, even in the presence of substantial amounts of humic substances. As a result, GAC could be expected to have a very long life (approximately 2 years) where removal of these specific compounds is concerned. Practical experience confirms that GAC can be used for 2 to 3 years for odor removal in certain cases.

CONCLUSION

The previous discussion indicate that ozone and GAC or a combination of the two are the most promising treatment processes for taste and odor control at the Peace River and North Port WTPs. Since both alternatives are capital-intensive and effective taste and odor removal processes are highly site-specific, a thorough investigation of these processes (including pilot testing) would be required prior to modifying either plant's process flow. Treatment alternatives should be evaluated in concert with source control measures.

SUMMARY

Current firm capacity of the North Port WTP is 4.4 mgd. Concerns about the quantity and quality of its raw water source, the Myakkahatchee Creek, make future usage uncertain. Secondary drinking water standards for TDS and sulfates are not met at times, a circumstance that is magnified because existing treatment processes at the plant increase concentrations of those substances. Hardness removal (when the softening operational mode is used) is limited because caustic soda can be added only to an extent that does not exceed the sodium primary drinking water standard. Taste and odor control is an ongoing problem, and new THM and turbidity standards under consideration as a result of the 1986 Amendments to the Safe Drinking Water Act will probably be too strict for existing facilities to meet.

The Peace River WTP, which has a current firm capacity of 12.0 mgd, can be expected to continue as the major treated water source for the GDU Port Charlotte service area. With adequate offstream storage capacity for both raw and finished waters, all current drinking water standards are typically met. Major concerns about water quality center on taste and odor control, and possibly stricter future THM and turbidity standards. GDU is committed to monitoring changes in the drinking water regulations and to taking appropriate action to ensure compliance.

Future studies must focus on sludge production, handling, and disposal for both WTPs, both for the large costs associated with these items and to address environmental concerns.

SECTION 7
Alternatives for Facility Expansion

Section 7
ALTERNATIVES FOR FACILITY EXPANSION

APPROACH

The following three potential sources for meeting water demands for the Port Charlotte service area through the year 2000 were identified and discussed in Section 3:

1. Expansion of raw water supply from the Myakkahatchee Creek, with treatment at the North Port WTP

2. Expansion of raw water supply from the Peace River, with treatment at the Peace River WTP

3. Development of brackish groundwater supplies, with treatment at new RO desalting facilities

The locations of these sources, including the areas of two potential well fields for brackish groundwater development, are shown in Figure 7-1.

This section provides an analysis of the facility requirements and planning level costs for expansion/development of each of the potential sources, to allow comparison of their cost-effectiveness. Planning level cost estimates include construction costs; indirect capital costs, assumed to be 30 percent of construction costs; and annual operation and maintenance (O&M) costs. Details for the basis of cost estimates are provided in Appendix C. Conventional water treatment plant costs are based on cost curves published by the EPA (Gumerman et al., August 1979). RO plant costs are estimated using a CH2M HILL-developed cost estimating program. Costs for ASR, surface reservoirs, O&M, and other components are estimated using recent cost data or costs updated from the 1985 Peace River ASR study (CH2M HILL, April 1985a).

Equivalent annual costs for each source include amortized capital cost plus O&M costs. The amortization is based on the following criteria supplied by GDU:

- Interest rate: 9.0 percent per year

- Conventional water treatment plant life:

    - Structures: 33 years
    - Equipment: 22 years

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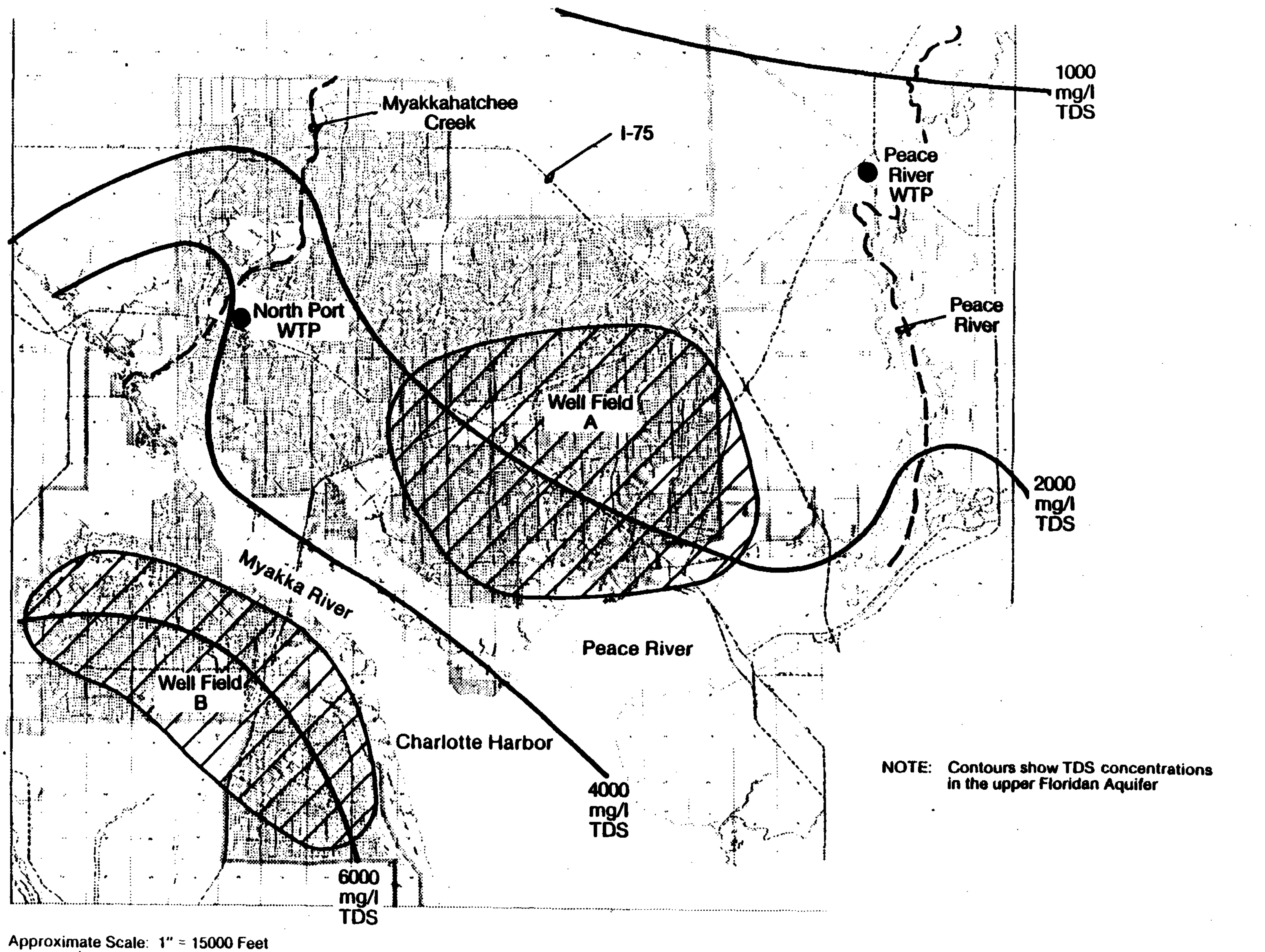

FIGURE 7-1.
Potential Raw Water Sources for the Port Charlotte Service Area.

CH2M HILL

- o RO treatment plant life:
  - Plant structure: 33 years
  - Plant equipment: 22 years
  - RO membrane: 5 years
  - Supply and disposal wells: 30 years
- o Offstream raw water reservoir life: 50 years
- o ASR well field life:
  - Pumps: 20 years
  - Wells: 30 years
  - Piping: 43 years
- o Major pump station life:
  - Structures: 33 years
  - Equipment: 20 years

All estimates are developed in March 1987 dollars.

EVALUATION OF RAW WATER SUPPLY SOURCES

PEACE RIVER

Expansion Requirements

The expansion path for the Peace River WTP was developed using the computer model PLANT, developed by CH2M HILL for GDU in 1985. PLANT simulates the operation of the water treatment plant and associated facilities on a monthly basis using available river flow and quality data, and the capacities of the various facilities. The components included are the treatment plant, the surface reservoir, and the ASR system. A thorough description of the program is found in the program documentation (CH2M HILL, April 1985b).

For this analysis, PLANT was modified to allow the input of a time variable demand function. Monthly demands and the year they are in effect (with year 1 as the first year of simulation) are input; the program then interpolates between these points to obtain demands for each month of simulation. The modification more realistically simulates the demands placed on the system than did the previous use of constant monthly demands for every year of a simulation.

The program was also modified so that a maximum storage volume for ASR can be input. If the amount of water in ASR storage at the beginning of a month is more than the specified amount, no treated water is injected that month, even if it is available. This modification enhances determination of the sensitivity of system reliability to available ASR storage. For this analysis, a maximum storage of 350 MG per mgd of ASR capacity was used with no decrease in system reliability. This is equivalent to storing enough treated water to allow withdrawal of acceptable quality water (less than 500 mg/l TDS) for 6 months. (Note: GDU's existing ASR wells operate under a 250-MG per mgd maximum storage capacity guideline; that is, 250 days of supply storage per year. For the computer simulations in this study, however, a 6-month water supply availability from ASR with water quality not exceeding 500 mg/l TDS was used, resulting in a maximum volume of 350 MG per mgd of ASR capacity. The new limit was placed in the model where no previous limit had existed. The overall system reliabilities for either the 250-MG or 350-MG storage limit per well should be equal or nearly equal and should meet or exceed a 99 percent reliability criteria.)

The PLANT model was updated based on ASR operational data obtained since completion of the 1985 project. The value of 0.961 used for the aquifer mixing parameters ALFA and BETA in the 1985 ASR simulation was based on the results of the first injection and recovery cycle. Analysis of additional site-specific injection and recovery data indicates a value of 0.90, which was used in the current expansion analysis. An ALFA value of 0.90 means that the initial blend of recovered water will be 90 percent injected water and 10 percent native water. An identical BETA value indicates that all injected water will be available for withdrawal and that none will be lost because of overall groundwater movement.

The PLANT model was used to simulate the operation of particular size configurations of Peace River facilities for a specified number of years and increasing demands. Each set of facilities was evaluated for 25 different but equally likely synthetic flow sequences generated with the PEACE program from 51 years of recorded flow data for the Peace River. The flow sequences, each of which consisted of a specified number of years of monthly flows, allowed the simulation to show system reliability for given demands under varying flow conditions. The need for expansion was indicated when a particular set of facilities failed to meet the demands with acceptable reliability.

For this analysis, the point of system failure was based on how many months the demand quality or quantity was not met in each of the 25 flow sequences. Acceptable reliability was defined as no monthly failures of either type for at least 20 of the 25 sequences. This was interpreted as an 80 percent (20/25) confidence level that the system would be 100 percent reliable (zero monthly failures) under an expected range of streamflow conditions for a given system demand and number of years. This confidence level means that the system is expected to be acceptable more than

99 percent of the time. When the system failed to achieve acceptable reliability as defined for a year, system expansion would occur and be online in that year, to maintain desired system reliability.

Expansions were assumed to occur in logical increments for each facility. The Peace River WTP was expanded in 6-mgd increments from current 12-mgd capacity, which is consistent with the current plant design. The ASR system was expanded in 3-mgd increments. The resulting expansion path for the Peace River facilities, using the most likely demand scenario discussed in Section 2, is listed in Table 7-1. The facilities shown for a specific demand are intended to be in place when that demand level is reached.

Economic Analysis

The cost for expansion of the Peace River facilities for any average day demand was developed based on the expansion path listed in Table 7-1. Construction costs for expansion of the Peace River WTP were based on existing treatment processes and cost curves published by the EPA (Gumerman et al., August 1979). Construction costs for ASR wells were based on actual bid amounts for the ongoing expansion to 5-mgd capacity near the Peace River WTP.

O&M costs were obtained from GDU for the existing plant (including the offstream reservoir and intake structure), and were assumed to be applicable for all plant configurations. O&M costs include direct labor, direct labor fringe benefits, chemicals, power, and equipment maintenance. Equivalent annual costs for capital costs were added to O&M costs to obtain total annual cost for any demand level. Production cost in dollars per thousand gallons was computed by dividing annual cost by annual production. Table 7-1 lists costs at the major expansion points for the Peace River facility; the total annual cost is plotted in Figure 7-2. The major expansion points listed in Table 7-1 are the peaks in the graph in Figure 7-2. A curve fitted to the values illustrated in Figure 7-2 resulted in the equation shown for annual cost in million dollars per year as a function of average daily demand in mgd.

MYAKKAHATCHEE CREEK

Expansion Requirements

The feasibility of using offstream storage to increase the yield and reliability of the Myakkahatchee Creek source through treatment at the existing North Port WTP was addressed in Section 5. The original PLANT computer model, without the modifications made for the Peace River system, was used to estimate the reliability of various system

Table 7-1
PEACE RIVER TREATMENT
FACILITIES EXPANSION REQUIREMENTS AND COSTS

Average Daily Yield (mgd)	Maximum Daily Yield (mgd)	Raw Water Intake (mgd)	Facility Capacity: Plant: Clarifier (mgd)	Facility Capacity: Plant: Filter (mgd)	Facility Capacity: ASR (mgd)	Facility Capacity: Maximum Delivery (mgd)	Facility Capacity: Surface Reservoir (MG)	Facility Capacity: Surface Reservoir Pump St (mgd)	Cumulative Capital Cost (Million $)	Direct Annual O&M Cost (Million $)	Equivalent Annual Cost (Million $)	Production Cost ($/1,000 gal)
6.5	10.4	22	12	15	5	17	625	24	0.0	1.5	1.5	0.64
8.2	13.1	22	12	15	8	20	625	24	0.9	1.9	2.0	0.67
10.1	16.2	22	12	15	11	23	625	24	1.9	2.4	2.6	0.69
12.0	19.2	22	18	18	14	32	625	24	7.6	2.8	3.6	0.82
14.5	23.2	22	18	18	17	35	625	24	8.6	3.4	4.3	0.80

NOTES:

1. The points listed above are the points where expansion occurs; therefore, the given yield is the lowest for that set of facilities.

2. Capital recovery factors are: Intake = 0.0998, Plant = 0.1018, ASR = 0.0969, Reservoir = 0.0912, Res. PS = 0.0998.

3. Total annual cost as a function of yield can be estimated as:

   Total annual cost = 0.1323 x $(\text{Yield})^{1.29}$
   where cost is in millions of 1987 dollars and yield is in mgd.
   This equation has an R-squared value of 0.998, which indicates that 99.8 percent of the variance in total annual cost can be explained by the variance in yield.

