CITY OF SULPHUR SPRINGS, TEXAS

COOPER RESERVOIR WATER SUPPLY STUDY

PREPARED
FOR
THE CITY OF SULPHUR SPRINGS, TEXAS

STATE OF TEXAS
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PROJECT NO. 14719.100
JULY 1988

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Sulphur Springs, Texas
Cooper Reservoir Water Study

B&V Project 14719.100
July 1988

Honorable Mayor and City Council
City of Sulphur Springs
125 South Davis
Sulphur Springs, Texas 75482

Council Members:

Black & Veatch is pleased to submit this report, "Cooper Reservoir Water Supply Study.

This report presents an evaluation of several alternatives and identifies the preferred plan for developing the City's water rights in Cooper Reservoir. The staged approach, as presented herein, offers an opportunity to implement the Cooper System on an affordable and cost-effective step-wise basis. While many details remain to be worked out through discussions with North Texas Municipal Water District, this report will provide the City with a basis for continuing those discussions.

Also evaluated were the existing water treatment facilities and distribution system. A staged approach is presented for expansion of the existing facilities.

We appreciate this opportunity to work with the City on this very significant project and look forward to our continued association.

Very truly yours,

BLACK & VEATCH

James C. Hesby, P.E.

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Enclosure

ACKNOWLEDGEMENTS

Mr. Dave Reed was coordinator for this project. Mr. Bill Farler assisted with inquiries about the design of the treatment facilities. Insight into operational and maintenance policies and problems was provided by Mr. Maxie Chester.

All of the gentlemen cited above are members of the City of Sulphur Springs staff.

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1.0 INTRODUCTION

1.1 BACKGROUND

The primary source of raw water for the City of Sulphur Springs is surface water from the combined pools of Century Lake and Lake Sulphur Springs. Both lakes are shown on Figure 1. According to a 1983 report by Freese and Nichols, Inc., the total storage volume of the combined lakes is 17,838 acre-feet, including an allowance for sediment accumulation.

In the early 1980's, drought conditions during three consecutive years caused the lake level to drop to dangerously low levels. The City was required to enforce water rationing.

In the mid 1980's, the City began expanding the existing water treatment and storage facilities. With these improvements, the present treatment plant has a rated treatment capacity of 7.0 million gallons per day.

The City's long-term source of raw water is Cooper Reservoir. This reservoir is currently under construction by the United States Army Corps of Engineers, with impountment scheduled for late 1991. The City of Sulphur Springs is a member of the Sulphur River Municipal Water District, which currently holds rights to 26.28 percent of the water in Cooper Reservoir. This equates to approximately 28.6 millon gallons per day available for withdrawal on an average day basis. The City's share of this is 13.2 mgd.

1.2 PURPOSE

The purpose of this report is to plan a regional water supply system for Hopkins County using Sulphur Springs' allocation of raw water from Cooper Reservoir, and to determine the improvements needed to satisfy present and future water requirements through the year 2040. The report provides the City with an implementation plan and a priority schedule for recommended improvements.

1.3 SCOPE

The planning period for this report is from the present to the year 2040. The study area, as shown on Figure 1, is Hopkins County including the City of Sulphur Springs.

Principal elements of this report include the following:

- Review of existing surface water and groundwater sources including quantity and quality, and the impact of 1986 Amendments to the Safe Drinking Water Act, and potential changes in regulatory agency standards.
- Analysis of the existing water treatment plant including physical facilities and processes.
- Development of future population and future water requirements for the planning area.
- Development of alternative intake/pumping station sizes and configurations at Cooper Reservoir, and alternative raw water pipeline sizes and routes to the existing water treatment plant.
- Development of alternative treatment processes for current and future water supplies.
- Development of recommended treatment facility improvements.
- Development of recommended distribution and storage facility improvements.
- Determination of probable construction costs for the recommended improvements.
- Development of a recommended improvements plan and a staged implementation schedule.
- Preparation of a water conservation plan to be submitted to the Texas Water Development Board and modified according to the Board's comments.

1.4 ABBREVIATIONS

The following abbreviations are used in this report:

AAD	Annual Average Day
ac ft	Acre-Foot
ac ft/year	Acre-Foot per year
Ag	Silver
$Al_2(SO_4)_3 14H_2O$	Alum
As	Arsenic
Ba	Barium
BAT	Best Available Technology
BOD	Biochemical Oxygen Demand
CO_3	Carbonate
C	Celsius
Cl	Chloride
Cd	Cadmium
Ca	Calcium
$CaCO_3$	Calcium Carbonate
CaO	Calcium Oxide (Quicklime)
cfs	Cubic Feet per second
COE	U.S. Corps of Engineers
Cr	Chromium
Cu	Copper
CT	Concentration (mg/l) Time (min)
El	Elevation
F	Fluoride
Fe	Iron
Ft	Feet
GAC	Granular Activated Carbon
gpd	Gallons per day
gpm/sq ft	Gallons per minute per square foot
HCO_3	Bicarbonate
HPC	Heterotrophic Plate Count

hp	Horsepower
IOC	Inorganic Compound
IH	Interstate Highway
K	Potassium
lb/mil gal	Pounds per million gallons
Log	Logarithim
MD	Maximum Day
Mg	Magnesium
ml	Milliliter
Mn	Manganese
MCL	Maximum Contaminant Level
MCLG	Maximum Contaminant Level Goal
Hg	Mercury
mg/l	Milligrams per liter
mgd	Million gallons per day
mil gal	Million gallons
Na	Sodium
NPDWR	National Primary Drinking Water Regulations
NO_3	Nitrate
NTMWD	North Texas Municipal Water District
NTU	Nephelometric Turbidity Units
Pb	Lead
pci/l	Picocuries per liter
ppd	Pounds per day
pph	Pounds per hour
psi	Pound per square inch
SDWA	Safe Drinking Water Act
Se	Selenium
SiO_2	Silicon Dioxide
SI	Langlier's Saturation Index
SO_4	Sulfate
SOC	Synthetic Organic Compound

Sq mi	Square Mile
SWTR	Surface Water Treatment Rule
TDH	Texas Department of Health
TDS	Total Dissolved Solids
THM	Thihalomethane
TOC	Total Organic Compounds
TON	Total Odor Number
TSS	Total Suspended Solids
TTHM	Total Trihalomethane
TWC	Texas Water Commission
TWDB	Texas Water Development Board
UMHOS	Micromhos
USEPA	U.S. Environmental Protection Agency
USGS	United States Geological Service
VOC	Volatile Organic Compound
VSS	Volatile Suspended Soilds
WSC	Water Supply Corporation
WTP	Water Treatment Plant
Zn	Zinc

2.0 SUMMARY

2.1 FINDINGS

Major findings of this report are presented below. The findings are presented for the City of Sulphur Springs and Hopkins County.

(1) Since 1970, the population of Sulphur Springs and Hopkins County has increased approximately 50 percent. The City's and County's 1987 population are estimated to be 16,000 and 30,000, respectively. The 1990 population is projected to be 19,000 for the City of Sulphur Springs and 33,000 for Hopkins County. The projected populations for the City and Hopkins County in the year 2040 are 57,500 and 84,460, respectively.

(2) On July 19, 1985, the highest recorded maximum day demand for raw water was recorded as 6.44 mgd. The projected maximum day demands for 1990 and 2040 are 10 and 26 mgd, respectively. The projected average annual day water requirements for all of Hopkins County in 1990 and 2040 are 6.41 and 14 mgd, respectively.

(3) The existing water treatment plant has a design capacity of 7 mgd. The filtering capacity of the plant is 13 mgd. The high service pump station has a firm capacity of 13.0 mgd. The existing site has adequate space to accommodate a 26 mgd water treatment plant. If the current method of sludge disposal is continued, additional land may have to be purchased for additional sludge handling facilities.

(4) The existing raw water intake/pump station on Lake Sulphur Springs has a firm pumping capacity of approximately 6 mgd because of hydraulic restrictions.

(5) Permitted average annual daily withdrawal of raw water from Lake Sulphur Springs is 8.75 mgd. The maximum diversion rate allowed by permit is 35 cfs, or 22.6 mgd.

(6) Cooper Reservoir is the long-term source of raw water for the planning area. This reservoir is currently under construction with impoundment scheduled to begin in 1991. The Sulphur River Municipal Water District (SRMWD) has a contract with the federal government for water supply storage in Cooper Reservoir in the amount of 26.282 percent of the total storage, or 71,750 acre-feet. The SRMWD has been issued Permit No. 2336 from the Texas Water Commission authorizing diversions not to exceed 26,960 ac ft/yr (24.01 mgd average annual day) for municipal purposes, and 11,560 ac ft/yr (10.32 mgd average annual day) for industrial purposes. The three member Cities of the SRMWD are Sulphur Springs, Commerce, and Cooper.

(7) We have estimated the annual average day flow to be 14 mgd in the year 2040. This water demand can be met with the City's share of the water rights at Cooper Reservoir (13.2 mgd) and their water rights on Lake Sulphur Springs. However, if the City wishes to use only water from Cooper Reservoir as their raw water source, then additional water rights would have to be purchased from the other member Cities of the SRMWD.

(8) Severe taste and odor problems have been experienced during the summer months from water in Lake Sulphur Springs. The major cause of this problem is increased growth of phytoplankton (primarily algae) during the summer.

(9) Water obtained from Cooper Reservoir will contain higher concentrations of inorganic dissolved solids than water from Lake Sulphur Springs, but it will have significantly lower concentrations of dissolved organic compounds. Therefore, it will be easier and less expensive to treat Cooper water than water from Lake Sulphur Springs. Waters from both sources could be intermixed in any proportion during the winter without producing problems in chemical quality. Successful

treatment of mixtures during the summer will require improvements at the existing water treatment plant. However, many of these improvements are also needed to satisfy new disinfection regulations, which are now being developed in response to the 1986 Amendments of the Safe Drinking Water Act.

(10) Chlorine and ammonia are added to the filter effluent to form chloramines as the primary disinfectant. The continued use of this method of disinfection will not be acceptable when requirements of the amendments to the Safe Drinking Water Act become effective in 1989.

(11) The water distribution system in the City of Sulphur Springs has a total ground and elevated storage capacity of 4.5 million gallons. It is anticipated that this storage capacity will keep the City within the State Health Department Standards for an "approved" water system until the year 2000, when additional ground storage and elevated storage will be required. Consideration should be given to construction of an additional elevated tank to correct low pressure problems identified by City staff at certain points in the distribution system.

(12) Distribution deficiencies in the City of Sulphur Springs have been identified in a report dated December 1985, by Bucher, Willis, and Ratliff. The City is currently conducting a water main replacement program for the smaller lines. An application has been filed with the Economic Development Administration for funding to construct larger lines.

(13) The water <u>transmission</u> mains from the City distribution system to the rural water districts in Hopkins County are of adequate capacity until the year 2000. At that time, the transmission mains to the North Hopkins and Brinker Water

Districts will have reached their capacities. The transmission main to the Brashear Water District will have adequate capacity until the year 2030 when an additional water main will be required.

2.2 RECOMMENDATIONS

Recommendations for the raw water conveyance system, water treatment plant, and water distribution system improvements are briefly summarized below.

(1) Proceed immediately with design and construction of high service pumping improvements at the existing high service pump station, to increase reliability and performance.

(2) Proceed immediately with design and construction of improvements and additions at the existing raw water pump station to increase its firm pumping capacity to 14 mgd by the year 1990.

(3) Proceed immediately with the design and construction of facilities at the water treatment plant to increase its capacity to 14 mgd by the year 1990.

(4) Proceed immediately with bench scale testing and plant pilot testing using ozone. After summertime operating data is available, proceed with detailed design and construction of ozone facilities to meet amendments to the Safe Drinking Water Act and to address taste and odor problems.

(5) Proceed with negotiations with the North Texas Municipal Water District (NTMWD) to share an intake/pumping station at the Finley Branch site on Cooper Reservoir. It is recommended that the structure be sized to provide an ultimate 26 mgd of firm pumping capacity to meet peak demands of the City of Sulphur Springs and Hopkins County. Initially install 14 mgd of firm pumping capacity.

(6) If shared intake/pump station facilities cannot be negotiated with NTMWD, proceed immediately with obtaining approval to construct a separate intake/pump station facility at Harpers Hill or Finley Branch. Upon site approval, proceed immediately with design and construction of facilities sized to provide an ultimate 26 mgd pumping capacity to allow completion by 1991.

(7) Once an intake/pump station site has been selected and approved at Cooper Reservoir, begin design of a 30-inch raw water pipeline to convey water from Cooper Reservoir to the water treatment plant. Provide an outlet to allow diversion of Cooper water into Lake Sulphur Springs. Use water from Cooper Reservoir as the primary source of raw water and maintain Lake Sulphur Springs as a standby source. As future demands increase, use Lake Sulphur Springs as a peaking reservoir, or construct a parallel 30-inch raw water pipeline from Cooper Reservoir to match the intake/pump station capacity.

(8) Continue to construct new water mains within the City's distribution system as recommended in the report by Bucher, Willis and Ratliff dated December 1985. The County distribution mains are of adequate capacity for the near future.

2.3 PROBABLE PROJECT COSTS

Probable project costs for the recommendations given in Section 2.2 are listed below. The costs included Engineering, Legal, and Administrative fees and an allowance for contingencies.

(1) Proceed immediately with design and construction of high service pumping improvements at the existing high service pump station, to increase reliability and performance.	$ 629,000

(2) Proceed immediately with design and construction of improvements and additions at the raw water pump station to increase its firm pumping capacity to 14 mgd by the year 1990. $ 152,000

(3) Proceed immediately with the design and construction of facilities at the water treatment plant to increase its capacity to 14 mgd by the year 1990. $3,929,000

(4) Proceed immediately with bench scale testing and plant pilot testing using ozone. After summertime operating data is available, proceed with detailed design and construction of ozone facilities to meet amendments to the Safe Drinking Water Act and to address taste and odor problems. $2,290,000

(5) Proceed with negotiations with the North Texas Municipal Water District (NTMWD) to share an intake/pumping station at the Finley Branch site on Cooper Reservoir. It is recommended that the structure be sized to provide an ultimate 26 mgd of firm pumping capacity to meet peak demands of the City of Sulphur Springs and Hopkins County. Initially install 14 mgd of firm pumping capacity. Negotiable

(6) If shared intake/pump station facilities cannot be negotiated with NTMWD, proceed immediately with obtaining approval to construct a separate intake/pump station

facility at Harpers Hill or Finley Branch. Upon site approval, proceed immediately with design and construction of facilities sized to provide an ultimate 26 mgd pumping capacity to allow completion by 1991. $3,800,000

(7) Once an intake/pump station site has been selected and approved at Cooper Reservoir, begin design of a 30-inch raw water pipeline to convey water from Cooper Reservoir to the water treatment plant. Provide an outlet to allow diversion of Cooper water into Lake Sulphur Springs. Use water from Cooper Reservoir as the primary source of raw water and maintain Lake Sulphur Springs as a standby source. As future demands increase, use Lake Sulphur Springs as a peaking reservoir, or construct a parallel 30-inch raw water pipeline from Cooper Reservoir to match the intake/pump station capacity. $6,900,000

3.0 SOURCES OF WATER

3.1 INTRODUCTION

Lake Sulphur Springs is the sole source of raw water for the City of Sulphur Springs at the present time. This lake is on White Oak Creek and is located northwest of the City.

The City is a member of the Sulphur River Municipal Water District, which is a local sponsor for and has water rights in Cooper Reservoir which is presently under construction on the South Sulphur River, approximately 11 miles north of the City. Cooper Reservoir will be a federal multipurpose impoundment operated by the U.S. Army Corps of Engineers (COE).

Groundwater is available in the southern half of Hopkins County, but it is very limited in the northern portion of the county. This has resulted in the City becoming a wholesale water purveyor to three smaller cities and nine water districts.

The objectives of this chapter are to characterize the chemical quality and the quantity of water available from each of these sources. This information is used for evaluation and design of water conveyance and treatment facilities in subsequent chapters.

3.2 SURFACE WATER

3.2.1 Lake Sulphur Springs

3.2.1.1 Quantity. Lake Sulphur Springs was constructed in 1971 and is located immediately downstream from Century Lake, which had served as the City's raw water supply since 1951. In 1983, the dam forming Century Lake was breached, allowing the two lakes to combine into a single pool.

A study, performed for the City in 1983 by Freese and Nichols, Inc., estimated the capacity of the combined pool to be 14,370 acre-feet (ac ft) at Elevation (El) 457 ft. This estimate included an allowance for the storage lost by sediment accumulation in the two reservoirs since their construction. The Freese and Nichols study recommended raising the spillway on the Lake Sulphur Springs Dam by two feet, to El 459. This improvement was completed in 1984 and increased the combined storage to 17,383 ac ft.

The Freese and Nichols study was also used as a basis for amending the original Texas Water Commission (TWC) Diversion Permit. The amended permit allows the City to divert 9,800 ac ft per year for municipal use. This is equivalent to 3,193 million gallons (mil gal) per year, or 8.75 million gallons per day (mgd) if the water is used at a uniform rate. The maximum diversion rate allowed by the permit is 35 cubic feet per second (cfs) which is equivalent to 22.6 mgd.

The drainage area for Lake Sulphur Springs includes approximately 66.4 square miles (sq mi). Average inflow to the lake during the 36 years (1943-1978) included in the Freese and Nichols study was 30,063 ac ft/yr, with a range from 5,300 ac ft/yr in 1956 to 85,160 ac ft/yr in 1957. Average evaporative loss from the lake was 3,160 ac ft/yr, or 10.5 percent of inflow. The average hydraulic detention time in the expanded lake is approximately 7.8 months, and the average water depth is approximately 7 ft when the water surface is at El 459. This combination of short detention time and shallow depth has a very significant impact on water quality.

3.2.1.2 Quality. The City measures turbidity, total alkalinity, and pH in the water obtained from Lake Sulphur Springs three times per day, (12 p.m., 8 a.m., 4 p.m.). These three observations are averaged to produce the daily values summarized in Appendix A.

Variations in monthly average and maximum daily turbidity from January 1985 through December 1987 are illustrated on Figure 3-1. Average turbidities during this period ranged from a high of 38

Nephelometric Turbidity Units (NTU) in June and July 1985, down to a low of 20 NTU in January 1986. A general trend of lower turbidities during the winter months (December, January, February, and March), followed by higher turbidities in the summer (June, July, August), is exhibited in all three years, but there is a great deal of variation from year to year. This variability is also present in the maximum daily turbidity observed each month. There are two primary causes of turbidity in the lake: suspension of sediment by wind-induced wave action, and increased growth of phytoplankton during warmer weather. The phytoplankton are responsible for the severe taste and odor problems which occur every summer.

Minimum daily turbidity values are not included on Figure 3-1, but they track the monthly values very closely and are usually 6 to 12 NTU below the average for each month. This correlation is also an indication of the major role played by phytoplankton in producing the turbidity observed in this lake.

The range in pH and total alkalinity observed during each month is shown on Figure 3-2. The pH is very uniform, with an overall average of 7.0 for the three-year period. Total alkalinity exhibits more variability, but the range in values does not pose significant treatment problems.

The U.S. Geological Survey (USGS) collected one or two water samples per year from the lake from 1975 through 1983 (13 samples total), and conducted analyses for the common inorganic ions found in surface waters. The average and range in concentration for each of these parameters is summarized in Table 3-1. The water is very low in Total Dissolved Solids (TDS), with calcium, bicarbonate and sulfate being the principal ions.

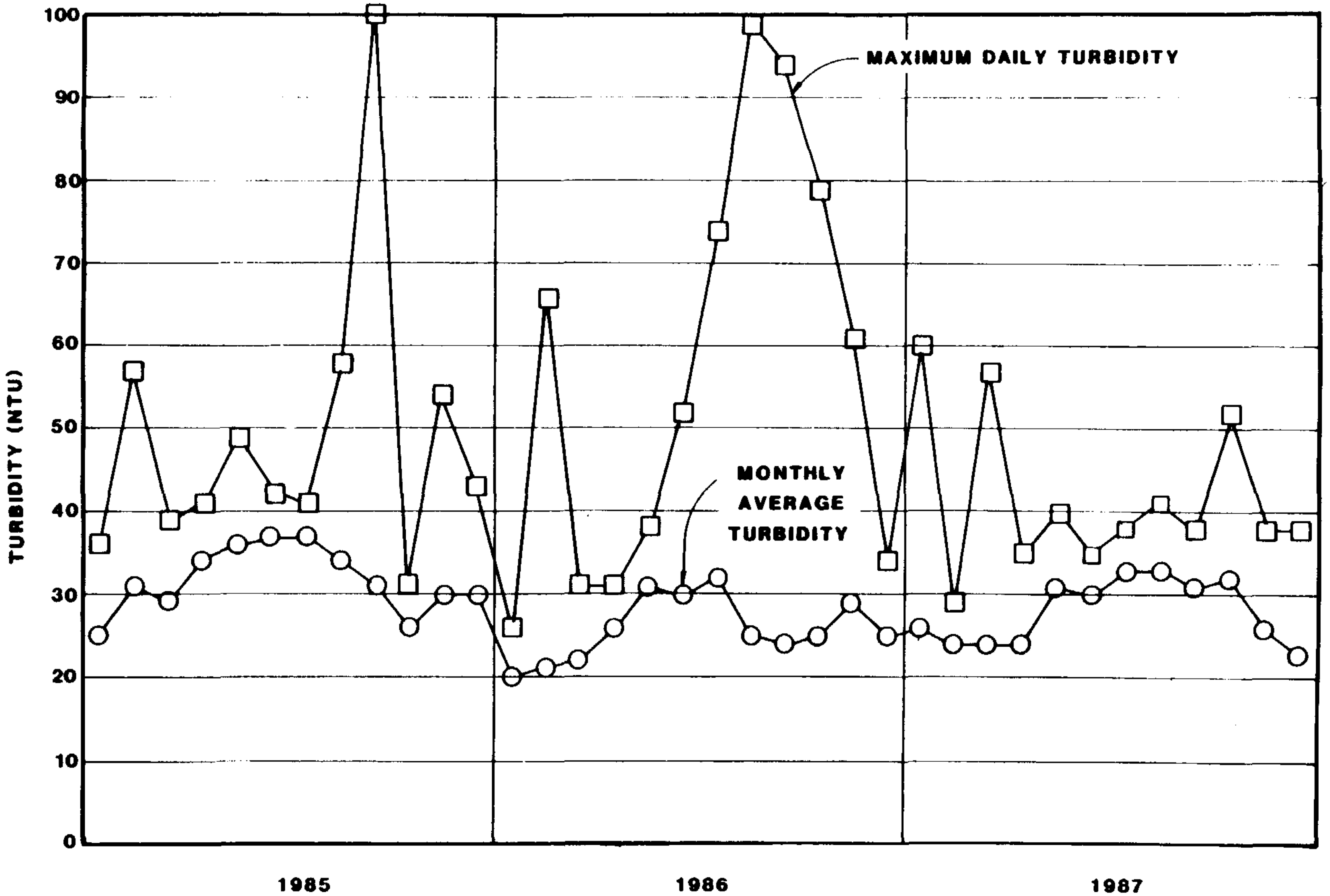

FIGURE 3-1

COOPER RESERVOIR
WATER STUDY
SULPHUR SPRINGS, TEXAS

TURBIDITY IN LAKE SULPHUR SPRINGS

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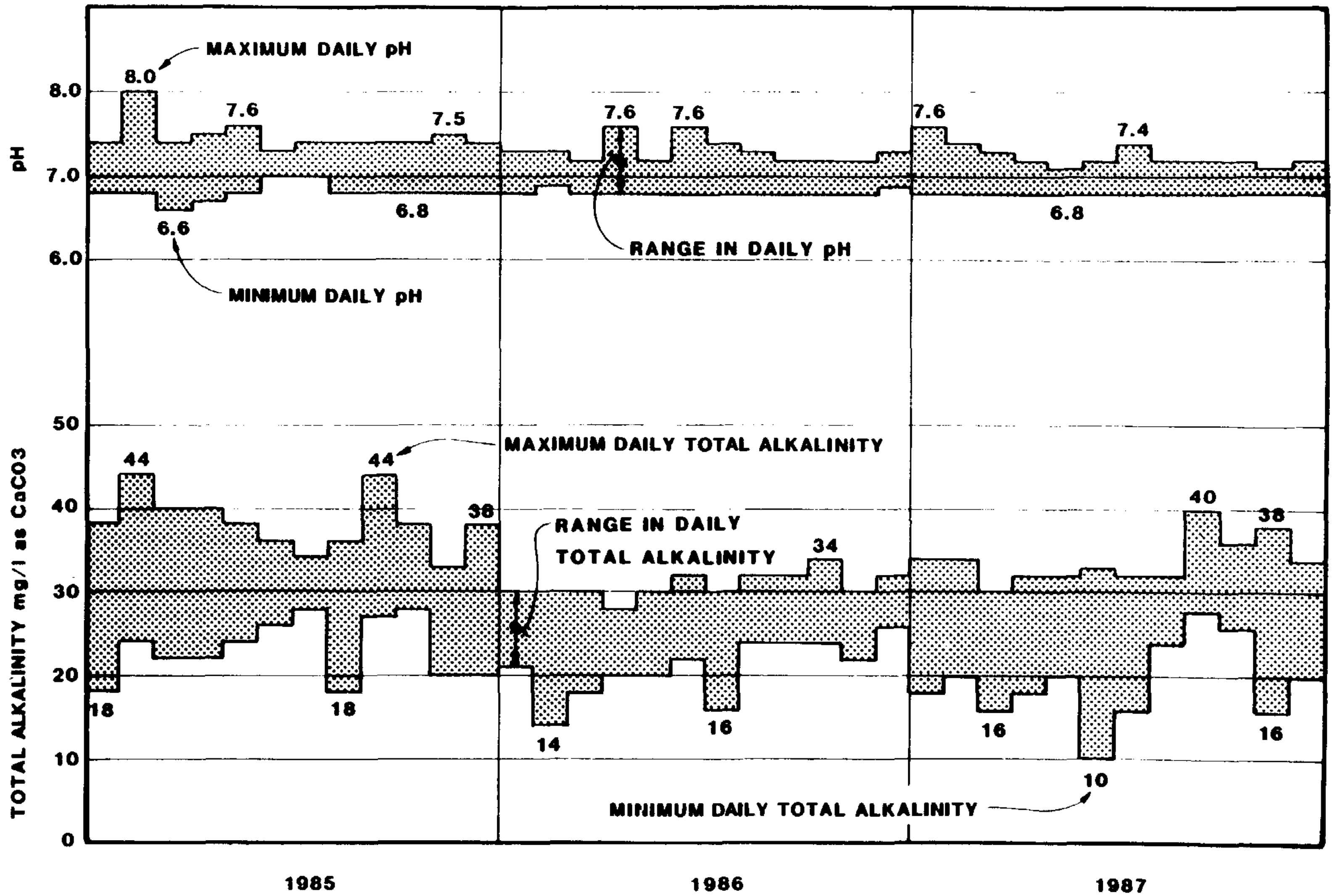


FIGURE 3-2

COOPER RESERVOIR
WATER STUDY
SULPHUR SPRINGS, TEXAS

pH & ALKALINITY
LAKE SULPHUR SPRINGS

BLACK & VEATCH DALLAS, TEXAS PROJECT NO. 14719.100

TABLE 3-1. WATER QUALITY IN LAKE SULPHUR SPRINGS

Constituent	Average	Range
Total Dissolved Solids, mg/l	65	53 - 81
Calcium, mg/l as Ca	9.1	5.8 - 14
Magnesium, mg/l as Mg	3.1	2.0 - 4.7
Sodium, mg/l as Na	5.3	3.4 - 8.3
Potassium, mg/l as K	4.2	3.3 - 5.6
Bicarbonate, mg/l as HCO_3	39	12 - 54
Sulfate, mg/l as SO_4	12	5.0 - 18
Chloride, mg/l as Cl	5.0	3.4 - 6.8
Fluoride, mg/l as F	0.2	0 - 0.3
Silica, mg/l as SiO_2	7.3	0.7 - 25
pH	7.0	6.6 - 8.0
Calcium Hardness, mg/l as $CaCO_3$	23	14 - 35
Magnesium Hardness, mg/l as $CaCO_3$	13	8 - 19
Total Alkalinity, mg/l as $CaCO_3$	32	10 - 44
Turbidity, NTU	29	12 - 100
Total Suspended Solids, mg/l	46	19 - 160
Volatile Suspended Solids, mg/l	15	6 - 51
Color, PCU	70	5 - 140
Total Organic Carbon, mg/l	12	6.3 - 20
Ammonia plus Organic Nitrogen, mg/l as N	1.4	0.9 - 2.2
Total Phosphorous, mg/l as P	0.30	0.1 - 0.5

Sources: Developed from data compiled by the USGS and the City of Sulphur Springs

Data are not available for the suspended solids and organic constituents in the lake, but the USGS operates a chemical monitoring station on the South Sulphur River near Cooper, which can be used to develop estimates of these parameters. Correlations between Total Suspended Solids (TSS), Volatile Suspended Solids (VSS), and turbidity in the river indicate the average ratio between TSS and turbidity is 1.6, and VSS are approximately 32 percent of TSS. These ratios, and the turbidity measurements made by the City, were used to develop the estimates for TSS and VSS in Table 3-1.

The USGS also monitors Color, Total Organic Carbon (TOC), Total Phosphorous, and the complete nitrogen series (nitrate, nitrite, ammonia, and organic nitrogen) on the South Sulphur River near Cooper. This data was used to develop estimates for these parameters in Lake Sulphur Springs.

Total phosphorous and ammonia, plus organic nitrogen concentrations observed in 43 samples, did not vary with the rate of flow in the river and were relatively constant year around. The averages and ranges in concentration for these parameters were, therefore, used as estimates for these constituents in Lake Sulphur Springs. Total Organic Carbon (TOC) concentrations had a much larger range in concentration, but there was very little correlation with flow or time of year. The average and range for the 36 TOC observations available from the river were used as estimates for TOC in the lake. Color measurements were available for 37 samples. There were very large increases in color when flows were high in the river. This produced seasonal variations because most of the high flows are runoff from spring rains, but high flows produced by thunderstorms later in the year also exhibited this characteristic to a lesser degree. Color measurements for all flows greater than 50 cfs were, therefore, excluded from computations used to produce the estimates given in Table 3-1.

Use of observations collected on the South Sulphur River for estimates of water quality in Lake Sulphur Springs is conservative because there is some sedimentation in the lake, and small decreases in nitrogen, phosphorous, and organics will occur. However, Lake Sulphur Springs is very shallow and the hydraulic detention time in this impoundment is quite short. Both of these conditions will limit the magnitudes of changes in the lake.

3.2.2 Cooper Reservoir

3.2.2.1 Quantity. Cooper Reservoir is under construction at the present time, with impoundment scheduled to begin in 1991. The dam site is located on the South Sulphur River at river mile 23.2, which is approximately 3.9 miles upstream from Texas State Highway No. 154 near Cooper, Texas. The drainage area is 476 sq mi and extends in a westerly direction from the dam site for approximately 38 miles.

The lake will be approximately 15 miles long and will provide 441,200 ac ft of storage at El 446.2, which will be the top of the flood control pool. The storage allocation will be 130,400 ac ft for flood control, 273,800 ac ft for water supply, and 37,000 ac ft for sediment. The top of the conservation pool will be at El 440.0 and water supply releases will be allowed down to El 415.5. Permission to obtain water below this elevation must be obtained in writing from the COE.

It is not known how the COE will regulate releases from the outlet works. However, the local interests contracting for water supply storage are required to furnish a low flow release of 5 cfs to maintain water quality downstream of the dam. Water supply releases, equal to a firm yield of 165 cfs (106.6 mgd), will be taken out of the lake upstream from the dam and will not pass through the outlet works.

Contracts for water supply storage in the lake have been consummated between the federal government and the Sulphur River Municipal Water District (SRMWD), the North Texas Municipal Water District, and the City of Irving. These contracts are written in terms of storage space rather than flow rates. The relationship between these parameters is summarized in Table 3-2.

TABLE 3-2. STORAGE CAPACITIES AND FIRM YIELDS FOR WATER SUPPLY FROM COOPER RESERVOIR

User	Percent of Total Storage	Usable Storage ac ft	Firm Yield cfs	Firm Yield mgd
Sulphur River Municipal Water District	26.282	71,750	43.365	28.02
North Texas Municipal Water District	36.859	100,625	60.817	39.31
City of Irving	36.859	100,625	60.817	39.31
Total	100.000	273,000	164.999	106.64

The SRMWD also holds Permit No. 2336 from the Texas Water Commission which is dated January 4, 1966. This permit authorizes diversions not to exceed 26,960 ac ft/yr (24.01 mgd) for municipal purposes, and 11,560 ac ft/yr (10.32 mgd) for industrial purposes within the service area of the District. Unconsumed water is to be returned to the Sulphur River Basin by the wastewater disposal plants operated by customers of the District. A maximum diversion rate is not specified in the permit.

At the present time, the SRMWD includes the cities of Sulphur Springs, Cooper, and Commerce. Division of the waters available to the SRMWD from Cooper Lake is summarized in Table 3-3. Water available to each of the member cities can be sold with the other members having the right of first refusal.

TABLE 3-3. DIVISION OF WATER AVAILABLE FROM COOPER RESERVOIR TO MEMBERS OF THE SULPHUR RIVER MUNICIPAL WATER DISTRICT

Member	Percent of Water Supply Storage	Firm Yield mgd
City of Sulphur Springs	47.1	13.20
City of Cooper	11.1	3.11
City of Commerce	41.8	11.71
Total	100	28.02

3.2.2.2 Quality. Initially, Cooper Reservoir will go through a period of transition, when water quality is changing in response to leaching of mineral and organic constituents from the soils, and decomposition of the vegetation being covered by water. With median inflow conditions, and normal variations in lake elevations, these initial effects should diminish after the conservation pool is filled in approximately five years. After this period, a more stable water quality regime will be established.

During the initial period, there will be an increase in Biochemical Oxygen Demand (BOD) in the overlying water, and this will produce a decrease in dissolved oxygen. However, there should be ample dissolved oxygen to support aquatic life in the top layer of water. With vegetative decay, there will also be an increase in phosphorous and total nitrogen which will stimulate algae growth. This will probably be more pronounced in the western end of the lake. Color in the lake during this period will also be greater than the intensity of color in the inflow. All of these factors will temporarily enhance biological productivity (zooplankton, phytoplankton, algae, benthos, fish, etc.).

The total effects on water quality, produced by changes occurring during initial filling and the times required for the water quality to stabilize, are dependent upon many physical, chemical, and climatological factors which cannot be controlled. Given the generally good quality of the inflowing waters, deterioration during this initial period should not be serious enough to pose insurmountable problems in water treatment. However, it will be necessary to continue the high level of plant operation being performed by the City at the present time.

Long-term changes in the lake will produce improvements to the quality observed in the South Sulphur River at the present time. There will be decreases in coliform bacteria, turbidity, color, suspended solids, BOD, nitrogen, and phosphorous. Variations in chemical quality observed in the river will also be decreased.

Data, collected by the USGS from the South Sulphur River near Cooper, was used to develop quantitative estimates of future inorganic chemical quantity. In addition to the collection of monthly grab samples, the USGS measures specific conductance continuously at this station. Correlations between specific conductance and the concentrations of TDS, chloride, sulfate, and total hardness have been prepared by the USGS, and are used by them to produce flow weighted estimates for concentrations of these ions. We used these flow weighted estimates to compute long-term annual means and ranges in concentration expected for these parameters in the water flowing into the lake, and then increased these values by five percent to account for evaporation while the water is in storage. (The average hydraulic retention time in the lake will be approximately 1.4 years).

Estimates, for dissolved ions not included in the USGS correlations with specific conductance, were developed by determining ratios between TDS/Sodium, total hardness/total alkalinity, calcium/total hardness, and calcium/magnesium. Potassium concentrations were essentially constant in the river water, and were averaged to determine the expected concentration in the lake. Chemical quality estimates for water in the lake are summarized in Table 3-4.

Table 3-4. ANTICIPATED WATER QUALITY IN COOPER RESERVOIR

Constituent	Average	Range
Total Dissolved Solids, mg/l	116	91 - 139
Calcium, mg/l as Ca	24	18 - 32
Magnesium, mg/l as Mg	2.4	1.8 - 3.3
Sodium, mg/l as Na	11	7.4 - 13
Potassium, mg/l as K	4.2	3.2 - 6.3
Bicarbonate, mg/l as HCO_3	87	45 - 89
Sulfate, mg/l as SO_4	16	12 - 21
Chloride, mg/l as Cl	7.5	3.9 - 9.9
Fluoride, mg/l F	0.2	0 - 0.3
Silica, mg/l as SiO_2	7.1	0.7 - 8.6
pH	7.0	6.5 - 8.1
Calcium Hardness, mg/l as $CaCO_3$	60	44 - 80
Magnesium Hardness, mg/l as $CaCO_3$	10	7 - 14
Total Alkalinity, mg/l as $CaCO_3$	71	55 - 109
Turbidity, NTU	9	4 - 20
Total Suspended Solids, mg/l	17	8 - 38
Volatile Suspended Solids, mg/l	13	6 - 29
Color, PCU	38	0 - 65
Total Organic Carbon, mg/l	8.7	5.6 - 14
Ammonia plus Organic Nitrogen, mg/l as N	1.2	0.7 - 1.9
Total Phosphorous, mg/l as P	0.12	0.08 - 0.17

Source: Developed from data compiled by the USGS

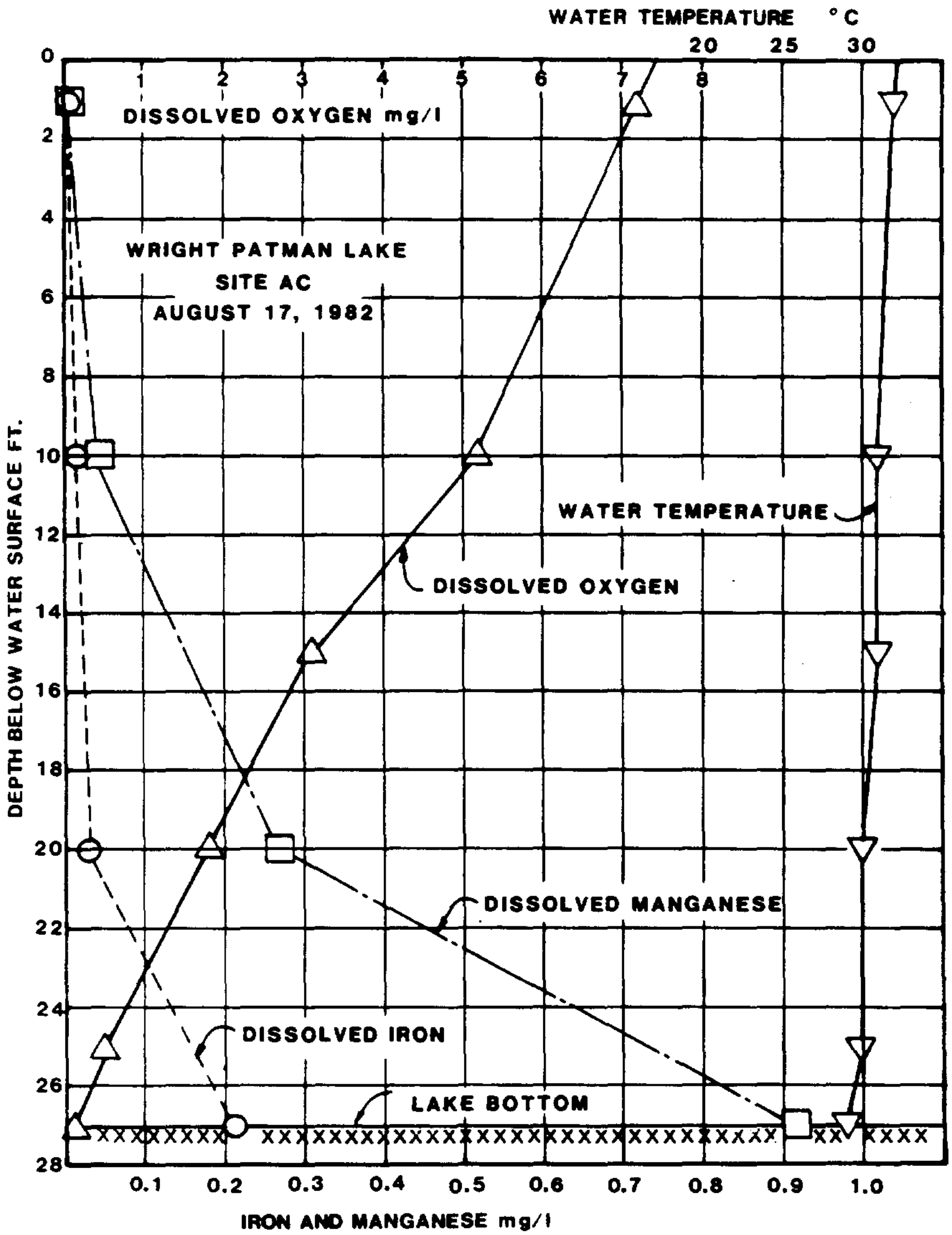
Estimates for turbidity, suspended solids, and the organic parameters TSS, VSS, Color, TOC, ammonia plus organic nitrogen, and total phosphorous included in Table 3-4, were developed by evaluating the changes which occurred in these parameters in Wright Patman Lake. Wright Patman Lake is located on the Sulphur River approximately 110 miles downstream of Cooper Reservoir. When necessary, this information was supplemented by data collected from the Sulphur River near Texarkana. This station is located approximately 0.4 mi downstream from the dam.