4. All costs are in March 1987 dollars.

5. Costs are order-of-magnitude estimates made without detailed engineering data. It is normally expected that estimates of this type are accurate within -30% to +50%.

6. Expansion requirements based on most likely demand scenario.

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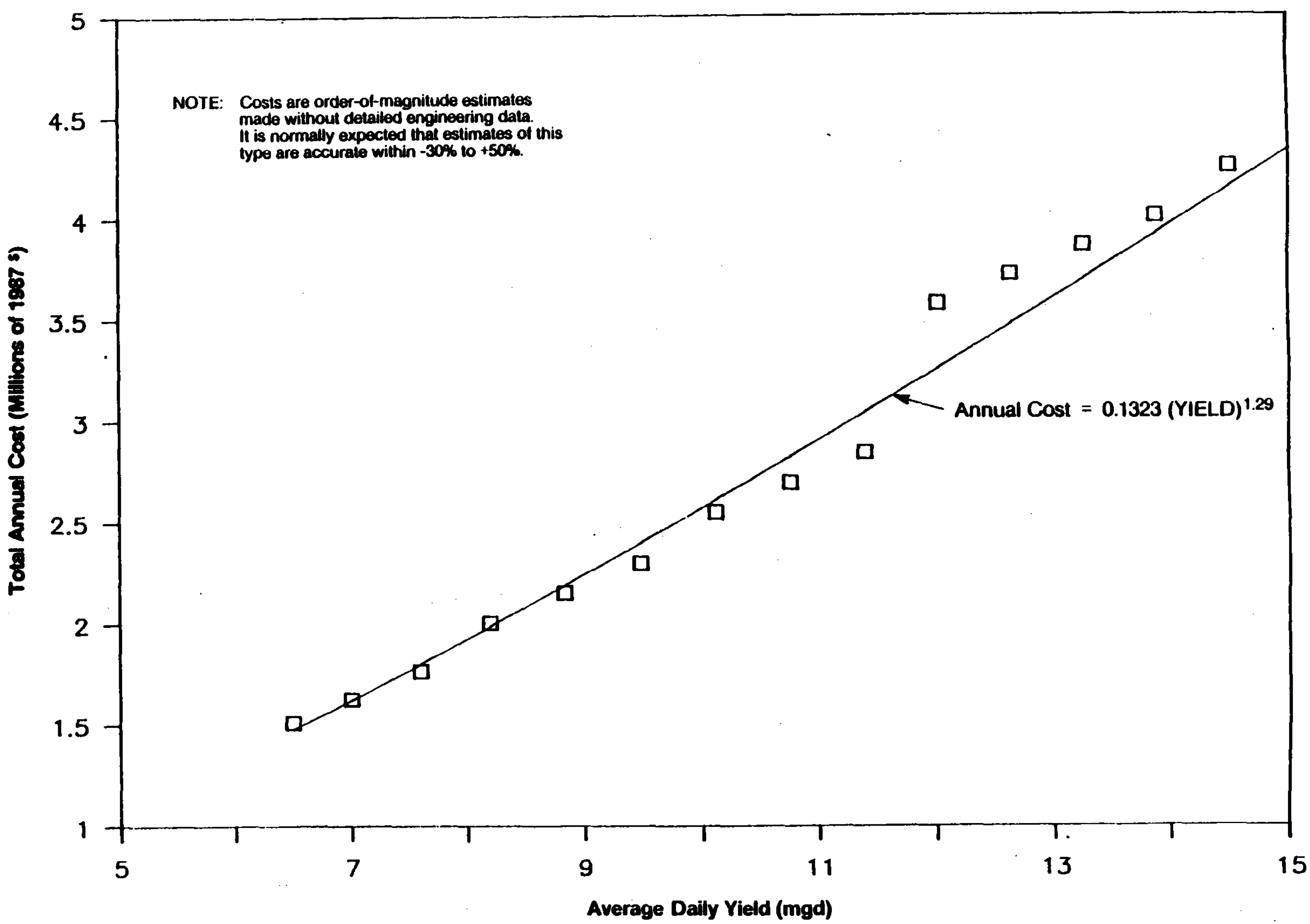

FIGURE 7-2.
Total Annual Cost for Peace River Water Supply System.

configurations. The conclusion presented in Section 5 was that the most appropriate system at the North Port WTP would include both ASR and surface storage to obtain quantity and quality reliability comparable to the Peace River source.

Table 7-2 shows the configurations required at the North Port treatment facilities for various yields. Basically, the plant capacity must be twice the average daily demand, ASR capacity should be equal to the maximum daily demand (1.6 times average daily demand), and the surface reservoir should be large enough to supply average daily demand for 3 months. These requirements result in over 98 percent reliability for the overall system.

<u>Economic Analysis</u>

Costs were developed for each North Port system configuration with the same methods and assumptions used for the Peace River WTP. Treatment plant O&M costs were provided by GDU and were assumed to remain the same for all plant sizes. Capital costs were amortized as described previously. Total annual costs, in 1987 dollars, for the different systems are tabulated in Table 7-2. For this case, total annual cost, as illustrated in Figure 7-3, is a linear function of average daily demand supplied, and is always at least twice Peace River development costs.

<u>BRACKISH GROUNDWATER</u>

<u>Expansion Requirements</u>

The potential for developing a brackish groundwater supply source for treatment by RO desalting was addressed in Section 3. The most favorable area identified for groundwater development was in the northeast corner of the study area near the Peace River WTP, where TDS concentrations in the upper Floridan aquifer average 1,000 mg/l (see Figure 7-1). This area is currently being developed as the ASR well field, however, and is not available for other water supply purposes.

Two areas to the south and west of the Peace River WTP and I-75 were subsequently identified for potential well field development for brackish groundwater supply and are shown in Figure 7-1. As the TDS contours in Figure 7-1 indicate, TDS concentrations in area groundwater increase toward the southwest. Groundwater in well field A, located between the Peace River and North Port WTPs, currently has TDS concentrations of approximately 2,000 mg/l. However, because substantial long-term withdrawals in the area are expected to increase raw water TDS content, groundwater in well field A was assumed to have a steady-state TDS concentration of 3,000 mg/l. Estimated maximum yield from the well field was 10 mgd. Well field B, in the far southwest near the

Table 7-2
NORTH PORT TREATMENT
FACILITIES EXPANSION REQUIREMENTS AND COSTS

Average Daily Yield (mgd)	Facility Capacity: Raw Water Intake (mgd)	Facility Capacity: Plant (mgd)	Facility Capacity: ASR (mgd)	Facility Capacity: Surface Reservoir (MG)	Cumulative Capital Cost (Million $)	Direct Annual O&M Cost (Million $)	Equivalent Annual Cost (Million $)	Production Cost ($/1,000 gal)
2.2	9	4.4	3.5	201	3.9	0.7	1.0	1.29
4.0	16	8.0	6.4	365	11.5	1.2	2.3	1.60
6.0	24	12.0	9.6	548	17.4	1.8	3.5	1.61
8.0	32	16.0	12.8	730	23.5	2.4	4.7	1.62
10.0	40	20.0	16.0	913	29.5	3.0	5.9	1.62
12.0	48	24.0	19.2	1095	35.4	3.7	7.1	1.62

NOTES:

1. Capital recovery factors are: Intake = 0.0998, Plant = 0.1018, ASR = 0.0969, Reservoir = 0.0912.

2. Total annual cost as a function of yield can be estimated as:
   Total annual cost = (0.613 x Yield) - 0.206
   where cost is in millions of 1987 dollars and yield is in mgd.
   This equation has an R-squared value of 0.999, which indicates that 99.9 percent of the variance in total annual cost can be explained by the variance in yield.

3. All costs are in March 1987 dollars.

4. Costs are order-of-magnitude estimates made without detailed engineering data. It is normally expected that estimates of this type are accurate within -30% to +50%.

5. Facilities required to provide finished water of the same quality and similar reliability as the Peace River system.

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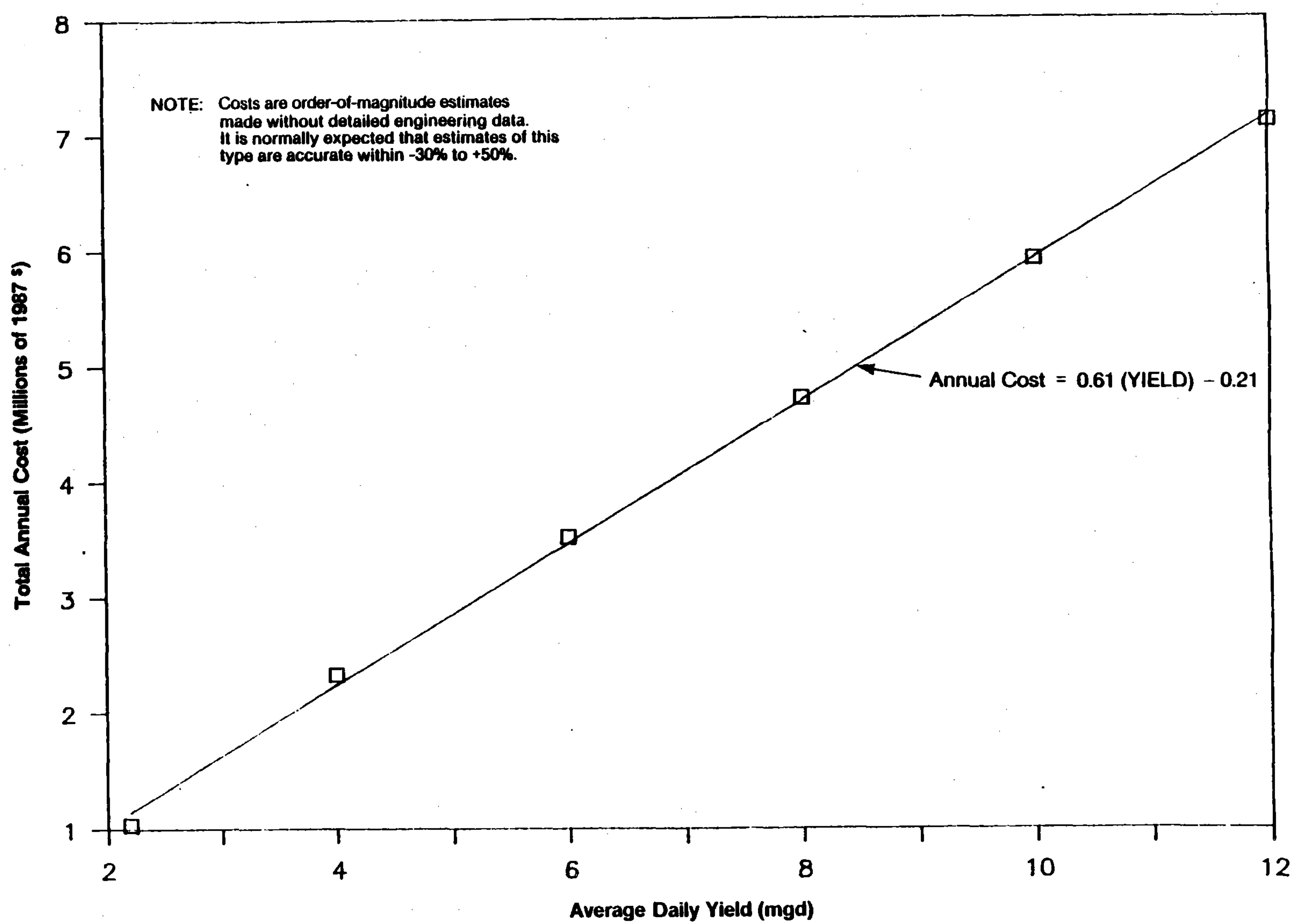

FIGURE 7-3.
Total Annual Cost for North Port Water Supply System.

CH2M HILL

Gulf Cove WWTP, was assumed to have a raw water TDS level of 6,000 mg/l with a maximum yield of approximately 6 mgd.

Economic Analysis

Estimated costs for RO desalting plants with capacities of 2, 5, and 10 mgd for the two well fields included construction and O&M costs. Previously listed assumptions for amortization, facility life, and other parameters were applied. Table 7-3 lists facility sizes and associated costs for desalting plants at each site. Figure 7-4 shows the total annual cost per average daily yield for each brackish water well field. Production costs for brackish groundwater were computed based on the assumption that this source would serve as a supplementary water supply and would operate at near steady-state conditions. Peaking capacity would not be provided by the desalting facilities.

As Table 7-3 and Figure 7-4 show, costs for developing well field B are 10 to 20 percent higher than those for well field A. The costs for developing well field A were also compared to cost data for the North Port facilities in Table 7-2, and found to be about 12 percent lower for comparable development.

EXPANSION ALTERNATIVES

The existing Peace River system can supply an average daily demand of at least 6.5 mgd. An additional 8 mgd of capacity is needed to meet the most likely demand of 14.5 mgd for the year 2000. Total annual costs for a 14.5-mgd system can be compared for each of the four sources, as shown below, to estimate the best expansion alternative based on cost:

System	Total Annual Cost (million $)
14.5 mgd Peace River	4.3
6.5 mgd Peace River + 8 mgd Myakkahatchee Creek	6.2
6.5 mgd Peace River + 8 mgd Well Field A	5.6
8.5 mgd Peace River + 6 mgd Well Field B	6.4

Besides costs, which are an important aspect in providing service to customers, technical and engineering considerations also indicate that the Peace River is the preferred source for future development. As determined in Section 3, the Peace River by itself has the capacity to meet Port Charlotte service area needs through the year 2000. Reliance on the Peace River would produce the facilities expansion path presented in Table 7-1. In addition, no

Table 7-3
BRACKISH GROUNDWATER DESALTING
FACILITIES REQUIREMENTS AND COSTS

Site	Raw Water TDS (mg/l)	Average Daily Yield (mgd)	Facilities Required: Raw Water Feed (mgd)	Facilities Required: Number of Reverse Osmosis Trains	Facilities Required: Number of Wells	Cumulative Capital Cost (Million $)	Direct Annual O&M Cost (Million $)	Equivalent Annual Cost (Million $)	Production Cost ($/1,000 gal)
Well Field A	3,000	1.9	2.6	3	3	5.4	0.7	1.2	1.84
	3,000	5.2	6.9	8	8	11.4	1.6	2.8	1.55
	3,000	10.3	13.8	16	16	20.7	3.1	5.2	1.44
Well Field B	6,000	2.3	3.0	3	6	6.3	0.8	1.6	2.06
	6,000	5.2	6.9	7	14	12.6	1.6	3.3	1.83
	6,000	10.3	13.8	14	28	23.4	3.8	6.2	1.74

NOTES:

1. Wells located in areas with 3000 mg/l TDS have 600 gpm capacity each.

2. Wells located in areas with 6000 mg/l TDS have 350 gpm capacity each.

3. Total annual cost as a function of yield can be estimated as:

   3,000 mg/l: Total annual cost = 0.69 X $(\text{Yield})^{0.855}$ R-squared = 0.999
   6,000 mg/l: Total annual cost = 0.78 X $(\text{Yield})^{0.887}$ R-squared = 0.9996
   where cost is in millions of 1987 dollars and yield is in mgd.
   An R-squared value of 0.999 indicates that 99.9 percent of the variance in total annual cost can be explained by the variance in yield.

4. Maximum yields are approximately 10 mgd from 3000 mg/l TDS source and 6 mgd from 6,000 mg/l TDS source.

5. All costs are in March 1987 dollars.

6. Costs are order-of-magnitude estimates made without detailed engineering data. It is normally expected that estimates of this type are accurate within -30% to +50%.

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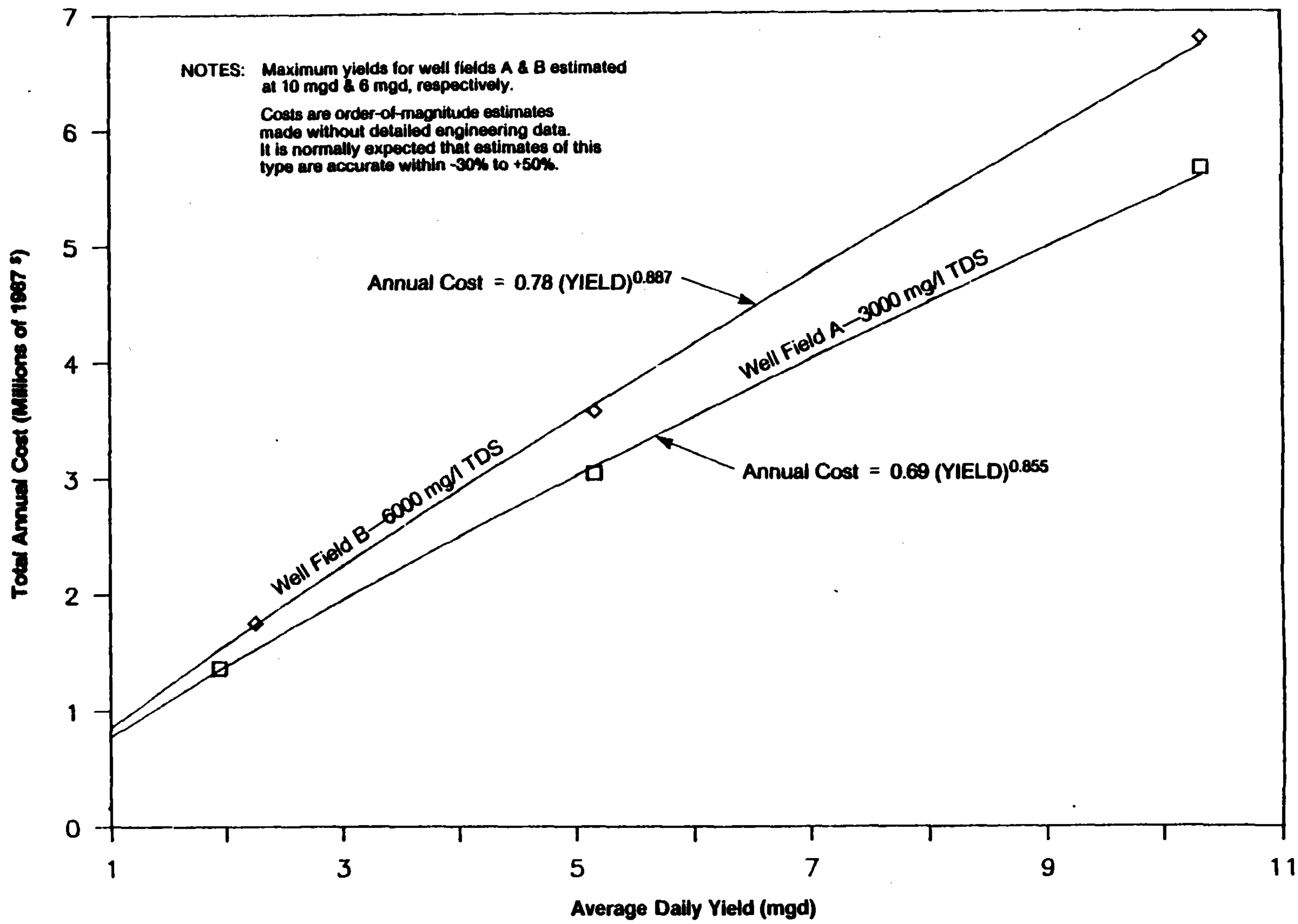

FIGURE 7-4.
Total Annual Cost for Groundwater Desalination.

groundwater sources would need to be developed. The existing North Port WTP would be gradually phased out of service, on an appropriate time table. This could be relatively soon, as the recent expansion of the Peace River WTP from 6 to 12 mgd and the ongoing ASR system expansion to 5 mgd would allow these facilities to easily accommodate current finished water production of the North Port plant.

DISCUSSION

Based on costs, technical and resource management considerations, and engineering judgment, the best alternative for meeting water supply needs through the year 2000 in the Port Charlotte service area is the use of the Peace River source alone. The North Port site would be phased out, but has potential for development as a major finished water storage and distribution center. This possibility should be considered by GDU when evaluating future distribution system needs. Table 7-4 presents the expansion path and associated costs for development of the Peace River, based on the most likely growth scenario.

The difference between the two demand scenarios presented in Section 2 (see Figures 2-8 and 2-9) defines the boundaries for expansion requirements. The expansion path developed for the most likely scenario (Table 7-4) was modified to estimate facilities requirements based on the minimum demand scenario. Table 7-5 summarizes the facilities requirements for each demand scenario and Table 7-6 summarizes the cost differences. Each of these analyses is based on development of the Peace River alone.

SUMMARY

The Peace River is the best overall raw water supply source for the Port Charlotte service area. If the service area demand follows the expected growth scenario, only one 6-mgd plant expansion and 12 mgd of additional ASR capacity would need to be constructed through the year 2000. Based on the results of this study, further development of the North Port WTP is not recommended and the Myakkahatchee Creek water supply source should be phased out.

Selection of the recommended alternative is based on monthly systems simulation, resource management considerations, planning level cost estimates, operational benefits, and current engineering and water treatment technology. The costs and feasibility of brackish groundwater development should be examined in more detail in the future, when average daily demand is within five years of reaching 18 mgd. This analysis should include systems simulation on a daily time step, revised desalting cost estimates, and both short- and long-term impacts on water distribution costs. The existing distribution system computer analyses (not part of the current project) can be updated and used at the time of the future analysis.

Table 7-4
PORT CHARLOTTE SERVICE AREA
EXPANSION PATH AND COST ESTIMATES FOR THE
PEACE RIVER

	Demand (mgd)		Peace River Facility Capacity (mgd)					Capital Cost (Million $)		Direct Annual O&M Cost	Equivalent Annual Cost	Production Cost
Year	Ave Day	Max Day	Intake	WTP	ASR	Surface Reservoir PS	Maximum Delivery	Total	Cum. Total	(Million $)	(Million $)	($/1,000 gal)
1987	6.5	10.4	22	12	5	24	17	0.0	0.0	1.5	1.5	0.64
1990	8.2	13.1	22	12	8	24	20	0.9	0.9	1.9	2.0	0.67
1993	10.1	16.2	22	12	11	24	23	0.9	1.9	2.4	2.6	0.69
1996	12.0	19.2	22	18	14	24	32	5.8	7.6	2.8	3.6	0.82
2000	14.5	23.2	22	18	17	24	35	0.9	8.6	3.4	4.3	0.80

NOTES:

1. The existing Peace River WTP surface reservoir remains through the year 2000.
2. The schedule shown is based on the most likely demand scenario.
3. All costs are in March 1987 dollars.
4. Costs are order-of-magnitude estimates made without detailed engineering data. It is normally expected that estimates of this type are accurate within -30% to +50%.

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Table 7-5
PORT CHARLOTTE SERVICE AREA FACILITIES EXPANSION SCHEDULE
UNDER DIFFERENT DEMAND SCENARIOS

	Minimum Demand Scenario				Expected Demand Scenario			
	Demand (mgd)		Facility Capacity (mgd)		Demand (mgd)		Facility Capacity (mgd)	
Year	Avg Day	Max Day	Peace River WTP	Peace River ASR	Avg Day	Max Day	Peace River WTP	Peace River ASR
1987	6.4	10.2	12	5	6.5	10.4	12	5
1989	7.2	11.5			7.6	12.2		
1990	7.7	12.3			8.2	13.1	12	8
1991	8.1	13.0	12	8	8.8	14.1		
1993	9.0	14.4			10.1	16.2	12	11
1995	9.9	15.8	12	11	11.4	18.2		
1996	10.3	16.5			12.0	19.2	18	14
1998	11.2	17.9			13.3	21.2		
2000	12.0	19.2	18	14	14.5	23.2	18	17

1. Existing river intake structure, surface reservoir, and reservoir pump station are adequate through the year 2000 for all demand scenarios.

Table 7-6
PORT CHARLOTTE SERVICE AREA COST ESTIMATES
UNDER DIFFERENT DEMAND SCENARIOS

Minimum Demand Scenario

Year	Demand (mgd) Avg Day	Demand (mgd) Max Day	Cum. Capital Cost (Million $)	Direct Annual O&M (Million $)	Equivalent Annual Cost (Million $)	Production Cost ($/1,000 gal)
1987	6.4	10.2	0.0	1.5	1.5	0.64
1989	7.2	11.5	0.0	1.7	1.7	0.64
1990	7.7	12.3	0.0	1.8	1.8	0.64
1991	8.1	13.0	0.9	1.9	2.0	0.67
1993	9.0	14.4	0.9	2.1	2.2	0.67
1995	9.9	15.8	1.9	2.3	2.5	0.69
1996	10.3	16.5	1.9	2.4	2.6	0.69
1998	11.2	17.9	1.9	2.6	2.8	0.68
2000	12.0	19.2	7.6	2.8	3.6	0.82

Expected Demand Scenario

Year	Demand (mgd) Avg Day	Demand (mgd) Max Day	Cum. Capital Cost (Million $)	Direct Annual O&M (Million $)	Equivalent Annual Cost (Million $)	Production Cost ($/1,000 gal)
1987	6.5	10.4	0.0	1.5	1.5	0.64
1989	7.6	12.2	0.0	1.8	1.8	0.64
1990	8.2	13.1	0.9	1.9	2.0	0.67
1991	8.8	14.1	0.9	2.1	2.2	0.67
1993	10.1	16.2	1.9	2.4	2.5	0.69
1995	11.4	18.2	1.9	2.7	2.8	0.68
1996	12.0	19.2	7.6	2.8	3.6	0.82
1998	13.3	21.2	7.6	3.1	3.9	0.80
2000	14.5	23.2	8.6	3.4	4.2	0.80

NOTES:

1. All costs are in March 1987 dollars.

2. Costs are order-of-magnitude estimates made without detailed engineering data. It is normally expected that estimates of this type are accurate within -30% to +50%.

SECTION 8
References

Section 8
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APPENDIXES

Appendix A
EXTENDED FLOW AND QUALITY RECORDS

Appendix A
EXTENDED FLOW AND QUALITY RECORDS

Terms appearing in this appendix are defined below.

MONTH - Month of year (10 = October)

HCQ - Monthly flow at Horse Creek--cfs

Q101 - Monthly flow for Myakkahatchee Creek--cfs

Q106 - Monthly flow for Cocoplum Waterway--cfs

TDS101 - TDS concentration for Myakkahatchee Creek--mg/l

TDS106 - TDS concentration for Cocoplum Waterway--mg/l

QSUM - Sum of monthly flows (Q101 + Q106)

TDSAVG - Flow weighted average TDS concentration for Myakkahatchee Creek and Cocoplum Waterway--mg/l

CUM101 - Cumulative flow (for each year) at Myakkahatchee Creek--1,000 acre feet

CUM106 - Cumulative flow (for each year) at Cocoplum Waterway--1,000 acre feet

CUMSUM - Total cumulative flow, for each year (CUM101 + CUM106)--1,000 acre feet

101<400 - Cumulative flow (for each year) for all flows with TDS less than 400 mg/l at Myakkahatchee Creek--1,000 acre feet

SUMMARY OF DATA FOR WATER YEAR 1951

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	169.	81.	48.	220.	505.	129.	326.	4.89	2.88	7.77	4.89
11	29.	10.	12.	389.	664.	23.	540.	5.50	3.63	9.13	5.50
12	24.	8.	11.	415.	679.	19.	571.	5.96	4.29	10.26	5.50
1	14.	3.	9.	530.	714.	11.	671.	6.12	4.81	10.93	5.50
2	29.	10.	12.	389.	664.	23.	540.	6.73	5.55	12.29	6.12
3	6.	1.	7.	741.	753.	7.	752.	6.77	5.95	12.71	6.12
4	183.	88.	51.	214.	498.	139.	318.	12.08	9.04	21.12	11.43
5	16.	4.	9.	492.	706.	13.	645.	12.30	9.59	21.88	11.43
6	2.	1.	5.	741.	782.	6.	778.	12.33	9.91	22.24	11.43
7	463.	230.	122.	137.	410.	352.	232.	26.19	17.28	43.47	25.29
8	319.	157.	86.	168.	445.	243.	266.	35.66	22.45	58.11	34.76
9	93.	43.	29.	270.	562.	71.	387.	38.23	24.17	62.40	37.33
AVERAGE	112.	53.	33.	191.	498.	86.	310.				