Observations obtained in Wright Patman Lake were also used to evaluate potential effects of thermal stratification in Cooper Reservoir.

This annual cycle is illustrated on Figure 3-3 which shows a typical summer profile, and Figure 3-4 which depicts winter conditions. A very small increase in temperature in the upper level of the lake will produce stratification and a marked decrease in dissolved oxygen at deeper depths. Dissolved oxygen levels below 1.5 to 1.0 mg/l will produce a dramatic increase in dissolved manganese, iron, and phosphorous during the summer. During the winter, the surface water cools and the lake will be completely mixed by the wind. This will bring dissolved oxygen back down to the bottom and precipitate the dissolved manganese and iron from solution. Thermal stratification will not pose problems in water treatment if the intake structure constructed in Cooper Lake is provided with multiple openings, which will allow the operator to select water from the depth which has the highest quality.

3.3 COMPARISONS OF WATER QUALITY

The information summarized in Tables 3-1 and 3-4 is essential to engineers designing municipal and industrial water treatment processes, but it is not particularly useful to interested citizens because the items of major importance tend to become lost in the details. The comparisons illustrated on Figures 3-5 and 3-6 should help in this regard.



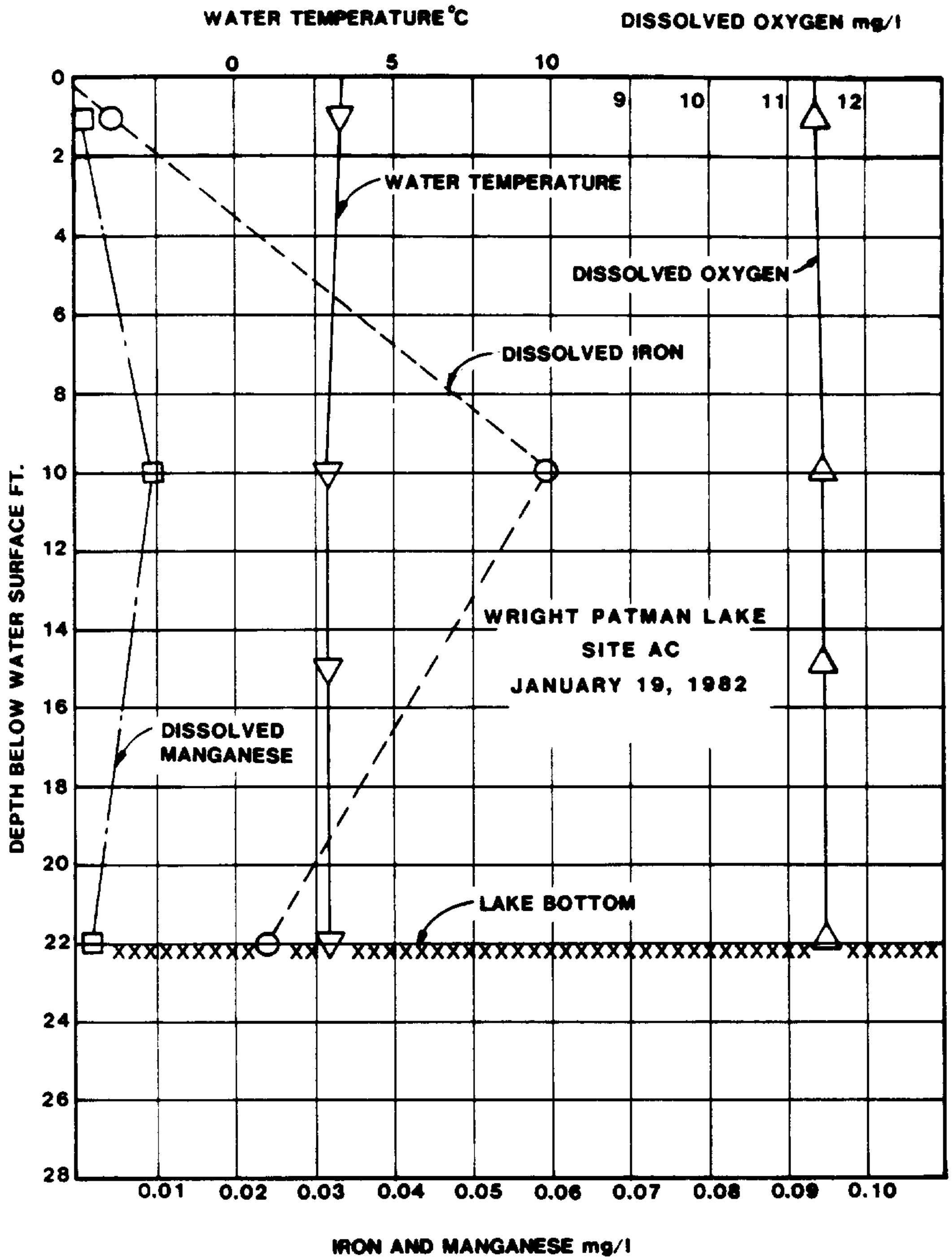
TYPICAL SUMMER PROFILE
EXPECTED IN COOPER LAKE

BLACK & VEATCH DALLAS, TEXAS

PROJECT NO. 14719.100

COOPER RESERVOIR
WATER STUDY
SULPHUR SPRINGS, TEXAS

FIGURE
3-3



NOTE: THE ENTIRE IRON AND MANGANESE SCALE ON THIS FIGURE IS EQUAL TO THE FIRST SUBDIVISION OF THE IRON AND MANGANESE SCALE IN FIGURE 3-3.

TYPICAL WINTER PROFILE EXPECTED IN COOPER LAKE

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BLACK & VEATCH DALLAS, TEXAS

COOPER RESERVOIR WATER STUDY SULPHUR SPRINGS, TEXAS

FIGURE 3-4

in the existing water plant can make the continued use of water from Lake Sulphur Springs, or use of a mixture of both waters, more feasible during this period. These improvements are discussed in Chapter 8.0.

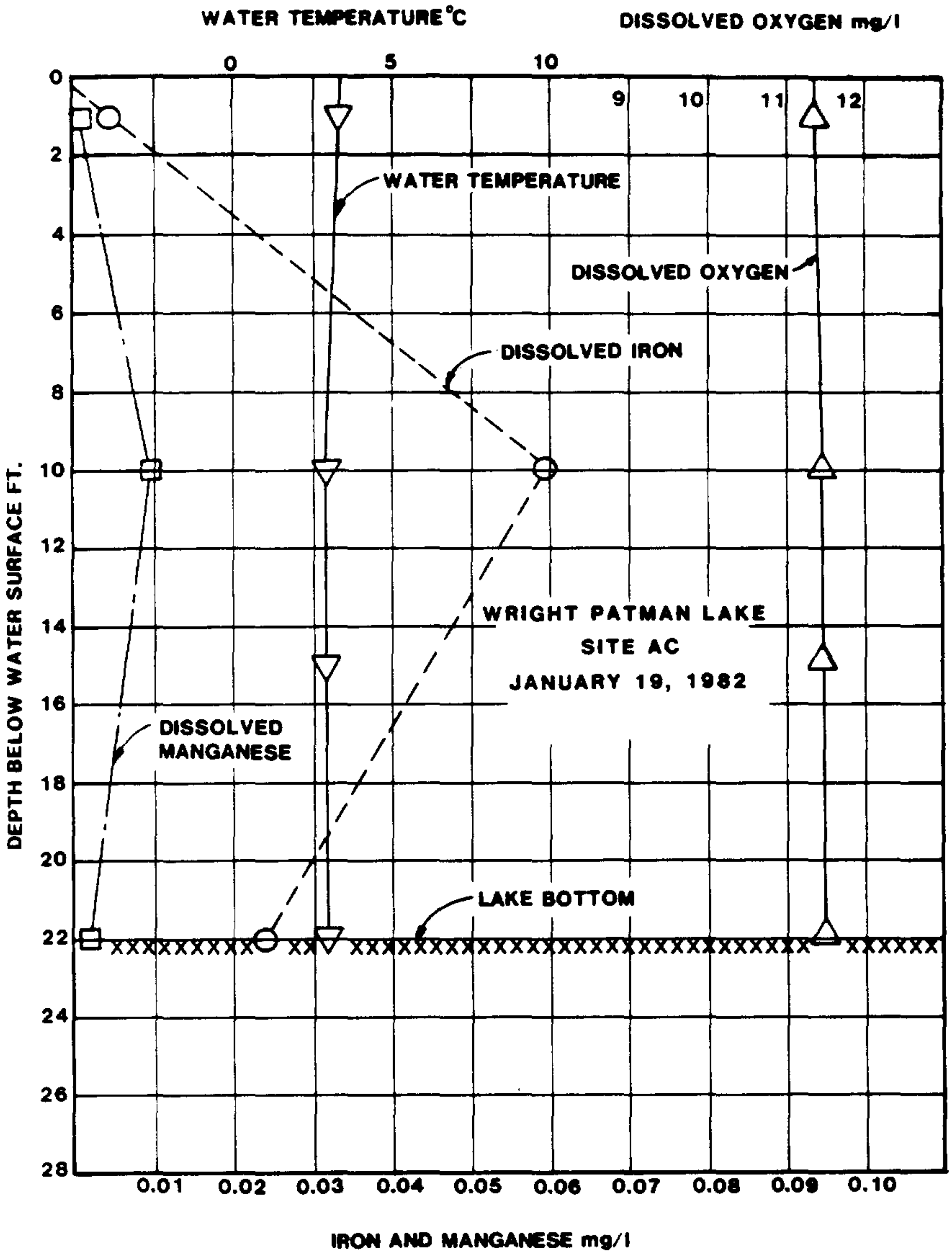
3.4 GROUNDWATER

3.4.1 Introduction

The major source of groundwater in Hopkins County is the Carrizo-Wilcox Aquifer, which outcrops in the south central portion of the county and downdips to the southeast. A minor aquifer, the Nacatoch Sand, outcrops to the north in Delta County and to the west in Hunt County. The downdip area for the Nacatoch Sand extends across the northern edge of Hopkins County, and it is found at lower depths in the western portion of the county. The locations of these aquifers are shown on Figure 3-7. Groundwater is now being used as a source of public water supply by two small cities and seven water supply corporations (WSC). In many instances, this water is being supplemented by treated surface water purchased from the City of Sulphur Springs. A total of 29 wells, with an aggregate capacity of 3.739 mgd, are now in use. The distribution of these wells is summarized in Table 3-5 and their locations are shown on Figure 3-8. Virtually all of the wells are located in the southern half of the county. They can be subdivided into wells tapping the Carrizo-Wilcox Aquifer and wells located in the southwestern corner of the county.

TABLE 3-5. PUBLIC WATER SUPPLY WELLS IN HOPKINS COUNTY.

Name of Owner	Number of Wells	Total Well Capacity mgd	Map Designation
Brinker WSC	3	0.468	D
City of Como	1	0.202	C
Cornersville WSC	2	0.317	A



NOTE: THE ENTIRE IRON AND MANGANESE SCALE ON THIS FIGURE IS EQUAL TO THE FIRST SUBDIVISION OF THE IRON AND MANGANESE SCALE IN FIGURE 3-3.

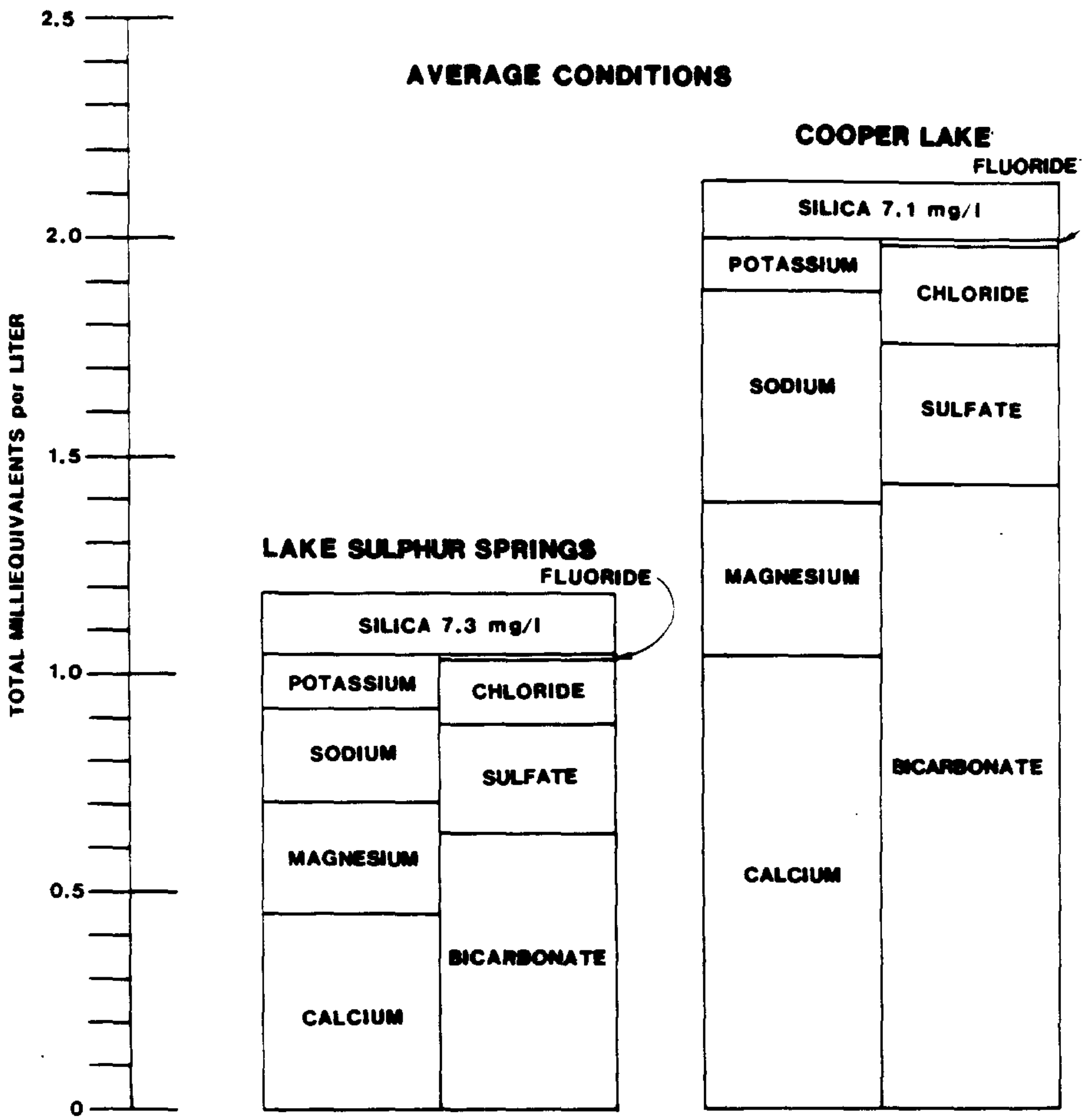
TYPICAL WINTER PROFILE
EXPECTED IN COOPER LAKE

BLACK & VEATCH DALLAS, TEXAS

PROJECT NO. 14719.100

COOPER RESERVOIR
WATER STUDY
SULPHUR SPRINGS, TEXAS

FIGURE
3-4



	Lake Sulphur Springs	Cooper Lake
TOTAL DISSOLVED SOLIDS	65 mg/l	116 mg/l
TOTAL HARDNESS	37 mg/l as $CaCO_3$	70 mg/l as $CaCO_3$
TOTAL ALKALINITY	32 mg/l as $CaCO_3$	71 mg/l as $CaCO_3$

CHEMICAL QUALITY, LAKE SULPHUR SPRINGS & COOPER LAKE

BLACK & VEATCH DALLAS, TEXAS

PROJECT NO. 14719.100

COOPER RESERVOIR WATER STUDY SULPHUR SPRINGS, TEXAS

FIGURE 3-5

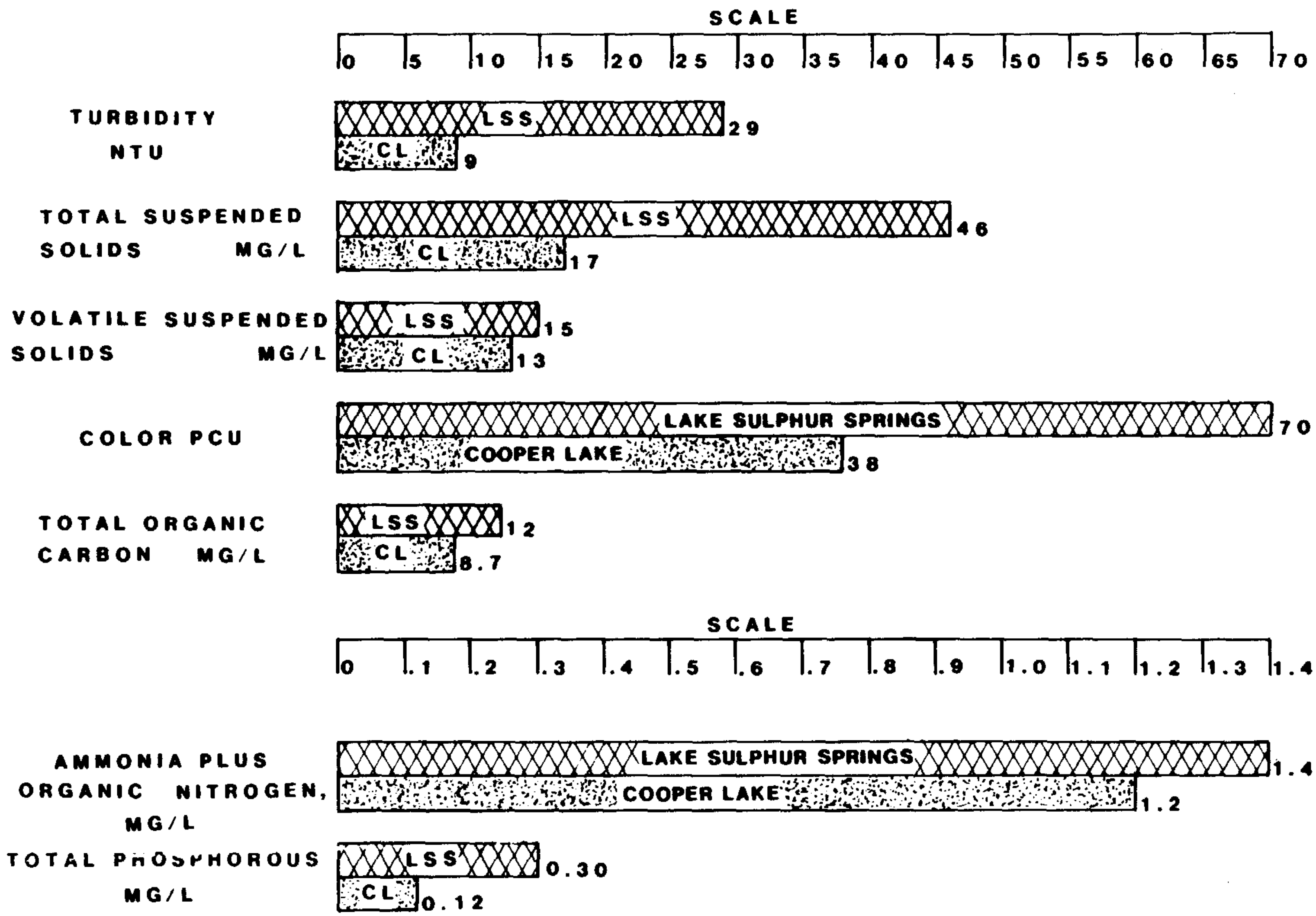


FIGURE 3-6

COOPER RESERVOIR
WATER STUDY
SULPHUR SPRINGS, TEXAS

SUSPENDED SOLIDS & ORGANIC QUALITY, LAKE SULPHUR SPRINGS & COOPER LAKE

BLACK & VEATCH DALLAS, TEXAS

PROJECT NO. 14719.100

The average concentrations of TDS, Total Hardness, and Total Alkalinity will be almost twice as large in Cooper Lake as they are in Lake Sulphur Springs. This is not a drawback! It will make the water from Cooper Reservoir easier to treat, more palatable, and more stable in the distribution system. The only change required in the treatment process now being used will be lowering the finished water pH from its current level of 8.6 to 8.8 down to approximately 8.0. This will not detract from the overall quality.

However, the big advantages in water from Cooper Reservoir versus water from Lake Sulphur Springs are shown on Figure 3-6. Turbidity and TSS will be about one third lower. There will not be much difference in VSS, but the color will only be about one half the intensity seen in Lake Sulphur Springs. The TOC will be lower by approximately one third. Ammonia plus organic nitrogen will be similar in both waters, but the concentration of total phosphorous in water obtained from Cooper Reservoir will be about one half the phosphorous concentration in Lake Sulphur Springs.

Water from both lakes can be intermixed in any proportion without producing problems in chemical quality. After the short-term organics conditions have stabilized in Cooper Reservoir (see discussion in Section 3.2.2.2), the frequency and intensity of tastes and odors in this water will be much less than they now are in Lake Sulphur Springs. The changes in disinfection practices required by the U.S. Environmental Protection Agency (USEPA), in the proposed Surface Water Treatment Rule (see Chapter 4.0), provides an opportunity to correct the problems which may occur in this area at both reservoirs.

The waters from both lakes can continue to be used during the winter months. However, the chemical costs for treatment of water from Cooper Reservoir will be lower than the costs for treatment of water from Lake Sulphur Springs. Treatment of water from Cooper Lake will be much easier and less expensive during the summer. However, improvements in the existing water plant can make the continued use of water from Lake Sulphur Springs, or use of a mixture of both waters, more feasible during this period. These improvements are discussed in Chapter 8.0.

3.4 GROUNDWATER

3.4.1 Introduction

The major source of groundwater in Hopkins County is the Carrizo-Wilcox Aquifer, which outcrops in the south central portion of the county and downdips to the southeast. A minor aquifer, the Nacatoch Sand, outcrops to the north in Delta County and to the west in Hunt County. The downdip area for the Nacatoch Sand extends across the northern edge of Hopkins County, and it is found at lower depths in the western portion of the county. The locations of these aquifers are shown on Figure 3-7. Groundwater is now being used as a source of public water supply by two small cities and seven water supply corporations (WSC). In many instances, this water is being supplemented by treated surface water purchased from the City of Sulphur Springs. A total of 29 wells, with an aggregate capacity of 3.739 mgd, are now in use. The distribution of these wells is summarized in Table 3-5 and their locations are shown on Figure 3-8. Virtually all of the wells are located in the southern half of the county. They can be subdivided into wells tapping the Carrizo-Wilcox Aquifer and wells located in the southwestern corner of the county.

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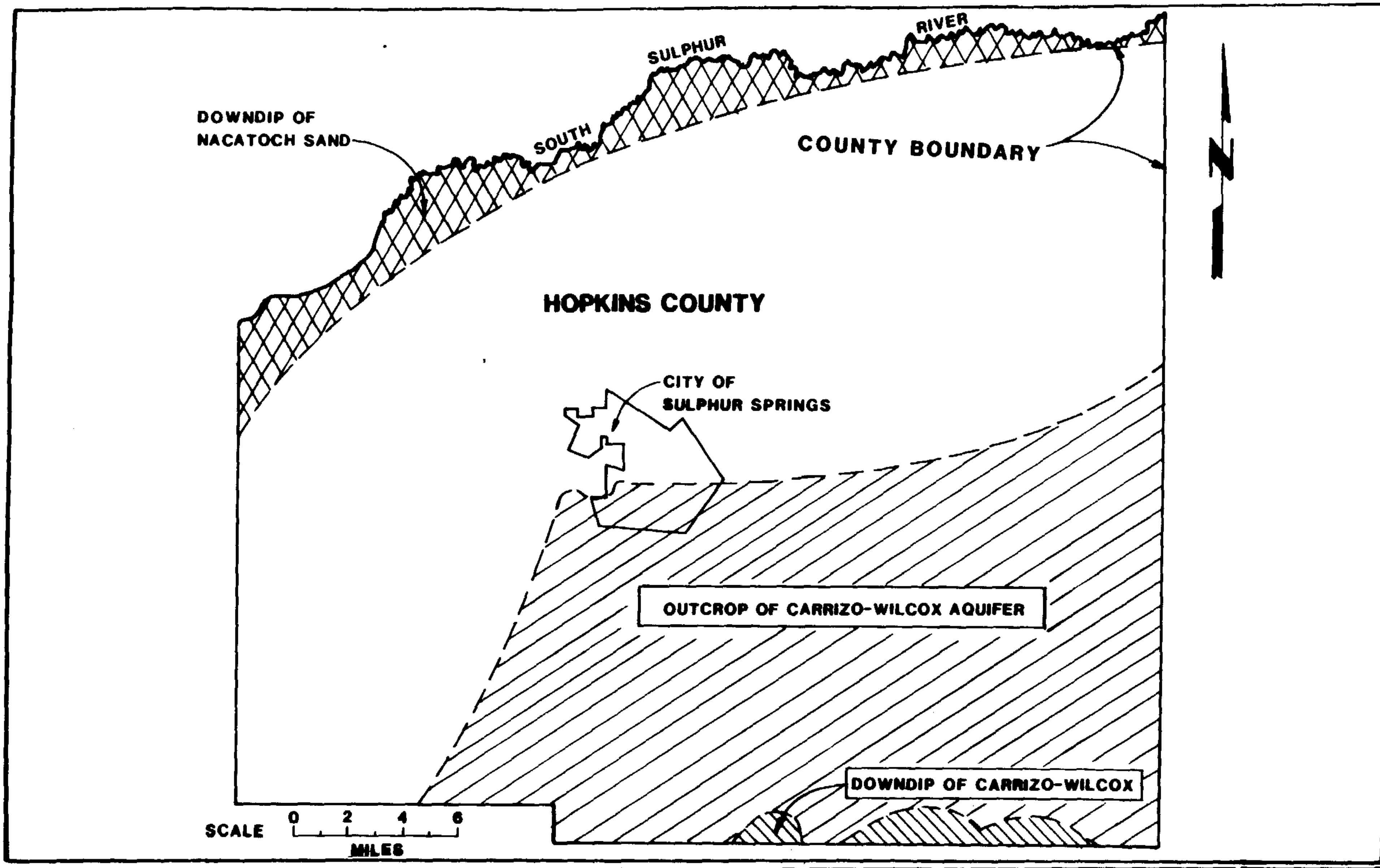


FIGURE 3-7

COOPER RESERVOIR
WATER STUDY
SULPHUR SPRINGS, TEXAS

GROUND WATER AQUIFERS
IN HOPKINS COUNTY

BLACK & VEATCH DALLAS, TEXAS PROJECT NO. 14719.100

TABLE 3-5. (Continued) PUBLIC WATER SUPPLY WELLS IN HOPKINS COUNTY.

Name of Owner		Number of Wells	Total Well Capacity mgd	Map Designation
City of Cumby		4	0.370	I
Gafford Chapel WSC		3	0.408	G
Martin Springs WSC		6	0.871	E
Miller Grove WSC		4	0.233	H
Pickton WSC		1	0.144	B
Shirley WSC		5	0.726	F
	Total	29	3.739	

3.4.2. Wells in the Carrizo-Wilcox Aquifer.

A total of 18 public water supply wells are currently extracting water from the Carrizo-Wilcox Aquifer. The minimum well depth is 300 ft, the average is 398 ft, and the maximum is 600 ft. The average capacity is 104, gpm with a range from 34 to 150 gpm.

Water quality in these wells is monitored by the Texas Department of Health (TDH). The latest analyses in their files are summarized in Table 3-6. The quality is remarkably uniform throughout this portion of the aquifer, with very low hardness, sulfate, and chloride. The average TDS for all of the wells is 229 mg/l, but this average is skewed upward by the well operated by Pickton WSC, which has a TDS of 417 mg/l. If this well is excluded, the average TDS drops to 191 mg/l.

The major water quality problems are occasional wells with high concentrations of iron or manganese, and corrosiveness produced by the very low hardness. The iron or manganese is typically oxidized with chlorine or sequestered.

The corrosiveness is minimized by the naturally high pH of the water which ranges from 8.0 to 8.8.

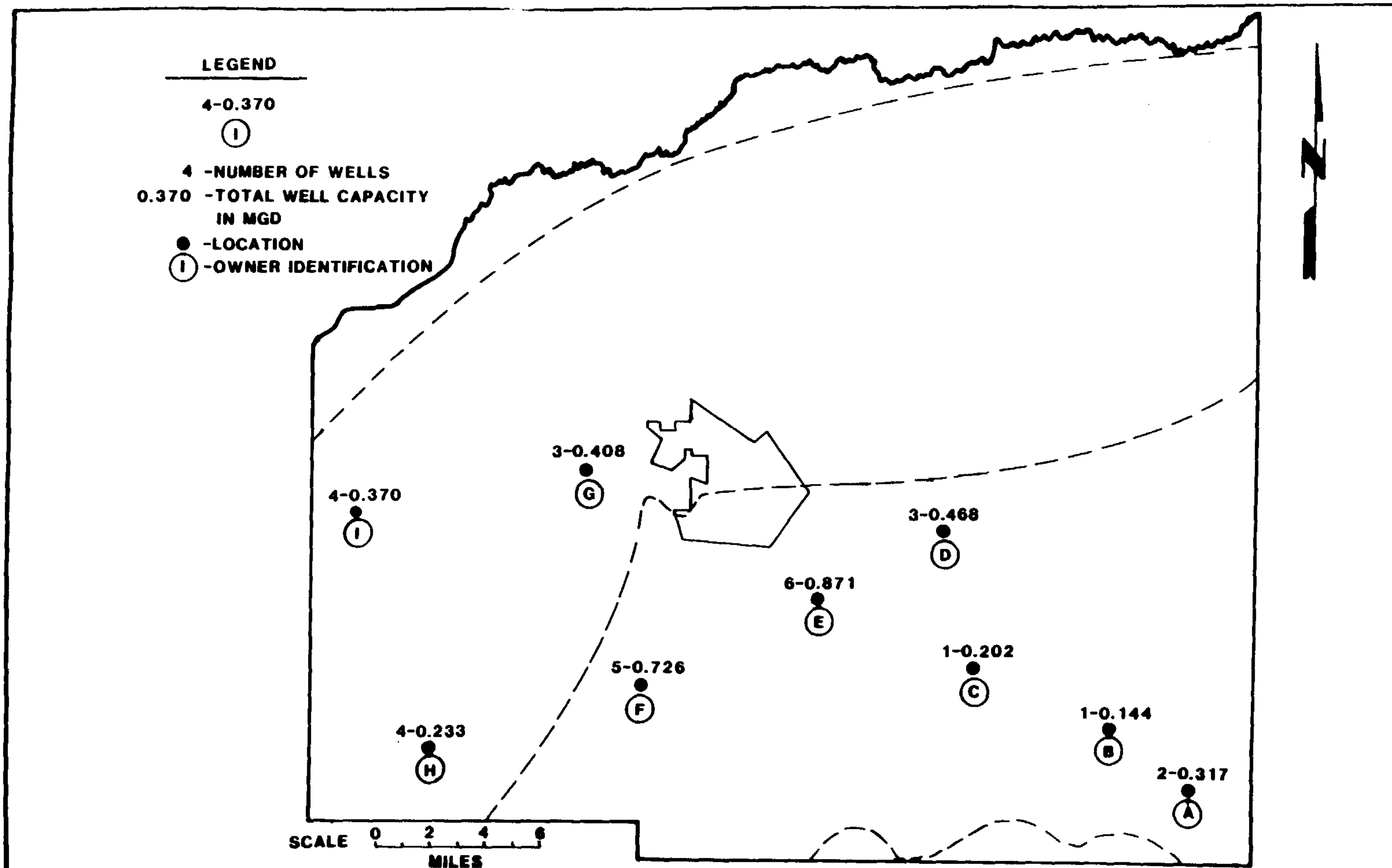


FIGURE 3-8

COOPER RESERVOIR
WATER STUDY
SULPHUR SPRINGS, TEXAS

PUBLIC WATER SUPPLY WELLS
IN HOPKINS COUNTY

BLACK & VEATCH DALLAS, TEXAS

PROJECT NO. 14719.100

Approximately 11,200 ac-ft of groundwater were used in the Sulphur River Basin in 1980, and over 50 percent of this total was obtained from the Carrizo-Wilcox Aquifer. Most of this demand occurred east of Hopkins County, and overdrafts were reported in Bowie, Cass, Franklin and Morris counties. Information is not available on the safe yield of this aquifer in Hopkins County, but the installed capacity of the public water supply wells is about 3,100 ac-ft/yr. This should not pose problems but additional development may be limited in specific locations.

3.4.3 Wells in Southwestern Hopkins County

The City of Cumby, Gafford Chapel WSC, and Miller Grove WSC have developed 11 wells in southwestern Hopkins County. These are deeper wells ranging from 640 to 1,060 ft, with an average depth of 824 ft. The capacities are also significantly less than those in the Carrizo-Wilcox Aquifer, with an average of 53 gpm and a range from 15 to 90 gpm.

Quality of the waters obtained from these wells is summarized in Table 3-7. The water is very low in hardness, but concentrations of most of the other dissolved solids are higher than they are in water obtained from the Carrizo-Wilcox. The pH is also higher, ranging from 8.4 to 9.2. Iron and manganese are quite low.

The newest public water supply wells in the county are Wells 4 and 5, which were completed by Miller Grove WSC in 1987. Both wells are approximately 1,000 ft deep and have capacities of 30 gpm or less. The TDH has ordered Well 4 abandoned because of the high fluoride concentration. However, it is being used temporarily until Well 5 is placed in production.

The Nacatoch Sand is reported to be overdrafted in Hunt County, which is updip of the wells in southwestern Hopkins County. The effect of this situation on the long-term yield from these wells is not known.

However, the chemical quality of all of the wells in this portion of the county is significantly lower than the quality of the surface water available from the City of Sulphur Springs. This should produce a gradual transfer from groundwater to surface water in this region.

4.0 WATER QUALITY REQUIREMENTS

4.1 BACKGROUND

Prior to passage of the Safe Drinking Water Act by the United States Congress in 1974, the individual states established their own standards for drinking water. Federal involvement in this area was limited to water used on interstate carriers. Federal standards were developed by the U.S. Public Health Service in 1914 and revised in 1925, 1942, 1946, and 1962. These standards formed the basis of almost all the regulations adopted by individual states. The federal standards specified physical, chemical, and radiological quality as well as bacterial limits. Two types of criteria were used:

(1) Limits which, if exceeded, were grounds for rejection of the supply because of potentially adverse health effects.

(2) Limits which should not be exceeded because they produced effects which were objectionable to users. These included substances which produced tastes, odors, and stains which were not desirable, but did not directly impair health.

The Safe Drinking Water Act of 1974 changed the federal/state relationship by making federal standards the minimum requirements for the entire country. The act also sharpened the distinction between primary and secondary drinking water regulations. These differences are:

(1) Primary standards are mandatory criteria designed to protect public health. They must be adopted by the states.

(2) Secondary standards are aesthetic criteria intended to make a water more desirable to the consumer. Concentrations in excess of the federal secondary standards may be established by individual states.

Initial action taken under the 1974 act was adoption of the 1962 U.S. Public Health Service Standards as interim regulations. These criteria were subsequently modified by adoption of total trihalomethane (TTHM) regulations, relaxation of fluoride limits, and adoption of

Maximum Contaminant Levels (MCLs) for eight volatile organic chemicals (VOCs). The primary standards currently in effect are summarized in Table 4-1. The federal secondary standards are shown in Table 4-2.

4.2 1986 AMENDMENTS TO THE SAFE DRINKING WATER ACT

The USEPA was moving toward adoption of additional standards under the 1974 law, but their rate of progress was not acceptable to the U.S. Congress. This dissatisfaction lead to passage of amendments to the Safe Drinking Water Act (SDWA) in 1984. These amendments are extremely broad in scope, and they require rapid implementation of the new regulations. The most significant changes for individual water supply utilities are:

- A greatly expanded list of contaminants, which are to be regulated, was identified by Congress, and a mandated schedule for increasing the number of regulated contaminants by 25 compounds every three years was included in the legislation.
- A schedule for adoption of filtration and disinfection requirements, as embodied in the proposed Surface Water Treatment Rule (SWTR), was established. This rule contains criteria for identifying water sources which must include filtration as part of the treatment provided, sets the maximum turbidity allowed in the finished water, establishes the level of disinfection required during treatment, and identifies the minimum concentrations of residual disinfectant required in the distribution system.
- Establishment of maximum concentrations for disinfectant residuals and maximum allowable concentrations of byproducts produced by disinfection.
- Reduction of the maximum contaminant level for lead to the lowest possible concentration and requirement of strict public notification procedures for this metal.
- Establishment of mandatory enforcement mechanisms and greatly increased fines for violations.