SUMMARY OF DATA FOR WATER YEAR 1952

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	640.	319.	167.	110.	380.	486.	203.	19.27	10.07	29.34	19.27
11	41.	16.	15.	348.	636.	32.	488.	20.25	11.00	31.24	20.25
12	18.	5.	10.	466.	699.	14.	623.	20.52	11.57	32.10	20.25
1	11.	1.	8.	644.	727.	9.	717.	20.59	12.04	32.63	20.25
2	55.	23.	19.	318.	610.	42.	449.	22.00	13.18	35.18	21.65
3	96.	44.	29.	267.	559.	73.	384.	24.65	14.95	39.61	24.31
4	41.	16.	15.	348.	636.	32.	488.	25.63	15.88	41.51	25.29
5	3.	1.	6.	741.	775.	6.	772.	25.67	16.22	41.89	25.29
6	5.	1.	6.	741.	759.	7.	757.	25.70	16.60	42.30	25.29
7	58.	25.	20.	312.	605.	45.	442.	27.20	17.78	44.99	26.79
8	272.	133.	74.	181.	460.	207.	281.	35.23	22.24	57.47	34.82
9	214.	10[illegible].	59.	201.	483.	163.	303.	41.49	25.81	67.30	41.08
AVERAGE	121.	57.	36.	177.	484.	93.	295.				

SUMMARY OF DATA FOR WATER YEAR 1953

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	1335.	671.	343.	52.	311.	1014.	140.	40.48	20.68	61.16	40.48
11	297.	146.	80.	174.	452.	226.	273.	49.28	25.51	74.79	49.28
12	81.	36.	25.	282.	575.	62.	402.	51.48	27.05	78.53	51.48
1	163.	78.	46.	223.	509.	124.	329.	56.18	29.84	86.02	56.18
2	238.	116.	65.	192.	473.	181.	293.	63.18	33.78	96.95	63.18
3	24.	8.	11.	415.	679.	19.	571.	63.64	34.44	98.08	63.18
4	73.	32.	23.	291.	584.	56.	414.	65.59	35.86	101.45	65.13
5	5.	1.	6.	741.	761.	7.	759.	65.63	36.23	101.86	65.13
6	706.	353.	184.	102.	371.	536.	194.	86.91	47.31	134.22	86.41
7	387.	191.	103.	152.	427.	294.	248.	98.45	53.52	151.97	97.96
8	905.	453.	234.	81.	348.	687.	172.	125.81	67.63	193.44	125.31
9	1427.	718.	366.	47.	305.	1084.	134.	169.10	89.72	258.81	168.60
AVERAGE	470.	234.	124.	92.	365.	357.	187.				

SUMMARY OF DATA FOR WATER YEAR 1954

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	1026.	515.	265.	71.	336.	779.	161.	31.05	15.96	47.01	31.05
11	431.	214.	114.	143.	417.	328.	238.	43.94	22.84	66.78	43.94
12	348.	172.	93.	161.	437.	265.	258.	54.29	28.46	82.74	54.29
1	183.	88.	51.	214.	498.	139.	318.	59.60	31.55	91.15	59.60
2	45.	18.	16.	338.	628.	35.	475.	60.70	32.54	93.24	60.70
3	45.	18.	16.	338.	628.	35.	475.	61.81	33.53	95.33	61.81
4	99.	46.	30.	265.	556.	76.	380.	64.56	35.34	99.90	64.56
5	155.	74.	44.	227.	513.	118.	334.	69.02	38.01	107.03	69.02
6	199.	96.	55.	207.	490.	152.	310.	74.82	41.35	116.17	74.82
7	471.	234.	124.	135.	409.	358.	230.	88.93	48.84	137.77	88.93
8	262.	128.	71.	184.	464.	199.	284.	96.65	53.14	149.79	96.65
9	345.	170.	92.	161.	438.	262.	259.	106.91	58.71	165.62	106.91
AVERAGE	301.	148.	81.	145.	425.	229.	245.				

SUMMARY OF DATA FOR WATER YEAR 1955

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	199.	96.	55.	207.	490.	152.	310.	5.80	3.34	9.14	5.80
11	64.	28.	21.	303.	597.	49.	430.	7.49	4.62	12.10	7.49
12	66.	29.	22.	300.	594.	51.	426.	9.23	5.93	15.16	9.23
1	41.	16.	15.	348.	636.	32.	488.	10.21	6.85	17.06	10.21
2	74.	33.	24.	290.	583.	57.	413.	12.20	8.29	20.48	12.20
3	16.	4.	9.	492.	706.	13.	645.	12.41	8.83	21.24	12.20
4	16.	4.	9.	492.	706.	13.	645.	12.63	9.38	22.01	12.20
5	2.	1.	6.	741.	778.	6.	775.	12.66	9.71	22.37	12.20
6	3.	1.	6.	741.	771.	6.	769.	12.70	10.06	22.75	12.20
7	10.	1.	8.	741.	732.	8.	733.	12.73	10.51	23.24	12.20
8	315.	155.	85.	169.	446.	240.	267.	22.08	15.62	37.70	21.54
9	701.	350.	182.	102.	372.	533.	195.	43.20	26.62	69.83	42.67
AVERAGE	126.	60.	37.	166.	473.	96.	283.				

SUMMARY OF DATA FOR WATER YEAR 1956

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	71.	31.	23.	294.	587.	54.	418.	1.90	1.39	3.28	1.90
11	5.	1.	6.	741.	759.	7.	757.	1.93	1.76	3.69	1.90
12	4.	1.	6.	741.	765.	7.	763.	1.96	2.13	4.09	1.90
1	4.	.	6.	741.	765.	7.	763.	2.00	2.49	4.49	1.90
2	13.	2.	8.	572.	720.	10.	693.	2.11	2.98	5.09	1.90
3	3.	1.	6.	741.	771.	6.	769.	2.14	3.33	5.47	1.90
4	1.	1.	5.	741.	788.	6.	784.	2.17	3.64	5.81	1.90
5	3.	1.	6.	741.	769.	6.	766.	2.21	3.99	6.20	1.90
6	0.	1.	5.	741.	791.	6.	786.	2.24	4.30	6.54	1.90
7	2.	1.	6.	741.	776.	6.	773.	2.28	4.63	6.91	1.90
8	66.	29.	22.	300.	594.	51.	426.	4.02	5.94	9.96	3.64
9	292.	143.	79.	175.	454.	222.	274.	12.66	10.70	23.36	12.28
AVERAGE	39.	17.	15.	226.	582.	32.	389.				

SUMMARY OF DATA FOR WATER YEAR 1957

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	247.	120.	67.	189.	469.	188.	290.	7.27	4.07	11.34	7.27
11	17.	4.	9.	478.	702.	13.	634.	7.52	4.63	12.15	7.27
12	2.	1.	6.	741.	776.	6.	773.	7.55	4.97	12.52	7.27
1	5.	1.	6.	741.	761.	7.	759.	7.58	5.34	12.93	7.27
2	12.	1.	8.	600.	724.	9.	704.	7.67	5.82	13.50	7.27
3	200.	97.	56.	206.	489.	152.	310.	13.51	9.18	22.68	13.10
4	94.	43.	29.	269.	561.	72.	386.	16.10	10.91	27.02	15.70
5	338.	167.	91.	163.	440.	257.	261.	26.15	16.37	42.53	25.75
6	148.	70.	42.	231.	518.	113.	339.	30.40	18.94	49.33	29.99
7	358.	177.	96.	158.	434.	272.	255.	41.05	24.70	65.76	40.65
8	612.	305.	160.	114.	385.	465.	207.	59.47	34.34	93.81	59.06
9	650.	324.	169.	108.	379.	494.	201.	79.04	44.57	123.60	78.63
AVERAGE	224.	109.	62.	152.	440.	171.	256.				

SUMMARY OF DATA FOR WATER YEAR 1958

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	639.	319.	167.	110.	380.	485.	203.	19.24	10.06	29.29	19.24
11	20.	6.	10.	445.	692.	16.	603.	19.57	10.66	30.24	19.24
12	66.	29.	22.	300.	594.	51.	426.	21.32	11.97	33.29	20.98
1	682.	341.	178.	104.	374.	518.	197.	41.87	22.68	64.55	41.53
2	334.	165.	90.	164.	441.	254.	262.	51.79	28.08	79.87	51.45
3	789.	395.	205.	92.	361.	599.	184.	75.60	40.43	116.03	75.27
4	330.	162.	88.	165.	442.	251.	263.	85.41	45.77	131.17	85.07
5	137.	65.	40.	237.	525.	104.	347.	89.32	48.16	137.48	88.98
6	44.	18.	16.	340.	630.	34.	478.	90.39	49.13	139.52	90.05
7	346.	171.	93.	161.	438.	263.	258.	100.68	54.71	155.39	100.34
8	187.	90.	52.	212.	496.	142.	316.	106.12	57.87	163.99	105.78
9	42.	17.	16.	345.	634.	32.	484.	107.13	58.81	165.94	106.79
AVERAGE	301.	148.	81.	139.	420.	229.	238.				

SUMMARY OF DATA FOR WATER YEAR 1959

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	33.	12.	13.	372.	654.	26.	520.	0.74	0.81	1.54	0.74
11	90.	41.	28.	273.	565.	69.	391.	3.20	2.48	5.68	3.20
12	62.	27.	21.	306.	599.	48.	434.	4.82	3.72	8.55	4.82
1	97.	45.	30.	266.	558.	74.	382.	7.51	5.51	13.02	7.51
2	55.	23.	19.	318.	610.	42.	449.	8.92	6.65	15.57	8.92
3	712.	356.	185.	101.	370.	541.	193.	30.38	17.82	48.20	30.38
4	133.	63.	39.	240.	528.	101.	350.	34.17	20.15	54.32	34.17
5	22.	7.	11.	429.	685.	17.	586.	34.57	20.79	55.36	34.17
6	691.	345.	180.	103.	373.	525.	196.	55.40	31.63	87.03	55.00
7	820.	410.	212.	89.	357.	623.	181.	80.16	44.45	124.61	79.76
8	1138.	571.	293.	64.	326.	864.	153.	114.63	62.12	176.75	114.23
9	1616.	813.	414.	39.	292.	1227.	125.	163.69	87.09	250.78	163.29
AVERAGE	456.	226.	120.	88.	361.	346.	182.				

SUMMARY OF DATA FOR WATER YEAR 1960

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	495.	246.	130.	131.	404.	376.	226.	14.84	7.86	22.70	14.84
11	88.	40.	27.	275.	567.	67.	393.	17.25	9.50	26.76	17.25
12	41.	16.	15.	348.	636.	32.	488.	18.23	10.43	28.66	18.23
1	28.	10.	12.	393.	667.	22.	545.	18.82	11.16	29.98	18.82
2	181.	87.	51.	214.	499.	138.	319.	24.07	14.22	38.29	24.07
3	304.	149.	82.	172.	450.	231.	270.	33.08	19.16	52.24	33.08
4	103.	48.	31.	261.	552.	79.	376.	35.95	21.04	56.99	35.95
5	34.	13.	14.	369.	652.	26.	515.	36.72	21.86	58.58	36.72
6	41.	16.	15.	348.	636.	32.	488.	37.70	22.79	60.49	37.70
7	855.	428.	221.	86.	353.	649.	177.	63.53	36.14	99.67	63.53
8	1571.	790.	402.	41.	295.	1193.	127.	111.22	60.42	171.63	111.22
9	1696.	854.	434.	36.	288.	1288.	121.	162.72	86.61	249.33	162.72
AVERAGE	453.	225.	120.	82.	352.	344.	176.				

SUMMARY OF DATA FOR WATER YEAR 1961

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	680.	340.	177.	105.	375.	517.	197.	20.49	10.68	31.17	20.49
11	120.	56.	35.	248.	538.	92.	360.	23.88	12.81	36.69	23.88
12	21.	6.	10.	437.	688.	16.	594.	24.25	13.44	37.68	23.88
1	50.	21.	18.	327.	619.	38.	461.	25.50	14.50	40.00	25.13
2	177.	85.	50.	216.	501.	135.	321.	30.63	17.50	48.14	30.27
3	45.	18.	16.	338.	628.	35.	475.	31.74	18.49	50.23	31.37
4	44.	18.	16.	340.	630.	34.	478.	32.81	19.47	52.27	32.44
5	4.	1.	6.	741.	766.	7.	764.	32.84	19.83	52.67	32.44
6	5.	1.	6.	741.	761.	7.	759.	32.88	20.20	53.08	32.44
7	138.	65.	40.	237.	524.	105.	346.	36.82	22.61	59.42	36.38
8	275.	135.	75.	180.	459.	209.	280.	44.94	27.11	72.05	44.50
9	264.	129.	72.	184.	463.	201.	283.	52.73	31.44	84.17	52.29
AVERAGE	152.	73.	43.	176.	473.	116.	287.				

SUMMARY OF DATA FOR WATER YEAR 1962

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	8.	1.	7.	741.	743.	8.	743.	0.03	0.42	0.45	0.00
11	3.	1.	6.	741.	774.	6.	771.	0.07	0.76	0.83	0.00
12	3.	1.	6.	741.	772.	6.	769.	0.10	1.11	1.21	0.00
1	4.	1.	6.	741.	762.	7.	761.	0.13	1.48	1.61	0.00
2	4.	1.	6.	741.	767.	6.	765.	0.17	1.83	2.00	0.00
3	4.	1.	6.	741.	764.	7.	762.	0.20	2.20	2.40	0.00
4	39.	15.	15.	354.	641.	30.	496.	1.11	3.09	4.20	0.91
5	2.	1.	5.	741.	781.	6.	777.	1.14	3.42	4.56	0.91
6	186.	90.	52.	212.	496.	142.	317.	6.55	6.56	13.11	6.31
7	236.	115.	65.	193.	474.	180.	294.	13.48	10.46	23.94	13.25
8	392.	194.	104.	151.	426.	298.	247.	25.18	16.75	41.92	24.94
9	1531.	770.	392.	43.	298.	1163.	129.	71.64	40.42	112.06	71.41
AVERAGE	201.	99.	56.	94.	387.	155.	200.				

SUMMARY OF DATA FOR WATER YEAR 1963

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	192.	93.	54.	210.	493.	146.	313.	5.59	3.23	8.82	5.59
11	62.	27.	21.	306.	599.	48.	434.	7.21	4.48	11.69	7.21
12	18.	5.	10.	466.	699.	14.	623.	7.49	5.06	12.55	7.21
1	20.	6.	10.	445.	692.	16.	603.	7.83	5.66	13.49	7.21
2	220.	107.	61.	199.	480.	167.	301.	14.27	9.32	23.60	13.66
3	232.	113.	64.	194.	475.	177.	296.	21.08	13.17	34.25	20.47
4	6.	1.	7.	741.	753.	7.	752.	21.12	13.56	34.68	20.47
5	5.	1.	6.	741.	761.	7.	759.	21.15	13.93	35.08	20.47
6	38.	14.	14.	357.	643.	29.	501.	22.02	14.81	36.83	21.34
7	143.	68.	41.	234.	521.	109.	342.	26.12	17.29	43.41	25.43
8	338.	167.	91.	163.	440.	257.	261.	36.16	22.75	58.91	35.48
9	457.	227.	121.	138.	412.	347.	233.	49.84	30.03	79.87	49.16
AVERAGE	144.	69.	41.	189.	486.	110.	300.				

SUMMARY OF DATA FOR WATER YEAR 1964

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	125.	59.	37.	245.	534.	95.	356.	3.54	2.21	5.75	3.54
11	171.	82.	48.	219.	504.	130.	325.	8.49	5.12	13.61	8.49
12	66.	29.	22.	300.	594.	51.	426.	10.24	6.43	16.67	10.24
1	214.	104.	59.	201.	483.	163.	303.	16.50	10.00	26.50	16.50
2	337.	166.	90.	163.	440.	256.	261.	26.51	15.44	41.96	26.51
3	155.	74.	44.	227.	513.	118.	334.	30.97	18.11	49.09	30.97
4	122.	57.	36.	247.	536.	93.	358.	34.43	20.28	54.70	34.43
5	14.	2.	8.	537.	715.	11.	676.	34.57	20.79	55.36	34.43
6	13.	2.	8.	572.	720.	10.	693.	34.68	21.28	55.96	34.43
7	43.	17.	16.	342.	632.	33.	481.	35.72	22.24	57.96	35.47
8	179.	86.	50.	215.	500.	136.	320.	40.92	25.27	66.19	40.66
9	363.	179.	97.	157.	433.	276.	254.	51.73	31.11	82.84	51.47
AVERAGE	150.	71.	43.	204.	496.	114.	314.				

SUMMARY OF DATA FOR WATER YEAR 1965

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	27.	9.	12.	398.	670.	21.	551.	0.56	0.71	1.27	0.55
11	6.	1.	7.	741.	753.	7.	752.	0.59	1.11	1.69	0.55
12	6.	1.	7.	741.	751.	7.	750.	0.62	1.50	2.12	0.55
1	7.	1.	7.	741.	746.	7.	746.	0.65	1.92	2.57	0.55
2	17.	4.	9.	485.	704.	13.	639.	0.89	2.47	3.36	0.55
3	95.	44.	29.	268.	560.	73.	385.	3.51	4.22	7.74	3.18
4	6.	1.	6.	741.	755.	7.	754.	3.55	4.61	8.16	3.18
5	1.	1.	5.	741.	788.	6.	784.	3.58	4.92	8.50	3.18
6	140.	66.	40.	236.	523.	107.	344.	7.58	7.36	14.94	7.18
7	10[illegible]1.	503.	259.	73.	338.	762.	163.	37.93	22.97	60.90	37.53
8	832.	416.	215.	88.	356.	632.	179.	63.06	35.97	99.03	62.66
9	103.	48.	31.	261.	552.	79.	376.	65.93	37.85	103.78	65.53
AVERAGE	187.	91.	52.	111.	410.	143.	220.				

SUMMARY OF DATA FOR WATER YEAR 1966

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	499.	248.	131.	131.	403.	379.	225.	14.96	7.92	22.88	14.96
11	42.	17.	16.	345.	634.	32.	484.	15.97	8.86	24.83	15.97
12	21.	6.	10.	437.	688.	16.	594.	16.34	9.48	25.82	15.97
1	107.	50.	32.	258.	549.	82.	372.	19.34	11.42	30.75	18.97
2	363.	179.	97.	157.	433.	276.	254.	30.15	17.26	47.41	29.78
3	137.	65.	40.	237.	525.	104.	347.	34.06	19.65	53.71	33.69
4	39.	15.	15.	354.	641.	30.	497.	34.96	20.54	55.50	34.59
5	11.	1.	8.	644.	727.	9.	717.	35.03	21.01	56.04	34.59
6	552.	275.	145.	122.	394.	419.	216.	51.60	29.74	81.34	51.17
7	230.	112.	63.	195.	476.	175.	296.	58.35	33.65	91.91	57.92
8	345.	170.	92.	161.	438.	262.	259.	68.61	39.12	107.73	68.18
9	214.	104.	59.	201.	483.	163.	303.	74.88	42.69	117.56	74.44
AVERAGE	213.	103.	59.	167.	454.	162.	271.				

SUMMARY OF DATA FOR WATER YEAR 1967

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	314.	154.	84.	169.	447.	239.	267.	9.31	5.09	14.41	9.31
11	15.	3.	9.	509.	710.	12.	658.	9.50	5.63	15.13	9.31
12	6.	1.	7.	741.	753.	7.	752.	9.53	6.02	15.55	9.31
1	8.	1.	7.	741.	742.	8.	742.	9.57	6.44	16.01	9.31
2	23.	7.	11.	422.	682.	18.	578.	10.00	7.09	17.09	9.31
3	11.	1.	8.	644.	727.	9.	717.	10.06	7.56	17.63	9.31
4	1.	1.	5.	741.	785.	6.	781.	10.10	7.88	17.98	9.31
5	0.	1.	5.	741.	791.	6.	786.	10.13	8.19	18.32	9.31
6	154.	73.	44.	228.	514.	117.	335.	14.56	10.84	25.40	13.74
7	190.	92.	53.	211.	494.	145.	315.	20.09	14.04	34.13	19.27
8	868.	435.	225.	85.	352.	659.	176.	46.31	27.59	73.91	45.50
9	392.	194.	104.	151.	426.	298.	247.	58.01	33.88	91.89	57.19
AVERAGE	165.	80.	47.	141.	441.	127.	251.				

SUMMARY OF DATA FOR WATER YEAR 1968

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	215.	104.	59.	200.	482.	164.	303.	6.29	3.58	9.88	6.29
11	13.	2.	8.	572.	720.	10.	693.	6.40	4.08	10.48	6.29
12	18.	4.	9.	472.	700.	14.	628.	6.66	4.64	11.31	6.29
1	12.	2.	8.	591.	723.	10.	701.	6.76	5.13	11.89	6.29
2	7.	1.	7.	741.	748.	7.	748.	6.79	5.53	12.33	6.29
3	12.	2.	8.	591.	723.	10.	701.	6.89	6.02	12.91	6.29
4	2.	1.	5.	741.	779.	6.	776.	6.92	6.35	13.27	6.29
5	10.	1.	7.	741.	734.	8.	735.	6.96	6.80	13.75	6.29
6	543.	270.	142.	124.	396.	413.	217.	23.26	15.39	38.65	22.60
7	1742.	877.	446.	35.	285.	1323.	119.	76.17	42.28	118.44	75.50
8	234.	114.	64.	193.	474.	178.	295.	83.04	46.15	129.19	82.38
9	600.	299.	157.	115.	386.	456.	208.	101.08	55.61	156.69	100.42
AVERAGE	284.	140.	77.	88.	371.	216.	188.				

SUMMARY OF DATA FOR WATER YEAR 1969

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	158.	75.	45.	226.	512.	120.	332.	4.56	2.71	7.27	4.56
11	201.	97.	56.	206.	489.	153.	309.	10.42	6.08	16.50	10.42
12	25.	8.	11.	409.	676.	19.	564.	10.91	6.77	17.67	10.42
1	66.	29.	22.	300.	594.	51.	426.	12.66	8.08	20.73	12.16
2	40.	16.	15.	350.	638.	31.	491.	13.60	8.99	22.59	13.11
3	428.	212.	113.	144.	418.	325.	239.	26.39	15.82	42.22	25.90
4	52.	22.	18.	323.	615.	40.	456.	27.71	16.92	44.63	27.22
5	7.	1.	7.	741.	749.	7.	749.	27.74	17.32	45.06	27.22
6	451.	224.	119.	139.	413.	343.	234.	41.24	24.51	65.75	40.72
7	301.	148.	81.	173.	451.	229.	271.	50.16	29.40	79.56	49.63
8	441.	219.	117.	141.	415.	335.	236.	63.35	36.43	99.78	62.82
9	446.	221.	118.	140.	414.	339.	235.	76.69	43.54	120.24	76.17
AVERAGE	218.	106.	60.	166.	453.	166.	270.				

SUMMARY OF DATA FOR WATER YEAR 1970

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	503.	250.	132.	130.	403.	382.	224.	15.08	7.98	23.06	15.08
11	174.	84.	49.	218.	502.	133.	323.	20.12	10.94	31.06	20.12
12	109.	51.	33.	256.	547.	83.	370.	23.18	12.90	36.08	23.18
1	276.	135.	75.	180.	459.	210.	279.	31.33	17.42	48.75	31.33
2	97.	45.	30.	266.	558.	74.	382.	34.02	19.20	53.22	34.02
3	491.	244.	129.	132.	405.	373.	227.	48.74	26.99	75.74	48.74
4	155.	74.	44.	227.	513.	118.	334.	53.20	29.66	82.86	53.20
5	15.	3.	9.	509.	710.	12.	658.	53.39	30.19	83.58	53.20
6	229.	111.	63.	195.	476.	174.	297.	60.11	33.99	94.10	59.92
7	190.	92.	53.	211.	494.	145.	315.	65.64	37.19	102.83	65.45
8	331.	163.	89.	165.	442.	252.	263.	75.47	42.55	118.01	75.28
9	340.	168.	91.	163.	439.	259.	260.	85.58	48.04	133.61	85.39
AVERAGE	243.	118.	66.	173.	456.	185.	275.				

SUMMARY OF DATA FOR WATER YEAR 1971

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	134.	63.	39.	239.	527.	102.	349.	3.82	2.35	6.17	3.82
11	16.	4.	9.	492.	706.	13.	645.	4.04	2.89	6.93	3.82
12	7.	1.	7.	741.	748.	7.	748.	4.07	3.30	7.37	3.82
1	6.	1.	6.	741.	756.	7.	755.	4.10	3.68	7.79	3.82
2	27.	9.	12.	396.	669.	21.	549.	4.67	4.40	9.07	4.38
3	7.	1.	7.	741.	745.	7.	745.	4.70	4.82	9.52	4.38
4	2.	1.	5.	741.	782.	6.	778.	4.74	5.14	9.88	4.38
5	0.	1.	5.	741.	790.	6.	785.	4.77	5.45	10.22	4.38
6	11.	1.	8.	674.	729.	9.	724.	4.82	5.91	10.74	4.38
7	56.	24.	19.	317.	609.	43.	447.	6.24	7.06	13.31	5.81
8	178.	86.	50.	216.	500.	136.	321.	11.41	10.08	21.49	10.97
9	483.	240.	127.	133.	407.	367.	228.	25.88	17.76	43.64	25.44
AVERAGE	77.	36.	25.	189.	517.	60.	322.				

SUMMARY OF DATA FOR WATER YEAR 1972

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	367.	181.	98.	156.	432.	279.	253.	10.93	5.90	16.84	10.93
11	124.	58.	36.	246.	535.	95.	357.	14.45	8.10	22.54	14.45
12	137.	65.	40.	237.	525.	104.	347.	18.36	10.49	28.85	18.36
1	17.	4.	9.	484.	704.	13.	638.	18.59	11.05	29.64	18.36
2	125.	59.	37.	245.	534.	95.	356.	22.14	13.26	35.39	21.90
3	26.	9.	12.	404.	673.	20.	558.	22.66	13.95	36.61	21.90
4	57.	25.	20.	313.	606.	44.	443.	24.14	15.13	39.27	23.38
5	13.	2.	8.	556.	718.	10.	686.	24.26	15.63	39.90	23.38
6	185.	89.	52.	213.	497.	141.	317.	29.64	18.76	48.40	28.76
7	69.	30.	22.	296.	590.	53.	421.	31.48	20.11	51.59	30.59
8	273.	134.	74.	181.	460.	208.	280.	39.54	24.58	64.12	38.65
9	423.	210.	112.	144.	419.	322.	240.	52.18	31.34	83.52	51.30
AVERAGE	151.	72.	43.	195.	489.	115.	306.				

SUMMARY OF DATA FOR WATER YEAR 1973

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	79.	35.	25.	284.	577.	60.	405.	2.14	1.51	3.65	2.14
11	29.	10.	12.	389.	664.	23.	540.	2.75	2.25	5.01	2.75
12	96.	44.	29.	267.	559.	73.	384.	5.41	4.02	9.43	5.41
1	433.	215.	115.	143.	417.	329.	238.	18.36	10.93	29.29	18.36
2	291.	143.	79.	176.	454.	221.	274.	26.97	15.67	42.64	26.97
3	76.	34.	24.	288.	581.	58.	410.	29.02	17.13	46.15	29.02
4	157.	75.	45.	226.	512.	120.	333.	33.54	19.83	53.37	33.54
5	7.	1.	7.	741.	746.	7.	745.	33.58	20.25	53.82	33.54
6	9.	1.	7.	741.	738.	8.	738.	33.61	20.68	54.29	33.54
7	156.	74.	44.	227.	513.	119.	334.	38.10	23.36	61.46	38.03
8	384.	190.	102.	153.	428.	292.	249.	49.55	29.53	79.08	49.48
9	341.	168.	91.	162.	439.	259.	260.	59.69	35.03	94.72	59.62
AVERAGE	172.	82.	48.	184.	476.	131.	292.				

SUMMARY OF DATA FOR WATER YEAR 1974

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	78.	35.	25.	285.	578.	60.	407.	2.11	1.49	3.60	2.11
11	6.	1.	7.	741.	751.	7.	750.	2.14	1.89	4.03	2.11
12	5.	1.	6.	741.	758.	7.	756.	2.18	2.27	4.45	2.11
1	3.	1.	6.	741.	770.	6.	767.	2.21	2.62	4.83	2.11
2	5.	1.	6.	741.	758.	7.	756.	2.24	3.00	5.25	2.11
3	2.	1.	5.	741.	782.	6.	778.	2.28	3.33	5.61	2.11
4	0.	1.	5.	741.	790.	6.	785.	2.31	3.63	5.95	2.11
5	1.	1.	5.	741.	789.	6.	784.	2.35	3.94	6.29	2.11
6	284.	139.	77.	178.	456.	216.	277.	10.74	8.58	19.32	10.51
7	870.	436.	225.	84.	352.	661.	176.	37.03	22.16	59.19	36.79
8	745.	372.	193.	97.	366.	566.	189.	59.50	33.83	93.34	59.27
9	149.	71.	43.	230.	517.	114.	338.	63.78	36.41	100.19	63.54
AVERAGE	179.	88.	50.	120.	419.	138.	229.				