TABLE 4-1. NATIONAL PRIMARY DRINKING WATER REGULATIONS (NPDWRs)

Constituent	Maximum Contaminant Level
Arsenic (As), mg/l	0.05
Barium (Ba), mg/l	1.0
Cadmium (Cd), mg/l	0.01
Chromium (Cr), mg/l	0.05
Lead (Pb), mg/l	0.05
Mercury (Hg), mg/l	0.002
Nitrate (as N), mg/l	10.0
Selenium (Se), mg/l	0.01
Silver (Ag), mg/l	0.05
Fluoride (F), mg/l	4
Endrin, mg/l	0.0002
Lindane, mg/l	0.004
Toxaphene, mg/l	0.005
2,4-D, mg/l	0.1
2,4, 5-TP (Silvex), mg/l	0.01
Methoxychlor, mg/l	0.1
Total Trihalomethanes	0.10
Turbidity, NTU	1
Coliforms, per 100 ml	1
Gross Alpha particle Activity, pCi/l	15
Gross Beta and Proton Activity, pCi/l	50
Benzene, mg/l	0.005
Vinyl Chloride, mg/l	0.002
Carbon Tetrachloride, mg/l	0.005
1,2-Dichloroethane, mg/l	0.005
Trichloroethylene, mg/l	0.005
p-Dichlorobenzene, mg/l	0.075
1,1-Dichloroethylene, mg/l	0.007
1,1,1-Trichloroethane, mg/l	0.2

Source: USEPA

TABLE 4-2. NATIONAL SECONDARY DRINKING WATER STANDARDS

Constituent	Maximum Contaminant Level
Chloride (Cl), mg/l	250
Color (Color Units)	15
Copper (Cu), mg/l	1
Corrosivity	Non Corrosive
Fluoride (F), mg/l	2
Foaming Agents, mg/l	0.5
Iron (Fe), mg/l	0.3
Manganese (Mn), mg/l	0.05
Odor (TON)	3
pH	4.5 - 8.5
Sulfate (SO_4), mg/l	250
Total Dissolved Solids (TDS), mg/l	500
Zinc (Zn), mg/l	5

Source: USEPA

4.2.1 National Primary Drinking Water Regulation

There were 22 parameters included in the National Primary Drinking Water Regulation (NPDWR) when the 1986 amendments were passed. The law called for nine additional parameters by June 1987 to produce the 31 contaminants now included in the NPDWR. These are summarized in Table 4-1. An additional 40 parameters are scheduled for promulgation by June 1988, and 34 more will follow by June 1989. These two groups of contaminants, plus the nine announced in June 1987, are the 83 contaminants specifically identified in the 1986 amendments. The USEPA was given some latitude to make changes in this list, and announced seven substitutions in July 1987; but there will be at least 83 contaminants covered by the NPDWR in the near future.

Identifying the individual compounds to be included in the 1988 and the 1989 lists is difficult because the evaluation process is going on at the present time. However, there are some indications of the relative status of individual parameters.

Individual utilities were required to start monitoring for the 36 contaminants included in Table 4-3 on January 1, 1988. (In Texas, this monitoring is being done by the TDH.) Each state also had the option of requiring monitoring for any or all of the 15 contaminants included in Table 4-4. Contaminants scheduled for regulation are identified in Tables 4-4 and 4-5. This sequence of tables (4-3 through 4-6) is a rough gradation of the relative positions these contaminants occupy in the regulatory process.

In addition to the contaminants listed in Tables 4-1, 4-3, 4-4, 4-5, and 4-6, the 1986 amendments also require the USEPA to regulate 25 additional contaminants every three years starting in January 1988. This requirement for mandatory additions does not have a cutoff date at the present time, so it could go on until there are several hundred regulated contaminants.

TABLE 4-3. REQUIRED MONITORING FOR UNREGULATED CONTAMINANTS

Parameter	Parameter
Chloroform*	1,1,2,2 - Tetrachloroethane
Bromodichloromethane*	Ethylbenzene
Chlorodibromomethane*	1,3 - Dichloropropane
Bromoform*	Styrene
Trans - 1,2 - Dichloroethylene	Chloromethane
Chlorobenzene	Bromomethane
m - Dichlorobenzene	1,2,3 - Trichloropropane
Dichloromethane	1,1,1,2 - Tetrachloroethane
cis - 1,2 - Dichloroethylene	Chloroethane
Dibromonethande	2,2 - Dichloropropane
o - Dichlorobenzene	1,1,2 - Trichloroethane
1,1 - Dichloropropene	o - Chlorotoluene
Tetrachloroethylene	p - Chlorotoluene
Toluene	Bromobenzene
p - Xylene	1,3 - Dichloropropene
o - Xylene	Ethylene Dibromide (EDB)**
m - Xylene	1,2 - Dibromo - 3 - Chloropropane (DBCP)**
1,1 - Dichloroethane	
1,2 - Dichloropropane	

* Data should be available from THM monitoring.

** Required only for vulnerable systems as identified by the state. Mandatory detection level of 0.00002 mg/l.

TABLE 4-4. ADDITIONAL MONITORING FOR UNREGULATED CONTAMINANTS*

Parameter
1,2,4 - Trimethylbenzene
1,2,4 - Trichlorobenzene
1,2,3 - Trichlorobenzene
n - Propylbenzene
n - Butylbenzene
Naphthalene
Hexachlorobutadiene
1,3-5 - Trimethylbenzene
p - Isopropyltoluene

Parameter
Isopropylbenzene
Tert - Butylbenzene
Sec - Butylbenzene
Fluorotrichloromethane
Dichlorodifluoromethane
Bromochloromethane

---

* Required at the discretion of the state.

TABLE 4-5. ADDITIONAL CONTAMINANTS WHICH ARE SCHEDULED FOR REGULATION

Parameter	Parameter
Methylene Chloride	Alachlor
Aldicarb Sulfoxide	Epichlorohydrin
Aldicarb Sulfone	Adipates
Antimony	2,3,7, 8 - TCDD (Dioxin)
Asbestos	Vydate
Sulfate	Simazine
Copper	PAH's
Heptachlor	PCB's
Heptachlor Epoxide	Atrazine
Nickel	Phthalates
Thallium	Acrylamide
Beryllium	Pentachlorophenol
Cyanide	Pichloram
Aldicarb	Dinoseb
Chlordane	Hexachlorocyclopentadiene
Dalapon	Nitrite
Diquat	Uranium
Endothall	Radon
Glyphosate	
Carbofuran	

TABLE 4-6. CONTAMINANTS WHICH MAY BE REGULATED (USEPA SAFE DRINKING WATER PRIORITY LIST)

Parameter	Parameter
Zinc	Chloralhydrate
Silver*	2,4 - Dichlorophenol
Sodium	Chloropichrin
Aluminum	2,4 - Dinitrotoluene
Molybdenum	1,3 - Dichloropropane
Vanadium	Bromobenzene**
Dibromomethane**	Chloromethane**
Chlorine	Bromomethane**
Hypochlorite Ion	Chloroethane**
Chlorine Dioxide	1,1 - Dichloroethane**
Chlorite	1,2,3 - Trichloropropane**
Chloramine	1,1,1,2 - Tetrachloroethane**
Ammonia	2,2 - Dichloropropane**
Ozone	2,2 - Dichloropropane**
Chloroform**	o - Chlorotoluene**
Bromoform**	p - Chlorotoluene**
Bromodichloromethane**	1,1 - Dichloropropene**
Dibromochloromethane**	1,3 - Dichloropropene**
Dichloroiodomethane	2,4,5 - T
Bromochloroacetonitrile	Isophorone
Dichloroacetonitrile	Ethylene Thiourea
Dibromacetonitrile	Boron
Monochloroacetic Acid	Strontium*
Dichloroacetic Acid	Cryptosporidium
Trichloroacetic Acid	

* Currently regulated.

** On unregulated contaminant monitoring list.

The 1986 amendments also change the administrative process used to establish allowable concentrations of contaminants. Each compound is evaluated in terms of its potential effects to health, and a Maximum Contaminant Level Goal (MCLG) is established which represents the level which produces zero-risk. A MCL is also established. This is the lowest concentration which can be achieved using the Best Available Technology (BAT) for removal of the contaminant. The BAT used to establish the MCL must be identified by the USEPA, and the MCL must be as close to the MCLG as possible. Cost is a criterion in establishing a MCL, but it is not an overriding factor. Granular activated carbon (GAC) and packed-tower aeration have been identified as BAT for the VOCs included in Table 4-1. GAC will probably also be considered BAT for many of the synthetic organic compounds (SOCs) in Tables 4-3 through 4-6.

4.2.2 Surface Water Treatment Rule

The SWTR establishes criteria for determining when filtration is required in water treatment, the maximum turbidities allowable in the finished water, and the levels of disinfection which must be provided to this water. A draft of this rule was published in the Federal Register on November 3, 1987, for comment. The comment period was originally scheduled for 60 days after publication but was extended in January. The final SWTR should be promulgated by the end of 1988. All water utilities must be in compliance with the final rule by December 1992.

Raw water obtained from both of the sources available to the City of Sulphur Springs will require filtration. The proposed SWTR will require filtered water turbidities less than or equal to 0.5 NTU in 95 percent of the samples collected each month. This is the proposed MCL; a MCLG has not been proposed. Sampling must be done at least once every four hours, and the values obtained must be reported to the state regulatory agency. Continuous monitoring may be substituted for grab samples, if the monitoring instrument is calibrated at least twice per week. The sampling point can be located where the combined filter effluent enters the clearwell, the clearwell effluent, or the discharge from the high service pump station immediately prior to entry into the distribution system.

The proposed SWTR includes provisions for individual states to relax the turbidity standard to 1.0 NTU or less in 95 percent of the samples collected. However, doing this requires onsite studies which demonstrate effective removal and/or inactivation of Giardia lambia cyst sized particles at filtered turbidity levels above 0.5 NTU. This provision allows the state to take disinfection into account in determining the overall performance of the system. However, the maximum filtered water turbidity cannot exceed 5 NTU at any time, and all systems are expected to optimize treatment to achieve the lowest turbidity possible.

Disinfection is handled in the proposed SWTR by requiring compliance with "CT" values, where C represents the concentration of the disinfectant in milligrams per liter (mg/l) and T is the contact time in minutes (min) prior to delivery of water to the first customer. In addition, a detectable disinfectant residual must be present in more than 95 percent of the samples collected in the distribution system each month, for two consecutive months. Sites that do not have detectable residuals but have Heterotrophic Plate Count (HPC) measurements of less than 500/milliliter (ml) would be equivalent to sites with detectable residuals for purposes of determining compliance. (Recent changes in the Texas Department of Health [TDH] rules will require a minimum residual of 0.5 mg/l in the distribution system).

The CT values must be computed daily. The C value is the concentration measured at the end of the contact basin (if one is used), or the concentration immediately prior to the first customer. The T value is the hydraulic detention time, during peak hourly flow, between addition of the disinfectant and the location where C is measured. The T must be measured by tracer studies when open basins are used for disinfection. The minimum allowable product produced when C is multiplied by T is specified in the proposed rule. It is dependent on the disinfectant used, water temperature, pH (if chlorine is the disinfectant), and the removal and/or inactivation required for Giardia cysts and enteric viruses. At a minimum, the overall treatment system must achieve a 3 log (99.9 percent) removal and/or inactivation of Giardia cysts, and a 4 log (99.99 percent) removal and/or inactivation of enteric viruses. However, the level of disinfection required is commensurate with the degree of contamination in the source water. The parameter used to measure contamination is the Total Coliform concentration. The minimums cited above are for raw waters containing a geometric mean of less than 100 Total Coliform per 100 milliliters (ml) of water. Higher removal/inactivation levels are required when the Total Coliform concentration is greater than this value. Filtration, which meets the turbidity performance levels discussed earlier, is assumed to achieve 2 log (99 percent) removal of Giardia cysts and a 1 log (90 percent) removal of enteric viruses. The remaining organisms must be inactivated by disinfection.

The proposed SWTR contains CT tables for free chlorine, chlorine dioxide, and ozone. All three of these disinfectants can be used against both Giardia cysts and enteric viruses. The CT values required for inactivation of viruses by chloramines are extremely large, so it is not practical to use chloramines as primary disinfectants. Chloramines can be used to maintain a residual in the distribution system after the initial kill is obtained with chlorine, chlorine dioxide, or ozone.

4.2.3 Disinfectant Residuals and Byproducts

Proposed regulations covering maximum concentrations for disinfectant residuals and byproducts are scheduled for release in January 1990. Final regulations will be promulgated by January 1991 and are scheduled to become effective by January 1992.

All of the disinfectants available to the water supply industry at the present time pose some risk; however, the greatest potential problem is associated with chlorine dioxide. The chemical itself, and its byproducts, chlorite and chlorate, have produced hemolytic anemia in test animals. Experimental studies with laboratory test animals have also reported reproductive problems and changes in serum chemistry from these compounds; however, tests for mutagenic potential have been negative. Additional testing is being conducted, but concern about the potential side effects produced by these chemicals has led the USEPA to suggest a limit of 1.0 mg/l total residual for chlorine dioxide plus chlorite and chlorate at the consumer's tap. It is possible the use of chlorine dioxide will be discontinued in the future. Contact basins constructed for use of this compound should be designed so they can be used with ozone, if a change is required.

The only disinfection byproducts regulated at the present time are the THMs. These compounds will continue to be of interest, and it is very probable the MCL for TTHMs will be lowered. The magnitude of the future MCL is not known, but estimates of 0.05 mg/l are common. Facilities installed prior to promulgation of the new MCL should be designed so they can easily be modified to produce significantly lower concentrations of total trihalomethanes in the future.

A number of other byproducts are also being evaluated, and many of them are potentially more difficult to control. These include chloroacetic acid, haloacetonitriles, chlorinated benzenes, total organic halides (TOX), trichloroacetone, chlorinated aldehydes, and chlorophenols. The common thread in the list of compounds is the presence of organic chemicals in the water when it is being disinfected, and the use of halogens (chlorine, bromine, or iodine) as the disinfectant. The ultimate destination in water treatment appears to be the use of ozone for primary disinfection, coupled with the use of GAC for organics removal when this is necessary. This trend may change when additional information becomes available, but designs prepared for future facilities should be compatible with eventual use of these processes.

4.2.4 Lead

Lead is a toxic metal that tends to accumulate in the tissues of humans. The major toxic effects of lead include anemia, neurological disfunction, and renal impairment. The current MCL is 0.05 mg/l. This concentration is readily attainable with conventional treatment. However, the addition of lead to drinking water occurs chiefly in the distribution system, including household plumbing; and this is most likely to occur when the water is "corrosive". This situation lead to a 1986 ban on lead solders, flux, and pipe, and consideration of moving the sampling location for this metal to the consumer's faucet.

The proposed rule now being discussed establishes a MCL for lead of 0.005 mg/l for finished water leaving the treatment plant. In addition, morning first-draw samples at the tap must have a lead concentration less than 0.010 mg/l, a pH greater than 8.0, and a total alkalinity greater than 30 mg/l as $CaCO_3$. The pH and total alkalinity criterion will make production of a consistently suitable finished water from Lake Sulphur Springs more difficult than it has been in the past.

4.2.5 Enforcement

The original SDWA contained enforcement mechanisms, but they were not geared toward rapid corrective actions, and they provided a wide range of discretionary latitude to state and federal regulatory authorities. This situation has been changed by the 1986 amendments. The USEPA must now initiate corrective action if the state does not do so within 30 days of a violation. The maximum fine has also been increased from $5,000 to $25,000 per day, and the violation no longer has to be willful to merit the fine. However, of even more significance from an enforcement standpoint is the administrative power given to the USEPA to order compliance, and to fine violators without going through the court system. The law still provides for variances and exemptions to specific rules under certain conditions. However, variances and exemptions must include a schedule for future compliance when they are granted. The emphasis of the 1986 amendments is clearly oriented toward prompt and widespread compliance with the law.

5.0 EXISTING FACILITIES

5.1 SURFACE WATER CONVEYANCE FACILITIES

The raw water conveyance system consists of the intake/pumping station for Century Lake and the intake/pumping station for Lake Sulphur Springs. The two pump stations are interconnected by a 24 inch pipe. Various raw water pipelines convey the raw water to the water treatment plant.

The intake located in Lake Sulphur Springs is an offshore tower with multiple ports for various drawoff levels. A 48 inch pipe connects the tower to an onshore pump station.

The raw water pump stations consist of five vertical turbine pumps. Pumps 4 and 5 are mounted in 36 inch diameter cans connected to each other and to the intake tower in Lake Sulphur Springs by a 48 inch pipe. The remaining pumps share a common wetwell that is connected to the can pumps with a 24 inch pipe. The wetwell is connected to Century Lake by a 24 inch pipe. The capacities of the pumps are listed below.

Pump	Capacity mgd
No. 1	5.0
No. 2	3.0
No. 3	3.0
No. 4	5.0
No. 5	5.0

Due to hydraulic restrictions in the suction and discharge piping, the installed capacity of the pump station is 10 mgd, and the firm capacity (maximum capacity with the largest pump out of service) is 6 mgd. A surge relief valve on the pump discharge header provides surge control.

5.2 SULPHUR SPRINGS WATER TREATMENT PLANT

The existing Sulphur Springs Water Treatment Plant was placed in service in 1967. It is a conventional clarification plant consisting of two 1.5 mgd modules, one 1.0 mgd module, and one 3.0 mgd module, for a rated plant capacity of 7 mgd. The two 1.5 mgd modules are identical in size and design. The layout of the existing plant is shown on Figure 5-1.

5.2.1 Treatment Process

The treatment process involves rapid mixing, flocculation, sedimentation and filtration. Alum is added in the rapid mix basins for coagulation, and lime is added in Sedimentation Basin 2 for pH adjustment. Disinfection is accomplished by the addition of chlorine and ammonia after filtration. Fluoride is also added after filtration. Activated carbon is available for taste and odor control.

A process schematic of the facility is given on Figure 5-2.

5.2.2 Chemical Storage and Feeding

Liquid alum is stored in three above-ground 10,000 gal fiberglass tanks located next to the plant drive, west of the filter building. Two roto-dip feeders and a four-way flow splitter are housed in a room located within the filter building. Alum is transferred to the roto-dip feeders by gravity. Only one roto-dip feeder is used at a time, with the other being a standby. The feeders are alternated monthly. The roto-dip feeder delivers the alum through the four-way splitter box, which directs the flow through a small PVC pipe, which dumps the alum directly into each rapid mix basin. The alum is not diffused into the water below the impeller of the mixer.

Two 7.3 cubic feet per hour (cfh) and one 5.3 cfh dry alum gravimetric feeders and solution tanks, located in the filter building, are maintained as standby for the liquid feed system. A 30 day supply of dry alum is stored in bags.

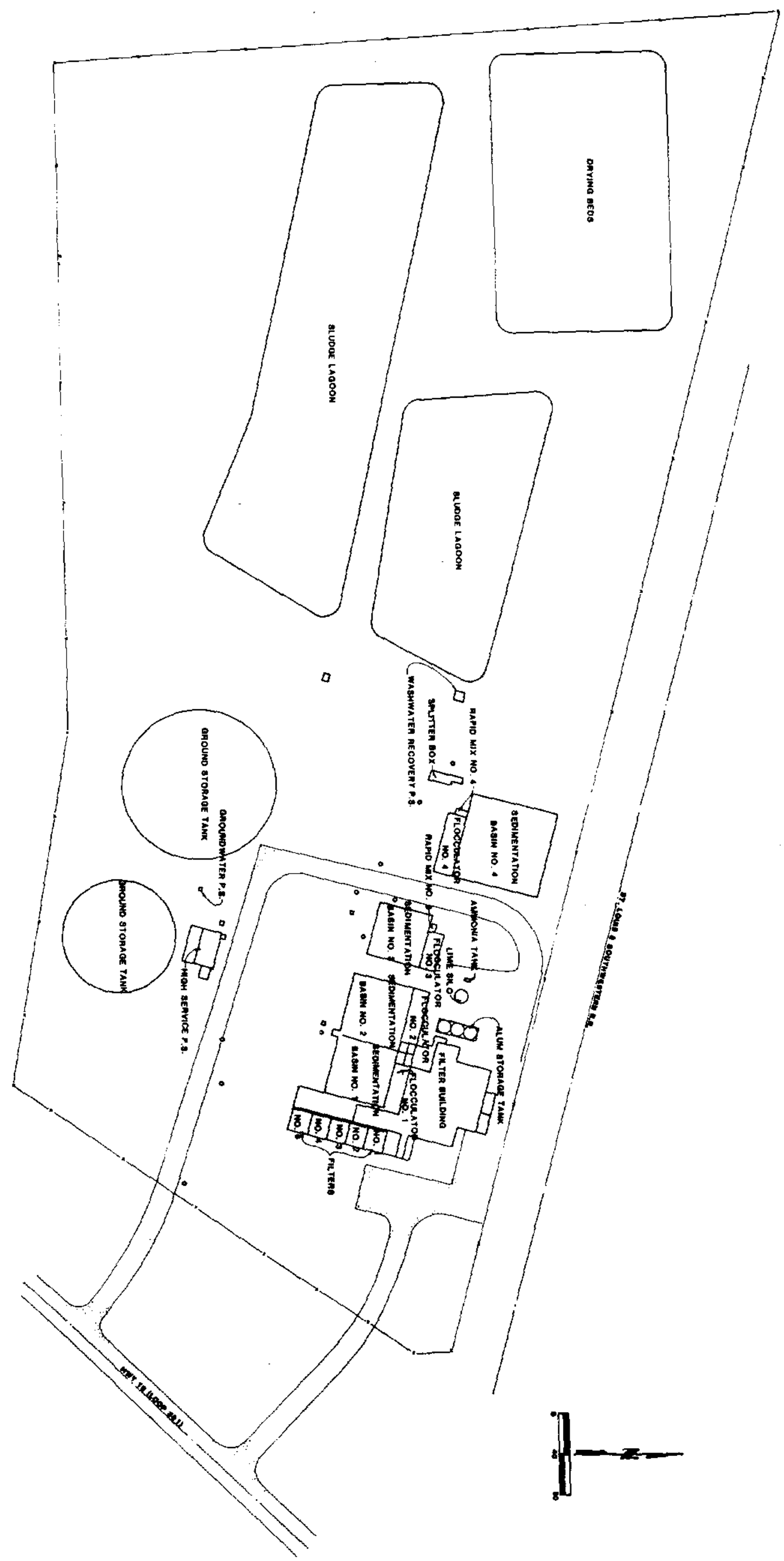
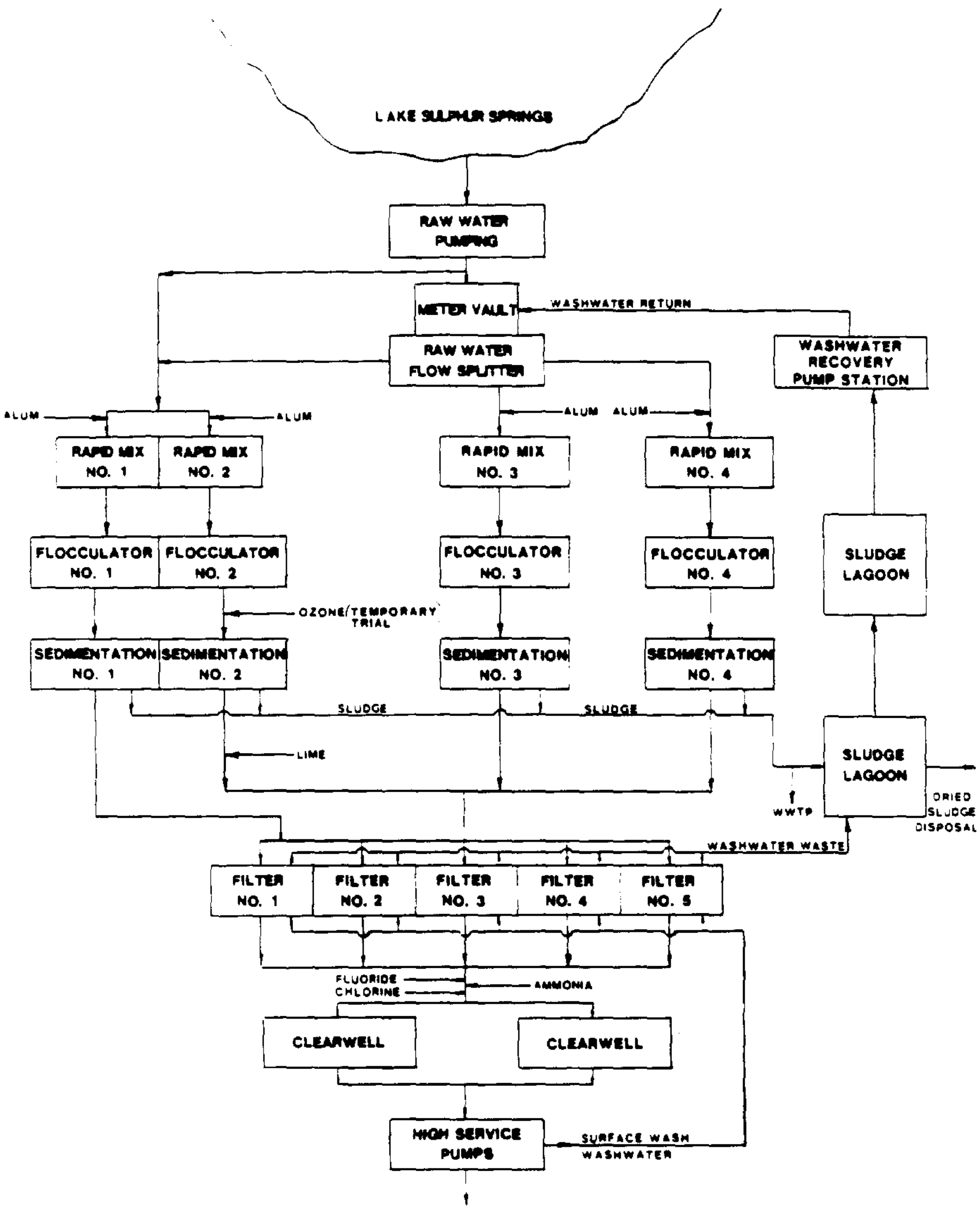


FIGURE 5-1		COOPER RESERVOIR WATER STUDY SULPHUR SPRINGS, TEXAS	EXISTING SITE LAYOUT WATER TREATMENT PLANT BLACK & VEATCH DALLAS, TEXAS PROJECT NO. 14719.100	



PROCESS SCHEMATIC

PROJECT NO. 14719.100

BLACK & VEATCH DALLAS, TEXAS

COOPER RESERVOIR
WATER STUDY
SULPHUR SPRINGS, TEXAS

FIGURE
5-2

Anhydrous ammonia is stored in a 500 gal steel tank, west of the lime silo. Ammonia is fed at a rate of 70 ppd by a 400 ppd feeder to the injection point in the finished water pipeline ahead of the clearwells.

Chlorine is purchased in one-ton containers which are stored outside in a covered area adjacent to the filter building. The area has space for a total of seven containers. Five to seven containers are currently maintained in the plant inventory. There are two 400 ppd chlorinators located in the chlorine room, one of which is used as a standby. The chlorine can be fed to the filters and upstream of the clearwell. Currently, 160 ppd of chlorine is added after the filters at the 24 inch filter effluent line.

Ozone fed from a portable ozonator is presently being used on Sedimentation Basin 1. The ozone is being experimentally applied in the clarifier influent.

Bulk pebble lime is stored in a 1700 cu ft tank located west of the filter building. The stored lime is transferred by gravity into two-day tanks that supply two 500 pph gravimetric feeders and slakers. The lime slurry flows by gravity through a metal trough from the slakers to the effluent launder trough of Sedimentation Basin 2. One feeder and one slaker serve as a standby.

The existing polymer system is not currently in use. The polymer system consists of a 175 ppd dry chemical feeder, a 50 gal mixing tank, a 120 gal feed tank, and two 57 gph polymer feed pumps. Polymer can be applied to each of the existing rapid mix basins in the same manner as the alum is fed. The polymer cannot be diffused into the water below the mixer impeller.

Powdered activated carbon is stored in 50 lb bags at the raw water flow splitter at the head of the plant. The activated carbon is poured manually into the raw water influent at the splitter box, in quantities based on past experience and performance.

The existing fluoridation system includes two 3.2 gph feed pumps which pump hydrofluosilicic acid from 150 lb drums. Twenty-five to 35 drums are kept in storage outside in the chlorine storage area. The fluoride is applied to the 24 inch filter effluent line.

Chemical handling and feeding data are summarized in Table 5-1.

5.2.3 Raw Water Metering

An 18-inch cast iron venturi flow tube, located in the raw water flow splitter box, measures raw water flow to the plant. The flow tube has an approximate capacity of 12 mgd.

5.2.4 Rapid Mixing

In the rapid mix basins, alum is combined with incoming raw water for coagulation.

5.2.4.1 Rapid Mix Basins 1 and 2. Rapid Mix Basins 1 and 2 have a common inlet which divides flow to two parallel rapid mix basins, which are separated by a concrete wall. Water exits each mixing basin through a 24 inch pipe to its respective flocculation basin. Both basins have one mixing compartment. One mechanical mixer is installed in each mixing compartment.

The physical characteristics of Rapid Mix Basins 1 and 2 are indicated in Table 5-2.

5.2.4.2 Rapid Mix Basin 3. Rapid Mix Basin 3 has a 14 inch pipe inlet and a wooden baffle wall across the inlet area. The upper two thirds of the wall is solid, forcing the inlet water down. Water exits the mixing basin through a 30 inch diameter port to the flocculation basin, located adjacent to the mixing basin.

The physical characteristics of Rapid Mix Basin 3 are indicated in Table 5-2.

5.2.4.3 Rapid Mix Basin 4. The raw water enters Rapid Mix Basin 4 through a 24 inch pipe. The mixed water flows over a concrete weir to Flocculation Basin 4. The physical characteristics of Rapid Mix 4 are indicated in Table 5-2.

TABLE 5-1. CHEMICAL HANDLING AND FEEDING

Chemical (Purity)	Point of Application	Average Dosage	Dosage Range	Average Usage @ 7 mgd	Days Storage @ 7 mgd	Method of Feeding	Method of Storage
Chlorine (100%)	Filter Effluent	58 lb/MG	39-76 lb/MG	406 ppd	35	Chlorine Feeders	7 Container[2]
Hydro-fluosilicic Acid (23-30%)	Filter Effluent	41.5 lb/MG	28.6-54.2 lb/MG	291 ppd	18	Metering Pumps	35 Drums[3] (inside)
Lime (90%)	Sedimentation Basin 2	264 lb/MG	159-343 lb/MG	1848 ppd	35	Gravimetric Feeders	Bulk[4]
Alum	Rapid Mix	106 gal/MG	60-190 gal/MG	742 gpd	40	Existing roto-dip	3 Fiberglass[5] Tanks
Ammonia	Filter Effluent	20 lb/MG	14-29 lb/MG	140 ppd	6 mo.	Direct feed Ammoniator	500 Gallon Tank
Powdered Activated Carbon	Flow Splitter	250 lb/MG[1]	180-320 lb/MG	1750 ppd	30	Manually Applied bags	50 lb. Bags

(1) Fed Only in Summer Months
(2) 1 ton each
(3) 150 pounds each
(4) 1700 cu. ft Storage Silo
(5) 10,000 gal each

TABLE 5-2. PHYSICAL CHARACTERISTICS - RAPID MIX BASINS

	Module No. 1	Module No. 2	Module No. 3	Module No. 4
Number of rapid mix basins	1	1	1	1
Number of mixing compartments	1	1	1	1
Size of each mixing compartment, ft.	7.0 x 9.0 x 7.4	7.0 x 9.0 x 7.4	6.0 x 4.5 x 7.0	6.0 x 6.0 x 7.5
Retention time each rapid mix, sec.	199 @ 1.5 mgd	199 @ 1.5 mgd	122 @ 1.0 mgd	58 @ 3.0 mgd
Mixer Power, each, hp	1.0	1.0	2.0	1.5
Mixer G factor, sec -1	149	149	328	532

5.2.5 Flocculation

5.2.5.1 Flocculation Basins 1 and 2. Flocculation Basins 1 and 2 are separated by two effluent channels, which flow to their respective sedimentation basins. Both flocculation basins consist of four vertical flocculators, which are separated by baffle walls open at the bottom.

Water enters each flocculation basin through a 24 inch pipe. Each basin contains one channel arranged to allow water to flow in an axial configuration through the basin. The flocculator effluent flows through a 24 inch pipe to the center of each sedimentation basin. Flocculator paddle speed can be adjusted by changing the speed manually on the drive units.

The physical characteristics of Flocculation Basins 1 and 2 are indicated in Table 5-3.

5.2.5.2 Flocculation Basin 3. The mixed water from the rapid mix basin enters Flocculation Basin 3 through a 30 inch diameter port in the concrete wall, which separates the two basins. A redwood baffle separates the mixed raw water from the flocculation basin. There are two vertical flocculators with no baffles separating them. The flocculator effluent exits the basin through an effluent launder trough, which flows to a 24 inch pipe leading to the center of Sedimentation Basin 3. The speed of the flocculators can be manually adjusted at each drive unit.

The physical characteristics of Flocculation Basin 3 are indicated in Table 5-3.

5.2.5.3 Flocculation Basin 4. Flocculation Basin 4 is separated from Rapid Mix Basin 4 by an overflow weir over which the mixed water flows. The flocculation basin consists of three vertical flocculators, each separated by a concrete baffle wall open at the bottom. The flocculator

TABLE 5-3. PHYSICAL CHARACTERISTICS - FLOCCULATION BASINS

	Module No. 1	Module No. 2	Module No. 3	Module No. 4
Number of basins	1	1	1	1
Number of channels, each basin	1	1	1	1
Flocculator type	Vertical Turbine	Vertical Turbine	Vertical paddle Wheel	Vertical Turbine
Number of flocculators	4	4	2	3
Dimensions, each basin, ft				
Overall size	12.0 x 46.0	12.0 x 46.0	15.0 x 32.0	18.5 x 57.5
Average sidewater depth	13.0	13.0	13.0	12.0
Design G factor, sec -1	125 to 105 tapered	125 to 105 tapered	61	60
Retention time, each basin, min	52	52	67	46

effluent flows over a concrete overflow weir into the effluent launder trough. The trough flows to a 36 inch pipe which leads to the center of Sedimentation Basin 4. The speed of the flocculators can be adjusted manually at the drive unit.

The physical characteristics of Flocculation Basin 4 are indicated in Table 5-3.

5.2.6 Sedimentation

5.2.6.1 Sedimentation Basins 1 and 2. Sedimentation Basins 1 and 2 are located adjacent to their respective flocculation basins. Each basin is square and has one sedimentation zone. The basins are provided with circular sludge collection equipment with corner sweeps. Water exits the sedimentation basins over steel weir plates and flows through the collection troughs to the filters.

The physical characteristics of the sedimentation basins are indicated in Table 5-4.

5.2.6.2 Sedimentation Basin 3. Sedimentation Basin 3 is square and is located adjacent to Flocculation Basin 3. The basin is provided with circular sludge collecting equipment and corner sweeps. Water exits the basin over a steel weir plate and flows through the collection troughs to the filters.

The physical characteristics of Sedimentation Basin 3 are indicated in Table 5-4.

5.2.6.3 Sedimentation Basin 4

The layout of Sedimentation Basin 4 is similar to Sedimentation Basin 3 except for the differences shown in Table 5-4.

TABLE 5-4. PHYSICAL CHARACTERISTICS - SEDIMENTATION BASINS

	Module No. 1	Module No. 2	Module No. 3	Module No. 4
Number of basins	1	1	1	1
Number of zones, each basin	1	1	1	1
Dimensions, each basin, ft				
Overall size	60.0 x 60.0	60.0 x 60.0	50.0 x 50.0	85.0 x 85.0
Sidewater depth	15.0	15.0	10.5	13.0
Retention time, each basin, hr	6.5	6.5	4.7	6.0
Weir loading rate, gpm/ft	4.7 @ 1.5 mgd	4.7 @ 1.5 mgd	3.9 @ 1.0 mgd	5.6 @ 3.0 mgd
Surface loading rate, gpm/sq ft	0.33 @ 1.5 mgd	0.33 @ 1.5 mgd	0.41 @ 1.0 mgd	0.35 @ 3.0 mgd

5.2.7 Filtration

Five constant rate dual media filters are located east of the filter building. Each filter is equipped with surface wash equipment, concrete washwater troughs, and anthracite and sand media supported on gravel and tile underdrains. The filters are designed to operate at a maximum filtration rate of 5 gpm/sq ft of filter surface area. This filtration rate yields a nominal flow of 2.6 mgd per filter, for a maximum plant filtering rate of 13 mgd.

Settled water from the sedimentation basins flows through a 36 inch pipe header and enters each filter through a 16 inch pipe. Finished water exits each filter through individual 18 inch effluent pipes connected to a common 24-inch pipe located in the pipe gallery. Once outside the gallery, the water is conveyed through a 30 inch pipe to the clearwell.

Filter backwashing is accomplished by using two washwater pumps which take suction from the existing 1.0 mil gal finished water clearwell. Each unit is a vertical turbine pump and is rated at a nominal flow capacity of 5400 gpm at 35 ft of head. This provides washwater at a rate of approximately 15.0 gpm/sq ft. Backwash waste is drained to the sludge lagoon. Filter surface washwater is provided by two 5400 gpm washwater pumps. Both pumps are located at the high service pumping station.

Physical characteristics of the filters are indicated in Table 5-5.

TABLE 5-5. PHYSICAL CHARACTERISTICS - FILTERS

Number of filters	5
Size, each ft	23.25 x 15.5
Overall depth, ft	11.0
Surface loading, gpm/sq ft	
Design Plant Flow	
7 mgd	2.70 for five filters

TABLE 5-5 (continued). PHYSICAL CHARACTERISTICS - FILTERS

Bed depth, inches	
Anthracite	20
Sand	10
Gravel Support	12

5.2.8 Finished Water Storage

Finished water is stored in one 2 mil gal, 175 ft diameter clearwell and one 1 mil gal, 106 ft diameter clearwell. Both reservoirs are buried concrete tanks with normal water depths of 16.00 and 16.75 ft, respectively. The 1 mil gal clearwell is located south, and the 2 mil gal clearwell is west, of the high service pumping station. The clearwells supply the high service, washwater, and filter surface wash pumps. Provisions have been made in the design of this facility for an adjacent 2 mil gal clearwell.

5.2.9 High Service Pumping

High service pumping is provided to fill 0.25, 0.50 and a 0.75 mil gal elevated storage tanks, all located in the distribution system.

Three 5.0 and one 3.0 mgd vertical turbine pumps are located in the high service pumping station. No space is available for additional pumps. The pump capacities are listed below.

Pump	Capacity mgd
No. 1	5.0
No. 2	3.0
No. 3	5.0
No. 4	5.0

The pumps are currently run individually in periods of low demand. In periods of high demand, any combination of pumps can be used for a maximum capacity of 18 mgd. The firm capacity (maximum capacity with the largest pump out of service) is 13 mgd.

An electrically operated butterfly valve in each pump discharge provides "stop/check" control.

High service discharge flow is measured with a 18 inch buried venturi flow tube, with a maximum metering capacity of approximately 12 mgd.

5.2.10 Washwater Waste and Sludge Disposal

Washwater waste from the filters and sludge from the sedimentation basins are conveyed through individual pipes to the sludge lagoons. Approximately one half of the sludge is discharged to the sanitary sewer through a valved interconnection with the sewer.

The washwater waste from the filters flows by gravity through a 24 inch line to the south sludge lagoon.

Two earthen lagoons are used for storing sludge. The north lagoon is approximately 90 by 220 ft and is 4 to 6 ft deep. The south lagoon is approximately 500 by 120 ft and is 2 to 4 ft deep. Sludge and washwater waste enter the south lagoon first and the supernatant is drained by a 12 inch pipe to the north lagoon. The washwater recovery station pumps from the north lagoon.

Settled sludge is collected in the bottom hoppers of the sedimentation basins. Sludge collected by hoppers in Sedimentation Basins 1 and 2 is conveyed through an 8 inch line to a 12 inch sludge drain line, where it is carried to the south sludge lagoon or sent to the sanitary sewer system. The sludge from Sedimentation Basin 4 flows by gravity through a 6 inch line to the 12 inch sludge drain line, and the sludge from Sedimentation Basin 3 is pumped to the 12 inch line. A 12 inch tee and two 12 inch valves have been added to the 12 inch sanitary sewer line, to allow the operator to divert the sludge to the lagoons or the sanitary sewer in any combination of flows. When a lagoon becomes full, the sludge is allowed to air dry. It is then removed from the lagoon with a front end loader and spread on City property west of the lagoons. The present contract for the wastewater treatment plant expansion includes a third sludge storage basin. It is intended, that upon completion of the WWTP expansion, all sludge from the water treatment plant will be conveyed to the WWTP. Sludge is wasted to the WWTP during low flow periods (at night) because of hydraulic limitations in the sewer collection system.