SUMMARY OF DATA FOR WATER YEAR 1975

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	15.	3.	9.	509.	710.	12.	658.	0.19	0.53	0.72	0.00
11	3.	1.	6.	741.	772.	6.	769.	0.22	0.88	1.10	0.00
12	9.	1.	7.	741.	738.	8.	739.	0.25	1.31	1.56	0.00
1	5.	1.	6.	741.	761.	7.	759.	0.29	1.68	1.97	0.00
2	3.	1.	6.	741.	769.	6.	767.	0.32	2.04	2.36	0.00
3	1.	1.	5.	741.	785.	6.	781.	0.35	2.35	2.71	0.00
4	0.	1.	5.	741.	791.	6.	786.	0.39	2.66	3.05	0.00
5	0.	1.	5.	741.	790.	6.	785.	0.42	2.97	3.39	0.00
6	8.	1.	7.	741.	742.	8.	742.	0.46	3.39	3.85	0.00
7	134.	63.	39.	239.	527.	102.	349.	4.28	5.74	10.01	3.82
8	216.	105.	60.	200.	482.	164.	302.	10.60	9.34	19.93	10.14
9	435.	216.	115.	142.	416.	331.	238.	23.61	16.28	39.88	23.15
AVERAGE	69.	33.	22.	183.	518.	55.	320.				

SUMMARY OF DATA FOR WATER YEAR 1976

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	241.	117.	66.	191.	472.	183.	292.	7.09	3.98	11.07	7.09
11	181.	87.	51.	214.	499.	138.	319.	12.34	7.04	19.38	12.34
12	12.	2.	8.	579.	721.	10.	696.	12.44	7.53	19.98	12.34
1	9.	1.	7.	741.	737.	8.	737.	12.48	7.97	20.45	12.34
2	6.	1.	6.	741.	755.	7.	754.	12.51	8.36	20.87	12.34
3	3.	1.	6.	741.	769.	6.	767.	12.54	8.71	21.26	12.34
4	1.	1.	5.	741.	787.	6.	782.	12.58	9.03	21.60	12.34
5	6.	1.	7.	741.	751.	7.	750.	12.61	9.42	22.04	12.34
6	84.	38.	26.	279.	571.	64.	398.	14.90	11.01	25.91	14.63
7	70.	31.	23.	295.	588.	54.	419.	16.77	12.38	29.15	16.50
8	134.	63.	39.	239.	527.	102.	349.	20.59	14.72	35.31	20.32
9	153.	73.	44.	228.	515.	117.	336.	24.99	17.36	42.35	24.72
AVERAGE	75.	35.	24.	231.	547.	58.	361.				

SUMMARY OF DATA FOR WATER YEAR 1977

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	45.	18.	16.	338.	628.	35.	475.	1.10	0.99	2.09	1.10
11	22.	7.	11.	429.	685.	17.	586.	1.50	1.63	3.13	1.10
12	24.	8.	11.	415.	679.	19.	571.	1.96	2.29	4.26	1.10
1	39.	15.	15.	353.	640.	30.	495.	2.88	3.19	6.07	2.02
2	26.	9.	12.	404.	673.	20.	558.	3.40	3.89	7.29	2.02
3	12.	2.	8.	591.	723.	10.	701.	3.50	4.37	7.87	2.02
4	1.	1.	5.	741.	784.	6.	780.	3.53	4.69	8.23	2.02
5	1.	1.	5.	741.	787.	6.	783.	3.57	5.01	8.57	2.02
6	0.	1.	5.	741.	790.	6.	785.	3.60	5.31	8.91	2.02
7	48.	20.	17.	331.	622.	37.	466.	4.79	6.35	11.14	3.21
8	99.	46.	30.	265.	556.	76.	380.	7.54	8.16	15.71	5.97
9	538.	268.	141.	124.	396.	409.	218.	23.70	16.67	40.37	22.12
AVERAGE	71.	33.	23.	191.	520.	56.	327.				

SUMMARY OF DATA FOR WATER YEAR 1978

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	54.	23.	19.	319.	612.	41.	451.	1.38	1.13	2.50	1.38
11	20.	6.	10.	447.	692.	16.	605.	1.71	1.73	3.44	1.38
12	138.	65.	40.	237.	524.	105.	346.	5.65	4.14	9.79	5.32
1	166.	79.	47.	222.	507.	126.	328.	10.45	6.97	17.42	10.11
2	294.	144.	79.	175.	453.	224.	273.	19.15	11.76	30.91	18.82
3	221.	107.	61.	198.	480.	168.	300.	25.63	15.44	41.06	25.29
4	9.	1.	7.	741.	735.	8.	735.	25.66	15.88	41.54	25.29
5	4.	1.	6.	741.	766.	7.	764.	25.69	16.24	41.93	25.29
6	40.	16.	15.	350.	638.	31.	491.	26.64	17.15	43.80	26.24
7	495.	246.	130.	131.	404.	376.	226.	41.48	25.01	66.49	41.08
8	597.	298.	156.	116.	387.	454.	209.	59.44	34.43	93.86	59.03
9	34.	13.	14.	369.	652.	26.	515.	60.20	35.25	95.45	59.80
AVERAGE	173.	83.	49.	167.	462.	132.	276.				

SUMMARY OF DATA FOR WATER YEAR 1979

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	40.	16.	15.	350.	638.	31.	491.	0.95	0.91	1.86	0.95
11	7.	1.	7.	741.	745.	7.	745.	0.98	1.33	2.31	0.95
12	10.	1.	8.	741.	732.	8.	733.	1.02	1.78	2.80	0.95
1	306.	150.	82.	171.	449.	233.	270.	10.09	6.75	16.84	10.02
2	108.	50.	32.	257.	548.	82.	371.	13.11	8.70	21.81	13.04
3	87.	40.	27.	276.	568.	67.	394.	15.50	10.33	25.83	15.43
4	8.	1.	7.	741.	740.	8.	740.	15.53	10.76	26.29	15.43
5	47.	19.	17.	333.	624.	36.	469.	16.69	11.78	28.47	16.59
6	18.	4.	9.	472.	700.	14.	628.	16.96	12.35	29.31	16.59
7	138.	65.	40.	237.	524.	105.	346.	20.90	14.76	35.65	20.53
8	336.	166.	90.	164.	440.	256.	261.	30.88	20.19	51.07	30.52
9	927.	465.	240.	79.	346.	704.	170.	58.91	34.64	93.55	58.55
AVERAGE	169.	81.	48.	148.	446.	129.	258.				

SUMMARY OF DATA FOR WATER YEAR 1980

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	471.	234.	124.	135.	409.	358.	230.	14.11	7.49	21.60	14.11
11	26.	9.	12.	404.	673.	20.	558.	14.63	8.19	22.82	14.11
12	51.	21.	18.	325.	617.	39.	458.	15.92	9.27	25.19	15.40
1	36.	14.	14.	361.	646.	28.	505.	16.76	10.13	26.89	16.24
2	117.	55.	35.	250.	540.	89.	363.	20.06	12.22	32.28	19.54
3	83.	37.	52.	257.	801.	89.	572.	22.31	15.33	37.64	21.79
4	218.	10.	23.	482.	737.	34.	658.	22.94	16.73	39.67	21.79
5	33.	1.	16.	440.	742.	17.	733.	22.97	17.72	40.69	21.79
6	35.	0.	14.	460.	741.	14.	735.	22.99	18.55	41.54	21.79
7	42.	0.	16.	605.	791.	16.	786.	23.02	19.50	42.51	21.79
8	21.	10.	73.	185.	587.	83.	539.	23.62	23.90	47.51	22.39
9	81.	53.	183.	187.	440.	236.	383.	26.84	34.91	61.75	25.62
AVERAGE	101.	37.	48.	198.	542.	85.	392.				

SUMMARY OF DATA FOR WATER YEAR 1981

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	26.	17.	79.	303.	568.	96.	520.	1.04	4.76	5.80	1.04
11	6.	8.	43.	346.	530.	51.	501.	1.53	7.33	8.85	1.53
12	7.	6.	42.	431.	643.	48.	615.	1.91	9.86	11.77	1.53
1	5.	2.	6.	515.	666.	8.	629.	2.03	10.25	12.28	1.53
2	94.	40.	72.	389.	716.	112.	599.	4.45	14.60	19.05	3.95
3	17.	11.	50.	473.	633.	61.	605.	5.10	17.61	22.71	3.95
4	4.	0.	21.	603.	507.	21.	507.	5.11	18.86	23.97	3.95
5	3.	1.	6.	673.	564.	7.	578.	5.16	19.22	24.38	3.95
6	128.	27.	75.	529.	686.	102.	644.	6.81	23.75	30.56	3.95
7	16.	16.	9.	282.	452.	25.	344.	7.78	24.30	32.07	4.91
8	239.	116.	65.	361.	540.	182.	425.	14.80	28.25	43.05	11.94
9	769.	528.	369.	54.	216.	897.	121.	46.67	50.51	97.18	43.80
AVERAGE	110.	64.	70.	159.	438.	134.	304.				

SUMMARY OF DATA FOR WATER YEAR 1982

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	58.	33.	60.	196.	294.	93.	260.	1.96	3.64	5.60	1.96
11	8.	1.	24.	419.	618.	24.	613.	1.99	5.06	7.06	1.96
12	5.	3.	20.	545.	678.	23.	659.	2.20	6.26	8.45	1.96
1	6.	5.	14.	595.	656.	19.	641.	2.47	7.10	9.57	1.96
2	10.	3.	9.	646.	767.	12.	739.	2.64	7.65	10.28	1.96
3	61.	12.	5.	456.	742.	16.	537.	3.35	7.93	11.27	1.96
4	81.	25.	23.	271.	797.	48.	520.	4.87	9.30	14.17	3.49
5	78.	15.	16.	284.	727.	32.	515.	5.79	10.29	16.08	4.40
6	1854.	512.	320.	48.	488.	832.	217.	36.69	29.58	66.27	35.31
7	426.	211.	113.	105.	232.	324.	149.	49.42	36.39	85.81	48.04
8	296.	191.	139.	168.	290.	330.	219.	60.95	44.78	105.74	59.57
9	519.	208.	108.	199.	392.	316.	265.	73.52	51.28	124.80	72.13
AVERAGE	283.	102.	71.	123.	424.	172.	246.				

SUMMARY OF DATA FOR WATER YEAR 1983

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	726.	394.	234.	58.	111.	628.	78.	23.80	14.13	37.93	23.80
11	52.	32.	18.	247.	458.	50.	324.	25.70	15.22	40.92	25.70
12	17.	7.	43.	469.	653.	50.	627.	26.12	17.80	43.92	25.70
1	43.	15.	44.	562.	715.	59.	677.	27.00	20.48	47.48	25.70
2	543.	278.	121.	284.	640.	398.	392.	43.75	27.76	71.50	42.44
3	690.	319.	110.	100.	281.	429.	146.	63.00	34.37	97.37	61.70
4	308.	74.	36.	169.	457.	110.	263.	67.45	36.53	103.98	66.15
5	15.	2.	16.	329.	604.	18.	569.	67.59	37.49	105.08	66.29
6	97.	37.	35.	237.	653.	72.	439.	69.82	39.59	109.41	68.52
7	139.	73.	62.	130.	477.	135.	289.	74.22	43.31	117.53	72.92
8	438.	258.	104.	185.	415.	363.	251.	89.81	49.60	139.41	88.50
9	604.	413.	193.	118.	335.	606.	187.	114.73	61.26	175.99	113.43
AVERAGE	306.	158.	85.	148.	382.	243.	229.				

SUMMARY OF DATA FOR WATER YEAR 1984

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	129.	98.	101.	161.	315.	198.	239.	5.89	6.07	11.96	5.89
11	60.	47.	56.	165.	487.	103.	341.	8.70	9.48	18.18	8.70
12	201.	206.	69.	248.	588.	275.	333.	21.13	13.63	34.75	21.13
1	167.	72.	48.	152.	472.	120.	280.	25.49	16.52	42.01	25.49
2	89.	40.	33.	304.	640.	73.	455.	27.93	18.51	46.44	27.93
3	216.	105.	70.	208.	623.	175.	375.	34.25	22.76	57.01	34.25
4	68.	30.	45.	230.	453.	74.	364.	36.04	25.46	61.50	36.04
5	26.	13.	27.	345.	489.	41.	442.	36.85	27.11	63.96	36.85
6	38.	15.	34.	313.	558.	49.	483.	37.77	29.17	66.94	37.77
7	314.	51.	51.	174.	518.	102.	345.	40.88	32.23	73.11	40.88
8	218.	130.	60.	140.	434.	190.	233.	48.69	35.86	84.55	48.69
9	22.	19.	48.	186.	393.	66.	335.	49.81	38.74	88.55	49.81
AVERAGE	129.	69.	54.	201.	485.	122.	325.				

SUMMARY OF DATA FOR WATER YEAR 1985

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	5.	6.	6.	385.	512.	12.	452.	0.34	0.38	0.72	0.34
11	18.	5.	22.	450.	532.	27.	517.	0.65	1.72	2.37	0.34
12	9.	1.	7.	332.	560.	8.	544.	0.68	2.16	2.85	0.37
1	7.	1.	29.	433.	587.	29.	584.	0.72	3.89	4.61	0.37
2	6.	1.	24.	469.	542.	25.	540.	0.75	5.34	6.10	0.37
3	7.	1.	25.	636.	614.	26.	614.	0.79	6.83	7.62	0.37
4	6.	8.	20.	629.	630.	27.	630.	1.24	8.03	9.26	0.37
5	1.	1.	5.	754.	527.	6.	549.	1.27	8.34	9.61	0.37
6	1.	0.	2.	731.	514.	2.	514.	1.27	8.49	9.76	0.37
7	13.	2.	22.	189.	452.	24.	429.	1.40	9.80	11.20	0.50
8	207.	60.	34.	239.	477.	94.	325.	5.02	11.84	16.86	4.12
9	273.	90.	111.	186.	336.	201.	269.	10.47	18.53	29.00	9.57
AVERAGE	46.	14.	26.	243.	468.	40.	387.				