5.2.11 Electrical System

Power to the High Service Pumping Station is supplied from an onsite electrical substation located west of the high service pump station. This substation contains a 750 kVA pad-mounted transformer and receives power from an overhead feeder. Power to the remainder of the plant is supplied from an offsite feeder. There is no standby power source.

5.2.12 Instrumentation and Controls

The main instrument control panel for the plant is located in the filter building. All major items of equipment are controlled and monitored from this panel. Individual filter controls are also located in the filter building to provide the operator visual access to these facilities when they are backwashed.

The main control panel also serves as the central control and monitoring point for the water distribution system. The 0.75 mil gal and the 0.50 mil gal elevated storage tanks are monitored at this main control panel. There are no means at present for monitoring the 0.25 mil gal elevated storage tank.

5.2.13 Laboratory, Office, and Maintenance Facilities

The plant includes one building, the filter building, that houses various facilities and equipment, offices, and work areas. This building contains the administrative office, laboratory, main control room, instrument repair shop, classroom/breakroom, a warehouse for storage of miscellaneous items, and the filter control console. The chemical rooms in the building contain feed facilities for chlorine, ammonia, a backup dry alum feed system, and polymer.

5.3 PLANT OPERATIONS

5.3.1 Plant Balance

The objective of this section is to analyze flows through the existing treatment plant. This analysis is based on data collected in 1987 and the first three months in 1988.

Raw water enters the plant from Lake Sulphur Springs and flows through the rapid mix, flocculation, and the sedimentation basins. A portion of the flow entering the sedimentation basins is used to convey sludge out of these basins to either the onsite lagoons or the WWTP lagoons. This flow is commonly called "underflow" or "blowdown". The second term will be used in this discussion.

Normal operational procedure at the plant is to remove sludge from the sedimentation basins at night on a daily basis. A portion of this sludge is discharged into a sanitary sewer and is handled at the WWTP. The remainder is diverted into the sludge lagoons located at the water treatment plant.

Settled water discharged from the sedimentation basins flows through the dual media filters and into the clearwells. The vast majority of water entering the clearwells is pumped into the distribution system for use in the City. A small portion of the water in the clearwells is used to backwash the filters when they need to be cleaned. Dirty backwash water is discharged into the onsite lagoons.

Suspended solids in the waters (blowdown and backwash) entering the lagoons settle, and the clarified water is recycled back to the treatment plant. The solids collected in the lagoons are periodically removed and spread on City owned fields located adjacent to the treatment plant.

The quantities of raw water pumped from the lake, pumped into the distribution system, used for backwashing, and recycled from the lagoon are measured. Blowdown flows are not metered but they can be calculated. The calculated flows are based on the following assumptions: (1) the volume of water stored in the lagoons does not change; (2) the quantity of rain falling on the surface of the water in the lagoons is equal to the water lost by seepage and evaporation from these lagoons; and (3) water lost from the lagoons when sludge is removed for land disposal is insignificant compared to the quantity of water flowing through the lagoons.

These assumptions are not correct on a daily basis, but they are reasonable for longer time intervals. The largest potential error is the assumption regarding water withdrawn from the lagoons during sludge removal. However, the only impact of a departure from this assumption is a decrease in the flow going to the WWTP.

Plant flows for the past 15 months are summarized in Table 5-6 and illustrated on Figure 5-3. During this time interval:

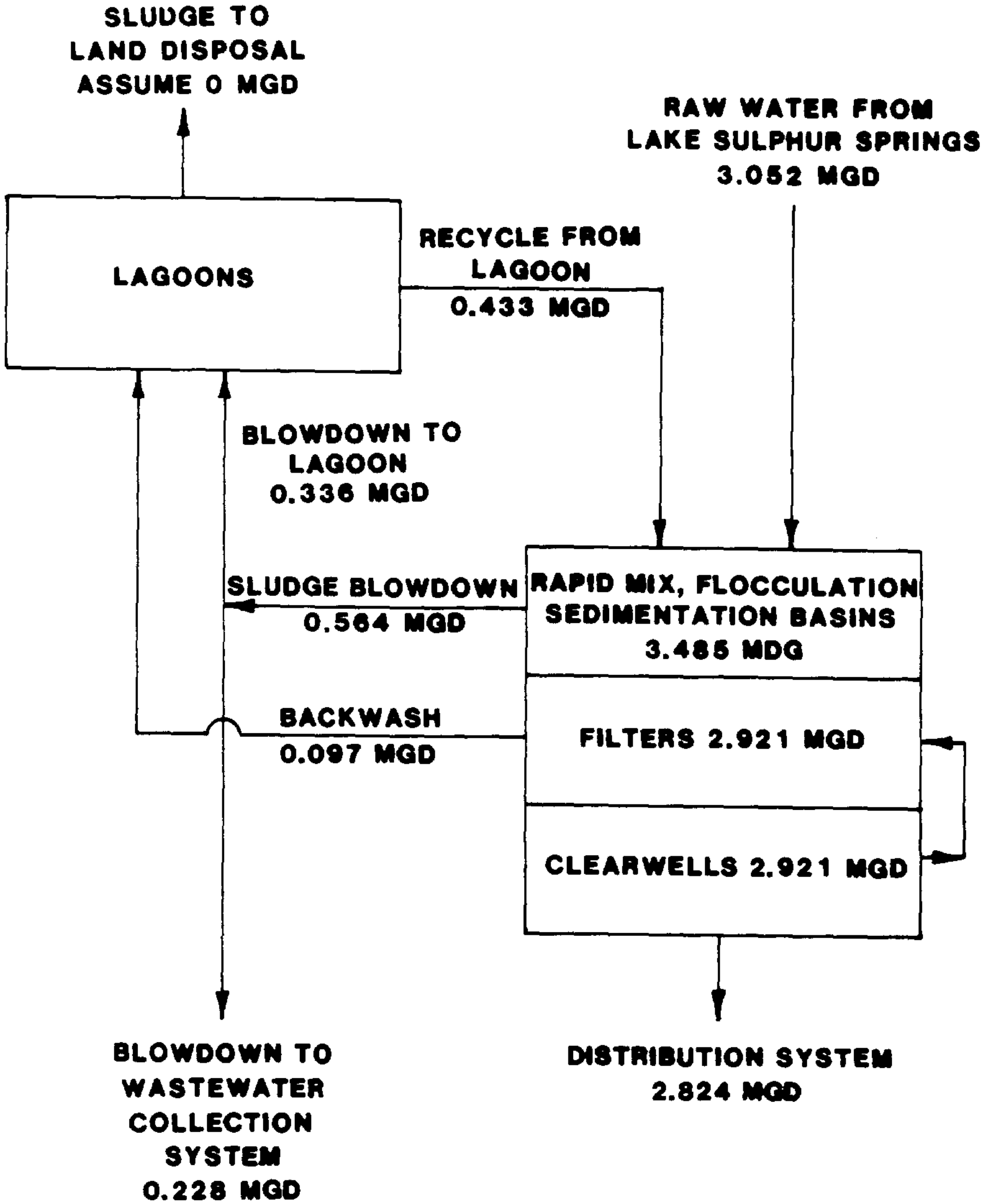
- o 92.5 percent of the raw water pumped from the lake entered the City water distribution system. The remaining 7.5 percent was used to convey sludge to the WWTP.
- o Water recycled from the lagoon was equal to 15.2 percent of the raw water pumped from the lake.
- o Average flow through the rapid mix, flocculation, and sedimentation basins was 3.485 mgd. This is 115.2 percent of the raw water obtained from the lake.
- o Average flow through the filters was 2.921 mgd. This is 95.7 percent of raw water pumpage.

The lagoon system now in use at the plant is functioning as a water conservation measure, and it provides a small decrease in electrical power consumption because the lagoon recycle pump station is operating against less head than the raw water pump station. Both of these benefits could be increased by enlarging the system so it would handle all of the sludge blowdown. However, there is one major drawback to this approach. Part of the organic material trapped in the sludge is

TABLE 5-6. WATER BALANCE FOR THE TREATMENT PLANT
(All flows are in mgd)

Date	Raw Water from Lake (1)	Finished Water to Distribution (1)	Recycled Water from Lagoon (1)	Backwash (1)	Sludge Blowdown (2)	Sludge to Lagoon (2)	Sludge To Wastewater Treatment (2)
January 87	2.682	2.340	0.582	0.129	0.795	0.453	0.342
February	2.841	2.251	0.507	0.130	0.967	0.377	0.590
March	2.850	2.468	0.671	0.134	0.919	0.537	0.382
April	3.242	3.117	0.417	0.113	0.429	0.304	0.125
May	3.028	2.795	0.517	0.096	0.654	0.421	0.233
June	3.013	2.830	0.578	0.088	0.673	0.490	0.183
July	3.688	3.458	0.457	0.049	0.638	0.408	0.230
August	4.909	4.347	0.551	0.107	1.006	0.444	0.562
September	3.415	3.193	0.432	0.094	0.560	0.338	0.222
October	2.936	2.806	0.234	0.088	0.276	0.146	0.130
November	2.663	2.586	0.282	0.102	0.257	0.180	0.077
December	2.536	2.494	0.234	0.080	0.196	0.154	0.042
January 88	2.695	2.635	0.384	0.089	0.355	0.295	0.060
February	2.695	2.507	0.349	0.084	0.453	0.265	0.188
March	2.581	2.518	0.293	0.086	0.270	0.207	0.063
Average	3.052	2.824	0.433	0.097	0.564	0.336	0.228

(1) Measured
(2) Calculated



Time Period
Jan. '87 - March '88

WATER BALANCE,
EXISTING TREATMENT PLANT
SULPHUR SPRINGS, TEXAS

BLACK & VEATCH DALLAS, TEXAS

PROJECT NO. 14719.100

COOPER RESERVOIR
WATER STUDY
SULPHUR SPRINGS, TEXAS

FIGURE
5-3

converted to soluble compounds by biological metabolism in the lagoons, and is recycled back into the plant. This is not a major concern during the winter, but it is intensifying the tastes and odors now occurring during the summer. Future lagoon systems should be designed to operate without flow recycle during this period. This can be done by providing supplemental irrigation facilities in fields near the water treatment plant, or by increasing the volume of sludge handled at the WWTP. The City is now constructing a third sludge storage basin at the WWTP. It is intended that all sludge blowoff from the WTP will be handled at the WWTP when this basin is completed. The only flow passing through the lagoons would be filter backwash. Use of this approach will increase the average annual raw water pumpage by 18.5 percent, but it will eliminate recycling tastes and odors from these lagoons during the summer.

5.3.2 Chemical Use

The objectives of treatment are to produce a clear, palatable water which is attractive and safe to use. The chemicals used to reach these goals in Sulphur Springs are alum, lime, powdered activated carbon, chlorine, ammonia, and hydrofluosilicic acid.

Alum is used to coagulate influent suspended solids and remove color. It is added at each of the four rapid mix basins in quantities based on the plant operator's assessment of the minimum amount needed. Jar tests performed in the laboratory can provide guidance on the dosages required, but the ultimate decision must be based on plant performance.

Powdered activated carbon (PAC) is used to adsorb organic compounds which are present in the raw water and the recycle flow from the lagoons. The objective is to remove compounds which produce tastes and odors, and this goal has not been satisfactorily achieved in the existing plant. Part of the problem is a complete absence of facilities to handle the carbon. Plant operators have tried to compensate for this by purchasing bagged PAC and emptying these bags into the raw water distribution structure as needed. This approach is the only method now feasible; however, it does not make full use of the PAC, and the purchase price for the carbon is higher than it would be if PAC were obtained in bulk quantities. The PAC should be added to the raw water 10 to 15 minutes before this water enters the rapid mix basins. The best location available in the existing plant is the raw water pump station. If a decision is made to continue to use PAC in the future, the carbon could be purchased in bulk and stored as a water slurry in an underground concrete vault. The PAC would then be injected into the discharge header for the pumps when it is needed.

Monthly average alum and PAC feed rates from January 1985 through December 1987 are shown on Figure 5-4. Alum use was lowered in January 1986 and good coagulation has been achieved since that time with 134 to 190 gallons of alum/mil gal of raw water entering the plant. However, the alum dosage is still very high. One of the advantages of using the higher TDS water impounded in Cooper Reservoir will be a significant decrease in the amounts of alum required to obtain satisfactory clarification. This is discussed in Chapter 8.0.

PAC additions start in May of each year, peak in August or September, and are discontinued in October or November. Peak dosages have declined from 481 pounds of PAC/mil gal of raw water in 1985 to 320 lb/mil gal in 1987. This decrease partially reflects differences in raw water quality in Lake Sulphur Springs (discussed in Chapter 3.0), but is also an indication of the poor performance of PAC in the present plant. Adsorption time available before the carbon particles are coated with alum floc is too short to obtain significant additional benefit from higher dosages. Relocation of the carbon addition point would improve PAC performance. However, complete correction of the taste and odor problem will also require use of a stronger disinfectant. This is discussed below.

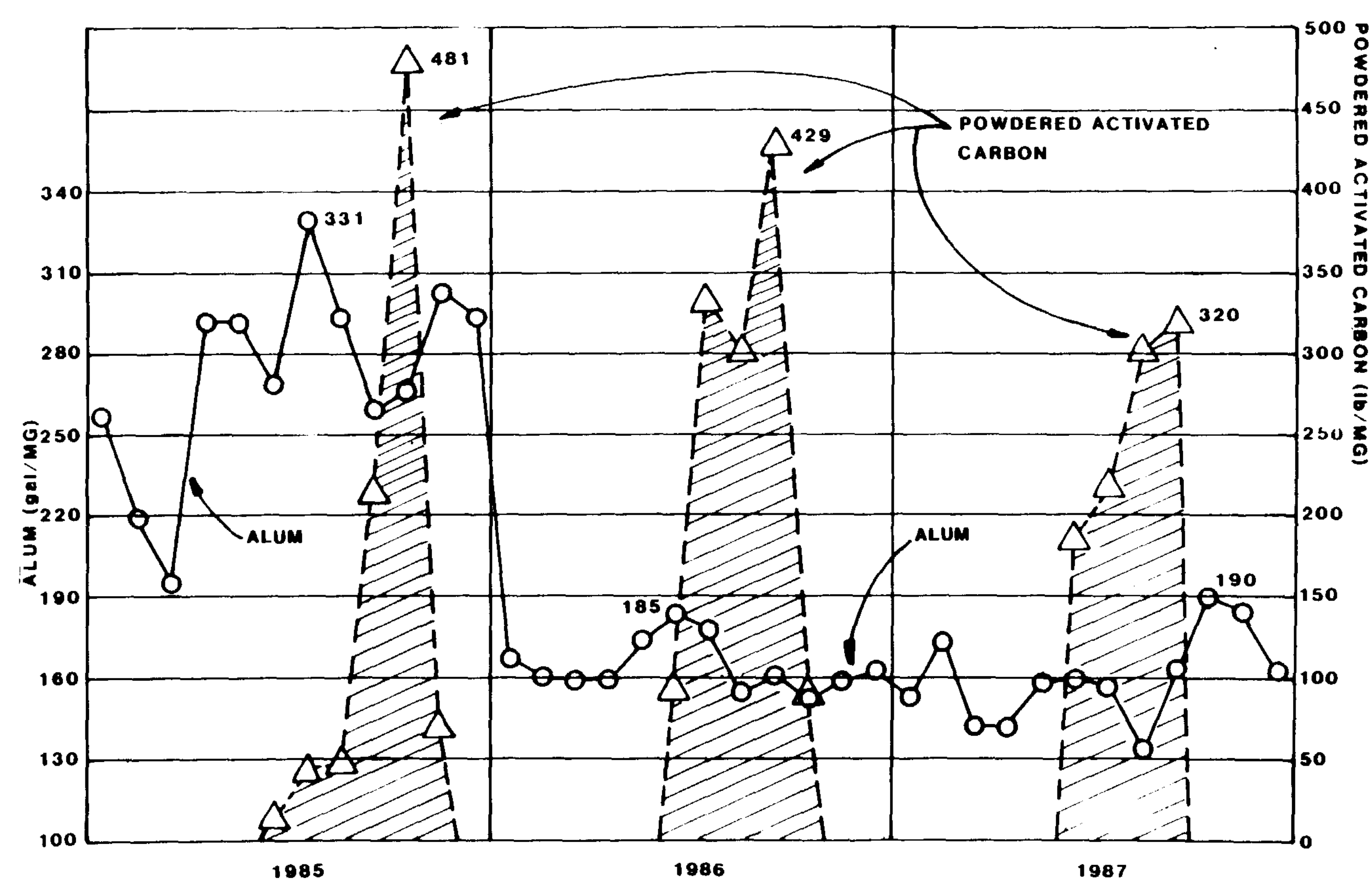


FIGURE 5-4

COOPER RESERVOIR
WATER STUDY
SULPHUR SPRINGS, TEXAS

ALUM & CARBON FEED RATES
SULPHUR SPRINGS WTP

BLACK & VEATCH DALLAS, TEXAS

PROJECT NO. 14719.100

Disinfection is now being achieved by addition of chlorine and ammonia to form chloramines in the filtered water flowing into the clearwells. This approach was adopted in 1983 to reduce the formation of total trihalomethane (TTHM) compounds in the finished water (discussed in the next section), and has been successful in this regard. However, chloramines are relatively weak oxidizing agents, and their limited reactions with the organic nitrogen compounds in the water have intensified the taste and odor problem. Correction of this situation will require use of a stronger oxidant, such as ozone or chlorine dioxide, as the primary disinfectant. This approach is also mandated by the proposed Surface Water Treatment Rule (SWTR) discussed in Chapter 4.0, so changes in the existing system will be required in the near future.

Chlorine and ammonia dosages used during the past three years are shown on Figure 5-5. The cyclic changes in chlorine dosage shown in this figure are produced, in part, by the method used to record chlorine consumption. Plant records are based on the number of new, one ton, chlorine containers placed in service each month, without accounting for the chlorine remaining in a partly used container at the end of the month. Thus, the entire contents of a container placed in service during the latter part of a month are charged to that month despite the fact that most of the chlorine is actually used in the following month. On an annual basis, this accounting procedure does not introduce a significant discrepancy, but it does introduce some of the monthly variation shown on Figure 5-5. This factor is also partly responsible for the monthly variations in the chlorine to ammonia ratios shown on Figure 5-6. The recommended ratio for monochloramine formation is 4:1 (weight of chlorine: weight of ammonia). Underfeeding ammonia will produce dichloramine (8:1) or nitrogen trichloride (12.5:1), and both of these compounds will produce tastes and odors in the finished water. Ammonia present in the raw water also enters into the reaction, and the ammonia nitrogen concentrations in Lake Sulphur Springs range from 0.01

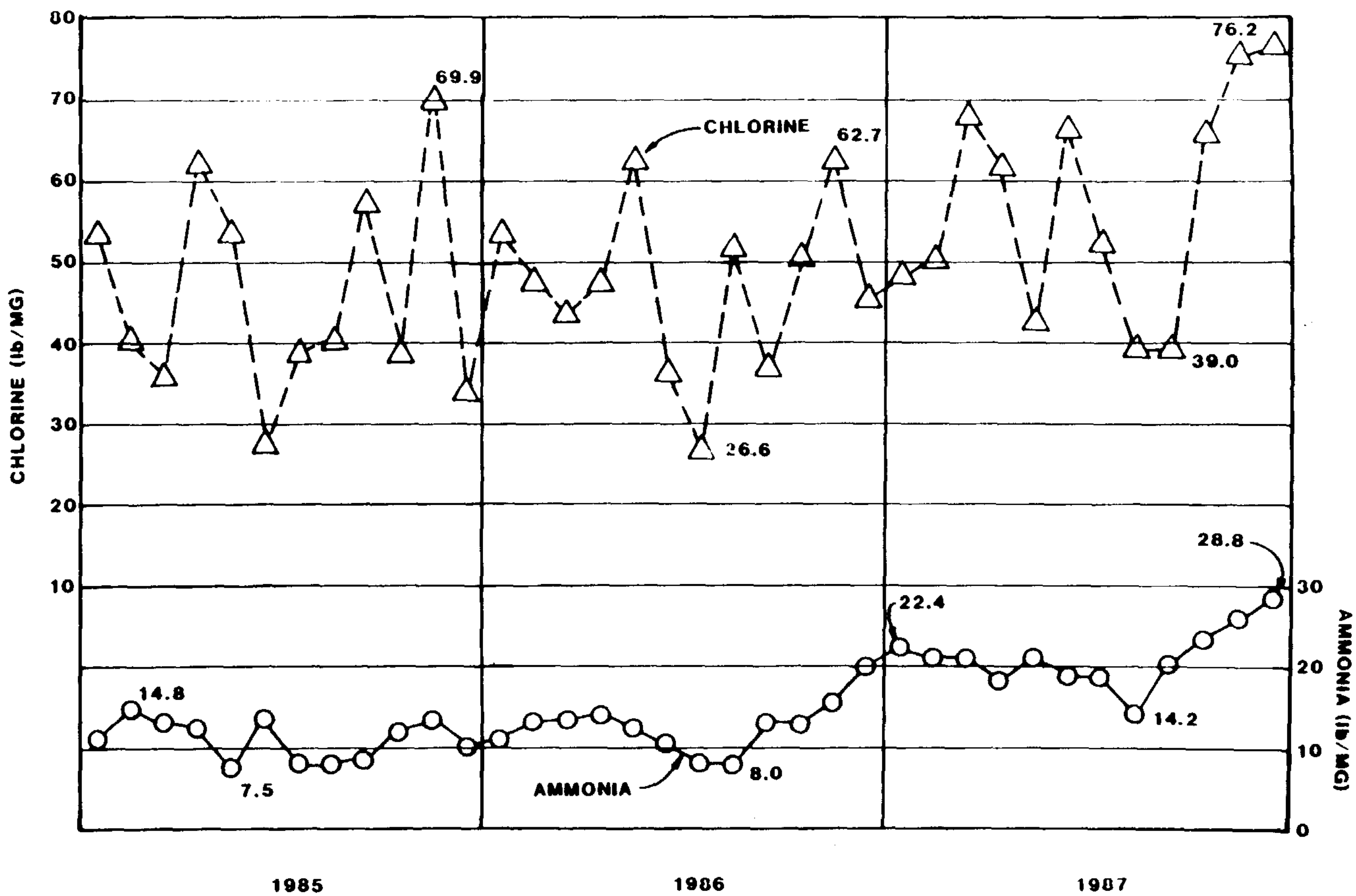


FIGURE 5-5

COOPER RESERVOIR
WATER STUDY
SULPHUR SPRINGS, TEXAS

CHLORINE & AMMONIA DOSAGES SULPHUR SPRINGS WTP

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PROJECT NO. 14719.100

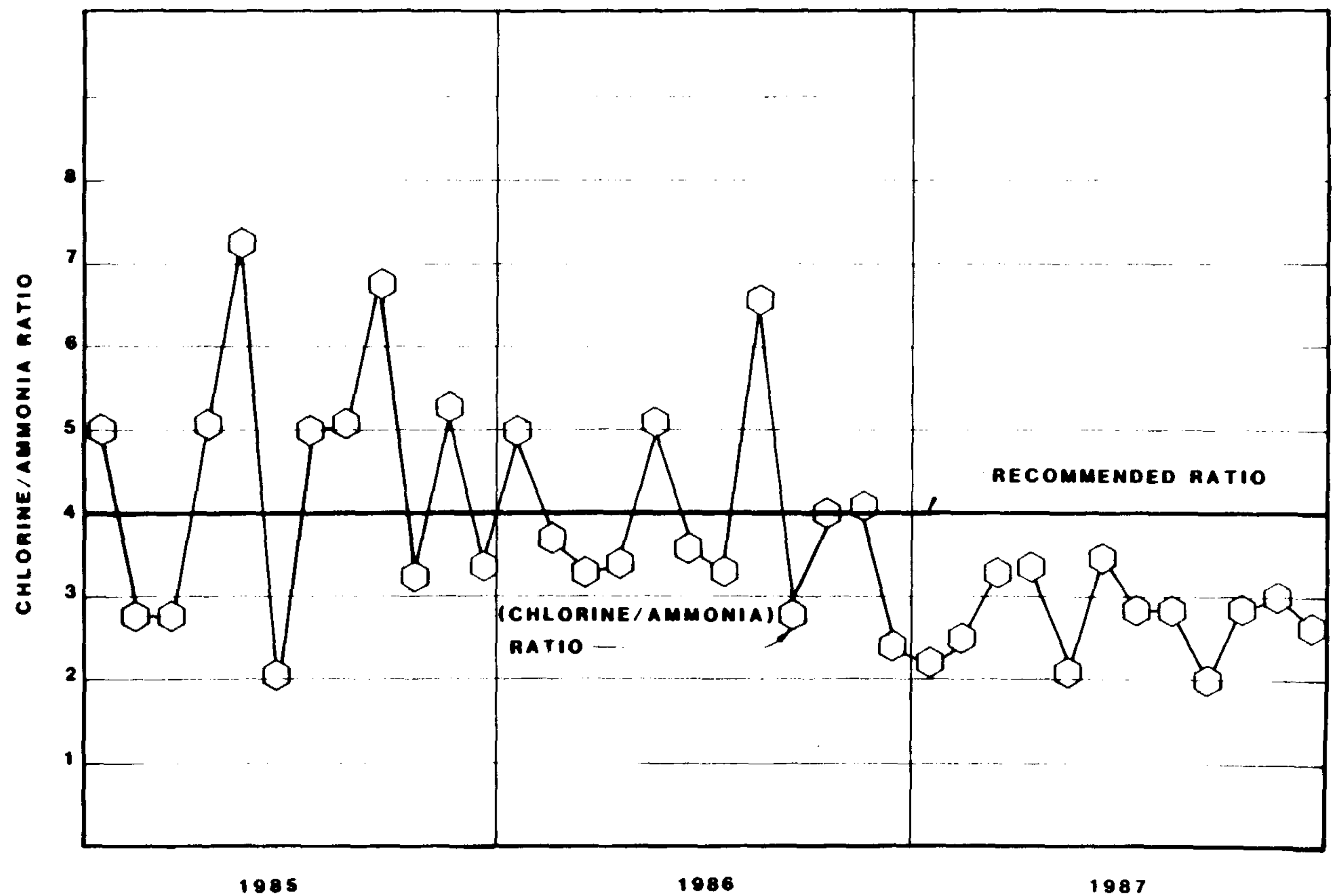

FIGURE 5-6

COOPER RESERVOIR
WATER STUDY
SULPHUR SPRINGS, TEXAS

CHLORINE/AMMONIA RATIO
SULPHUR SPRINGS WTP

BLACK & VEATCH DALLAS, TEXAS

PROJECT NO. 14719.100

mg/l to 0.4 mg/l. This will lower the ratios shown on Figure 5-6 somewhat. The major effect of chloramines on tastes and odors in the finished water is due to chlorine reactions with organic nitrogen compounds, and the failure to completely oxidize taste and odor compounds present in the raw water.

Lime is used to adjust the pH of the finished water. It is purchased as calcium oxide, slaked onsite, and added in the outlet channel of Sedimentation Basin 2. This is a convenient location because of the physical layout of the plant. Addition of lime downstream of sedimentation is also desirable from a treatment standpoint because it allows the alum to decrease the pH in the flocculation and sedimentation basins. The low pH removes color from the raw water.

Hydrofluosilicic acid is added in small quantities to raise the fluoride ion concentration in the finished water to approximately 1.0 mg/l. This level is maintained to reduce the formation of dental caries in individuals drinking the water, and is particularly effective in children who are forming their permanent teeth.

Lime and hydrofluosilicic acid feed rates for the past three years are shown on Figure 5-7. The large variations in lime dosage reflect changes in alum use, variations in alkalinity in the raw water, and the recent decision to raise the pH in the finished water to accommodate industrial customers. Changes in the hydrofluosilicic acid dosage are produced by using a relatively constant pumping rate for the acid, regardless of the rate of water flow through the treatment plant.

Average chemical quantities and the costs of treatment are summarized in Table 5-7. All of the cost computations included in this table are based on 1988 chemical prices, so the different years can be compared. The two largest expenditures are for alum and PAC, and the major reason for the decrease in total cost between 1985 and 1986 is due to decreased alum use in 1986. Expenditures for PAC increased in 1986 because large quantities of carbon were fed for three months during the summer. Total PAC use declined by approximately 44,000 lb in 1987 be-

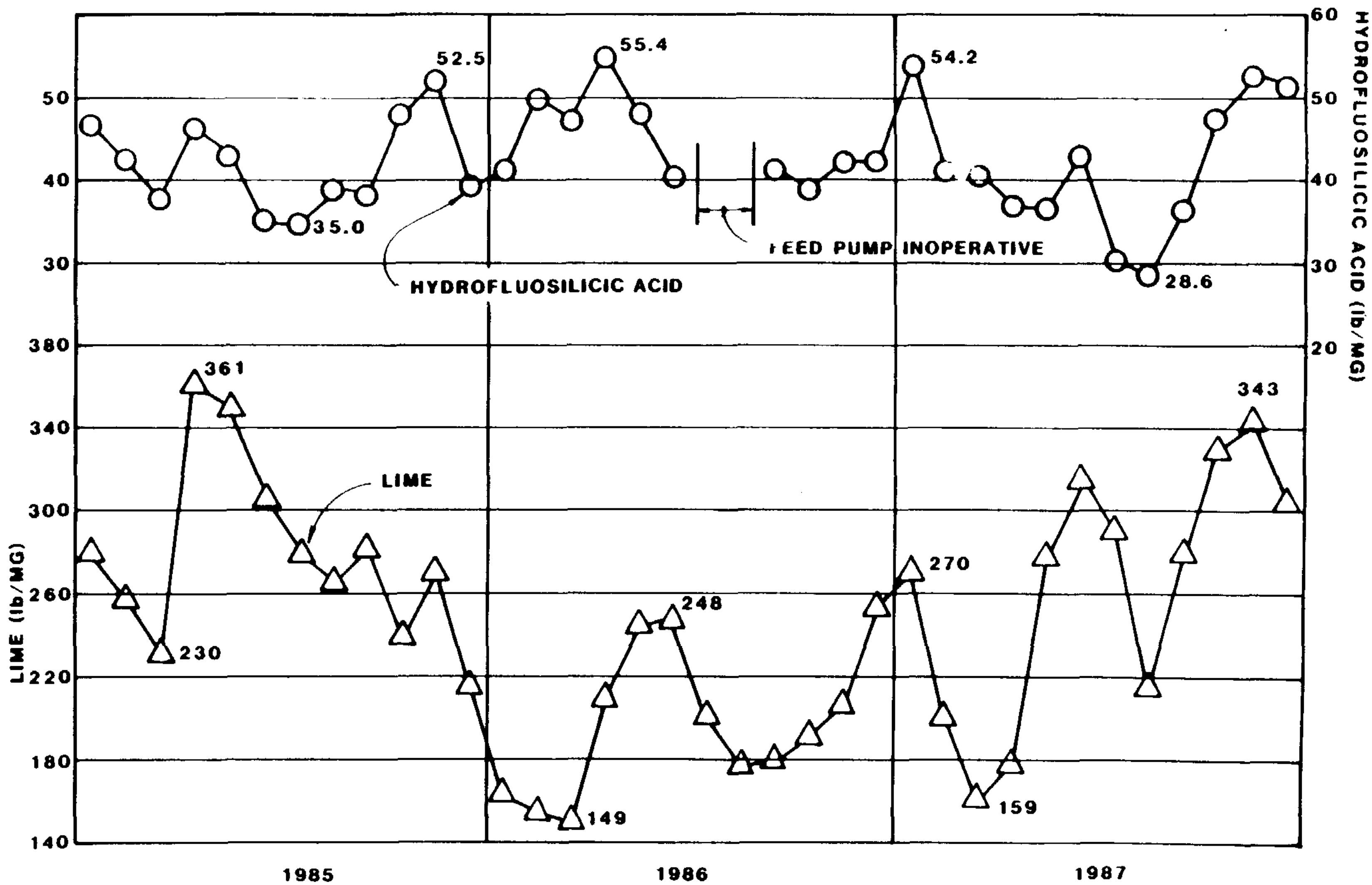

FIGURE 5-7

COOPER RESERVOIR
WATER STUDY
SULPHUR SPRINGS, TEXAS

LIME & HYDROFLUOSILICIC ACID FEED RATES SULPHUR SPRINGS WTP

BLACK & VEATCH DALLAS, TEXAS PROJECT NO. 14719.100

TABLE 5-7. AVERAGE CHEMICAL USE AND COST OF TREATMENT BASED ON RAW WATER PUMPAGE AND 1988 CHEMICAL PRICES

	Chemical	1985		1986		1987	
	Price $/lb	Quantity lb/MG	Cost $/MG	Quantity lb/MG	Cost $/MG	Quantity lb/MG	Cost $/MG
Alum	0.0457	1483.5	67.80	888.8	40.61	854.1	39.03
Lime	0.0326	277.9	9.06	196.4	6.40	260.5	8.49
Hydrofluosilicic Acid	0.1755	41.2	7.23	44.4*	7.79	40.4	7.09
Powdered Activated Carbon	0.3500	71.4	24.99	126.9	44.42	104.6	36.61
Chlorine	0.2145	44.9	9.63	46.4	9.95	55.6	11.93
Ammonia	0.2400	10.8	2.59	12.3	2.95	20.8	4.99
Total			121.30		112.12		108.14
Change From Previous Year					-9.18		-3.98

* July and August omitted because pump was inoperative

cause peak rates of use were lower, and carbon was used for only four months (it was used for five months in 1986). Chlorine and ammonia use, per million gallons of raw water entering the plant, were significantly higher in 1987 than the previous two years.

The average chloramine residual leaving the plant has been essentially constant at 3.0 mg/l (as chlorine) for all three years; however, there was increased emphasis in maintaining the minimum daily residual at this level in 1987. Chloramine residuals are also measured at the WWTP. There is practically no decrease during the winter, but there is a large chloramine demand in the distribution system in the summer. This is produced by the tastes and odor compounds present in the finished water during these periods.

Additional information on chemical use is summarized in Appendix A.

5.3.3 Finished Water Quality

Raw and finished water turbidity and total alkalinity are measured three times per day (12:00 a.m., 8:00 a.m., 4:00 p.m.). Raw water pH and finished water fluoride concentration and chloramine residual are also measured on this schedule. Finished water pH is measured at three-hour intervals. Raw water turbidity, alkalinity, and pH have been previously discussed in Chapter 3.0. This section will focus on finished water quality.