SUMMARY OF DATA FOR WATER YEAR 1986

MONTH	HCQ	Q101	Q106	TDS101	TDS106	QSUM	TDSAVG	CUM101	CUM106	CUMSUM	101<400
10	142.	22.	44.	426.	485.	66.	465.	1.33	2.63	3.96	0.00
11	35.	22.	34.	449.	461.	56.	456.	2.67	4.69	7.35	0.00
12	9.	9.	31.	699.	577.	40.	605.	3.21	6.54	9.76	0.00
1	12.	9.	45.	499.	559.	54.	549.	3.76	9.26	13.02	0.00
2	17.	8.	25.	808.	581.	33.	637.	4.26	10.76	15.02	0.00
3	109.	46.	28.	796.	591.	74.	718.	7.02	12.44	19.46	0.00
4	17.	7.	9.	546.	519.	16.	531.	7.45	13.00	20.45	0.00
5	3.	0.	15.	975.	558.	15.	558.	7.45	13.90	21.34	0.00
6	171.	123.	71.	620.	575.	194.	603.	14.85	18.20	33.05	0.00
7	353.	159.	76.	268.	310.	234.	282.	24.42	22.77	47.18	9.57
8	463.	230.	122.	195.	382.	352.	260.	38.28	30.13	68.42	23.43
9	424.	210.	112.	180.	327.	322.	231.	50.95	36.91	87.86	36.11
AVERAGE	146.	70.	51.	330.	444.	121.	378.				

Appendix B
SLOW THE FLOW PROGRAM

S L O W T H E F L O W

A Public Awareness Program

On

Water Conservation

GENERAL DEVELOPMENT UTILITIES, INC.

February 1982

INTRODUCTION

In February of 1982, General Development Utilities, Inc., (GDU) began its public awareness program on water conservation. This program was created and implemented for three important reasons:

1. Regulatory requirements
2. Customer requests for information on how to reduce water consumption
3. GDU's concern for the waste of water, a precious resource, for the future of water supply in Florida

It is GDU's plan to supply its customers with helpful information on such topics as lawn irrigation, planting and landscaping with native flora, watering with soaking hoses, how to find water leaks, the use of water displacement devices in toilets, and other such information -- all which stress the reduction of water. These topics will be explained further as this report progresses.

In addition, GDU will be working with the water management districts on water reduction plans as the dry season comes about. Customers will be notified in their bills of any voluntary or mandated reductions they are to accomplish.

This report will outline the program's goals, target publics, key ingredients, and measuring devices for effectiveness.

PROGRAM GOALS

Six goals were established, and all efforts were planned to reach these goals and the program's ultimate success. The goals are as follows:

1. To effect a reduction in home water use, by a measurable percentage, during the test period. This percentage is fluid at present, and depends largely on the required reductions imposed by the water management districts.
2. To be made aware of ordinances issued by the various water districts, or other governing body, and to alter required percentage reductions in water use accordingly.
3. To establish awareness of the need for water conservation in the minds of young people to carry into present home situations and later into life.
4. To raise general citizen awareness in General Development communities, about the need to conserve water in Florida, with emphasis upon newcomers.
5. To reinforce that General Development Utilities, Inc. is a concerned corporate citizen.

Customers

Customer information emphasis will be provided largely by use of bill stuffers (a list of suggested stuffers is attached). Other efforts will include a public service slide show for use with groups, posters placed throughout the communities, and other special programs such as home audit instruction plans for customers' own use.

Children's Education

Children's educational programs will be held during the first week in May, in most communities, through the planned activity "Better Water for People Week". Sponsored nationally by the American Waterworks Association, GDU has elected to participate.

Key ingredients include the issuance of city ordinances praising the water utility for providing the excellent service and product, comic books for school children on where water comes from and the types of treatment, and tours for school children with written scripts used to correlate with questions provided to teachers as a learning tool.

Emphasis on school children is important to a successful community program because as the kids strive to conserve, their parents will too. While this event is planned as a "fun" event, every effort is made to stress water conservation, certification of operating personnel, professionalism, and the importance of a water company to the well-being of everyone living in that community.

Newcomers

New customer orientation is especially vital to acquaint people with conditions in our particular areas of Florida. If the new resident is not from Florida originally, even more is the importance. So many things are different from what they are accustomed: effects of the sun on all living things (humans, animals and plant life too); and growing lawns, plant and vegetables — when to plant and how to maintain them during the different seasons; to name a few. This information will be made available as it is prepared, because these are some of the bill stuffer subjects.

The Utility Company will also be providing these brochures to the parent company's Housing Division for use by existing customers as the questions arise relating to these subjects.

Business

Emphasis will be important here too, though not as much. Most businesses, other than restaurants, nurseries or laundromats use water primarily for landscaping irrigation and flushing. Special efforts will be made to work with restaurants, nurseries, laundromats and industrial concerns in order to plan water conservation programs specific to their needs.

KEY INGREDIENTS

To effect a successful program, the following key ingredients must be thoroughly researched and accomplished.

A. Of the pertinent regulators, determine the roles they will assume towards water management, their legal enforcement powers, the media campaigns they will pursue, their modes of notifying the utility when specific action is to be taken, and their probability of cooperation with the utility in seeking rate relief when water consumption, thus revenue, does decline.

The pertinent regulators are:

1. Water management districts
   a. Southwest Florida Water Management District - Charlotte, Sarasota and DeSoto County operations
   b. South Florida Water Management District - St. Lucie, Hendry, and Glades County operations
   c. St. Johns River Water Managment District, Brevard, Indian River, and Marion County operations

2. Other regulators
   a. City and county governments
   b. Florida Public Service Commission

B. Determine budget restrictions and cost out program based on regulatory requirements and funds available.

C. Review available conservation data. Much informational material is already written, and can be used by the utility in a cost-saving measure.

D. Plan bill stuffers; assign writing duties.

E. Create measuring device to meet regulatory guidelines (reporting form for field location to use).

Example:

Month of __________, 198_

Number Active Customers	Percent Reduction	Day	Plant Flow	Remarks	Rainfall

Note: Depending on the specific water management district, the utility may not be required to provide this information. The District's efforts will include determining the level of the actual water source before any additional action is taken. The District may require a reduction in pressure of the utility's distribution system, as a drastic measure, if community residents don't conserve as requested. In that case, this measuring device form still would not be used.

F. Coordinate all written material and on-going publicity with the Corporate Communications Department of the parent company.

G. Enlist cooperation of other divisions of the parent company to effect an all-company effort towards water conservation. (In some communities, water saving devices and low flush toilets are already being installed).

H. Coordinate the availability of water saving devices with local plumbers and hardware stores; and provide a list of those participating in our program to customers upon request.

SUMMARY

Customer interest in water conservation generally leans more towards a reduction in the amount of their water bills. They don't usually wish to alter their life styles enough to accomplish this cost reduction. Obviously, a change in daily habits has to occur.

In addition, many customers don't feel there could be a water shortage anyway. "After all, didn't it rain last night?" "Look at all the water in the ocean surrounding Florida", they say.

With these customers' feelings in mind, we are planning and providing our water conservation program. GDU's Slow the Flow is an honest and thoroughly-planned effort toward customers' information and subsequent water reductions. Whether customers put conservation methods into practice remains a choice they must make.

SUGGESTED BROCHURE SUBJECT

1) Lawn Watering
2) How Much Water Do you Use/How to Read Your Meter
3) Home Self Water Audit
4) What a Water Leak Costs You (Usage not dollars)
   Don't Leave a Drip When You Go on a Trip
5) Flush with Less: Toilet Water Displacement Devices
6) Editorial on Rainy Season and Year-Long Conservation Plan (reinforcement of conservation)
7) Lawn Fertilizing/Soaking Hoses
8) Planting/Landscaping
9) Listen for Leaks/Are You Wasting Water
10) Call Us in an Emergency
11) Be a Leak Seeker
12) Utility Speakers Available for Group Presentations/ Conservation Slide Show

SLOW THE FLOW

March, 1982

ARE YOU WATERING TOO MUCH?

Most of us have developed bad habits as we water lawns and plants. We not only water too much, but at the wrong times.

You can save money on your water bill by following these simple rules.

1. Let your plants tell you when they are thirsty. Water only when plant leaves begin to droop and curl. Lawns need water when leaf blades are folded and footprints remain in the grass.

2. Check the weather report before watering. You may be able to save the water–plus the effort required to sprinkle–by getting the forecast. It's silly to water just before a thunderstorm.

3. Avoid the need to water as often by mulching around plants. Putting down a layer of mulch will slow evaporation of moisture and discourage weed growth too.

General Development Utilities, Inc.

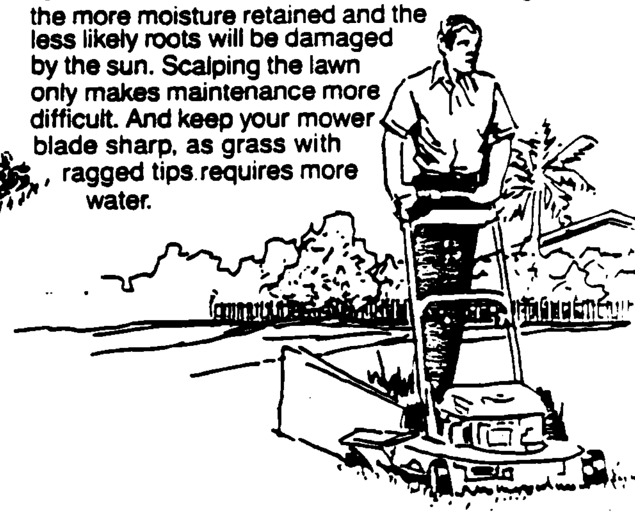
4. **If you must water, soak your lawn to a depth of one-half to three-quarters of an inch.** To tell how long the sprinkler must run, collect water in a pie plate. Soaking the roots makes them grow deeper so that plants won't need watering as often. Light, frequent waterings do not encourage roots to stretch.

5. **Water in the early morning hours.** If you have a timer, between 4 and 6 a.m. is best. Avoid watering in the midday heat or when windy, because of evaporation. Lawns watered at night remain moist too long, creating the risk of pest invasion.

6. **Water your plants rather than the street or driveway.** Sprinkler heads should have an even spray pattern aimed in the right direction and valves should open and close properly. Replace heads if they are leaking.

7. **Fertilize your lawn twice yearly, in early spring (April-May) and early autumn (October-November).** A fertilized lawn in healthy condition requires less water. It will also remain green longer.

8. **Set your mower for a height of at least two to two and one half inches.** The taller the grass, the more moisture retained and the less likely roots will be damaged by the sun. Scalping the lawn only makes maintenance more difficult. And keep your mower blade sharp, as grass with ragged tips requires more water.



Note to newcomers. Bahiagrass common to this area requires less irrigation than other types. Prolonged watering is detrimental to turf quality and leads to weed problems.

General Development Utilities, Inc.

WE USE TOO MUCH WATER

Pressures to conserve water in Florida are becoming more and more intense.

The State's population is increasing three percent each year, and urban water consumption is rising more than twice as fast.

True, most parts of Florida get 50 inches of rain a year. Yet there are simply no guarantees that sufficient water will always be where we need it, when we need it. As we must go further and further to find water, the cost of bringing it to our homes likely will rise.

We Americans have always used rivers of water in our daily lives. The average Florida resident uses approximately 150 gallons per day.

Water experts say that many of us could cut our daily use way down without feeling any serious inconvenience.

To conserve water and ultimately save money too, we simply must do better.

IT'S A MATTER OF HABIT

Reducing the amount of water we use around the home requires that we change our wasteful habits.

As any smoker who has tried to quit can tell you, habits are the most difficult things in the world to change.

We were successful in changing our habits to conserve more gasoline when world-wide scarcity threatened. And with a little bit of effort from each of us, we will be equally effective in conserving Florida's vital water resources.

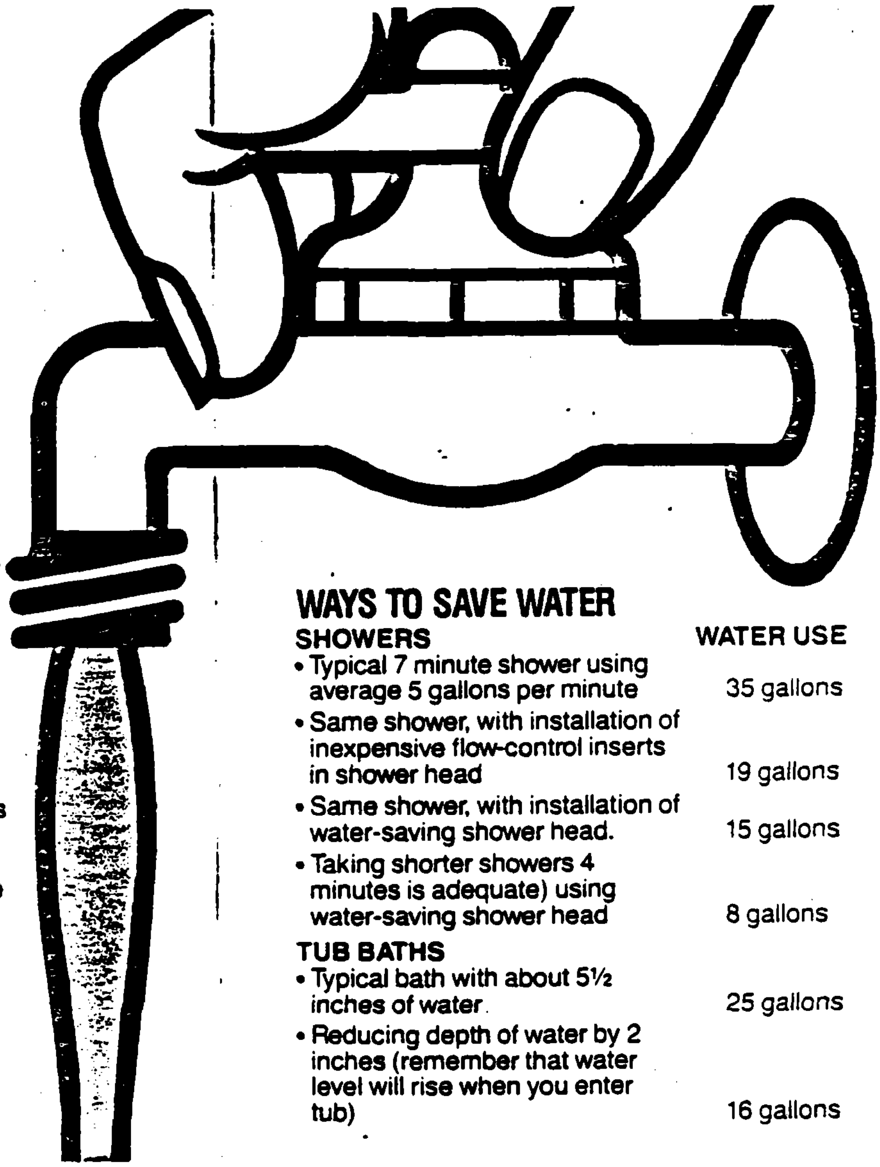
HERE ARE SOME HELPFUL REMINDERS.