Monthly average total alkalinty and pH in the finished water is shown on Figure 5-8. Sufficient lime is being added to replace the raw water alkalinity consumed by the alum and elevate the pH. Average pH values ranging from 8.6 to 8.8 have been maintained since August 1986. Larger variations occurred prior to that time, but the average monthly pH has consistently been above 8.3. Providing a finished water with a high pH has reduced the corrosive properties of the water, and it will not produce a scale buildup on the walls of pipes in the distribution

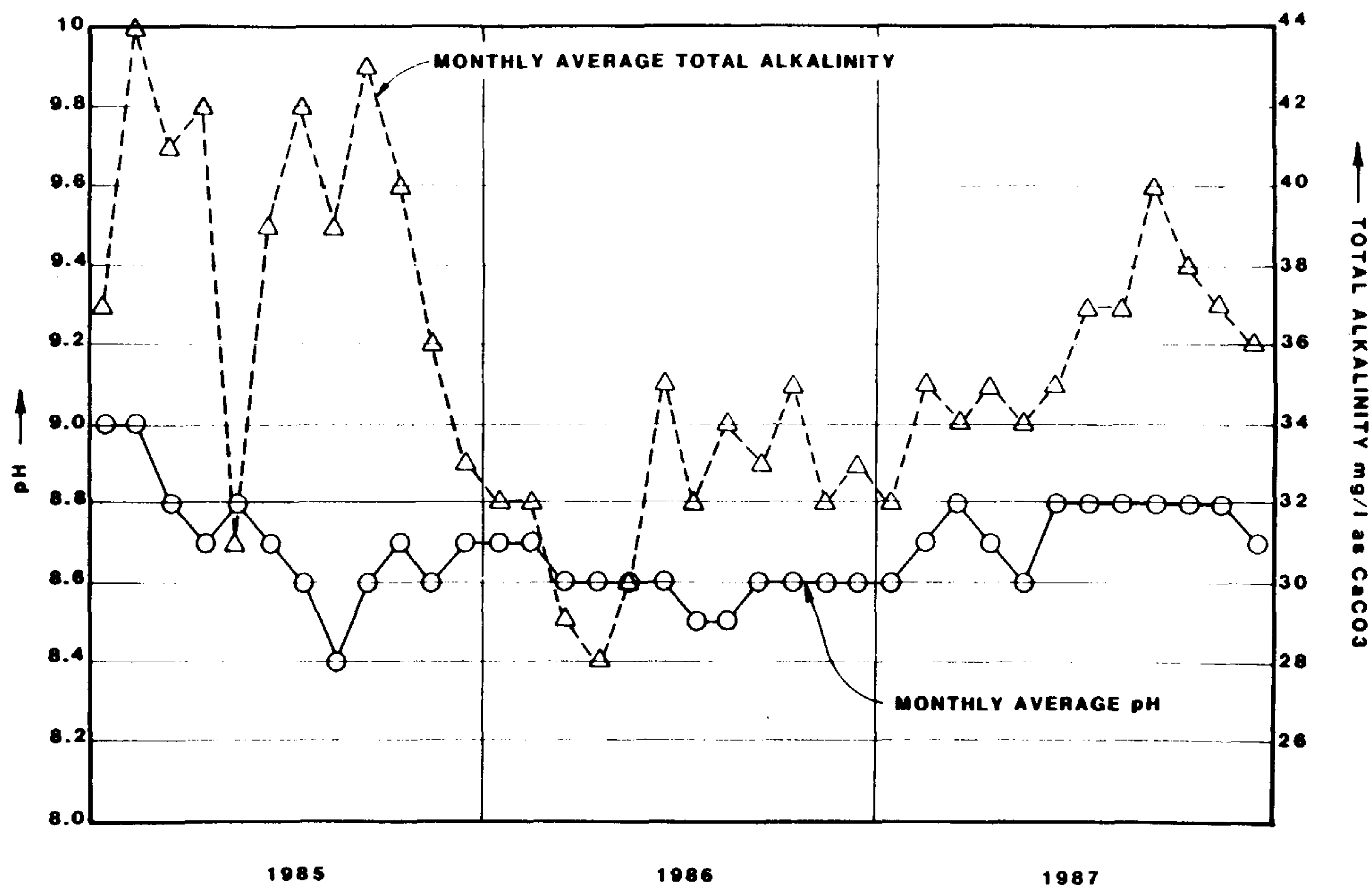

FIGURE 5-8

COOPER RESERVOIR
WATER STUDY
SULPHUR SPRINGS, TEXAS

FINISHED WATER pH & TOTAL ALKALINITY SULPHUR SPRINGS WTP

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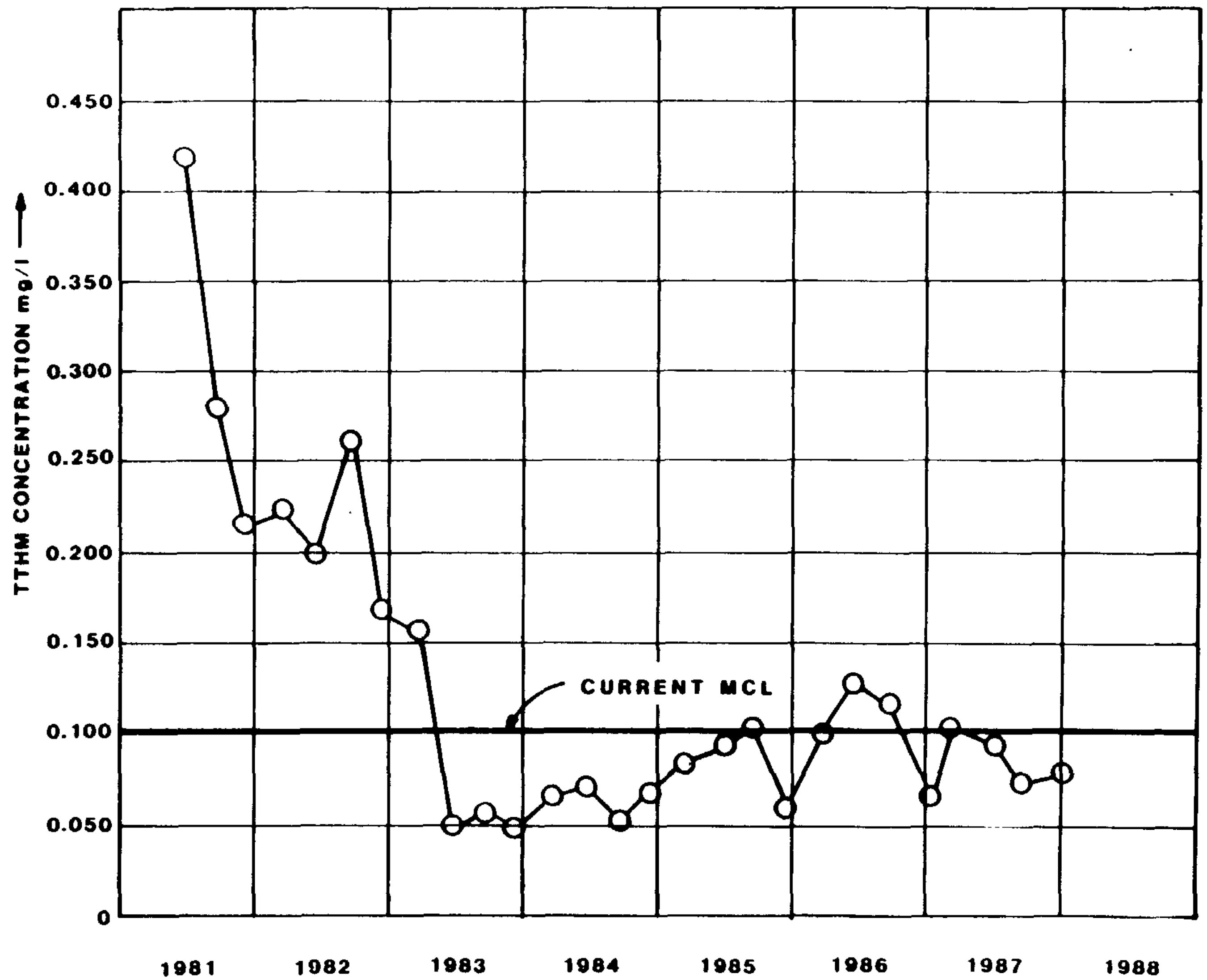

FIGURE 5-9

COOPER RESERVOIR
WATER STUDY
SULPHUR SPRINGS, TEXAS

TOTAL TRIHALOMETHANES
1981 - 1988
SULPHUR SPRINGS, TEXAS

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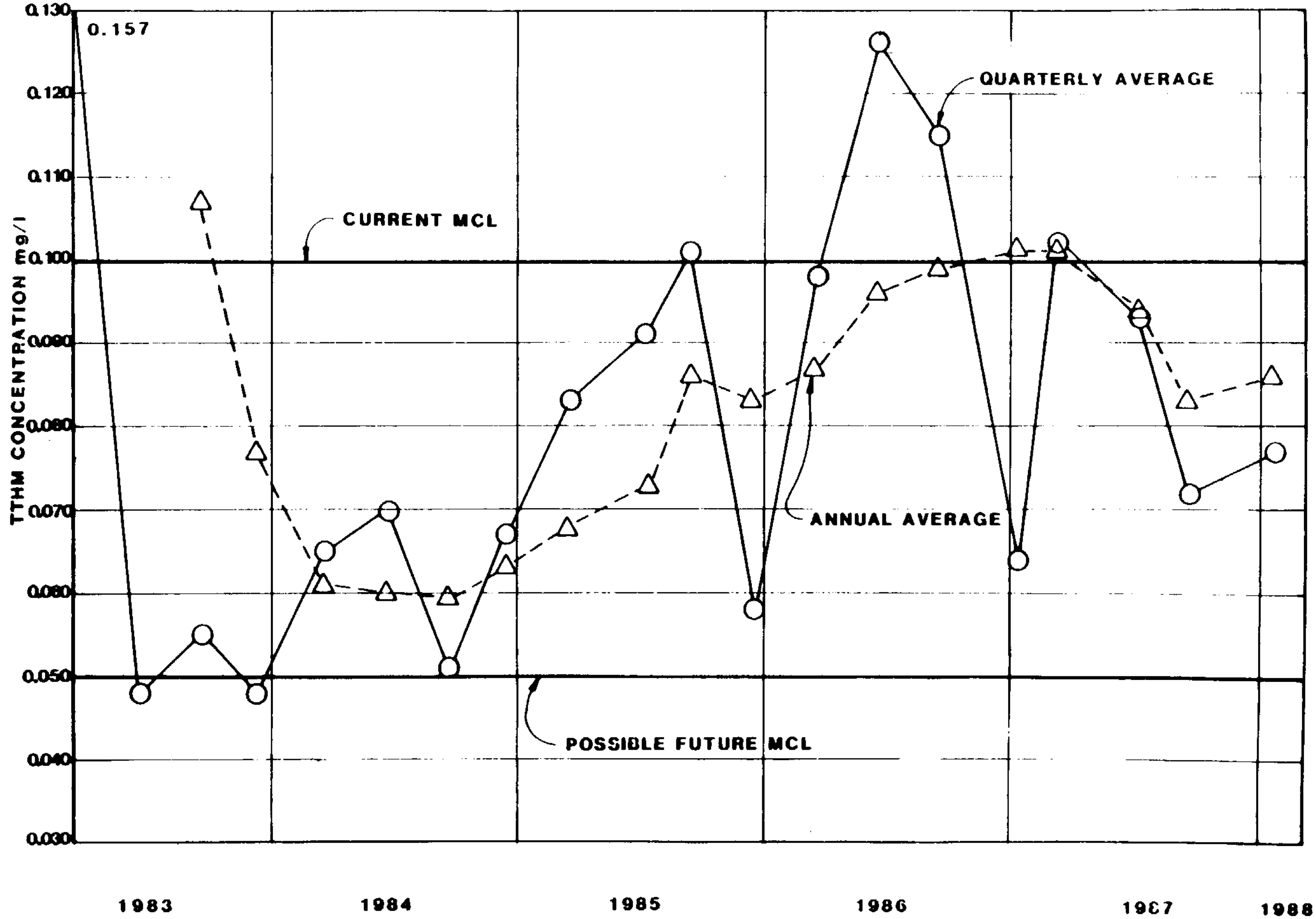

FIGURE 5-10

COOPER RESERVOIR
WATER STUDY
SULPHUR SPRINGS, TEXAS

TOTAL TRIHALOMETHANES
1983 - 1987
SULPHUR SPRINGS, TEXAS

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TABLE 5-9. CHEMICAL QUALITY OF THE FINISHED WATER

Constituent	MCL	02/01/84 (1)	10/24/85 (1)	10/19/87 (1)	02/19/88 (2)
Calcium, mg/l	----	28	32	33	---
Chloride, mg/l	250	11	13	12	---
Fluoride, mg/l	4	1.2	1.3	1.1	1.2
Magnesium, mg/l	----	4	4	2	<0.001
Nitrate, mg/l as N	10	0.06	0.19	0.33	1.2
Sodium, mg/l	----	8	7	7	9
Sulfate, mg/l	250	73	75	71	---
Potassium, mg/l	----	5	5	----	---
Total Hardness, mg/l as $CaCo_3$	----	85	97	92	13.7
pH	6.5-8.5	6.9	7.2	7.4	8.2
Conductivity, umhos/cm	----	256	286	282	---
Total Alkalinity, mg/l as $CaCo_3$	----	16	19	21	8
Bicarbonate, mg/l	----	20	23	26	---
Carbonate, mg/l	----	0	0	0	---
Total Dissolved Solids, mg/l	500	140	150	145	120
Nitrite, mg/l	----	----	----	----	1.5
Arsenic, mg/l	0.05	<0.01	<0.01	<0.01	---
Barium, mg/l	1.0	<0.5	<0.5	<0.5	---
Cadmium, mg/l	0.01	<0.005	<0.005	<0.005	<0.001
Chromium, mg/l	0.05	<0.02	<0.02	<0.02	---
Copper, mg/l	----	<0.02	<0.02	<0.02	---
Iron, mg/l	0.3	0.11	0.08	0.05	0.042
Lead, mg/l	0.05	<0.02	<0.02	<0.02	0.023
Manganase, mg/l	0.05	0.02	<0.02	<0.02	---
Mercury, mg/l	0.002	<0.0002	<0.0002	0.0004	<0.001
Selenium, mg/l	0.01	<0.002	<0.002	<0.002	<0.001
Silver, mg/l	0.05	<0.01	<0.01	<0.01	<0.001
Zinc, mg/l		<0.02	<0.02	<0.02	---
Endrin	0.0002	<0.0002	----	----	<0.01
Lindane	0.004	<0.00003	----	----	<0.01
Methoxychlor	0.1	<0.0005	----	----	<0.01
Toxaphene	0.005	<0.005	----	----	<0.01
2,4-D	0.1	<0.020	----	----	<0.01
2,4,5-TP	0.01	<0.005	----	----	<0.01

(1) Source: Texas Department of Health
(2) Source: Gymnurs Laboratories, Inc.

TABLE 5-9. CHEMICAL QUALITY OF THE FINISHED WATER

Constituent	MCL	02/01/84 (1)	10/24/85 (1)	10/19/87 (1)	02/19/88 (2)
Calcium, mg/l	----	28	32	33	---
Chloride, mg/l	250	11	13	12	---
Fluoride, mg/l	4	1.2	1.3	1.1	1.2
Magnesium, mg/l	----	4	4	2	<0.001
Nitrate, mg/l as N	10	0.06	0.19	0.33	1.2
Sodium, mg/l	----	8	7	7	9
Sulfate, mg/l	250	73	75	71	---
Potassium, mg/l	----	5	5	----	---
Total Hardness, mg/l as $CaCo_3$	----	85	97	92	13.7
pH	6.5-8.5	6.9	7.2	7.4	8.2
Conductivity, umhos/cm	----	256	286	282	---
Total Alkalinity, mg/l as $CaCo_3$	----	16	19	21	8
Bicarbonate, mg/l	----	20	23	26	---
Carbonate, mg/l	----	0	0	0	---
Total Dissolved Solids, mg/l	500	140	150	145	120
Nitrite, mg/l	----	----	----	----	1.5
Arsenic, mg/l	0.05	<0.01	<0.01	<0.01	---
Barium, mg/l	1.0	<0.5	<0.5	<0.5	---
Cadmium, mg/l	0.01	<0.005	<0.005	<0.005	<0.001
Chromium, mg/l	0.05	<0.02	<0.02	<0.02	---
Copper, mg/l	----	<0.02	<0.02	<0.02	---
Iron, mg/l	0.3	0.11	0.08	0.05	0.042
Lead, mg/l	0.05	<0.02	<0.02	<0.02	0.023
Manganase, mg/l	0.05	0.02	<0.02	<0.02	---
Mercury, mg/l	0.002	<0.0002	<0.0002	0.0004	<0.001
Selenium, mg/l	0.01	<0.002	<0.002	<0.002	<0.001
Silver, mg/l	0.05	<0.01	<0.01	<0.01	<0.001
Zinc, mg/l		<0.02	<0.02	<0.02	---
Endrin	0.0002	<0.0002	----	----	<0.01
Lindane	0.004	<0.00003	----	----	<0.01
Methoxychlor	0.1	<0.0005	----	----	<0.01
Toxaphene	0.005	<0.005	----	----	<0.01
2,4-D	0.1	<0.020	----	----	<0.01
2,4,5-TP	0.01	<0.005	----	----	<0.01

(1) Source: Texas Department of Health
(2) Source: Gymnurs Laboratories, Inc.

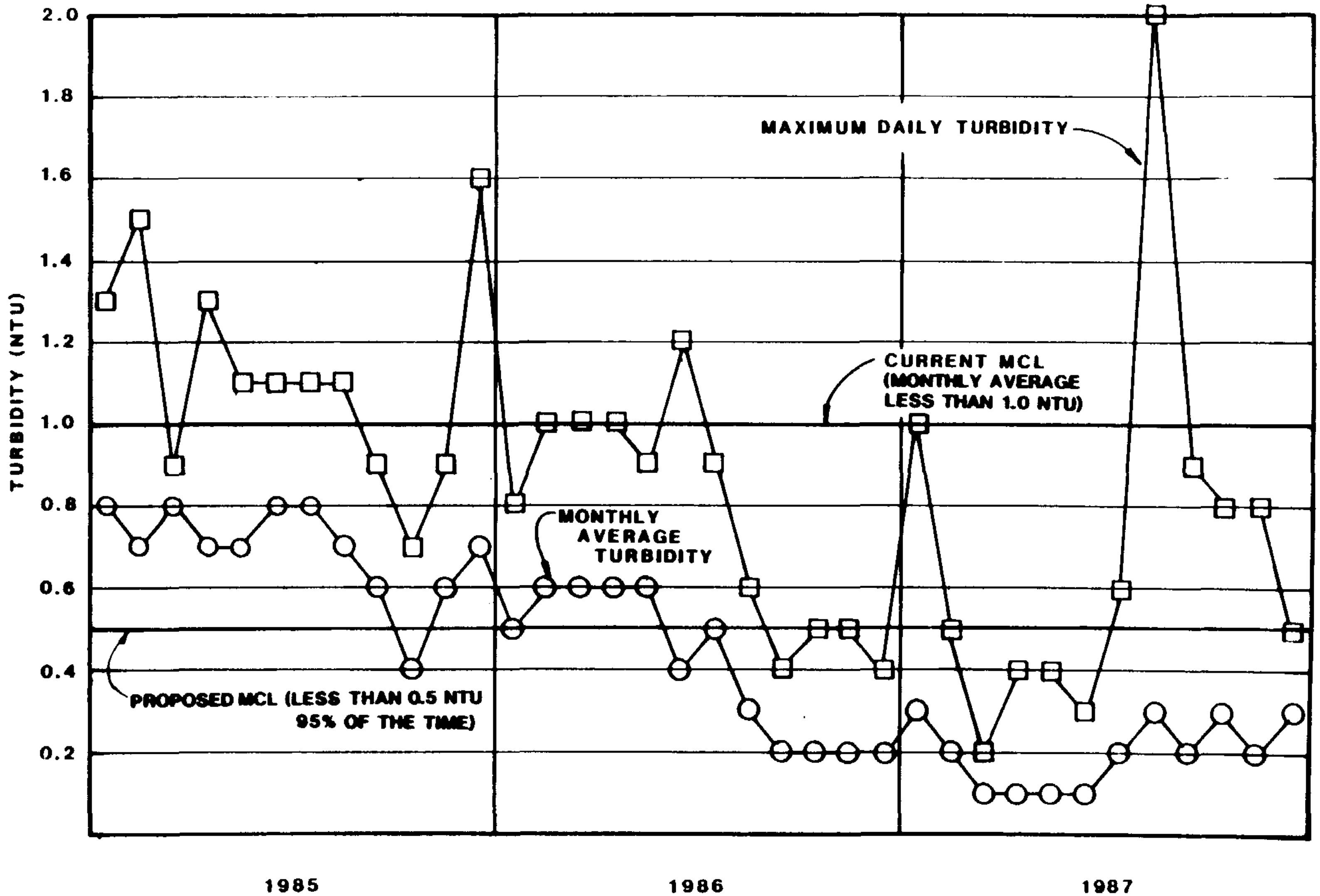


FIGURE 5-11

COOPER RESERVOIR
WATER STUDY
SULPHUR SPRINGS, TEXAS

FINISHED WATER TURBITY
SULPHUR SPRINGS WTP

BLACK & VEATCH DALLAS, TEXAS

PROJECT NO. 14719.100

successfully reduced the average turbidity below the proposed MCL of 0.5 NTU. However, the maximum daily turbidity is frequently above this value. If we assume all of the readings taken on the day when the maximum turbidity is greater than 0.5 NTU are above this value, and all of the readings taken on days when the average turbidity is less than 0.5 NTU are below this value, the existing filters are producing an effluent below 0.5 NTU 97 percent of the time. This is greater than the proposed minimum criteria which requires the effluent to be below 0.5 NTU 95 percent of the time. The existing filters will not require upgrading to meet the proposed MCL, but the present high standard of operation must be continued in the future.

5.4 GROUNDWATER PRODUCTION FACILITIES

The City of Sulphur Springs obtains all raw water from Lake Sulphur Springs and has no groundwater production facilities. Many of the smaller towns within Hopkins County do have groundwater production facilities, and buy water from the City of Sulphur Springs only in times of peak demand.

Groundwater production facilities used by the wholesale water users are listed in Table 5-10. The facilities are also shown on Figure 3.

TABLE 5-10. GROUNDWATER PRODUCTION FACILITIES - HOPKINS COUNTY

Water District or City	Population	Number of Connections	Number of Wells	Total Capacity mgd
Martin Springs W.D.	2220	740	6	0.871
Brinker W.D.	1524	N/A	3	0.468
Cornersville W.D.	627	209	2	0.331
North Hopkins W.D.	4080	1360	0	0*
Gafford Chapel W.D.	990	330	3	0.408***
Shady Grove No. 2 W.D.	483	155	0	0*
Brashear WSC.	780	254	0	0*
Pleasant Hill W.D.	180	60	0	0*
City of Como	750	250	1	0.202

TABLE 5-10 (continued). GROUNDWATER PRODUCTION FACILITIES - HOPKINS COUNTY

Water District or City	Population	Number of Connections	Number of Wells	Total Capacity mgd
City of Cumby	1080	366	4	0.370
Miller Grove W.D.	1140	371	4	0.233
City of Tira	273	90	0	0**
Pickton WSC	528	176	1	0.144
Saltillo WSC	339	113	0	****
Shirley WSC	1365	455	5	0.726

* All treated water is purchased from City of Sulphur Springs.
** All treated water is purchased from North Hopkins W.D.
*** Includes 0.216 mgd well under construction.
**** Water supplied by City of Mt. Vernon.

5.5 TREATED WATER STORAGE AND DISTRIBUTION FACILITIES

5.5.1 City of Sulphur Springs

The existing Sulphur Springs water distribution system is comprised of a network of various size pipelines interconnected with each other and with three elevated storage tanks.

Sizes and lengths of pipe used in the distribution system are listed in Table 5-11.

TABLE 5-11. PIPING NETWORK - CITY OF SULPHUR SPRINGS

Pipe Size in	Total Length ft
1-1/2	4,845
2	48,025
3	740
4	14,955
6	293,030
8	112,205
10	1,490
12	90,470
14	5,450
18	2,530
20	1,770
27	420

Elevated water storage in the City distribution system is provided as listed below.

Tank Location	Capacity, gallons
Main at Tomlison	250,000
Carter at Whitworth	500,000
Morris at College	750,000

The existing water lines and elevated storage tanks in the City are shown on Figure 2.

5.5.2 Hopkins County

The water distribution system supplying treated water to water districts and cities in Hopkins County has pipes ranging in size from 1-1/2 to 12 inches in diameter. The County distribution system is shown on Figure 3.

The treated water storage facilities located in the County are listed in Table 5-12 below.

TABLE 5-12. TREATED WATER STORAGE FACILITIES - HOPKINS COUNTY

District or City	Capacity of Ground Storage 1000 gal	Capacity of Elevated Storage 1000 gal	Total Capacity 1000 gal
Martin Springs W.D.	0	200	200
Brinker W.D.	247	200	447
Cornersville W.D.	3	100	103
North Hopkins W.D.	65	334	399
Gafford Chapel W.D.	3	146	149
Shady Grove No. 2 W.D.	106	0	106
Brashear W.D.	106	0	106
Pleasant Hill W.D.	0	34	34
City of Como	100	70	170
City of Cumby	100	50	150
Miller Grove W.D.	105	38	143
City of Tira	0	0	0
Pickton WSC	42	100	142
Saltillo WSC	-----	-----	-----
Shirley WSC	199	150	349

The water distribution system in Hopkins County is supplied from the City of Sulphur Springs distribution system, and from wells in some locations.

6.0 WATER REQUIREMENTS

6.1 POPULATION

6.1.1 Hopkins County

The population of Hopkins County dropped during the years of 1940 to 1960, according to U.S. Census Bureau data. During the 1960's and 1970's, however, the population increased by approximately 36 percent. Growth has continued over the past seven years, with an estimated 1987 population for Hopkins County of approximately 30,000. U.S. Census Bureau population data for Hopkins County are shown in Table 6-1.

TABLE 6-1. HISTORICAL POPULATION HOPKINS COUNTY

Year	Population
1930	29,410
1940	30,274
1950	23,494
1960	18,594
1970	20,710
1980	25,247
1987[(1)]	30,000

(1) Estimate from Texas Water Development Board Projections.

It is expected that Hopkins County will experience considerable population growth over the next 50 years. Population projections for the years 1990 to 2030 were supplied by the Texas Water Development Board (TWDB). These TWDB projections include a high and a low population projection for the County. The City has also recently updated its Comprehensive Plan and predicts larger increases in population for the years 1990 to 2005 than projected by the TWDB. The TWDB figures do not account for the impact on population growth from Cooper Reservoir and the new South Sulphur State Park. Therefore, the TWDB population projections were adjusted. For the years 1990 to 2000, the percent increase in population developed in the City's Comprehensive Plan was used in this study. From the years 2000 to 2030, the percent increases used were those developed by the TWDB for the high population projection. These percents of increase were then projected from the year 2000 population developed by the City's data. The projected population for the year 2040 was estimated using the projections described above. The projected population for Hopkins County is shown in Table 6-2 and shown graphically on Figure 6-1.

TABLE 6-2. PROJECTED POPULATION - HOPKINS COUNTY

Year	Population
1990	33,009
2000	46,934
2010	52,047
2020	59,898
2030	68,934
2040	84,460

6.1.2 City of Sulphur Springs

The population of the City of Sulphur Springs, in contrast to the population of Hopkins County, has steadily increased since 1930. The years from 1960 to 1980 showed the largest growth, with a 36 percent increase. Sulphur Springs has continued to grow through the 1980's, with an estimated population of approximately 16,000 in 1987. U.S. Census Bureau population data for the City are listed in Table 6-3.

TABLE 6-3. HISTORICAL POPULATION - SULPHUR SPRINGS

Year	Population
1930	5,417
1940	6,742
1950	8,991
1960	9,160
1970	10,642
1980	12,804
1985 (1)	15,000

(1) Estimate from City's Comprehensive Plan.

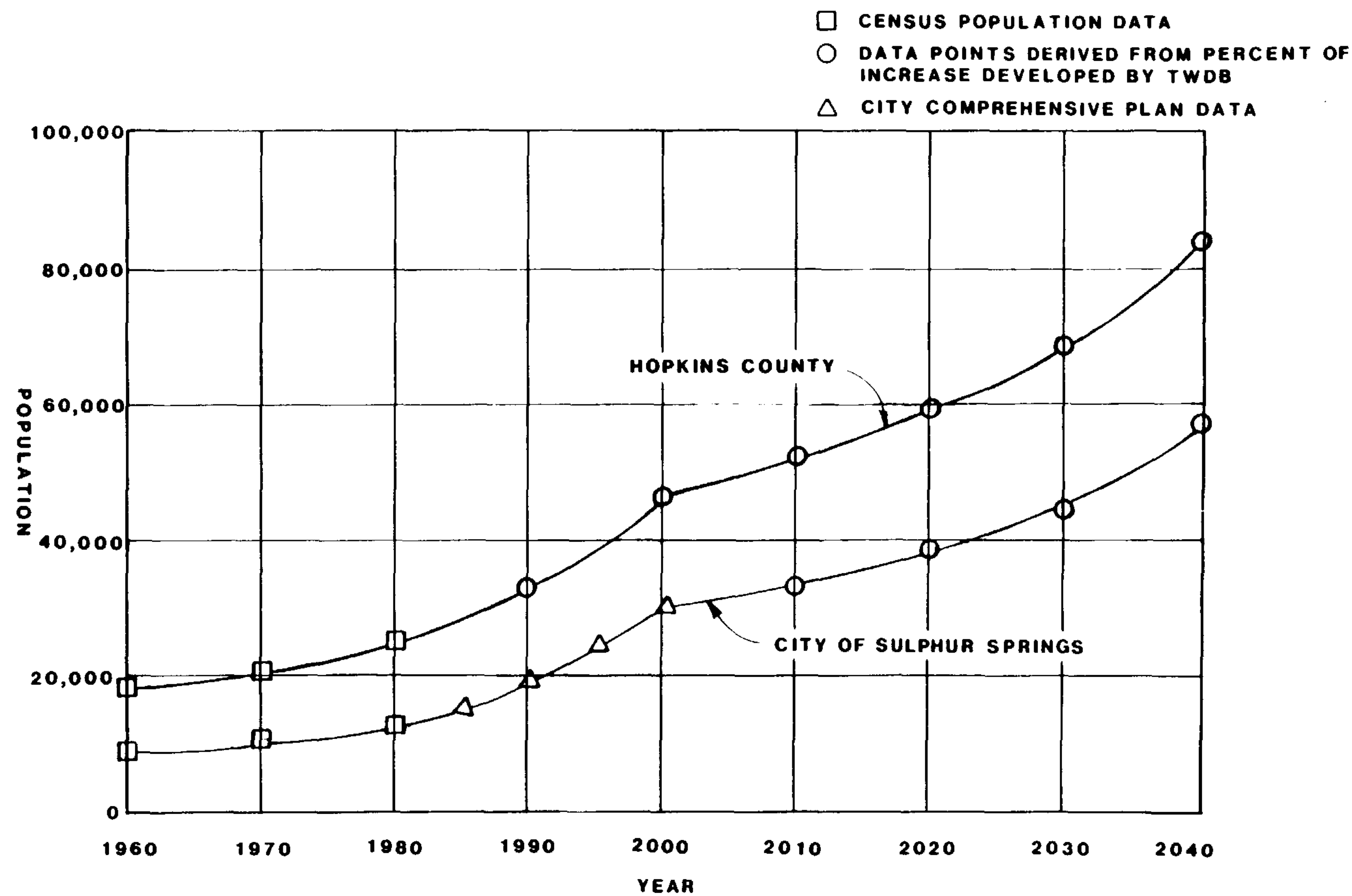


FIGURE 6-1

COOPER RESERVOIR
WATER STUDY
SULPHUR SPRINGS, TEXAS

POPULATION PROJECTIONS
CITY OF SULPHUR SPRINGS
& HOPKINS COUNTY

BLACK & VEATCH DALLAS, TEXAS PROJECT NO. 14719.100

The City of Sulphur Springs, like Hopkins County, is expected to experience considerable growth over the next 50 years. Population projections for the years 1990 to 2000 were taken from the City's Comprehensive Plan. The years from 2000 to 2030 were projected using the percent increases developed for that time period by the TWDB. The population for the year 2040 was projected using the populations described above.

The population projections for the City of Sulphur Springs are listed in Table 6-4 and are shown graphically on Figure 6-1.

TABLE 6-4. PROJECTED POPULATION - CITY OF SULPHUR SPRINGS

Year	Population
1990	19,000
2000	30,000
2010	33,402
2020	38,726
2030	44,900
2040	57,501

6.2 WATER DEMAND

A water distribution system must be able to supply water at rates which fluctuate over a wide range. Rates most important to the design and operation of a water supply and treatment system are annual average day (AAD) and maximum day (MD). The maximum hour demand is usually the most critical rate; however, maximum hour records are not kept at the treatment plant and cannot be addressed here.

Annual average day use is the total annual volume of water delivered to the distribution system divided by the number of days in the year. This rate is used as a basis for projecting maximum day and for estimating revenues and operating costs. The firm yield of the City's water supply must be able to meet the annual average day demand.

Maximum day use is the largest quantity of water used on any day of the year. The maximum day rate is used to size water supply and treatment facilities and to evaluate distribution system capability.

6.2.1 Historical

Daily raw water pumpage records for years 1980 through 1987 were furnished by the City staff. Historical annual average day and maximum day raw water pumpages and demand ratios are shown in Table 6-5.

TABLE 6-5. HISTORICAL RAW WATER PUMPAGE BY THE CITY OF SULPHUR SPRINGS

Year	Annual Average Day mgd	Maximum Day mgd	MD to AAD Ratio
1980	3.64	5.86	1.61
1981	3.56	5.83	1.64
1982	3.42	5.03	1.47
1983	4.05	6.13	1.51
1984	3.94	6.03	1.53
1985	3.91	6.44	1.65
1986	3.51	6.23	1.77
1987	3.15	6.44	2.04
		Average =	1.65

Daily treated water pumpage records for years 1960 through 1987 were also furnished. Treated water annual average day and maximum day pumpages, and the corresponding ratios for years 1980 through 1987, are shown in Table 6-6.

TABLE 6-6. HISTORICAL TREATED WATER PUMPAGE BY CITY OF SULPHUR SPRINGS

Year	Annual Average Day mgd	Maximum Day mgd	MD to ADD Ratio
1980	3.03	4.74	1.56
1981	3.03	5.51	1.82
1982	3.16	4.65	1.47
1983	3.34	5.90	1.77
1984	3.32	5.57	1.68
1985	3.29	5.38	1.64
1986	2.88	5.17	1.80
1987	2.97	5.37	1.81
		Average =	1.73

Per capita water use was computed using the 1980 treated water records and the 1980 U.S. Census Bureau data. The average day water demand for wholesale water used outside of the City was 0.93 mgd from records supplied by City staff. The water demand for the City was 3.03 mgd minus 0.93 mgd, or 2.10 mgd. The 1980 population was 12,804; therefore, average water demand was 164 gallons per capita per day (gpcd).

6.2.2 Projected

Future water requirements are based on the population projections, historical pumpage, and the average water demand discussed previously. The maximum day to annual average day (MD/AAD) ratio is expected to peak at 1.85 and maintain that level throughout the planning period because of water conservation measures.

The projected average annual day flows for the years 1990 to 2040 were developed by multiplying the population at a given year by 164 gpcd. The projected maximum day flows were obtained by multiplying the projected AAD by the MD/AAD factor of 1.85. The projected water demands for Hopkins County are shown in Table 6-7. The projected demands are shown graphically on Figure 6-2.

TABLE 6-7. PROJECTED WATER USE - HOPKINS COUNTY

Year	Average Annual Day mgd	Maximum Day mgd
1990	6.41	10.01
2000	7.70	14.25
2010	8.53	16.78
2020	9.82	18.17
2030	11.31	20.92
2040	13.85	26.62

The water demands for the City were also developed using the same method as described for the County projections. The City's water demand projections are shown in Table 6-8 and shown graphically on Figure 6-3.

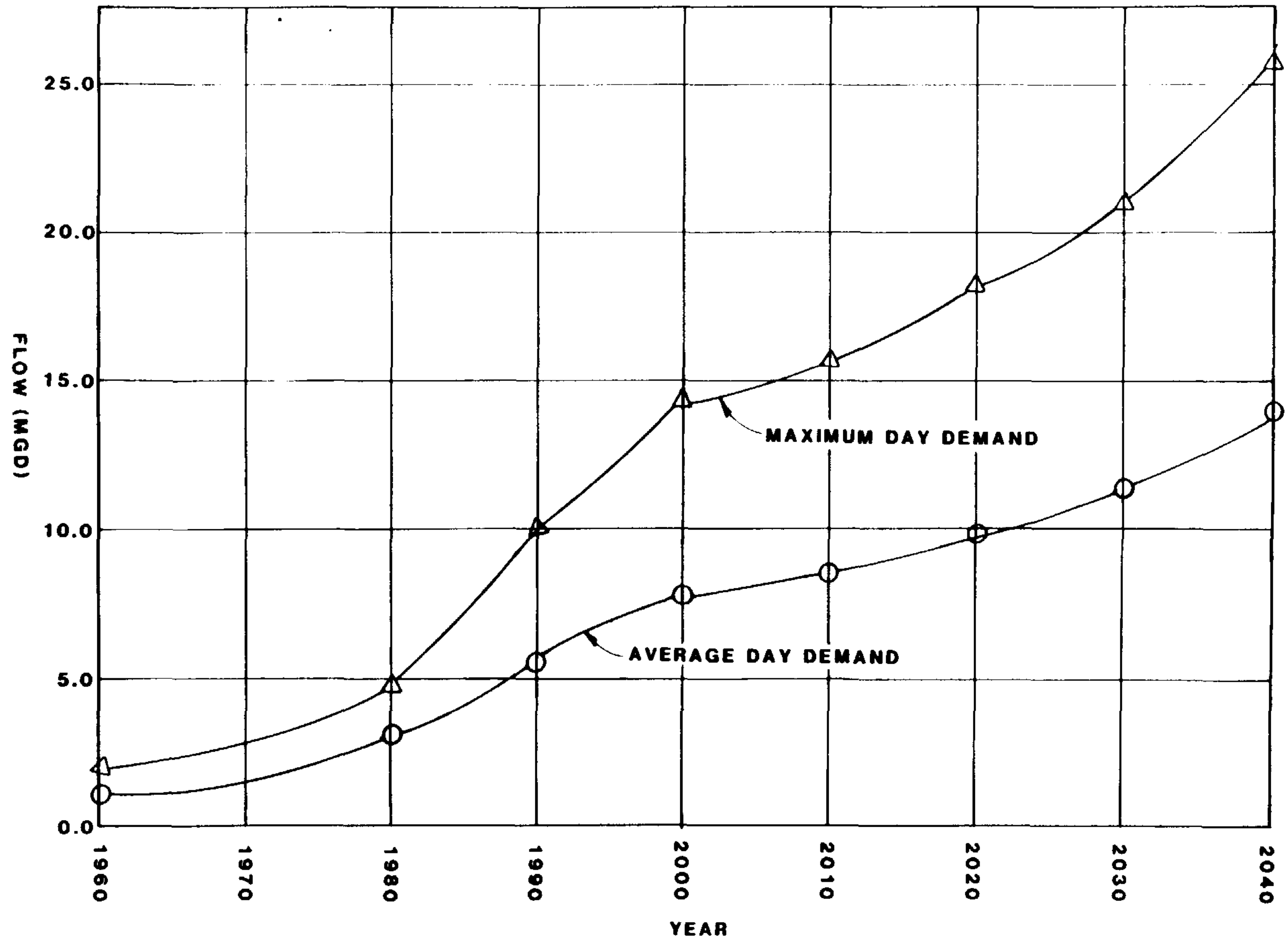


FIGURE 6-2

COOPER RESERVOIR
WATER STUDY
SULPHUR SPRINGS, TEXAS

WATER DEMAND PROJECTION
HOPKINS COUNTY

BLACK & VEATCH DALLAS, TEXAS

PROJECT NO. 14719.100

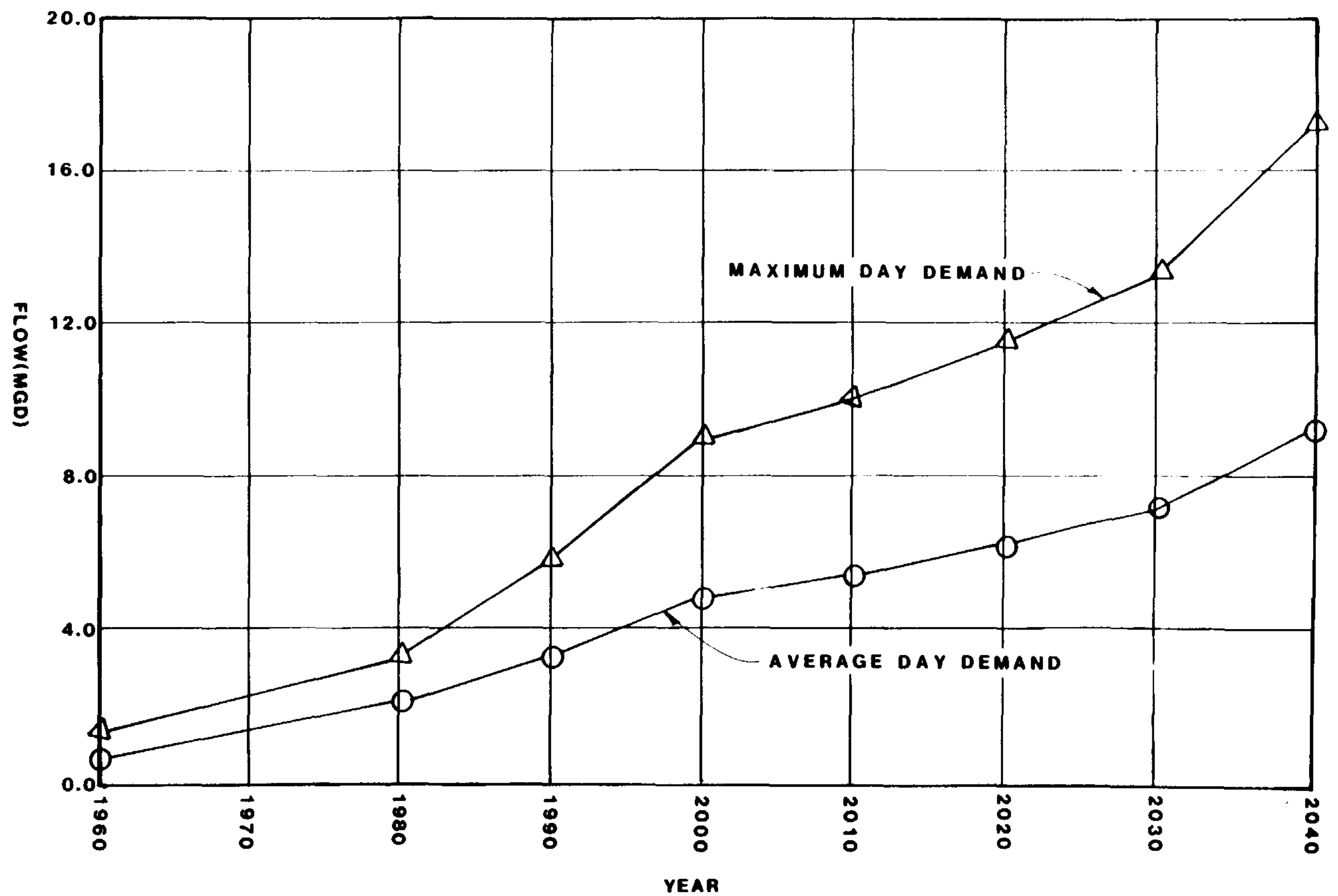


FIGURE 6-3

COOPER RESERVOIR
WATER STUDY
SULPHUR SPRINGS, TEXAS

WATER DEMAND PROJECTION
CITY OF SULPHUR SPRINGS

BLACK & VEATCH DALLAS, TEXAS

PROJECT NO. 14719.100

TABLE 6-8. PROJECTED WATER USE - CITY OF SULPHUR SPRINGS

Year	Average Annual Day mgd	Maximum Day mgd
1990	3.20	5.92
2000	4.92	9.10
2010	5.48	10.14
2020	6.35	11.75
2030	7.36	13.62
2040	9.43	17.45

7.0 COOPER RESERVOIR CONVEYANCE FACILITIES

A primary objective of this study is to determine the most economical alternative for developing the City's share of Cooper Reservoir water to meet Sulphur Springs' and Hopkins County's future water needs. The conveyance facilities are the most expensive part of this development. The facilities consist of the raw water intake, pumping station, and pipeline. Described below are various alternatives for achieving the conveyance of Cooper Reservoir water to Sulphur Springs.

7.1 INTAKE/PUMPING STATION ALTERNATIVES

Three locations have been identified on Cooper Reservoir for the proposed raw water intake/pumping station. The locations are Finley Branch (FB), Harpers Hill (HH), and East Bank (EB) as indicated on Figure 4. The intake/pumping station at each location has been conceptually developed for two alternative pumping rates: (1) the projected average annual day flow (14 mgd), and (2) the projected maximum day flow (26 mgd). For each of the above pumping capacities, each intake/pumping station location has also been evaluated for two alternative types: (1) channel type intake and, (2) a submerged onshore intake. The intake/pump station developed for the maximum day flow (26 mgd) involves using only the water from Cooper Reservoir as a raw water source. The annual average day alternative involves continued use of Lake Sulphur Springs as a secondary raw water source.

The water level in Cooper Reservoir will not be static, but will change with time. The conservation pool is between elevations 415.00 and 440.00 msl, and the high water elevation is elevation 446.00. The intake structure must be able to divert water when the lake level is at any point in this range.

It is also important to be able to obtain the best quality water in the lake. During summer months, intense solar radiation warms the water near the surface of the lake, and produces thermal stratification which prevents vertical mixing. Oxygen entering the water from the atmosphere is not transferred downward, and the dissolved oxygen eventually becomes depleted near the bottom of the lake. The anaerobic conditions produced in this zone convert insoluble iron and manganese in the sediments to soluble ions, which must be removed if this water enters the treatment plant. Water obtained from this level will also contain high concentrations of compounds which produce taste and odors. Sunlight intensity decreases in the fall and the water cools. Eventually, the temperature becomes uniform and the water is mixed by the winter winds. This brings dissolved oxygen down to the bottom of the lake and the soluble iron and manganese is precipitated from solution. Suitable raw water can be obtained from greater depths in the lake after this occurs.

Typical summer and winter profiles expected in Cooper Lake are illustrated on Figures 3-3 and 3-4 and discussed in Section 3.2.2.2. An intake having multiple drawoff elevations will be required to consistently obtain the best water available in this lake.

Another lake feature that affects intake station design is the topography of the lake bed and shoreline. The most desirable intake structure is located closest to deep water. This condition usually exists at the dam, and a common location for the intake station is at or adjacent to the dam. The U.S. Army Corps of Engineers has expressed concerns that placing the intake structure near the dam may interfere with the outlet works operations. They have also expressed concern that the East Bank location may interfere with the expeditious construction of the dam.

Sites on the north shore of the reservoir have been ruled out because of the additional length of transmission main required to reach these locations. Another reason the north shore sites were not investigated further was because deep water is not close to the shore. The sites identified on the south side of the lake (Finley Branch, Harpers Hill, and East Bank) have been selected as possible sites due to the steep embankments along the shore and the closeness to deep water.

The routing of electrical power to the sites has been discussed with TU Electric. The electrical utility is awaiting a final site selection before proceeding with a detailed cost evaluation. Because the intake/pumping station will be located at a remote site, and will be operated automatically for extended periods of time, the station must be designed to resist vandalism and must be provided with a backup supply of electrical power in the event of a power outage.

The Texas Parks and Wildlife Department is currently negotiating a lease with the Corps of Engineers (COE) for property along the reservoir for a State Park. The Corps of Engineers has developed a Park Master Plan which identifies planned recreation areas within the proposed state park on the south side of the lake. The location of the intake/pumping station and raw water pipeline should be coordinated with the Master Plan in order to avoid interfering with the planned marinas, picnic areas, and entrance and access roads.

In addition to coordinating the selection of intake/pumping station sites with the COE Park Master Plan, site access is a concern. As much as possible, sites were chosen in close proximity to existing access roads. The East Bank intake/pumping station location was evaluated because of its proximity to the existing COE south access road. There is an existing road to the Harpers Hill intake/pumping station site; however, it would require paving to provide an all weather surface. There are no existing roads to the Finley Branch intake/pumping station alternative, and a right-of-way would have to be obtained, the area cleared, and a new road constructed to the site.

7.1.1 Channel Intake Structure

This type intake configuration consists of a trapezoidal channel excavated from the shoreline into the lake. The bottom elevation of the channel is approximately flat at El 410 msl. This elevation is 5.5 feet below the minimum elevation of the permitted storage space, and will allow the channel to accumulate silt and still withdraw water from the reservoir during an extended drought.

The wetwell/pumping station is constructed on shore, with the operating floor above the high water elevation of the reservoir. The 26 mgd station will house four vertical turbine pumps with associated piping and valves, three travelling water screens, and control and instrumentation equipment. Each pump will be sized for one third of the station's rated capacity, to provide a "firm pumping capacity" equal to the rated capacity of the system. The "firm pumping capacity" is the discharge capacity of a station with its largest pump out of service. Travelling water screens would be self-cleaning mechanical screens that operate automatically when the head loss through the screen increases because of trapped debris. The intake is integral to the station structure and will consist of three motor operated sluice gates to control the elevation at which water is withdrawn from the lake.

The 14 mgd station will have three vertical turbine pumps, each designed for one half of the station's rated capacity, to provide a firm pumping capacity equal to the rated capacity of the system. The station will also include two travelling water screens, control and instrumentation equipment, and three motor operated sluice gates. The channel intake configuration is indicated on Figure 7-1.

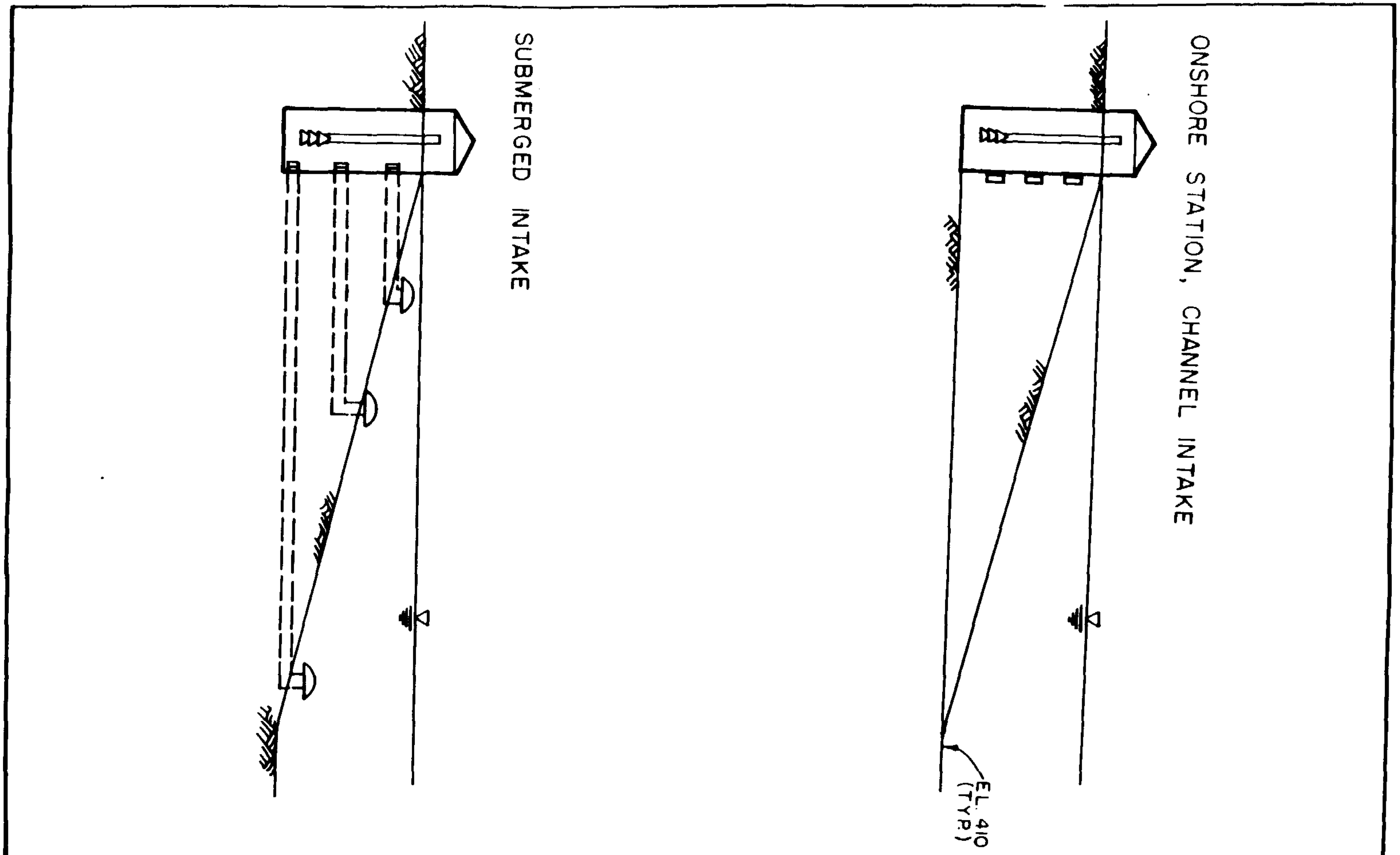


FIGURE 7-1

COOPER RESERVOIR
WATER STUDY
SULPHUR SPRINGS, TEXAS

INTAKE/PUMPING STATION CONFIGURATIONS

BLACK & VEATCH DALLAS, TEXAS

PROJECT NO. 14719.100

7.1.2 Submerged Intake Structure

The submerged intake configuration includes an onshore pumping station and wetwell, with a finished floor elevation above the high water elevation of the reservoir. The layout of the 26 mgd and 14 mgd stations are similar to the channel intake configuration, with one exception. The submerged configuration uses static screens in place of the travelling water screens.

As indicated on Figure 7-1, water is conveyed through submerged pipes from various depths in the reservoir to the onshore pumping station wetwell. A static screen is attached to the reservoir end of each pipe to prevent debris from entering the intake pipe and pump wetwell. Motor operated sluice gates are connected on the wetwell end of each pipe to allow the desired water to be withdrawn. The static screens are cleaned of the accumulated debris by inducing air or water into the pipe and reversing the flow through the screen openings.

7.2 PIPELINE ALTERNATIVES

Three pipeline routes have been identified to convey the raw water from Cooper Reservoir to the City of Sulphur Springs. Items evaluated in considering the pipeline routes included total pipeline length, available right-of-way, existing obstructions, and construction access. The pipeline is sized 36 inches in diameter to convey the maximum day demand (26 mgd) and 30 inches in diameter to convey the average day demand (14 mgd). Pipeline routes are shown on Figure 4.

- Raw Water Pipeline - Route 1. Pipeline route 1 originates at the Finley Branch intake/pumping station and terminates at the existing water plant. It includes the construction of a new access road across undeveloped land from Finley Branch south to the intersection of Highway 71 and F.M. 2285. From that intersection, the pipeline is constructed within state right-of-way along F.M. 2285 southerly to the point where it intersects Highway 19. The pipeline is constructed within the state right-of-way of Highway 19 southwesterly to the City of Sulphur Springs Water Treatment Plant. The cost of the right-of-way for the access road north of Highway 71 includes purchasing land, tree clearing, and grubbing.

- <u>Raw Water Pipeline</u> - <u>Route 1A</u>. Pipeline route 1A follows the same alignment as route 1 until it turns southerly about 2 miles north of Century Lake Dam. The pipeline then crosses Century Lake Dam on a drilled pier pipe bridge. Once across the dam, the pipeline is routed through open farm land, following the easement of the existing raw water lines, to the water treatment plant.
- <u>Raw Water Pipeline</u> - <u>Route 1B</u>. The alignment of route 1B is identical to route 1 until it terminates in Lake Sulphur Springs on the north side of the reservoir.
- <u>Raw Water Pipeline</u> - <u>Route 2</u>. This route alternative originates at the Harpers Hill intake/pumping station and follows the alignment of the existing county road that runs southerly toward Sulphur Springs, to a point approximately 4 miles north of the City. At this point, the pipeline will be constructed in a purchased easement across open farm land to another county road which runs southerly to F.M. 2285. The pipeline then continues to the water treatment plant as described in alternative 1.
- <u>Raw Water Pipeline</u> - <u>Route 2A</u>. The alignment of route 2A is identical to route 2 until reaching the north side of Lake Sulphur Springs. The pipeline continues southerly across Lake Sulphur Springs with a submerged pipe located on the lake bed. After crossing the lake, it continues to the water plant.

- Raw Water Pipeline - Route 2B. Pipeline route 2B follows the alignment of route 2 until it terminates into the north side of Lake Sulphur Springs on the north side of the reservoir.
- Raw Water Pipeline - Route 3. Pipeline route 3 originates at the East Bank intake station and continues easterly along the existing Corps of Engineers south access road to State Highway 19. At this point, the pipeline turns south and is constructed within the right-of-way for Highway 19 to the water treatment plant. No right-of-way acquisition is necessary for this alternative.

7.3 ESTIMATED CONSTRUCTION COST OF ALTERNATIVES

Construction costs for the raw water intake/pumping station and the raw water pipeline alternatives were developed using current unit prices, and compared to historical construction costs on similar projects. Detailed cost opinions are presented in Appendix B.

7.3.1 Intake Alternatives

The preliminary construction cost opinions (including 10 percent for contingencies) for the raw water intake/pumping station alternatives are listed in Table 7-1. The costs include access roads and related site work as detailed in Appendix B. The cost for incoming power is not included.

TABLE 7-1. SUMMARY OF CONSTRUCTION COST OPINIONS FOR INTAKE/PUMPING STATION ALTERNATIVES

	14 mgd		26 mgd	
Location	Channel Intake $	Submerged Intake $	Channel Intake $	Submerged Intake $
Finley Branch	2,825,000	3,250,000	3,307,000	3,660,000
Harpers Hill	2,679,000	2,813,000	3,161,000	3,232,000
East Bank	2,690,000	2,952,000	3,185,000	3,340,000

7.3.2 Pipeline Alternatives

The preliminary cost opinions (including 10 percent for contingencies) for the construction of the pipeline alternatives are listed in Table 7-2 below. The costs include easement acquisition, road repairs, and access road construction as detailed in Appendix B.

TABLE 7-2. SUMMARY OF CONSTRUCTION COST OPINIONS FOR PIPELINE ALTERNATIVES

Alternative Route	14 mgd, 30 inch Pipe $	26 mgd, 36 inch Pipe $
1. Finley Branch to WTP	6,009,000*	8,213,000*
1A. Finley Branch to WTP Along Century Dam	6,887,000	N/A
1B. Finley Branch to Lake SS	4,507,000	N/A
2. Harpers Hill to WTP	6,031,000	8,884,000
2A. Harpers Hill to WTP Across Lake SS	9,899,000	N/A
2B. Harpers Hill to Lake SS	4,591,000	N/A
3. East Bank to WTP	6,161.000	8,622,000

*The probable project cost to construct Alternative Route 1 from Finley Branch to Highway 71 is $1,065,000 for a 30 inch pipe, and $1,722,000 for a 36 inch pipe. These costs are given as a consideration in the event of a combined intake with North Texas Municipal Water District and the City of Irving.

8.0 FUTURE WATER TREATMENT FACILITIES

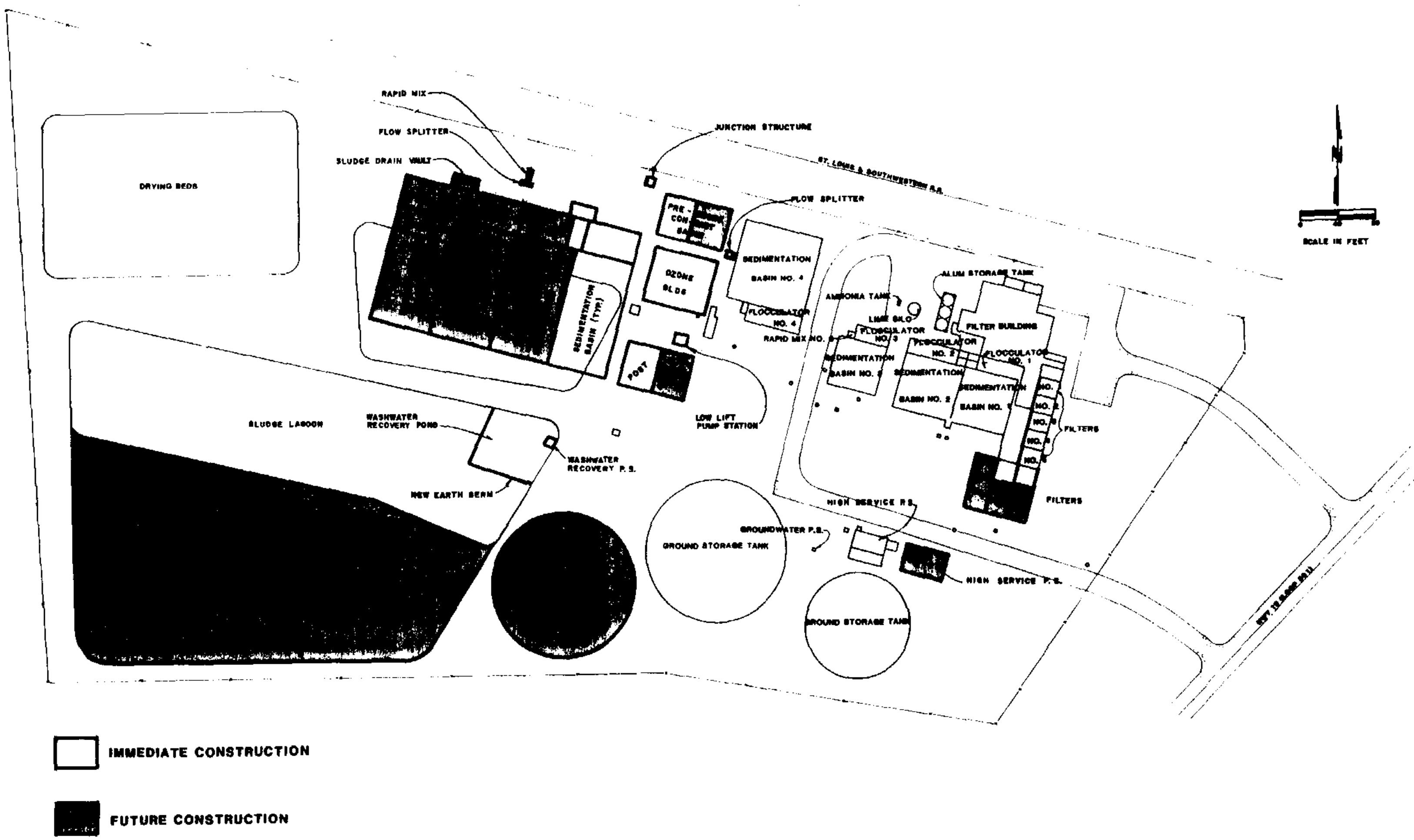
8.1 INTRODUCTION

The purpose of this chapter is to discuss the treatment processes needed to meet anticipated future drinking water standards, and to make recommendations for plant improvements. Treatment processes will be addressed first. Then, the plant improvements will be discussed in general terms. This will be followed by specific discussions on two alternatives for plant expansion. Alternative A is based on construction of four 4 mgd plant expansions. Alternative B involves construction of two 8 mgd plant expansions. Both alternatives include uprating the existing plant from 7 to 10 mgd. An economic analysis is also included for both alternatives.

8.2 TREATMENT PROCESSES

The material discussed in Chapters 3.0, 4.0, and 5.0 provides the bases for developing future treatment processes. Changes are needed to handle the different quality of raw water obtained from Cooper Reservoir and the higher finished water standards mandated by the 1986 Amendments of the SDWA. However, these changes must make maximum use of the existing treatment facilities. A method for accomplishing this goal is illustrated on Figure 8-1. All of the existing facilities will remain in service, with the exception of the small sludge lagoon located east of the existing raw water splitter box. This lagoon will lose about one fourth of its existing volume in the first plant expansion, and lose additional volume with each additional expansion. However, this loss can be replaced by expansion of the existing large lagoon.

The recommended process diagram for this plant layout is shown on Figure 8-2. Prior to detailed design of ozone facilities, pilot testing is recommended. The process diagram and plant layout

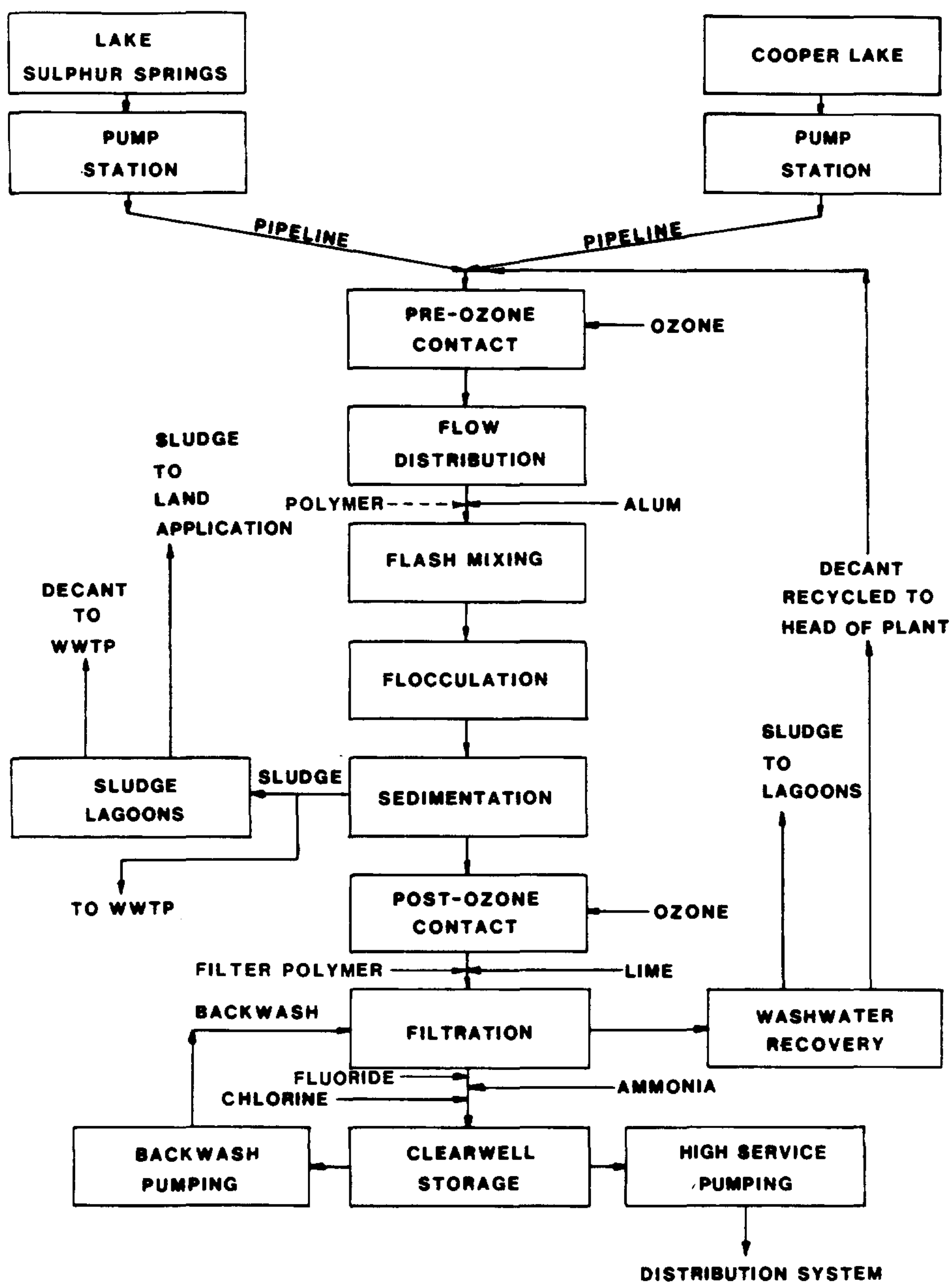


WATER TREATMENT PLANT EXPANSION
ALTERNATIVE A

BLACK & VEATCH DALLAS, TEXAS PROJECT NO. 14719.100

COOPER RESERVOIR
WATER STUDY
SULPHUR SPRINGS, TEXAS

FIGURE 8-1



PROCESS DIAGRAM
FUTURE TREATMENT PLANT

PROJECT NO. 14719.100

BLACK & VEATCH DALLAS, TEXAS

COOPER RESERVOIR
WATER STUDY
SULPHUR SPRINGS, TEXAS

FIGURE
8 - 2

herein assume certain design parameters for ozone. These assumptions are based upon Black & Veatch's experience with similar waters.

Raw water from Lake Sulphur Springs and Cooper Lake will first enter pre-ozone contact basins for oxidation of organic compounds (taste, odor, color, and trihalomethane precursors), and preconditioning of suspended solids so they can be settled easier. The ozone in this basin will also produce partial destruction of bacteria, viruses, algae, and protozoa in the raw water, but disinfection is not the objective of this treatment stage.

Flow leaving the pre-ozone contact basin will enter one of two flow distribution structures (the one now serving the existing plant or a new one designed to handle future plant expansions). These structures will divide the flow into the quantities needed in parallel treatment modules, which are based on the processes used in the existing treatment plant (See Figure 5-2). These processes consist of a flash mixing basin, flocculation basin, and a sedimentation basin operating in series.

Flocculation will be achieved by the addition of alum. Facilities will also be provided for polymer addition, when this is needed to provide satisfactory clarification. Based on past experience at the existing plant, polymer use will be infrequent (probably needed in winter).

Sludge removed from the sedimentation basins will be pumped to a sanitary sewer for disposal in the lagoons provided at the wastewater treatment plant (WWTP), or to one of the onsite lagoons shown on Figure 8-1.

Decant from the onsite lagoons will also be routed to a sanitary sewer and flow to the WWTP. This is being done to prevent recycling of organic contaminants back into the WTP. Ultimate disposal of the sludge collected in these lagoons will be accomplished by land application.

Clarified water discharged from each sedimentation basin will flow to a low head pump station, located south of the ozone building, and be lifted into post-ozone contact basins for disinfection prior to entering the filters. The two stages of ozonation achieve disinfection with a lower total dose of ozone and reduce operating costs.

Water leaving the post-ozone contact basin will flow eastward toward the existing filters. Lime will be used for pH adjustment prior to the water going onto the filters. A filter polymer will also be added at this location, when this is needed to reduce finished water turbidity below 0.5 NTU. The new filters will be dual media units similar to the existing facilities. Additional capacity will be provided by expanding the existing filter gallery southward.

A chloramine residual will be maintained in the distribution system by adding ammonia and chlorine to the filter effluent. Hydrofluosilicic acid will also be added at this location.

The treated water will then flow to ground storage tanks located south of the existing plant. These tanks will continue to provide influent to the high service pumps and the filter backwash pumps. Backwash water will be recovered in a pond constructed in the northeast corner of the existing large lagoon. Decant from this pond will be recycled to the head of the plant. The small amounts of sludge collected in the pond will be pumped into the lagoon for disposal. It will be critical to keep the quantities of sludge in the pond to a minimum, so soluble organic compounds are not released from the sludge and recycled back through the plant.

The use of ozone, for oxidizing soluble organic compounds in the raw water, coagulating suspended particles, and disinfection, is based on the treatment requirements discussed in Chapter 4.0, and prior experience of Black & Veatch with the use of chlorine dioxide in similar raw waters. It is not possible to consistently obtain satisfactory results with chlorine dioxide, when the maximum dosage rate for this compound is limited by the quantities of chlorite and chlorate allowed in the finished water. This, plus the uncertainty over the long-term health effects of these compounds, makes ozone a preferable solution despite its higher initial cost.

The multiple effects of ozone also make it attractive on the basis of chemical operating costs. Its use on raw water obtained from Lake Sulphur Springs will eliminate the need for PAC, reduce the average alum dose from 100 to approximately 80 mg/l, and lower the chlorine and ammonia dosages to the quantities needed to produce a 0.5 mg/l chloramine residual at the far reaches of the distribution system. The effect of these changes on the average cost of treatment is illustrated in Table 8-1.

TABLE 8-1. EXPECTED AVERAGE CHEMICAL USE AND COST OF TREATMENT WHEN OZONE IS USED ON WATER OBTAINED FROM LAKE SULPHUR SPRINGS

	Dosage mg/l	Quantity lb/mil gal	Chemical Price $/lb (1)	Cost $/mil gal
Ozone	6.5	54.2	0.96(2)	52.03
Alum	8.0	667.2	0.0457	30.49
Lime	25	208.5	0.0326	6.79
Hydrofluosilicic Acid	4.85	40.4	0.1755	7.09
Chlorine	1.0	8.34	0.2145	1.79
Ammonia	0.25	2.09	0.2400	0.50
			Total	$98.69

(1) Based on 1988 Chemical Prices for Sulphur Springs.
(2) Based on the use of 12 kWh to generate one pound of ozone and a power cost of $0.08/kWh.

Estimating future treatment costs for water obtained from Cooper Lake is more difficult because the there is no experience with this water. However, it will be similar to water obtained from Wright Patman Lake by the City of Texarkana. Average chemical requirements from April 1986 through March 1988 for this water were 57 mg/l for

8.3 TREATMENT FACILITIES

8.3.1 Introduction

To meet future water demand as projected in Chapter 6, additional water treatment plant capacity will be needed by early 1990. This can be accomplished with a combination of additional treatment units for plant expansion, and improvements to the existing facilities to increase their capacity. The expansion will consist of flow splitting improvements, additional rapid mix, flocculation, sedimentation, filtration facilities, washwater recovery, and high service pumping. New ozone facilities for disinfection and process enhancement will also be added. Existing facilities will be upgraded to increase their capacity and improve their performance. Space will be reserved on site for future treatment facilities that may be necessary to comply with future regulations.

8.3.2 New Treatment Facilities

To meet future water demand as projected in Chapter 6, the water treatment plant will ultimately be expanded to 26 mgd. The recommended facilities to increase treatment capacity are discussed below. The layout for the treatment plant expansion for Alternative A is shown on Figure 8-1.

8.3.2.1 Raw Water Metering and Flow Splitting. It is important to accurately meter the flow to each rapid mix basin, in order to optimize chemical usage in the process. Metering will be provided at the raw water junction box ahead of the pre-ozone contact basin. A meter will also be installed after the flow distribution structure, for purposes of setting chemical dosages to each rapid mix structure. Venturi insert tubes will be used.

To provide an even distribution of flow to each rapid mix basin, for optimum chemical usage, raw water will be hydraulically split by overflow weirs in a distribution structure. These weirs will be adjustable and capable of metering flow to each rapid mix basin.

8.3.2.2 Raw Water Pump Station. A new raw water intake/pump station will be constructed at Cooper Reservoir as described in Chapters 7 and 9.

8.3.2.3 Rapid Mixing. Series type rapid mix basins with mechanical mixers will be provided to combine chemicals with the incoming raw water. One rapid mix basin will serve each flocculation basin.

The physical characteristics of the rapid mix basins are listed in Table 8-3.

TABLE 8-3. PHYSICAL CHARACTERISTICS - RAPID MIX BASINS

Number of mixing compartments	2
Size of each mixing compartment, ft	4.66 x 4.66 x 7.83
Retention time, sec	10 @ 16 mgd
Mixer power, each, hp	8
Mixer G factor, sec^{-1}	800 @ 35F

8.3.2.4 Flocculation. Flocculation basins will be cross flow type, consisting of three separate mixing zones for providing tapered mixing intensities. Variable speed paddle wheel flocculators will be installed in each zone.

The physical characteristics of the flocculation basins are listed in Table 8-4.

TABLE 8-4. PHYSICAL CHARACTERISTICS - FLOCCULATION BASINS

Number of mixing zones	3
Flocculator type	Horizontal, paddle wheel
Dimensions, ft	
Overall size	59.0 x 33.33
Average sidewater depth	11.33
Paddle diameter	10.0
Design G factor, sec^{-1}	50-30-10
Retention time, min	60

8.3.2.5 Sedimentation. Sedimentation basins will contain two separate settling zones. Circular sludge collecting equipment will be installed in the first zone. The second zone will contain the collection weirs and troughs.

The physical characteristics of the sedimentation basins are listed in Table 8-5.

TABLE 8-5. PHYSICAL CHARACTERISTICS - SEDIMENTATION BASINS

Number of zones	2
Dimensions, ft	
Overall size	66 x 132
Sidewater depth	12.83
Retention time, hr	5
Weir loading rate, gpm/ft	14.5
Surface loading rate, gpm/sq ft	0.32

8.3.2.6. Ozone Facilities. A pre-ozone contact basin, ozone generator building, and a post-ozone contact basin will be installed adjacent to the new sedimentation basins. There will ultimately be a total of four pre-ozone and four post-ozone contact basins with common wall construction.

After metering, the incoming raw water will flow through the pre-ozone contact basin for process enhancement, prior to entering the rapid mix basins. After sedimentation, the settled water will collect in a settled water flume and be pumped to the post-ozone contact basin for disinfection. The water from the post-ozone contact basin will flow to the filters.

The Ozone Building will contain controls, ozone generators, air compressors, and air dryers. The physical characteristics of the ozone contact basins are listed in Table 8-6. A detailed opinion of probable cost for the ozone facilities is shown in Table B-23 in Appendix B.

TABLE 8-6. PHYSICAL CHARACTERISTICS - PRE-OZONE AND POST-OZONE CONTACT CHAMBERS

Dimensions, ft	
Pre-ozone chamber	66 x 44 x 20 deep
Post-ozone chamber	66 x 44 x 20 deep
Retention time, min	
Pre-ozone chamber	12 @ 26 mgd
Post-ozone chamber	12 @ 26 mgd

8.3.2.7 Low Lift Pumping Station. A low lift pumping station will be constructed at the end of the existing sedimentation basins. These pumps will take suction from a wetwell and lift the water to the post-ozone contact basin. The station will have space for four vertical propeller pumps at ultimate development. Initially, firm pumping capacity will match the treatment plant capacity.

8.3.2.8 Filtration. Filters will be constant rate, dual media type, operating at a rate of 5 gpm/sq ft. Each filter will be provided with tile underdrains, surface wash equipment, and washwater troughs.

The physical characteristics of the filters are listed in Table 8-7.

TABLE 8-7. PHYSICAL CHARACTERISTICS - FILTERS

Size, ft	23.25 x 15.5
Overall depth, ft	11.0
Surface loading, gpm/sq ft	5.0
Filtration capacity, mgd/filter	2.6
Bed depth, in	
Gravel	12.0
Sand	11.0
Anthracite	13.0

8.3.2.9 Finished Water Storage. Existing ground storage is adequate for the initial expansion. Additional storage will be required in the future, as described in Chapter 10.

8.3.2.10 High Service Pumping. Additional pumps in a new pump building will ultimately be added, to provide a firm capacity that matches plant capacity.

8.3.2.11 Chemical Storage and Feeding. Chlorine and ammonia will be added to produce a chloramine residual after filtration. Chlorine and ammonia feed points will also be provided at each of the rapid mix basins, so intermittant use of choramines can be used to reduce algae growth in the flocculation and sedimentation basins. The existing ammonia and chlorine feeders are adequate for the feed requirements of a 26 mgd (average day demand) plant.

Filter polymer will be fed from 55 gal drums to the lime addition basin ahead of the filters. Two metering pumps will be provided; one will serve as standby.

Polymer for coagulation will be fed to each rapid mix basin. One metering pump will be provided for each rapid mix basin. A standby pump will also be provided.

Continue to use existing lime feeding facilities which have adequate storage.

Continue to feed alum into the rapid mix basins using the exisitng alum feed facilities which have adequate storage.

The flouride storage and feeding facilities are of adequate capacity, and the hydrofluosilicic acid will be fed ahead of the clearwells.

8.3.2.12 Washwater Reuse and Sludge Disposal. A washwater recovery basin will be constructed within the existing south sludge lagoon. An earth berm will be added to separate washwater recovery from the sludge lagoon. The existing lagoon decant pumps will continue to be used to pump washwater to the head of the plant.

A new sludge lagoon will be constructed south and adjacent to the existing sludge lagoon. The new lagoon will be approximately 93,250 sq ft and 8 ft deep.

Consideration should be given to purchasing lands adjacent to the water treatment plant for future sludge lagoon requirements.

8.3.2.13 Electrical System. Additional electrical equipment will be added as required. Further evaluation of the existing power supply will be necessary during design of the expansion facilities.

8.3.2.14 Instrumentation and Controls. Individual filter controls will be located in the Filter Building at the new filters, to allow visual observation of filter backwashing. Raw water and filtered water flow indication and controls for raw water valves and pumps will be located at the main control panel. Process equipment will be locally controlled. The City may wish to consider a computer for data acquisition and logging. The probable cost of such a system would be approximately $170,000 and a small study is normally required.

8.3.2.15 Laboratory, Office, and Maintenance Facilities. The existing laboratory facilities are adequate for the ultimate needs of the treatment plant. However, individual office areas and storage requirements will be evaluated during the design phase for space utilization to meet present and future needs. Interior renovations will be made to meet these needs. Additional laboratory instrumentation will be needed to monitor total organic carbon.

Additional analytical capability may also be required to meet future regulations.

8.3.3 Upgrading Existing Treatment Facilities.

The existing water plant facilities will be upgraded to increase the treatment capacity from 7 mgd to 10 mgd. Improvements to the existing facilities are discussed below.

8.3.3.1. High Service Pumping. A looped discharge header, including the necessary piping and valves, will be added to the existing high service pump station to increase reliability and performance. A 36 inch discharge pipe will be added parallel to the existing 27 inch pipe, from the high service pump station to the distribution lines at Highway 19.

8.3.3.2. Raw Water Pump Station. The firm capacity of the existing raw water pump station will be increased to match the treatment plant capacity. One pump will be replaced or a new pump will be added. Associated piping and valves will also be added.

8.3.3.3. Sludge Disposal. The existing south sludge lagoon will be dredged to a capacity of 8 feet, in order to increase capacity of sludge handling facilities.

8.4. STAGING ALTERNATIVES

Two alternatives have been identified for the expansion of the water treatment plant to 26 mgd. The existing plant will be upgraded to 10 mgd as a part of both alternatives. Alternative A consists of four 4.0 mgd expansions to be phased in over the next 50 years. It is recommended that the 4.0 mgd expansions be completed in 1990, 2000, 2020, and 2030 in order to maintain adequate plant capacity. Figure 8-3 shows the recommended staging of water treatment plant construction for Alternative A.