FIX FAUCET LEAKS.

A tiny stream can waste thousands of gallons a month. Fixing leaks is often a matter of spending a few cents on a new washer.

BECOME WATER CONSCIOUS.

GDU customers pay different rates depending upon where they live, but as a rule of thumb every thousand gallon reduction in water use can save you nearly $2 on your water bill. Little savings mount up quickly.

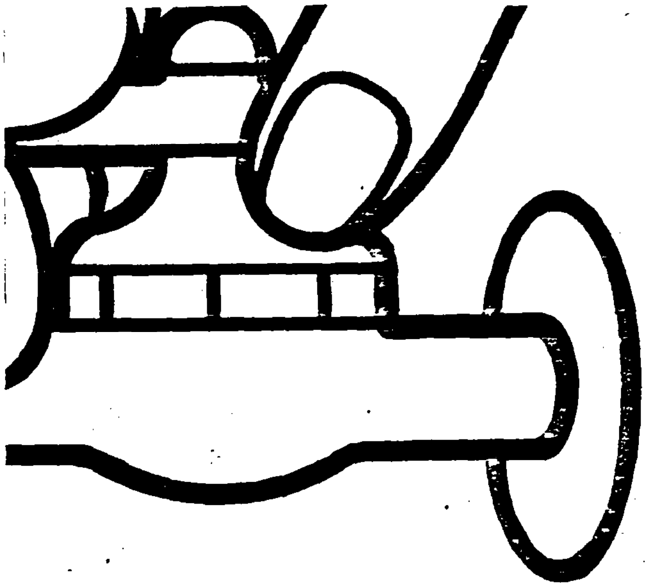


WAYS TO SAVE WATER

SHOWERS	WATER USE
• Typical 7 minute shower using average 5 gallons per minute	35 gallons
• Same shower, with installation of inexpensive flow-control inserts in shower head	19 gallons
• Same shower, with installation of water-saving shower head.	15 gallons
• Taking shorter showers 4 minutes is adequate) using water-saving shower head	8 gallons
TUB BATHS	
• Typical bath with about 5½ inches of water	25 gallons
• Reducing depth of water by 2 inches (remember that water level will rise when you enter tub)	16 gallons

WAYS TO SAVE WATER CONTINUED

TOILETS	WATER USE
• One flush of standard toilet	6 gallons
• One flush when two quart plastic bottles are filled with water and set in the toilet tank where they won't interfere with the flushing mechanism. Put a few clean stones in the bottles to hold them down.	5.5 gallons
• One flush when toilet dams (available at plumbing supply stores) are installed in tank to hold back water.	4.5 gallons
FAUCETS	
• Typical faucet running full blast for one minute	5 gallons
• One minute's use when aerators with screw-on adaptors are installed. Aerators mix air with water to reduce the flow.	2.5 gallons
SHAVING	
• Typical shave with water running	20 gallons
• Rinsing shaver in sink partially filled with water	1 gallon
BRUSHING TEETH	
• Ordinary tooth-brushing, letting water run	2 gallons
• Using a cup and running the tap just to rinse the toothbrush	Minimal
DISHWASHER	
• Full cycle	16 gallons
• Rinsing dishes in a stoppered sink allows bypass of pre-soak cycle	7 gallons



MAKE A COMMITMENT.

Set aside a couple of hours to install toilet dams. faucet aerators and either a water-saving shower head or flow restrictor inserts. While you're at the plumbing supply store you can pick up any replacement washers needed for faucets.

With water and sewer rates always rising because of the increased costs to the utility, the incentives to take water conservation seriously around the home are growing stronger. Why not begin now?

WAYS TO SAVE WATER CONTINUED

OUTDOORS	WATER USE
• Cleaning pool decks, sidewalks and driveways with a broom rather than hosing them down	SAVE 50 gallons
• Washing car for 20 minutes with a pistol grip nozzle on hose instead of allowing water to run continuously	SAVE 85 gallons

Why not take a moment to make a rough estimate of the amount of water you might save around the home in a month?

LOOK FOR LEAKS

You may have leaks around the home that are wasting hundreds – perhaps even thousands – of gallons each month. A little detective work may save you money on your water bill.

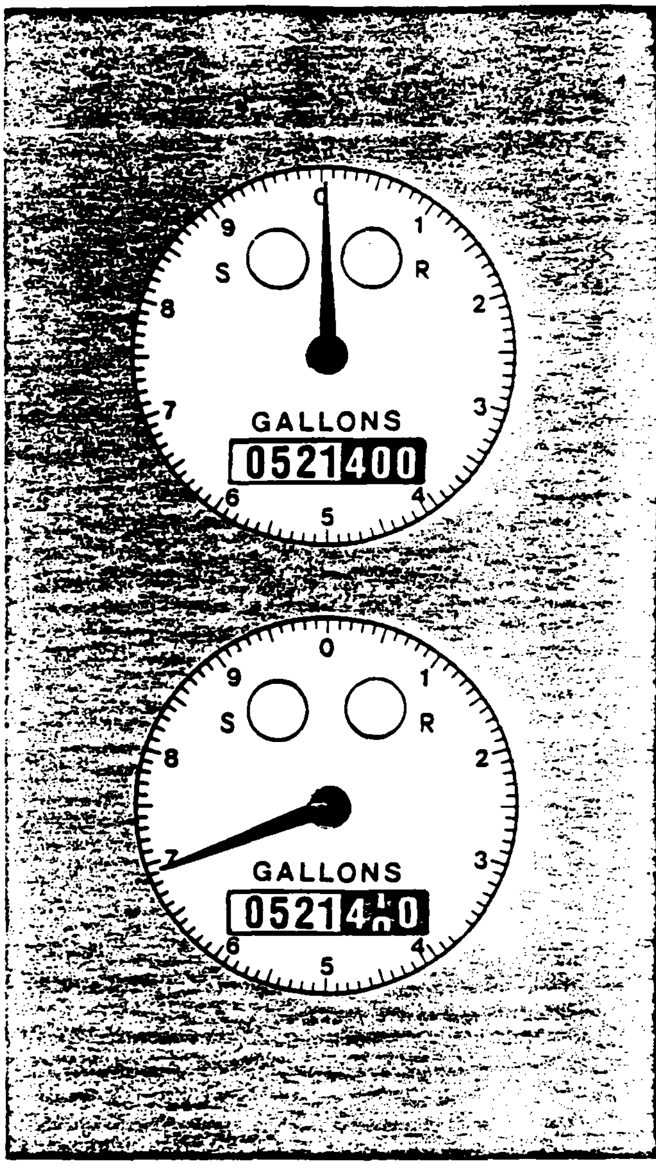
It's simple to conduct your own home water audit, but first you must learn how to read the meter.

The meter pictured to the right above reads 521,400 gallons. Each clockwise revolution of the sweep hand means that 10 gallons have been used. The meter at the right bottom shows the sweep hand has moved. When it reaches the top, the reading will be 521,410. Got it?

Some day when you plan to be gone for a few hours, shut off all the water around the house. Then make a note of the meter reading and time of day. If the sweep hand moves by the time you return home, you have a leak.

Project the water loss over an entire day, then a month. If a leak is shown, you will be surprised at the number of lost gallons.

Now don't you think it's worthwhile to fix leaky faucets?



Appendix C
BASIS OF COST ESTIMATES FOR PORT CHARLOTTE
WATER SUPPLY EXPANSION ALTERNATIVES

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BASIS OF COST ESTIMATES FOR PORT CHARLOTTE WATER SUPPLY EXPANSION ALTERNATIVES

All construction, capital, and O&M cost estimates are planning level estimates expressed in March 1987 dollars and based on an Engineering News Record Construction Cost Index (ENRCCI) of 4356. This appendix presents details of the basis of estimates for construction and O&M costs for each water supply expansion alternative. In all cases, non-construction capital costs were estimated at 30 percent of construction cost to account for additional capital items including administration, engineering, and legal expenses.

PEACE RIVER WATER SUPPLY SYSTEM

WATER TREATMENT PLANT

Construction cost estimates for expansion of the Peace River WTP were computed based on existing processes at the plant and using EPA cost curves (Gumerman et al., August 1979). Construction cost for each 6-mgd expansion of the existing plant was $3.72 million, up to a total installed capacity of 30 mgd. O&M costs for the Peace River WTP were estimated based on 1986 cost data supplied by GDU. O&M costs were $0.64 per 1,000 gallons produced.

ASR WELL FIELD

Construction costs for ASR well field expansion, including the wells, well development, piping, pumps, instrumentation, etc., were based on recent engineers' estimates and bid data for the current Peace River ASR facilities expansion. Total construction cost was estimated to be $240,000 per mgd of installed ASR capacity.

O&M costs for ASR were estimated at $3,550 per year per mgd. This estimate is based on the 1985 CH2M HILL evaluation of ASR at the Peace River WTP (CH2M HILL, April 1985a).

MYAKKAHATCHEE CREEK (NORTH PORT) WATER SUPPLY SYSTEM

WATER TREATMENT PLANT

Construction cost estimates for expansion of the North Port WTP were computed based on existing processes at the plant and using EPA cost curves (Gumerman et al., August 1979). These estimates are summarized as follows:

Plant Capacity (mgd)	Construction Cost (10^6 dollars)
4.4	3.85
8	5.80
12	7.80
16	9.88
20	11.71
24	13.96

These data were then used to develop the following construction cost equation:

$$\text{Construction Cost} = 1.667 + 0.51\ (Q)$$

where construction cost is expressed in million dollars and Q is plant capacity (above the existing capacity of 4.4 mgd) in mgd.

O&M costs for the North Port WTP were estimated based on 1986 cost data supplied by GDU. O&M costs were $0.72 per 1,000 gallons produced.

OFFSTREAM STORAGE RESERVOIR

Construction cost for offstream storage at North Port was based on the Peace River reservoir expansion cost estimate developed for GDU by CH2M HILL (CH2M HILL, June 1985). Unit cost was $2,412 per acre-foot. New land cost for the reservoir was included at $2,500 per acre.

The O&M cost of $72 per acre-foot per year was based on estimates developed in the 1985 CH2M HILL evaluation of ASR at the Peace River WTP.

RAW WATER TRANSMISSION PIPELINE

Construction costs for a raw water transmission pipeline to interconnect the raw water intake structure, the water treatment plant, and the offstream storage reservoir were based on the following unit costs:

Pipe Diameter (inches)	Unit Construction Cost ($/L.F.)
20	36.33
27	53.46
30	62.28
36	79.93
42	101.72

Total construction cost was based on an assumed pipeline length of 5,000 feet.

O&M costs for the raw water transmission pipeline are included in the intake structure O&M cost discussed below.

RAW WATER INTAKE STRUCTURE

Construction costs for a Myakkahatchee Creek raw water intake structure were estimated based on the cost for the Peace River intake structure of $41,820/mgd. O&M costs were estimated at $4,040 per year per mgd, an estimate originally developed during the 1985 CH2M HILL evaluation of ASR at the Peace River WTP.

ASR WELL FIELD

Construction and O&M cost estimates were developed using the unit costs previously discussed for the Peace River facility.

BRACKISH GROUNDWATER DESALTING

WELL FIELD A (3,000 mg/l TDS)

Construction costs for RO desalting, including associated well field and brine disposal costs, were developed for three capacities as follows:

RO Capacity (mgd)	Construction Cost ($\$10^6$)	Unit Construction Cost ($/gpd)
1.94	4.15	2.14
5.16	8.80	1.71
10.32	15.95	1.55

Major assumptions related to the above construction costs are:

Membrane Type	Low pressure, TFC type
RO Product Capacity/Train	0.645 mgd
RO Bypass Flow	None
RO Recovery (Product: Feed Flow Ratio)	75%
Plant Operating Factor	95%
Well Capacity	600 gpm each
Installed Equipment	$507,000/train
Installed Equipment Unit Cost	$0.79/gpd
Well Field Development	$140,000/well
Brine Disposal Well	$685,000
Finished Water Storage	100,000 gal

Other assumptions that influence construction costs and vary with RO plant and size are given below:

RO Capacity (mgd)	Land Area (acres)	Building Size (ft^2)
1.94	2.0	6,800
5.16	2.5	12,400
10.32	3.0	19,400

O&M costs were developed based on 6, 8, and 10 operators required for the 1.94-, 5.16-, and 10.32-mgd plants, respectively, at a raw annual salary of $20,000 per year each. Overhead costs were computed at 30 percent of raw salary. RO membrane life was assumed to be 5 years. Other unit costs used to compute O&M costs are summarized below:

Acid	$0.04/lb
Scale Inhibitor	$1.10/lb
Caustic	$0.10/lb
Chlorine	$0.11/lb
Power	$0.08/kWh

WELL FIELD B (6,000 mg/l TDS)

Construction costs for RO desalting, including associated well field and brine disposal costs, were developed for three capacities as follows:

RO Capacity (mgd)	Construction Cost ($\$10^6$)	Unit Construction Cost ($/gpd)
2.25	4.85	2.16
5.16	9.72	1.88
10.32	18.02	1.75

Major assumptions related to the above construction costs are:

Membrane Type	Medium Pressure, TFC type
RO Product Capacity/Train	0.75 mgd
RO Bypass Flow	None
RO Recovery (Product: Feed Flow Ratio)	75%
Plant Operating Factor	95%
Well Capacity	350 gpm each
Installed Equipment	$624,000/train
Installed Equipment Unit Cost	$0.83/gpd
Well Field Development	$140,000/well
Brine Disposal Well	$432,000
Finished Water Storage	100,000 gal