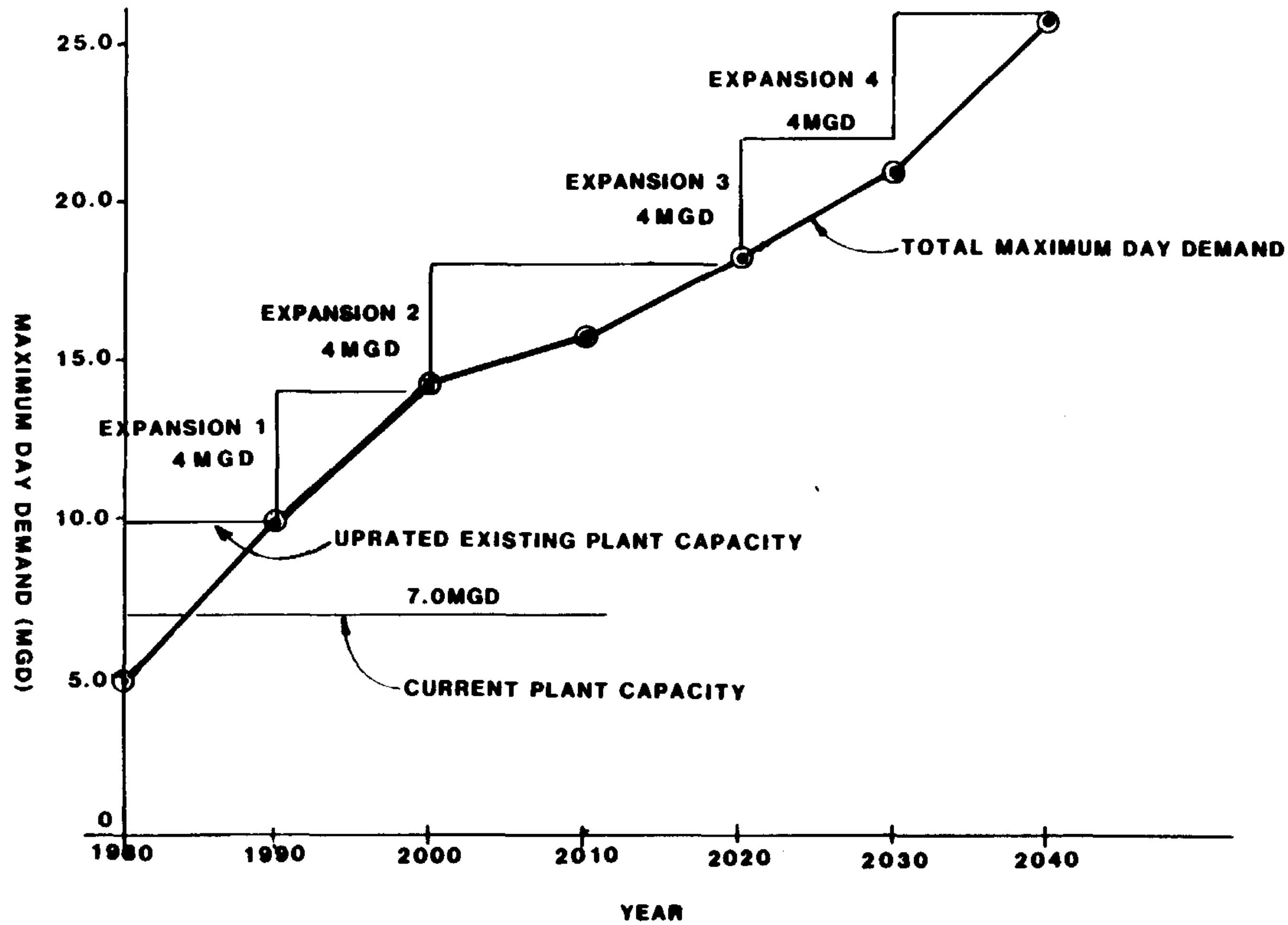


FIGURE 8 - 3

COOPER RESERVOIR
WATER STUDY
SULPHUR SPRINGS, TEXAS

STAGING OF WTP EXPANSION CONSTRUCTION FOR ALTERNATIVE A

BLACK & VEATCH DALLAS, TEXAS

PROJECT NO. 14719.100

Alternative B involves two 8.0 mgd expansions over the next 50 years. The expansions should be made in the years 1990 and 2020 in order to maintain adequate plant capacity. Figure 8-4 shows the recommended staging of water treatment plant construction for Alternative B.

8.4.1. Construction Schedules.

Projected time schedules for implementing construction of surface water treatment facilities for Alternatives A and B through the year 2040 are shown in Table 8-8 and 8-9, respectively.

TABLE 8-8. CONSTRUCTION SCHEDULE FOR ALTERNATIVE A

Construction Completion Date	Item Description
1990	Upgrade the existing treatment plant capacity from 7 mgd to 10 mgd by the following: Upgrade the controls for the pumps at the existing high service pump station to increase reliability and performance. A 36 inch pipe will be added from the high service pump station to the distribution line at Highway 19. Add a looped discharge header for increased reliability. Replace one raw water pump and improve discharge piping at the raw water pump station. Dredge the south sludge lagoon to a depth of 8 feet to increase capacity. Add three full body venturi tubes for flowmetering to existing rapid mixers.
	Construct a 4 mgd treatment module, including a junction structure, raw water metering, flow splitting, rapid mixing, flocculation and sedimentation. Construct ozone generator building, two pre- and post-ozone contact basins, washwater recovery facilities, associated site work, electrical, and chemical feed. Add one filter at the filter building.
2000	Replace one high service pump to increase firm capacity to meet treatment plant capacity.

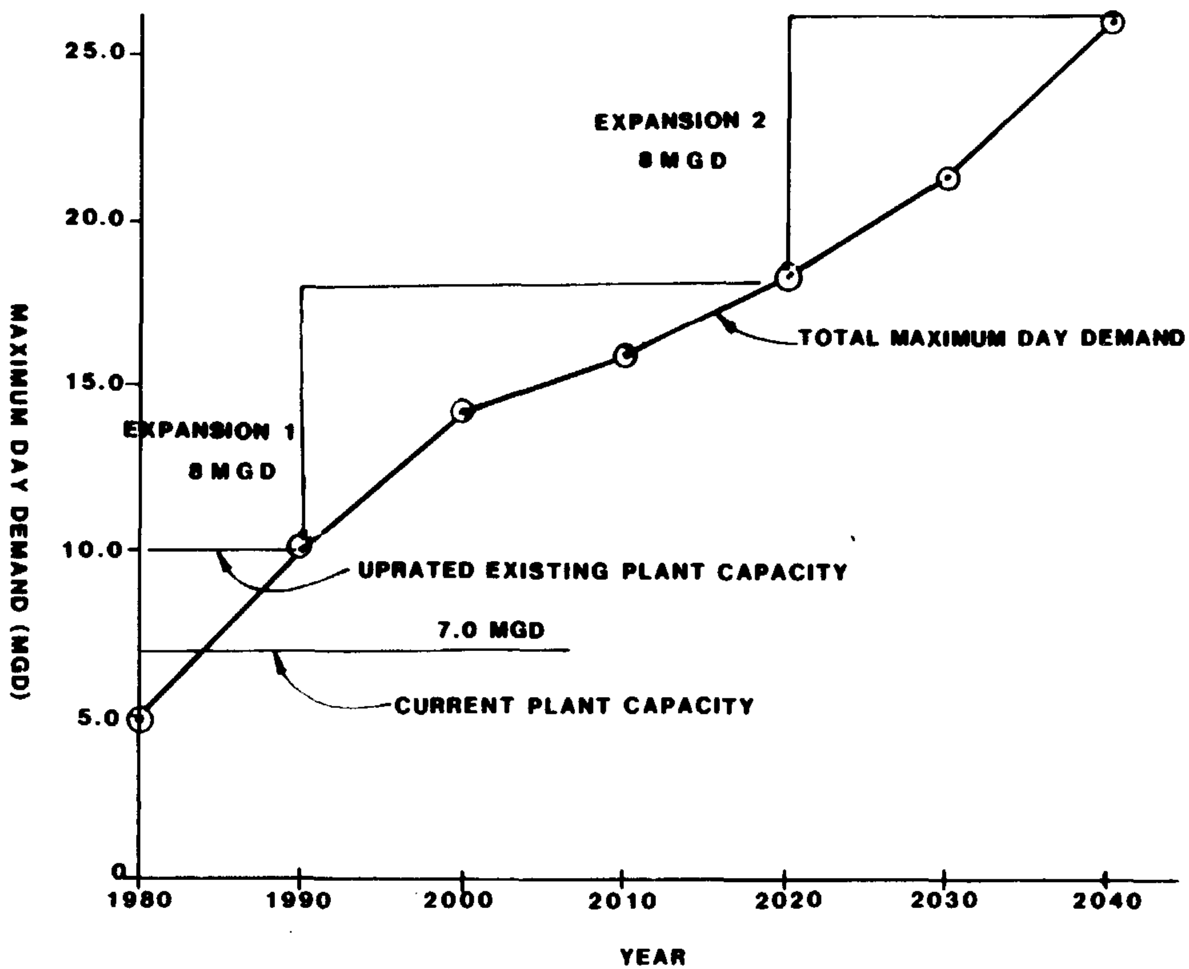


FIGURE 8 - 4

COOPER RESERVOIR
WATER STUDY
SULPHUR SPRINGS, TEXAS

STAGING OF WTP EXPANSION CONSTRUCTION FOR ALTERNATIVE B

BLACK & VEATCH DALLAS, TEXAS

PROJECT NO. 14719.100

TABLE 8-8 (continued). CONSTRUCTION SCHEDULE FOR ALTERNATIVE A

Construction Completion Date	Item Description
2000	Construct a 4 mgd treatment module including flow splitting, flocculation, sedimentation, associated site work, electrical and chemical feed. Add two filters at the existing filter building. Add a new sludge lagoon south and adjacent to the existing lagoon.
2020	Construct a 4 mgd treatment module, including flow splitting, flocculation, sedimentation, ozone facilities, new high service pump building and pumps, associated site work, electrical, and chemical feed. Add two filters at the existing filter building.
2030	Construct a 4 mgd treatment module, including flow splitting, flocculation, sedimentation, new high service pump, associated site work, electrical, and chemical feed. Add one filter at the filter building.

TABLE 8-9. CONSTRUCTION SCHEDULE FOR ALTERNATIVE B

Construction Completion Date	Item Description
1990	Upgrade the existing treatment plant capacity from 7 mgd to 10 mgd. Upgrade the controls of the pumps at the existing high service pump station to increase reliability and performance. A 36 inch pipe will be added from the high service pump station to the distribution line at Highway 19. Replace one raw water pump and improve discharge piping at the existing raw water pump station. Dredge the existing south sludge lagoon to a depth of 8 feet to increase capacity. Add three venturi insert flow tubes to meter flow to existing rapid mixers.

TABLE 8-9 (continued). CONSTRUCTION SCHEDULE FOR ALTERNATIVE B

Construction Completion Date	Item Description
	Construct an 8 mgd treatment module, including a junction structure, raw water metering, flow splitting, rapid mixing, flocculation, sedimentation, ozone generator building and contact basins, washwater recovery facilities, associated site work, electrical, and chemical feed. Add three filters at the filter building. Construct a new sludge lagoon south of and adjacent to the existing lagoon.
2020	Construct an 8 mgd treatment module, including flow splitting, flocculation, sedimentation, ozone contact basins, new high service pump building and pumps, associated site work, electrical, and chemical feed. Add three filters at the filter building.

8.5 CAPITAL COSTS

Preliminary opinions of probable construction costs for water treatment plant improvements/additions through 2040 for Alternatives A and B are indicated in Tables 8-10 and 8-11, respectively. All costs are based on June 1988 prices.

TABLE 8-10. OPINIONS OF PROBABLE CONSTRUCTION COSTS - ALTERNATIVE A

Construction Completion Date	Item Description	Probable Construction Cost ($)
1990	Upgrade existing treatment plant capacity from 7 mgd to 10 mgd	777,000
	4 mgd treatment module and associated plant work	4,823,000
	Subtotal - 1990	$5,600,000

TABLE 8-10 (continued). OPINIONS OF PROBABLE CONSTRUCTION COSTS - ALTERNATIVE A

Construction Completion Date	Item Description	Probable Construction Cost ($)
2000	Replace one high service pump in existing pump station	21,000
	4 mgd treatment module and associated plant work	1,897,000
	Subtotal - 2000	$1,918,000
2020	4 mgd treatment module and associated plant work	2,160,000
2030	4 mgd treatment module and associated plant work	1,630,000
TOTAL CONSTRUCTION COST - ALTERNATIVE A		$11,308,000

TABLE 8-11. OPINIONS OF PROBABLE COST - ALTERNATIVE B

Construction Completion Date	Item Description	Probable Construction Cost ($)
1990	Upgrade existing treatment plant capacity from 7 mgd to 10 mgd	777,000
	8 mgd treatment module and associated plant work	6,621,000
	Subtotal - 1990	$7,398,000
2020	8 mgd treatment module and associated plant work	3,557,000
TOTAL CONSTRUCTION COST - ALTERNATIVE B		$10,955,000

8.6. RECOMMENDED IMPROVEMENTS AND IMPLEMENTATION SCHEDULE

It is recommended that, by 1990, the existing water treatment plant be expanded to 14 mgd, as described by Alternative A in Section 8.4. The recommended improvements are shown on Figure 8-1.

Alternative A was chosen because its initial capital expenditures are approximately $1.5 million less than that of Alternative B. The probable costs for Alternative A are given in Table 8-12 below.

TABLE 8-12. RECOMMENDED IMPROVEMENTS AND OPINIONS OF PROBABLE COST - ALTERNATIVE A

Construction Completion Date	Item Description	Probable Construction Cost ($)
1990	Upgrade existing treatment plant to 10 mgd	777,000
	4 mgd treatment module	4,823,000
	Construction Cost - 1990	$5,600,000
	Engineering, Legal & Administrative @ 15%	840,000
	Contingencies @ 10%	560,000
Subtotal - 1990		$7,000,000
2000	Replace high service pump	21,000
	4 mgd treatment module	1,897,000
	Construction Cost - 2000	$1,918,000
	Engineering, Legal & Administrative @ 15%	288,000
	Contingencies @ 10%	192,000
Subtotal - 2000		$2,160,000
2020	4 mgd treatment plant expansion	2,160,000
	Construction Cost - 2020	$2,384,600
	Engineering, Legal & Administrative @ 15%	324,000
	Contingencies @ 10%	216,000
Subtotal - 2020		$2,700,000

TABLE 8-12 (continued). RECOMMENDED IMPROVEMENTS AND OPINIONS OF PROBABLE COST - ALTERNATIVE A

2030	4 mgd treatment plant expansion	1,630,000
	Construction Cost - 2030	$1,630,000
	Engineering, Legal & Administrative @ 15%	245,000
	Contingencies @ 10%	163,000
Subtotal - 2030		$2,038,000
TOTAL PROJECT COST - ALTERNATIVE A		$13,898,000

- Alternative 1A. This system alternative is the same as Alternative 1 except pipeline route 1A is used instead of pipeline route 1. Pipeline route 1A crosses the Century Lake Dam, as shown on Figure 4, then goes to the water treatment plant.
- Alternative 2. Pipeline route 2 and the Harpers Hill intake/pump station, as shown on Figure 4, convey 14 mgd to the water treatment plant. The existing raw water facilities at Lake Sulphur Springs are used to meet peak demands.
- Alternative 2A. This system alternative is the same as Alternative 2 except pipeline route 2A is used instead of pipeline route 2. Pipeline route 2A crosses Lake Sulphur Springs, as shown on Figure 4, then goes to the water treatment plant.
- Alternative 3. The Eastbank intake/pump station and pipeline route 3, as shown on Figure 4, convey 14 mgd to the water treatment plant. The existing raw water facilities on Lake Sulphur Springs are used to meet peak demand.

9.2 LAKE SULPHUR SPRINGS FOR TERMINAL STORAGE

These conveyance system operating alternatives are shown schematically on Figure 9-1. The raw water intake/pump station and raw water pipeline from Cooper Reservoir are sized for the average annual day flow of 14 mgd. The pipeline will convey the raw water to Lake Sulphur Springs where it will be discharged into the lake. Lake Sulphur Springs will act as the terminal reservoir for the raw water system. The existing raw water intake/pump station and raw water pipelines from Lake Sulphur Springs will continue to convey all raw water to the water treatment plant. The water impounded in Lake Sulphur Springs will provide raw water for peak day demands. The existing raw water facilities will be expanded or replaced at some point in the future, as discussed in Section 9.4, Economic Evaluation. The alternatives under this system are explained in detail below.

- Alternative 4. This system alternative includes the Finley Branch intake/pump station and pipeline route 1B, both sized to convey 14 mgd. The intake/pump station and pipeline route are shown on Figure 4. Pipeline route 1B conveys the raw water to Lake Sulphur Springs where it is discharged into the north side of the lake. The existing raw water conveyance facilities on Lake Sulphur Springs are used to convey all water to the water treatment plant. The water impounded in Lake Sulphur Springs is used to meet peak demands.
- Alternative 5. The Harpers Hill intake/pump station and pipeline route 2B, as shown on Figure 4, convey 14 mgd of raw water to the point of discharge on the north side of Lake Sulphur Springs. The existing raw water facilities on Lake Sulphur Springs are used to convey all water to the water treatment plant. The water impounded in Lake Sulphur Springs is used to meet peak demands.

9.3 COOPER RESERVOIR FOR TOTAL WATER NEEDS

These alternatives involve using the Cooper Reservoir water for both the average day demand and to meet peak day demands. Lake Sulphur Springs, and the existing raw water conveyance facilities on this lake, will be used only as a standby raw water source. The Cooper Reservoir intake/pump station and pipeline facilities are sized to convey the peak demand of 26 mgd to the water treatment plant. A schematic of this conveyance system alternative is shown on Figure 9-1. The alternatives for this conveyance system are described below.

- Alternative 6. The intake/pump station is located at Finley Branch and is sized to convey 26 mgd. Pipeline route 1, as shown on Figure 4, conveys the raw water to the water treatment plant. Lake Sulphur Springs and the facilities thereon will be used as a standby system.

- Alternative 7. Pipeline route 2 and the Harpers Hill intake/pump station, as shown on Figure 4, convey 26 mgd to the water treatment plant. The existing raw water facilities on Lake Sulphur Springs will be used as a standby system.
- Alternative 8. The Eastbank intake/pump station and pipeline route 3, as shown on Figure 4, convey 26 mgd to the water treatment plant. The existing raw water facilities on Lake Sulphur Springs will be used as a standby system.

9.4 ECONOMIC EVALUATIONS

The construction costs for each of the components of the conveyance systems are summarized in Chapter 7.0. In this section, the construction costs of all components in a conveyance system operating alternative are compared, in order to select the most cost-effective system. There is no cost comparison for the conveyance systems with a submerged intake structure, since the channel intake structure is the least expensive option at all locations on the reservoir.

The costs summarized in Chapter 7.0 are construction costs only (including 10 percent for contingencies). In this section, 15 percent is added to the construction costs to cover engineering, legal, and administrative fees, therefore giving the total project cost for each alternative. These costs are an important consideration in assessing the financial capabilities of the City to implement the project. Alternatives 1, 1A, 2, 2A, 3, 4, and 5 involve the use of the existing raw water conveyance facilities on Lake Sulphur Springs. Therefore, the cost of improving the existing raw water facilities and the estimated operation costs have been included in the appropriate alternatives. The costs for the existing raw water facilities are explained in detail in Section 9.4.4.

9.4.1 Total Project Cost. The total project cost for each conveyance system operating alternative is given in Table 9-1 below.

TABLE 9-1. RAW WATER CONVEYANCE SYSTEM - PROJECT COST SUMMARY

	Operating Alternative	Project Cost $
1.	Finley Branch to WTP, Lake SS for Peaking	10,269,000*
1A.	Finley Branch to WTP Across Century Dam, Lake SS for Peaking	11,279,000
2.	Harpers Hill to WTP, Lake SS for Peaking	10,126,000
2A.	Harpers Hill to WTP Across Lake SS, Lake SS for Peaking	14,574,000
3.	East Bank to WTP, Lake SS for Peaking	10,289,000
4.	Finley Branch to Lake SS, Lake SS for Terminal Storage	11,185,000
5.	Harpers Hill to Lake SS, Lake SS for Terminal Storage	11,112,000
6.	Finley Branch to WTP, Lake SS as Standby	13,249,000
7.	Harpers Hill to WTP, Lake SS as Standby	13,851,000
8.	Eastbank to WTP, Lake SS as Standby	13,578,000

* The probable project cost to construct Alternative 1 from Finley Branch to Highway 71 is $4,863,000. The probable project cost for Alternative 6 from Finley Branch is $6,287,000. These costs are given as a consideration in the event of a combined intake with North Texas Municipal Water District and the City of Irving.

9.4.2 Operation Cost. Annual costs for operation consist mainly of power costs for pumping. These costs must be met entirely from local revenue and, thus, will have a significant effect on user rates.

Operation costs were developed using a unit cost of \$0.08/kwh. The headloss across the pipeline in each alternative was found using the Hazen-Williams pipe flow formula. Once the headloss was determined, power requirements for pumping at the headloss were multiplied by the unit cost for power.

The estimated operation costs for the years 1990 and 2040 are shown in Table 9-2. The present worth of the operation costs are also given in the table.

The operation costs, shown in Table 9-2 include estimated costs to operate the new intake/pump station, the estimated power costs for the existing raw water pump station on Lake Sulphur Springs (where applicable), and the estimated chemical use costs. The operation cost calculations and data are shown in more detail in Appendix C.

TABLE 9-2. RAW WATER CONVEYANCE SYSTEM - OPERATION COST SUMMARY

	Operating Alternative	Operation and Chemical Cost 1990 \$	Operation and Chemical Cost 2040 \$	Present Worth Operation Costs \$
1.	Finley Branch to WTP, Lake SS for Peaking	309,000	1,055,000	5,030,000
1A.	Finley Branch to WTP Across Century Dam, Lake SS for Peaking	339,000	1,090,000	5,367,000
2.	Harpers Hill to WTP, Lake SS for Peaking	309,000	1,049,000	5,010,000
2A.	Harpers Hill to WTP Across Lake SS, Lake SS for Peaking	340,000	1,092,000	5,377,000
3.	East Bank to WTP, Lake SS for Peaking	311,000	1,084,000	5,107,000

TABLE 9-2 (continued). RAW WATER CONVEYANCE SYSTEM - OPERATION COST SUMMARY

Operating Alternative	Operation and Chemical Cost 1990 $	2040 $	Present Worth Operation Costs $
4. Finley Branch to Lake SS, Lake SS for Terminal Storage	305,000	1,156,000	4,847,000
5. Harpers Hill to Lake SS, Lake SS for Terminal Storage	305,000	1,156,000	4,836,000
6. Finley Branch to WTP, Lake SS as Standby	309,000	1,409,000	5,291,000
7. Harpers Hill to WTP, Lake SS as Standby	308,000	1,407,000	5,248,000
8. Eastbank to WTP, Lake SS as Standby	314,000	1,487,000	5,486,000

9.4.3 Total System Present Worth. The most cost-effective plan is defined as the operating alternative with the lowest present worth, that is capable of meeting the raw water conveyance demands. The present worth for each alternative is found by adding the project cost to the present worth of the operating costs (see sections 9.4.1 and 9.4.2) and the estimated chemical costs. The results of these analyses are summarized in Table 9-3.

TABLE 9-3. TOTAL SYSTEM PRESENT WORTH

Operating Alternative	Present Worth $
1. Finley Branch to WTP, Lake SS for Peaking	15,299,000
1A. Finley Branch to WTP Across Century Dam, Lake SS for Peaking	16,646,000
2. Harpers Hill to WTP, Lake SS for Peaking	15,136,000

10.0 FUTURE TREATED WATER STORAGE AND DISTRIBUTION FACILITIES

10.1 CITY OF SULPHUR SPRINGS

10.1.1 Water Storage Facilities. The primary purpose of water storage facilities is to supply water during the peak use period of the day, with subsequent refilling during the hours of limited demand at night. Other purposes include emergency supply, fire fighting, power failure, pressure equalization, and operational flexibility. Ground storage is used at the treatment plant to meet fluctuating water demands, while maintaining a relatively constant treatment rate. Elevated storage which "floats" on the the system is normally designed to meet a portion of the maximum-hour demands caused by lawn watering and to provide stored water for fire protection and other emergencies.

Several design criteria need to be considered to establish the proper amount of ground and elevated storage. The Key Rate Schedule of the State Board of Insurance provides one design consideration. The Schedule recommends that the total storage (ground and elevated) be adequate to supply each person in the city with water at a rate of 130 gallons per day (gpd) for 24 hours, and that elevated storage be able to supply water at this rate for 10 hours.

For an "Approved" public water supply, the Texas Department of Health requires that the total storage (ground and elevated) be equal to the average daily consumption, or 185 gallons per capita, whichever is less. The elevated storage shall be equivalent to 50 percent of the average daily consumption, or 55 gallons per capita, whichever is less, with a maximum of 5.0 mil gal required for each service level.

Present usable total storage is 4,500,000 gallons. The existing storage capacities indicated in Table 10-1 below can be compared with the computed storage requirements based on the design criteria in Table 10-2. By 2000, all of the criteria will exceed the existing storage capacity, assuming population growth occurs as projected.

TABLE 9-3 (continued). TOTAL SYSTEM PRESENT WORTH

	Operating Alternative	Present Worth $
2A.	Harpers Hill to WTP Across Lake SS, Lake SS for Peaking	20,131,000
3.	East Bank to WTP, Lake SS for Peaking	15,396,000
4.	Finley Branch to Lake SS, Lake SS for Terminal Storage	16,032,000
5.	Harpers Hill to Lake SS, Lake SS for Terminal Storage	15,948,000
6.	Finley Branch to WTP, Lake SS as Standby	18,540,000
7.	Harpers Hill to WTP, Lake SS as Standby	19,099,000
8.	Eastbank to WTP, Lake SS as Standby	19,064,000

Note: Costs do not include cost for operating or expanding existing raw water facilities on Lake Sulphur Springs.

9.4.4 Existing Raw Water Facilities. Alternatives 1 through 5 are conditional to the possibility of using the existing raw water intake/pump station and pipeline at Lake Sulphur Springs. The existing facilities would be used for peak flow demand with Alternatives 1, 1A, 2, 2A, and 3. In the case of Alternatives 4 and 5, where Lake Sulphur Springs is used for terminal storage, the existing raw water conveyance facilities would have to convey all raw water to the water treatment plant. Alternatives 6, 7, and 8 use the existing facilities as a standby only and, therefore, no expansion would be required.

The existing raw water conveyance facilities have a current firm capacity of approximately 6 mgd. Therefore, for Alternatives 1, 1A, 2, 2A, and 3, an expansion of approximately 8 mgd would be required before the year 2040 in order to meet peak demands. The capital cost of such an expansion would be approximately $110,000.

If Alternative 4 or 5 were chosen, the existing pump station would have to pump 26 mgd by the year 2040. This would require a 15 mgd expansion at the existing intake/pump station. The 15 mgd expansion would cost approximately $2,752,000.

The estimated power costs for pumping at the existing raw water intake/pump station are shown in Table 9-4.

TABLE 9-4. ESTIMATE OF POWER COSTS OF EXISTING PUMP STATION

Operation Cost 1990 $	Operation Cost 2040 $	Operation Cost Present Worth $
25,000	255,000	782,000

Data and calculations of operating costs are shown in detail in Appendix C.

9.4.5 Recommendations. The site location of the intake/pump station is dependent on discussions with the U.S. Army Corps of Engineers. The site location is also dependent on sharing facilities with North Texas Municipal Water District. The pipeline route is dependent on the intake/pump station location. The information contained in this chapter will aid in discussions and negotiations relating to intake/pump station location and size.

Because of the water quality differences between Lake Sulphur Springs and Cooper Reservoir, the preferred alternative is to construct a 26 mgd intake/pump station to use water only from Cooper Reservoir. However, that alternative would cost substantially more than the alternatives that utilize Lake Sulphur Springs as a secondary source of raw water. The best situation, then, is to build a 14 mgd intake/pump station (initially) at Cooper Reservoir and gain experience treating Cooper Reservoir Water. Later, a decision to expand the intake/pump station to 26 mgd can be made based on that experience.

10.0 FUTURE TREATED WATER STORAGE AND DISTRIBUTION FACILITIES

10.1 CITY OF SULPHUR SPRINGS

10.1.1 Water Storage Facilities. The primary purpose of water storage facilities is to supply water during the peak use period of the day, with subsequent refilling during the hours of limited demand at night. Other purposes include emergency supply, fire fighting, power failure, pressure equalization, and operational flexibility. Ground storage is used at the treatment plant to meet fluctuating water demands, while maintaining a relatively constant treatment rate. Elevated storage which "floats" on the the system is normally designed to meet a portion of the maximum-hour demands caused by lawn watering and to provide stored water for fire protection and other emergencies.

Several design criteria need to be considered to establish the proper amount of ground and elevated storage. The Key Rate Schedule of the State Board of Insurance provides one design consideration. The Schedule recommends that the total storage (ground and elevated) be adequate to supply each person in the city with water at a rate of 130 gallons per day (gpd) for 24 hours, and that elevated storage be able to supply water at this rate for 10 hours.

For an "Approved" public water supply, the Texas Department of Health requires that the total storage (ground and elevated) be equal to the average daily consumption, or 185 gallons per capita, whichever is less. The elevated storage shall be equivalent to 50 percent of the average daily consumption, or 55 gallons per capita, whichever is less, with a maximum of 5.0 mil gal required for each service level.

Present usable total storage is 4,500,000 gallons. The existing storage capacities indicated in Table 10-1 below can be compared with the computed storage requirements based on the design criteria in Table 10-2. By 2000, all of the criteria will exceed the existing storage capacity, assuming population growth occurs as projected.

TABLE 10-1. EXISTING CITY WATER STORAGE FACILITIES

Tank Type	Location	Capacity, mil gal
Elevated	Main at Tomlinson	0.25
Elevated	Carter at Whitworth	0.50
Elevated	Morris at College	0.75
Ground	Water Treatment Plant	1.0
Ground	Water Treatment Plant	2.0
Total		4.5

TABLE 10-2. CITY STORAGE REQUIREMENTS BASED ON DESIGN CRITERIA

Year	Type of Tank	Key Rate Schedule(1) mil gal	Texas Department of Health (2) mil gal
1990	Ground	1.5	2.2
	Elevated	1.0	1.0
2000	Ground	2.3	3.2
	Elevated	1.6	1.7
2010	Ground	2.5	3.7
	Elevated	1.8	1.8
2020	Ground	2.9	4.3
	Elevated	2.1	2.1
2030	Ground	3.4	4.9
	Elevated	2.4	2.5
2040	Ground	4.4	6.2
	Elevated	3.1	3.2

(1) From State Board of Insurance
(2) For "Approved" Water System

In order to meet the Texas Department of Health requirements for an approved system, it is recommended that the improvements shown in Table 10-3 be implemented for the water system storage requirements. Figure 10-1 shows the storage requirements and recommended improvements. The opinion of probable construction costs is included in Table 10-3.

10.2 HOPKINS COUNTY

A detailed analysis of the County treated water storage and distribution system is beyond the scope of this report. However, six major transmission lines in the system were briefly analyzed. The six lines convey water to the Brinker, North Hopkins, Brashear, Pleasant Hill, Gafford Chapel, and Martin Springs Water Districts from the City of Sulphur Springs distribution system.

The 1987 average daily flows to each water district were peaked by a factor of 1.85, and maximum day flows were projected through the year 2040. The future flows were projected using historical water demands and the percent of increase in water flow to Hopkins County developed in Chapter 6.0. Table 10-6 lists projected peak water demands and existing pipe capacities for the major water transmission mains. These future flows were compared to an estimated transmission main capacity, based on a nominal head loss of 3 feet per 1000 feet of pipe. Recommended improvements and their associated costs are shown in Table 10-7.

TABLE 10-6. PROJECTED PEAK FLOW RATES AND EXISTING CAPACITIES HOPKINS COUNTY TRANSMISSION MAINS

	MAXIMUM DAY FLOWS, MGD							
	1987*	1990	2000	2010	2020	2030	2040	Existing Capacity
North Hopkins W.D.	0.817	1.27	1.80	2.0	2.30	2.64	3.25	1.64
Brinker W.D.	0.129	0.20	0.28	0.32	0.36	0.42	0.51	0.26
Brashear W.D.	0.187	0.29	0.41	0.46	0.53	0.60	0.74	0.55
Pleasant Hill W.D.	0.041	0.06	0.09	0.10	0.12	0.13	0.16	0.26
Gafford Chapel W.D.	0.004	0.006	0.009	0.010	0.011	0.013	0.016	0.26
Martin Springs W.D.	0.093	0.14	0.20	0.23	0.26	0.30	.037	0.55

* 1987 Average day flow peaked by a factor of 1.85

TABLE 10-1. EXISTING CITY WATER STORAGE FACILITIES

Tank Type	Location	Capacity, mil gal
Elevated	Main at Tomlinson	0.25
Elevated	Carter at Whitworth	0.50
Elevated	Morris at College	0.75
Ground	Water Treatment Plant	1.0
Ground	Water Treatment Plant	2.0
Total		4.5

TABLE 10-2. CITY STORAGE REQUIREMENTS BASED ON DESIGN CRITERIA

Year	Type of Tank	Key Rate Schedule(1) mil gal	Texas Department of Health(2) mil gal
1990	Ground	1.5	2.2
	Elevated	1.0	1.0
2000	Ground	2.3	3.2
	Elevated	1.6	1.7
2010	Ground	2.5	3.7
	Elevated	1.8	1.8
2020	Ground	2.9	4.3
	Elevated	2.1	2.1
2030	Ground	3.4	4.9
	Elevated	2.4	2.5
2040	Ground	4.4	6.2
	Elevated	3.1	3.2

(1) From State Board of Insurance
(2) For "Approved" Water System

In order to meet the Texas Department of Health requirements for an approved system, it is recommended that the improvements shown in Table 10-3 be implemented for the water system storage requirements. Figure 10-1 shows the storage requirements and recommended improvements. The opinion of probable construction costs is included in Table 10-3.

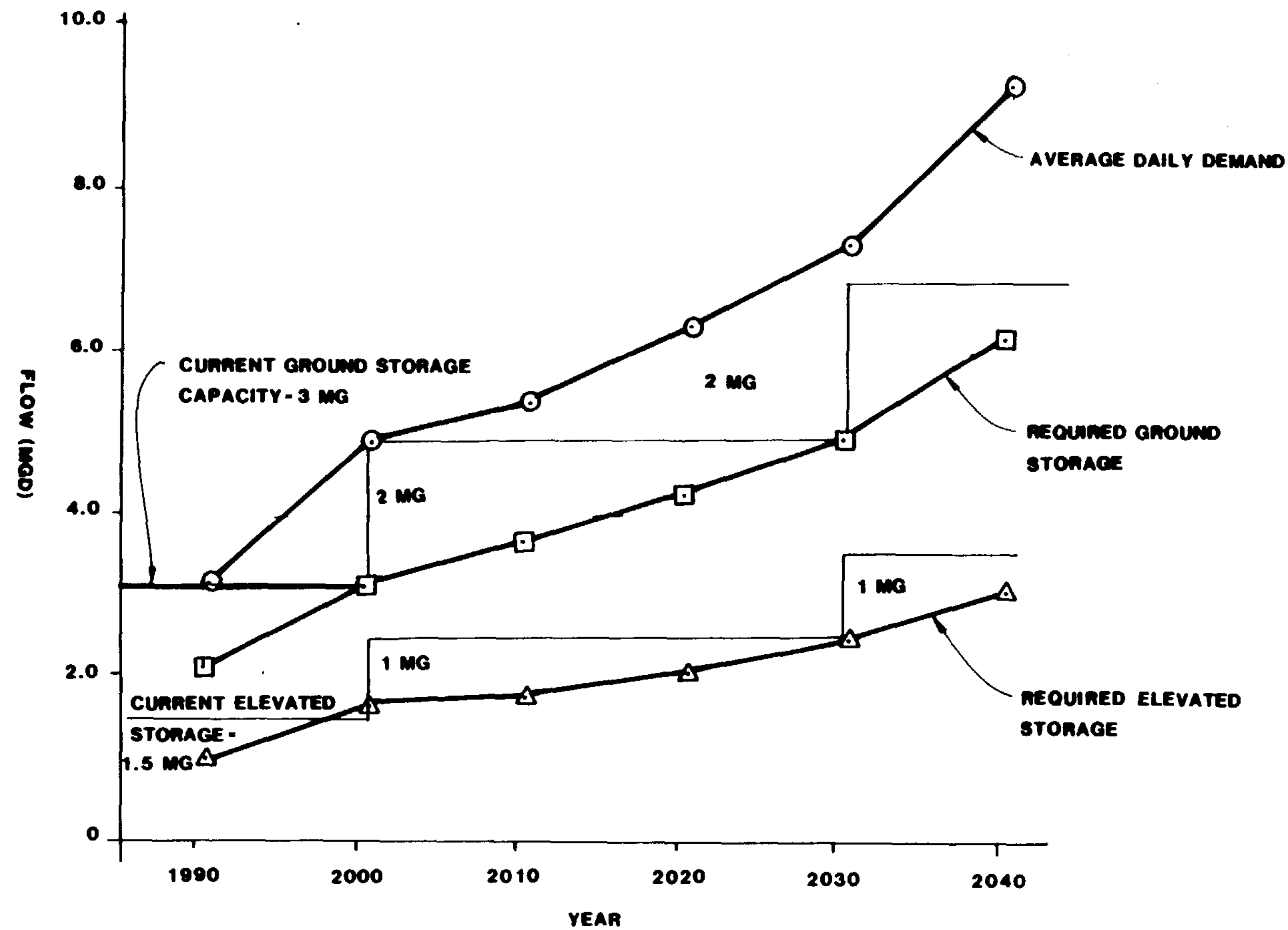


FIGURE 10 - 1

COOPER RESERVOIR
WATER STUDY
SULPHUR SPRINGS, TEXAS

REQUIRED STORAGE &
RECOMMENDED IMPROVEMENTS

BLACK & VEATCH DALLAS, TEXAS

PROJECT NO. 14719.100

TABLE 10-3. RECOMMENDED CITY STORAGE IMPROVEMENTS

Construction Completion Date	Item Description	Probable Construction Cost $
2000	Construct 2 mil gal ground storage tank at water treatment plant	640,000
2000	Construct 1 mil gal elevated tank in distribution system	880,000
2030	Construct 2 mil gal ground storage tank. Additional land adjacent to water treatment plant will have to be acquired.	640,000
2030	Construct 1 mil gal elevated tank in distribution system	880,000

10.1.2 Water Distribution System. A complete analysis of the water distribution system is beyond the scope of this report. However, several recommendations have been made in a report by Bucher, Willis and Ratliff, dated December 1985. Those recommendations are listed below, with their probable construction cost as estimated by Bucher, Willis and Ratliff.

Table 10-4 lists pipeline projects in order of recommended priority, which are necessary to provide fire flows and residual pressures.

TABLE 10-4. CITY DISTRIBUTION SYSTEM IMPROVEMENTS

Location	Size Line in	Probable Construction Cost $
1. Pipeline Road to IH 30 through proposed Industrial Park	12	918,000**
2. Pipeline Road - Davis to East Loop 301	12	227,000
3. Jefferson - Morris to East Loop 301	12	*
- East Loop 301 east to existing 8" line near Rockwell	8	158,000*

* Price for Jefferson improvements includes 8 inch and 12 inch lines.
** City applying to Economic Development Association for funding on construction of this portion of pipeline in Fall of 1988.

Table 10-5 lists projects which will replace existing lines with new or larger lines. The projects are listed in order of recommended priority.

TABLE 10-5. CITY DISTRIBUTION SYSTEM IMPROVEMENTS

Location	Size Line in	Probable Construction Cost $
1. Medical Drive - Airport to Church	8	38,000
2. Middle - Church to Jackson	6	19,000
3. IH 30 Southside - Crush Road to Helm	8	90,000
4. Holiday - Doris to McCann and McCann - Holiday to Broadway	8	329,000

Figure 2 shows the distribution system improvements recommended in Tables 10-4 and 10-5.

10.2 HOPKINS COUNTY

A detailed analysis of the County treated water storage and distribution system is beyond the scope of this report. However, six major transmission lines in the system were briefly analyzed. The six lines convey water to the Brinker, North Hopkins, Brashear, Pleasant Hill, Gafford Chapel, and Martin Springs Water Districts from the City of Sulphur Springs distribution system.

The 1987 average daily flows to each water district were peaked by a factor of 1.85, and maximum day flows were projected through the year 2040. The future flows were projected using historical water demands and the percent of increase in water flow to Hopkins County developed in Chapter 6.0. Table 10-6 lists projected peak water demands and existing pipe capacities for the major water transmission mains. These future flows were compared to an estimated transmission main capacity, based on a nominal head loss of 3 feet per 1000 feet of pipe. Recommended improvements and their associated costs are shown in Table 10-7.

TABLE 10-6. PROJECTED PEAK FLOW RATES AND EXISTING CAPACITIES HOPKINS COUNTY TRANSMISSION MAINS

	MAXIMUM DAY FLOWS, MGD							
	1987*	1990	2000	2010	2020	2030	2040	Existing Capacity
North Hopkins W.D.	0.817	1.27	1.80	2.0	2.30	2.64	3.25	1.64
Brinker W.D.	0.129	0.20	0.28	0.32	0.36	0.42	0.51	0.26
Brashear W.D.	0.187	0.29	0.41	0.46	0.53	0.60	0.74	0.55
Pleasant Hill W.D.	0.041	0.06	0.09	0.10	0.12	0.13	0.16	0.26
Gafford Chapel W.D.	0.004	0.006	0.009	0.010	0.011	0.013	0.016	0.26
Martin Springs W.D.	0.093	0.14	0.20	0.23	0.26	0.30	.037	0.55

* 1987 Average day flow peaked by a factor of 1.85

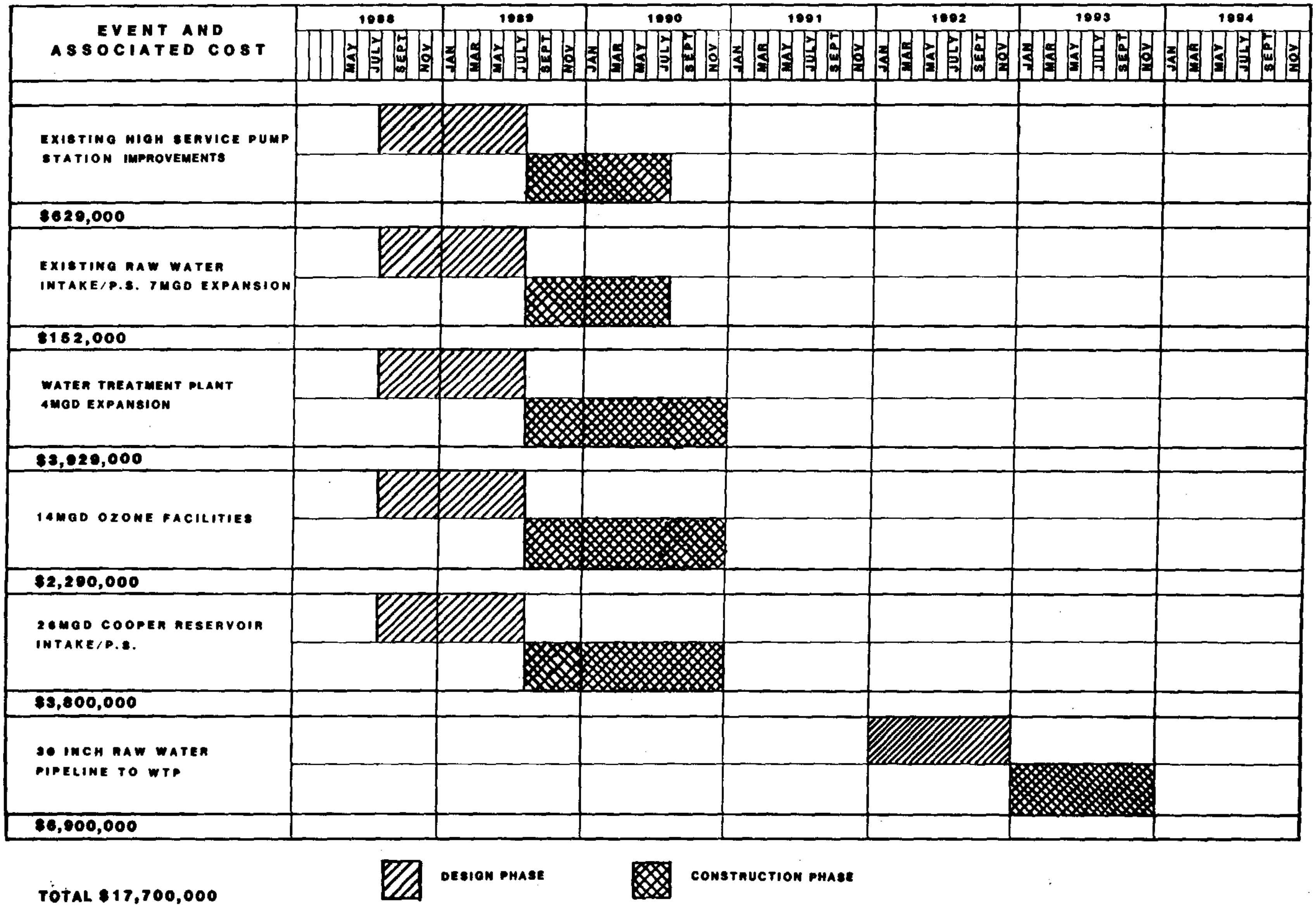
TABLE 10-7. RECOMMENDED COUNTY TRANSMISSION MAIN IMPROVEMENTS AND PROBABLE COSTS

Location	Yeaar Complete	Pipe Size in	Probable Construction Cost $
North Hopkins W.D.	2000	12	732,000
Brinker W.D.	2000	6	228,000
Brashear W.D.	2030	6	260,000

The recommended improvements for Hopkins County are shown on Figure 3.

11.0 RECOMMENDED IMPLEMENTATION SCHEDULE AND COSTS

The time schedule for implementing design and construction of the recommended projects discussed in Chapter 2 and probable project costs are shown on Figure 11-1 for the years 1988 to 1995.



TOTAL $17,700,000

PROJECT SCHEDULE AND ESTIMATED COSTS 1988 to 1995

COOPER RESERVOIR WATER STUDY SULPHUR SPRINGS, TEXAS

FIGURE 11-1

Appendix A

1985, 1986, & 1987
Plant Operating Records

1985 Plant Operating Records

	January Ave	January Range	Feburary Ave	Feburary Range	March Ave	March Range	April Ave	April Range
Raw Water								
Flow, mgd	3.62	3.23-4.15	3.55	2.99-4.59	3.59	2.98-4.60	3.22	2.62-4.18
Turbidity, NTU	25	16-36	31	20-57	29	22-39	34	25-41
Total Alkalinity, mg/l	35	18-38	32	24-44	31	22-40	29	22-40
pH	7.1	6.8-7.4	7.2	6.8-8.0	7.0	6.6-7.4	7.1	6.7-7.5
Finished Water								
Flow, mgd	3.15	2.63-3.67	3.14	2.55-4.08	2.91	2.44-3.28	2.90	2.54-3.37
Turbidity, NTU	0.8	0.5-1.3	0.7	0.3-1.5	0.8	0.3-0.9	0.7	0.3-1.3
Total Alkalinity, mg/l	37	28-46	44	30-66	41	26-60	42	32-54
pH	9.0	8.3-9.3	9.0	8.4-9.4	8.8	8.6-9.6	8.7	8.0-9.6
Fluoride, mg/l	1.0	0.7-1.4	1.0	0.6-2.0	1.0	0.6-1.4	1.2	0.9-1.7
Chloramine Residual								
WTP, mg/l	3.0	3.0-3.0	2.8	2.5-3.0	2.9	2.2-3.0	2.9	2.5-3.0
WWTP, mg/l	3.7	3.0-4.0	2.8	1.5-4.0	3.0	1.8-4.0	2.1	1.0-3.0
Alum, gallons	28,789		21,769		21,769		28,197	
Alum, mg/l	166	105-273	142	113-234	127	101-173	189	113-234
Lime, pounds	31,287		25,619		25,620		34,920	
Lime, mg/l	33.4	24.0-42.8	30.9	14.0-41.1	27.6	18.8-47.9	43.3	20.5-58.2
Chlorine, pounds	6,000		4,000		4,000		6,000	
Ammonia, pounds	1,237		1,475		1,475		1,198	
Chlorine: Ammonia Ratio	4.9/1		2.7/1		2.7/1		5.0/1	
Hydrofluosilicic Acid								
(25%), pounds	5,250		4,200		4,200		4,500	
Aclivated Carbon pounds	None		None		None		None	

Source: City of Sulphur Springs

1985 Plant Operating Records (Cont.)

	May		June		July		August	
	Ave	Range	Ave	Range	Ave	Range	Ave	Range
<u>Raw Water</u>								
Flow, mgd	3.59	3.13-5.32	4.84	4.28-5.68	4.98	3.20-6.44	4.81	3.67-5.61
Turbidity, NTU	36	26-49	37	31-42	37	28-41	34	20-58
Total Alkalinity, mg/l	31	24-38	30	26-36	31	28-34	31	18-36
pH	7.2	6.8-7.6	7.1	7.0-7.3	7.2	7.0-7.4	7.1	6.8-7.4
<u>Finished Water</u>								
Flow, mgd	3.10	2.54-3.90	3.64	2.86-4.29	4.00	2.84-4.87	4.56	3.42-5.38
Turbidity, NTU	0.7	0.4-1.1	0.8	0.5-1.1	0.8	0.4-1.1	0.7	0.2-1.1
Total Alkalinity, mg/l	31	24-52	39	30-50	42	32-50	39	22-44
pH	8.8	8.0-9.2	8.7	8.3-9.6	8.6	8.0-9.4	8.4	8.0-8.9
Fluoride, mg/l	1.1	0.6-1.7	0.9	0.5-1.3	1.0	0.5-1.5	0.9	0.5-1.2
Chloramine Residual								
WTP, mg/l	2.8	2.2-3.0	2.9	2.1-3.0	2.9	2.3-3.0	2.8	2.3-3.0
WWTP, mg/l	2.4	1.0-4.0	2.0	1.3-3.0	2.0	1.0-3.0	1.8	1.0-3.0
Alum, gallons	32,481		39,490		51,063		43,715	
Alum, mg/l	189	166-224	176	125-222	214	129-347	190	80-240
Lime, pounds	38,887		44,484		43,107		39,790	
Lime, mg/l	41.9	27.4-51.4	36.7	30.8-58.2	33.5	24.0-56.5	32.0	17.1-46.2
Chlorine, pounds	6,000		4,000		6,000		6,000	
Ammonia, pounds	837		2,000		1,237		1,196	
Chlorine: Ammonia Ratio	7.2/1		2.0/1		4.9/1		5.0/1	
Hydrofluosilicic Acid								
(25%), pounds	4,800		5,100		5,400		5,850	
Aclivated Carbon pounds	None		2,000		6,600		7,000	

Source: City of Sulphur Springs

1985 Plant Operating Records (Cont.)

	September Ave	September Range	October Ave	October Range	November Ave	November Range	December Ave	December Range
Raw Water								
Flow, mgd	4.67	3.04-6.08	3.31	2.31-4.53	2.86	1.98-3.63	3.80	2.52-6.31
Turbidity, NTU	31	20-100	26	23-31	30	19-54	30	12-43
Total Alkalinity, mg/l	35	27-44	32	28-38	29	20-33	27	20-38
pH	7.0	6.8-7.4	7.0	6.8-7.4	7.1	6.8-7.5	7.1	6.8-7.4
Finished Water								
Flow, mgd	3.90	2.38-5.38	2.81	2.19-3.34	2.46	2.00-2.95	2.84	2.18-4.26
Turbidity, NTU	0.6	0.3-0.9	0.4	0.2-0.7	0.6	0.2-0.9	0.7	0.4-1.6
Total Alkalinity, mg/l	43	32-50	40	32-48	36	26-44	33	20-42
pH	8.6	8.0-9.1	8.7	8.5-9.1	8.6	8.0-9.0	8.7	8.3-8.9
Fluoride, mg/l	0.9	0.2-1.6	1.1	0.8-1.8	1.0	0.6-1.4	1.1	0.7-1.6
Chloramine Residual								
WTP, mg/l	2.8	2.2-3.0	3.0	2.8-3.0	3.0	2.8-3.0	3.1	2.0-4.0
WWTP, mg/l	2.3	1.5-4.0	3.3	2.0-4.0	3.5	2.0-4.0	3.1	2.0-4.0
Alum, gallons	36,313		27,382		26,072		34,667	
Alum, mg/l	168	95-234	155	129-228	197	148-281	191	125-269
Lime, pounds	39,511		24,517		23,280		25,145	
Lime, mg/l	33.8	10.3-49.6	28.6	10.3-44.5	32.5	13.7-58.2	25.6	18.8-47.9
Chlorine, pounds	8,000		4,000		6,000		4,000	
Ammonia, pounds	1,197		1,237		1,158		1,198	
Chlorine: Ammonia Ratio	6.7/1		3.2/1		5.2/1		3.3/1	
Hydrofluosilicic Acid (25%), pounds	5,400		4,950		4,500		4,650	
Aclivated Carbon pounds	30,000		49,400		6,800		None	

Source: City of Sulphur Springs

1986 Plant Operating Records

	January Ave	January Range	Feburary Ave	Feburary Range	March Ave	March Range	April Ave	April Range
Raw Water								
Flow, mgd	3.61	2.40-4.00	3.00	2.27-3.86	2.96	2.31-4.13	2.80	2.04-3.51
Turbidity, NTU	20	17-26	21	14-66	22	17-31	26	19-31
Total Alkalinity, mg/l	26	21-30	25	14-30	24	18-30	23	20-28
pH	7.1	6.8-7.3	7.0	6.9-7.3	7.0	6.8-7.2	6.9	6.8-7.6
Finished Water								
Flow, mgd	2.60	2.22-2.92	2.46	2.07-2.88	2.57	2.16-3.07	2.52	1.77-3.16
Turbidity, NTU	0.5	0.3-0.8	0.6	0.3-1.0	0.6	0.3-1.0	0.6	0.3-1.0
Total Alkalinity, mg/l	32	26-36	32	30-38	29	20-34	28	22-40
pH	8.7	8.3-8.9	8.7	8.5-9.0	8.6	8.4-8.7	8.6	8.2-8.8
Fluoride, mg/l	1.0	0.6-1.9	1.1	0.7-1.3				
Chloramine Residual								
WTP, mg/l	3.0	3.0-3.0	3.0	2.8-3.0	3.0	3.0-3.0	3.0	2.6-3.0
WWTP, mg/l	2.7	2.0-3.0	2.8	2.0-4.0	2.7	2.0-3.5	2.3	1.5-4.0
Alum, gallons	18,698		13,438		14,620		13,407	
Alum, mg/l	108	43-230	104	74-138	103	84-121	103	72-207
Lime, pounds	18,197		12,913		13,697		17,488	
Lime, mg/l	19.5	8.6-41.1	18.4	0.2-36.0	17.9	5.1-36.0	25.0	12.0-41.1
Chlorine, pounds	6,000		4,000		4,000		4,000	
Ammonia, pounds	1,236		1,117		1,237		1.197	
Chlorine: Ammonia Ratio	4.9/1		3.6/1		3.2/1		3.3/1	
Hydrofluosilicic Acid								
(25%), pounds	4,650		4,200		4,350		4,650	
Aclivated Carbon pounds	None		None		None		None	

Source: City of Sulphur Springs

1987 Plant Operating Records

	January		Feburary		March		April	
	Ave	Range	Ave	Range	Ave	Range	Ave	Range
Raw Water								
Flow, mgd	2.68	2.15-3.48	2.84	2.40-4.20	2.85	2.25-3.31	3.24	2.37-4.70
Turbidity, NTU	26	20-60	24	19-29	24	18-57	24	18-35
Total Alkalinity, mg/l	28	18-34	27	20-34	24	16-30	26	18-32
pH	7.1	6.8-7.6	7.1	6.8-7.4	6.9	6.8-7.3	7.0	6.8-7.2
Finished Water								
Flow, mgd	2.34	1.86-2.77	2.25	2.01-3.30	2.47	1.93-2.86	3.12	2.15-3.67
Turbidity, NTU	0.3	0.1-1.0	0.2	0.0-0.5	0.1	0.0-0.2	0.1	0.0-0.4
Total Alkalinity, mg/l	32	24-44	35	22-42	34	22-44	35	28-46
pH	8.6	8.4-8.8	8.7	8.5-8.9	8.8	8.6-8.9	8.7	8.3-8.9
Fluoride, mg/l	1.1	0.9-1.5	1.1	0.5-1.7	1.2	0.9-1.4	1.1	0.8-1.9
Chloramine Residual								
WTP, mg/l	3.0	3.0-3.0	2.9	2.7-3.0	3.0	3.0-3.0	3.0	2.8-3.0
WWTP, mg/l	3.5	2.5-4.0	3.3	2.5-4.0	3.0	2.0-4.0	2.4	2.0-3.5
Alum, gallons	12,685		13,793		12,543		13,797	
Alum, mg/l	99	70-150	112	72-158	92	80-136	92	78-109
Lime, pounds	22,393		15,888		14,065		17,116	
Lime, mg/l	32.3	12.0-53.1	24.0	10.3-49.6	19.1	10.3-34.2	21.1	8.6-34.2
Chlorine, pounds	4,000		4,000		6,000		6,000	
Ammonia, pounds	1,857		1,676		1,856		1,796	
Chlorine: Ammonia Ratio	2.1/1		2.4/1		3.2/1		3.3/1	
Hydrofluosilicic Acid								
(25%), pounds	4,500		3,300		3,600		3,600	
Aclivated Carbon pounds	None		None		None		None	

Source: City of Sulphur Springs

1986 Plant Operating Records

	January		Feburary		March		April	
	Ave	Range	Ave	Range	Ave	Range	Ave	Range
Raw Water								
Flow, mgd	3.61	2.40-4.00	3.00	2.27-3.86	2.96	2.31-4.13	2.80	2.04-3.51
Turbidity, NTU	20	17-26	21	14-66	22	17-31	26	19-31
Total Alkalinity, mg/l	26	21-30	25	14-30	24	18-30	23	20-28
pH	7.1	6.8-7.3	7.0	6.9-7.3	7.0	6.8-7.2	6.9	6.8-7.6
Finished Water								
Flow, mgd	2.60	2.22-2.92	2.46	2.07-2.88	2.57	2.16-3.07	2.52	1.77-3.16
Turbidity, NTU	0.5	0.3-0.8	0.6	0.3-1.0	0.6	0.3-1.0	0.6	0.3-1.0
Total Alkalinity, mg/l	32	26-36	32	30-38	29	20-34	28	22-40
pH	8.7	8.3-8.9	8.7	8.5-9.0	8.6	8.4-8.7	8.6	8.2-8.8
Fluoride, mg/l	1.0	0.6-1.9	1.1	0.7-1.3				
Chloramine Residual								
WTP, mg/l	3.0	3.0-3.0	3.0	2.8-3.0	3.0	3.0-3.0	3.0	2.6-3.0
WWTP, mg/l	2.7	2.0-3.0	2.8	2.0-4.0	2.7	2.0-3.5	2.3	1.5-4.0
Alum, gallons	18,698		13,438		14,620		13,407	
Alum, mg/l	108	43-230	104	74-138	103	84-121	103	72-207
Lime, pounds	18,197		12,913		13,697		17,488	
Lime, mg/l	19.5	8.6-41.1	18.4	0.2-36.0	17.9	5.1-36.0	25.0	12.0-41.1
Chlorine, pounds	6,000		4,000		4,000		4,000	
Ammonia, pounds	1,236		1,117		1,237		1.197	
Chlorine: Ammonia Ratio	4.9/1		3.6/1		3.2/1		3.3/1	
Hydrofluosilicic Acid (25%), pounds	4,650		4,200		4,350		4,650	
Aclivated Carbon pounds	None		None		None		None	

Source: City of Sulphur Springs

1986 Plant Operating Records (Cont.)

	May Ave	May Range	June Ave	June Range	July Ave	July Range	August Ave	August Range
Raw Water								
Flow, mgd	3.10	2.24-4.73	3.68	2.00-5.60	4.85	3.34-6.23	4.99	3.50-6.16
Turbidity, NTU	31	22-38	30	17-52	32	20-74	25	17-99
Total Alkalinity, mg/l	25	20-30	29	22-32	25	16-30	28	24-32
pH	6.9	6.8-7.2	7.0	6.8-7.6	7.3	6.8-7.4	7.0	6.8-7.3
Finished Water								
Flow, mgd	2.55	2.01-3.18	2.70	1.65-3.89	3.80	2.44-4.89	4.26	3.37-5.17
Turbidity, NTU	0.6	0.3-0.9	0.4	0.1-1.2	0.5	0.2-0.9	0.3	0.0-0.6
Total Alkalinity, mg/l	30	22-36	35	28-38	32	24-38	34	30-38
pH	8.6	8.3-8.8	8.6	8.4-9.0	8.5	8.3-8.7	8.5	8.3-8.7
Fluoride, mg/l	1.0	0.7-1.6	1.0	0.8-1.3	0.8 (1)	0.0-1.1	0.1 (2)	0.0-1.1
Chloramine Residual								
WTP, mg/l	3.0	3.0-3.0	3.0	3.0-3.0	2.9	2.0-3.0	3.0	2.0-3.0
WWTP, mg/l	2.6	2.0-4.0	2.2	2.0-3.0	1.7	1.5-2.0	1.8	1.0-2.5
Alum, gallons	16,758		20,415		26,752		23,827	
Alum, mg/l	113	82-154	120	78-222	115	84-144	100	66-177
Lime, pounds	23,480		27,329		29,948		27,299	
Lime, mg/l	29.3	13.7-51.4	29.7	10.3-80.5	23.9	15.4-34.2	21.2	13.7-29.1
Chlorine, pounds	6,000		4,000		4,000		8,000	
Ammonia, pounds	1,200		1,157		1,235		1,235	
Chlorine: Ammonia Ratio	5.0/1		3.5/1		3.2/1		6.5/1	
Hydrofluosilicic Acid (25%), pounds	4,650		4,500		1,500 (1)		750 (2)	
Activated Carbon pounds	None		10,200		50,000		46,800	

Source: City of Sulphur Springs

(1) Hydrofluosilicic Acid pump inoperative for 20 days.
(2) Hydrofluosilicic Acid pump inoperative for 26 days.

1986 Plant Operating Records (Cont.)

	September		October		November		December	
	Ave	Range	Ave	Range	Ave	Range	Ave	Range
Raw Water								
Flow, mgd	3.61	2.48-5.07	3.83	2.37-5.22	3.19	2.43-4.63	2.84	2.05-3.59
Turbidity, NTU	24	15-94	25	17-79	29	18-61	25	19-34
Total Alkalinity, mg/l	29	24-32	28	24-34	27	22-30	29	26-32
pH	7.0	6.8-7.2	7.0	6.8-7.2	7.0	6.8-7.2	7.1	6.9-7.3
Finished Water								
Flow, mgd	3.13	2.11-3.63	3.00	2.48-3.95	2.61	2.08-3.29	2.37	1.80-2.66
Turbidity, NTU	0.2	0.1-0.4	0.2	0.0-0.5	0.2	0.0-0.5	0.2	0.0-0.4
Total Alkalinity, mg/l	33	32-36	35	28-58	32	30-38	33	24-38
pH	8.6	8.3-8.8	8.6	8.3-8.9	8.6	8.5-8.9	8.6	8.4-8.8
Fluoride, mg/l	0.9	0.7-1.5	1.1	0.7-1.4	1.0	0.4-1.6	0.9	0.2-1.3
Chloramine Residual								
WTP, mg/l	3.0	3.0-3.0	3.0	3.0-3.0	3.0	2.6-3.0	3.0	2.8-3.0
WWTP, mg/l	2.1	1.5-3.0	2.5	1.5-4.0	3.5	2.5-4.0	3.4	2.0-4.0
Alum, gallons	17,391		18,162		15,185		14,317	
Alum, mg/l	104	80-158	99	64-152	103	64-171	105	70-207
Lime, pounds	19,364		22,518		19,638		22,221	
Lime, mg/l	21.4	5.1-46.2	22.7	12.0-39.4	24.6	0.0-42.8	30.3	6.8-46.2
Chlorine, pounds	4,000		6,000		6,000		4,000	
Ammonia, pounds	1,494		1,544		1,498		1,757	
Chlorine: Ammonia Ratio	2.7/1		3.9/1		4.0/1		2.3/1	
Hydrofluosilicic Acid								
(25%), pounds	4,500		4,650		4,050		3,750	
Activated Carbon pounds	46,500		10,650		None		None	

Source: City of Sulphur Springs

1987 Plant Operating Records

	January		Feburary		March		April	
	Ave	Range	Ave	Range	Ave	Range	Ave	Range
Raw Water								
Flow, mgd	2.68	2.15-3.48	2.84	2.40-4.20	2.85	2.25-3.31	3.24	2.37-4.70
Turbidity, NTU	26	20-60	24	19-29	24	18-57	24	18-35
Total Alkalinity, mg/l	28	18-34	27	20-34	24	16-30	26	18-32
pH	7.1	6.8-7.6	7.1	6.8-7.4	6.9	6.8-7.3	7.0	6.8-7.2
Finished Water								
Flow, mgd	2.34	1.86-2.77	2.25	2.01-3.30	2.47	1.93-2.86	3.12	2.15-3.67
Turbidity, NTU	0.3	0.1-1.0	0.2	0.0-0.5	0.1	0.0-0.2	0.1	0.0-0.4
Total Alkalinity, mg/l	32	24-44	35	22-42	34	22-44	35	28-46
pH	8.6	8.4-8.8	8.7	8.5-8.9	8.8	8.6-8.9	8.7	8.3-8.9
Fluoride, mg/l	1.1	0.9-1.5	1.1	0.5-1.7	1.2	0.9-1.4	1.1	0.8-1.9
Chloramine Residual								
WTP, mg/l	3.0	3.0-3.0	2.9	2.7-3.0	3.0	3.0-3.0	3.0	2.8-3.0
WWTP, mg/l	3.5	2.5-4.0	3.3	2.5-4.0	3.0	2.0-4.0	2.4	2.0-3.5
Alum, gallons	12,685		13,793		12,543		13,797	
Alum, mg/l	99	70-150	112	72-158	92	80-136	92	78-109
Lime, pounds	22,393		15,888		14,065		17,116	
Lime, mg/l	32.3	12.0-53.1	24.0	10.3-49.6	19.1	10.3-34.2	21.1	8.6-34.2
Chlorine, pounds	4,000		4,000		6,000		6,000	
Ammonia, pounds	1,857		1,676		1,856		1,796	
Chlorine: Ammonia Ratio	2.1/1		2.4/1		3.2/1		3.3/1	
Hydrofluosilicic Acid (25%), pounds	4,500		3,300		3,600		3,600	
Aclivated Carbon pounds	None		None		None		None	

Source: City of Sulphur Springs

1987 Plant Operating Records (Cont.)

	May Ave	May Range	June Ave	June Range	July Ave	July Range	August Ave	August Range
Raw Water								
Flow, mgd	3.03	2.30-4.49	3.01	2.43-3.84	3.69	2.54-5.27	4.91	3.48-6.44
Turbidity, NTU	31	23-40	30	23-35	33	26-38	33	25-41
Total Alkalinity, mg/l	26	20-32	26	10-33	27	16-32	28	24-32
pH	6.9	6.8-7.1	7.0	6.8-7.2	7.0	6.8-7.4	7.0	6.8-7.2
Finished Water								
Flow, mgd	2.80	2.25-3.47	2.83	2.37-3.40	3.46	2.33-4.91	4.35	2.96-5.37
Turbidity, NTU	0.1	0.0-0.4	0.1	0.0-0.3	0.2	0.0-0.6	0.3	0.1-2.0
Total Alkalinity, mg/l	34	28-38	35	28-52	37	30-46	37	32-46
pH	8.6	8.5-8.8	8.8	8.2-9.0	8.8	8.5-9.0	8.8	8.5-8.9
Fluoride, mg/l	1.0	0.8-1.2	1.1	0.5-1.4	0.9	0.7-1.2	0.9	0.8-1.1
Chloramine Residual								
WTP, mg/l	3.0	2.5-3.0	2.9	2.1-3.0	3.0	3.0-3.0	3.0	3.0-3.0
WWTP, mg/l	1.8	1.0-2.0	1.3	0.2-2.5	1.0	0.2-2.0	0.8	0.2-1.5
Alum, gallons	14,853		14,422		17,944		20,464	
Alum, mg/l	102	80-138	103	82-112	102	84-125	87	62-105
Lime, pounds	26,048		28,520		33,218		32,707	
Lime, mg/l	33.2	18.8-56.5	37.9	25.7-58.2	34.8	8.6-47.9	25.8	18.8-36.0
Chlorine, pounds	4,000		6,000		6,000		6,000	
Ammonia, pounds	1,996		1,748		2,166		2,166	
Chlorine: Ammonia Ratio	2.0/1		3.4/1		2.8/1		2.8/1	
Hydrofluosilicic Acid								
(25%), pounds	3,450		3,900		3,450		4,350	
Aclivated Carbon pounds	None		16,816		24,800		46,050	

Source: City of Sulphur Springs

1987 Plant Operating Records (Cont.)

	September		October		November		December	
	Ave	Range	Ave	Range	Ave	Range	Ave	Range
Raw Water								
Flow, mgd	3.42	2.32-5.34	2.94	2.29-3.74	2.66	1.56-3.63	2.54	1.90-3.34
Turbidity, NTU	31	25-38	32	21-52	26	19-38	23	18-38
Total Alkalinity, mg/l	32	28-40	30	26-36	28	16-38	26	20-34
pH	7.0	6.8-7.2	7.0	6.8-7.2	7.0	6.8-7.1	6.9	6.8-7.2
Finished Water								
Flow, mgd	3.19	2.36-4.39	2.81	2.18-3.26	2.59	2.24-2.98	3.40	1.87-3.06
Turbidity, NTU	0.2	0.01-0.9	0.3	0.1-0.8	0.2	0.1-0.8	0.3	0.1-0.5
Total Alkalinity, mg/l	40	32-52	38	34-44	37	28-48	36	30-50
pH	8.8	8.6-9.0	8.8	8.7-8.9	8.8	8.5-8.9	8.7	8.5-8.9
Fluoride, mg/l	1.1	0.9-1.4	1.2	0.9-1.5	1.2	0.8-1.5	1.2	0.8-1.6
Chloramine Residual								
WTP, mg/l	3.0	3.0-3.0	3.0	3.0-3.0	3.0	3.0-3.0	3.0	3.0-3.0
WWTP, mg/l	0.3	0.2-1.0	1.1	0.2-2.5	1.9	1.5-3.0	2.8	2.0-4.0
Alum, gallons	16,719		17,345		14,794		12,730	
Alum, mg/l	106	88-129	123	107-146	120	97-228	105	88-129
Lime, pounds	28,645		29,978		27,372		23,950	
Lime, mg/l	33.5	25.7-42.8	39.4	30.8-53.1	41.1	25.7-59.9	36.5	29.1-46.2
Chlorine, pounds	4,000		6,000		6,000		6,000	
Ammonia, pounds	2,096		2,167		2,098		2,268	
Chlorine: Ammonia Ratio	1.9/1		2.8/1		2.9/1		2.6/1	
Hydrofluosilicic Acid								
(25%), pounds	3,750		4,350		4,200		4,050	
Aclivated Carbon pounds	32,800		None		None		None	

Source: City of Sulphur Springs

Appendix B

Opinions of Probable Cost

TABLE B-1
OPINION OF PROBABLE COST
14 MGD
CHANNEL INTAKE/PUMP STATION
FINLEY BRANCH

	ITEM	QUANTITY	UNIT	UNIT COST $	TOTAL COST $
1.0	SITE WORK				
1.1	Site Prep, Move-in, etc.	1	LS		25,000
1.2	Access Road	300	LF	28.00	8,400
1.3	Channel Excavation	95,650	CY	2.00	191,300
1.4	Site Fill	2,300	CY	15.35	35,305
1.5	Finish Grading	1	LS		5,000
	Subtotal Section 1.0				$265,005
2.0	PUMP STATION				
2.1	Sheet Piling	171	TN	425.00	72,675
2.2	Excavation	8,050	CY	10.00	80,500
2.3	Compacted Granular Fill	7,060	CY	13.00	91,780
CONCRETE					
2.4	Slab on Grade	65	CY	250.00	16,250
2.5	Walls	357	CY	350.00	124,950
2.6	Suspended Slabs	97	CY	450.00	43,650
3.0	METAL	1	LS		15,000
4.0	DOORS				
4.1	Overhead	1	EA	1,200.00	1,200
4.2	Hollow Core Metal	1	EA	400.00	400
5.0	EQUIPMENT				
5.1	Vertical Turbine Pumps	3	EA	126,000	378,000
5.2	Travelling Water Screens	2	EA	109,000	218,000
5.3	Sluice Gates	3	EA	81,000	243,000
6.0	MECHANICAL				
6.1	Steel Pipe	15,000	LB	3.00	45,000
6.2	Misc. Pipe	1,000	LB	3.00	3,000
7.0	CONVEYING SYSTEM				
7.1	10 Ton Travelling Bridge and Crane	1	LS		40,000
8.0	PIPE SUPPORTS	1	LS		2,000
9.0	AIR COMPRESSOR	1	LS		250,000
10.0	ROOFING	1,400	SF	3.00	4,200
11.0	VALVES				
11.1	24" Butterfly	4	EA	3,100	12,400
11.2	24" Pneumatic Ball	4	EA	20,000	80,000
12.0	FLOWMETERING	1	LS		50,000
	Subtotal Section 2-12				1,772,005
13.0	HEATING & VENTILATION	5%	PS		88,600
14.0	ELECTRICAL, INSTRUMENTATION AND CONTROLS	25%	PS		443,000
	Total Construction Costs				$ 2,568,610

TABLE B-2
OPINION OF PROBABLE COST
14 MGD
SUBMERGED INTAKE/PUMP STATION
FINLEY BRANCH

		ITEM	QUANTITY	UNIT	UNIT COST $	TOTAL COST $
1.0		SITE WORK				
	1.1	Site Prep, Move-in, etc.	1	LS		25,000
	1.2	Access Road	300	LF	28.00	8,400
	1.3	Excavation For Suction Pipe	58,900	CY	2.00	117,800
	1.4	48" Suction Piping	2,000	LF	91.00	182,000
	1.5	48" 90 Degrees Elbow	3	EA	4,210.00	12,630
	1.6	Compacted Backfill	46,800	CY	9.00	421,200
	1.7	Finish Grading	1	LS	5,000.00	5,000
		Subtotal Section 1.0				$772,030
2.0		PUMP STATION				
	2.1	Sheet Piling	150	TN	425.00	63,750
	2.2	Excavation	5,950	CY	10.00	59,500
	2.3	Compacted Granular Fill	5,500	CY	13.00	71,500
	CONCRETE					
	2.4	Slab on Grade	36	CY	250.00	9,000
	2.5	Walls	230	CY	350.00	80,500
	2.6	Suspended Slabs	59	CY	450.00	26,550
3.0		METAL	1	LS		15,000
4.0		DOORS				
	4.1	Overhead	1	EA	1,200.00	1,200
	4.2	Hollow Core Metal	1	EA	400.00	400
5.0		EQUIPMENT				
	5.1	Vertical Turbine Pumps	3	EA	126,000	378,000
	5.2	Sluice Gates	3	EA	81,000	243,000
	5.3	Johnson Screens	1	LS		252,000
6.0		MECHANICAL				
	6.1	Steel Pipe	15,000	LB	3.00	45,000
	6.2	Misc. Pipe	1,000	LB	3.00	3,000
7.0		CONVEYING SYSTEM				
	7.1	10 Ton Travelling Bridge and Crane	1	LS		40,000
8.0		PIPE SUPPORTS	1	LS		2,000
9.0		AIR COMPRESSOR	1	LS		250,000
10.0		ROOFING	790	SF	3.00	2,370
11.0		VALVES				
	11.1	24" Butterfly	4	EA	3,100	12,400
	11.2	24" Pneumatic Ball	4	EA	20,000	80,000
12.0		FLOWMETERING	1	LS		50,000
		Subtotal Section 2-12				1,685,170
13.0		HEATING & VENTILATION	5%	PS		84,300
14.0		ELECTRICAL, INSTRUMENTATION AND CONTROLS	25%	PS		421,300
		Total Construction Costs				$ 2,962,800

TABLE B-3
OPINION OF PROBABLE COST
14 MGD
CHANNEL INTAKE/PUMP STATION
HARPER'S HILL

		ITEM	QUANTITY	UNIT	UNIT COST $	TOTAL COST $
1.0		SITE WORK				
	1.1	Site Prep, Move-in, etc.	1	LS		25,000
	1.2	Access Road	600	LF	28.00	16,800
	1.3	Channel Excavation	42,400	CY	2.00	84,800
	1.4	Finish Grading	1	LS		5,000
		Subtotal Section 1.0				$131,600
2.0		PUMP STATION				
	2.1	Sheet Piling	171	TN	425.00	72,675
	2.2	Excavation	8,050	CY	10.00	80,500
	2.3	Compacted Granular Fill	7,060	CY	13.00	91,780
	CONCRETE					
	2.4	Slab on Grade	65	CY	250.00	16,250
	2.5	Walls	357	CY	350.00	124,950
	2.6	Suspended Slabs	97	CY	450.00	43,650
3.0		METAL	1	LS		15,000
4.0		DOORS				
	4.1	Overhead	1	EA	1,200.00	1,200
	4.2	Hollow Core Metal	1	EA	400.00	400
5.0		EQUIPMENT				
	5.1	Vertical Turbine Pumps	3	EA	126,000	378,000
	5.2	Travelling Water Screens	2	EA	109,000	218,000
	5.3	Sluice Gates	3	EA	81,000	243,000
6.0		MECHANICAL				
	6.1	Steel Pipe	15,000	LB	3.00	45,000
	6.2	Misc. Pipe	1,000	LB	3.00	3,000
7.0		CONVEYING SYSTEM				
	7.1	10 Ton Travelling Bridge and Crane	1	LS		40,000
8.0		PIPE SUPPORTS	1	LS		2,000
9.0		AIR COMPRESSOR	1	LS		250,000
10.0		ROOFING	1,400	SF	3.00	4,200
11.0		VALVES				
	11.1	24" Butterfly	4	EA	3,100	12,400
	11.2	24" Pneumatic Ball	4	EA	20,000	80,000
12.0		FLOWMETERING	1	LS		50,000
		Subtotal Section 2-12				1,772,005
13.0		HEATING & VENTILATION	5%	PS		88,600
14.0		ELECTRICAL, INSTRUMENTATION AND CONTROLS	25%	PS		422,800
		Total Construction Costs				$ 2,435,205

TABLE B-4
OPINION OF PROBABLE COST
14 MGD
SUBMERGED INTAKE/PUMP STATION
HARPER'S HILL

	ITEM	QUANTITY	UNIT	UNIT COST $	TOTAL COST $
1.0	SITE WORK				
1.1	Site Prep, Move-in, etc.	1	LS		25,000
1.2	Access Road	600	LF	28.00	16,800
1.3	Excavation For Suction Pipe	27,800	CY	2.00	55,600
1.4	48" Suction Piping	960	LF	91.00	87,360
1.5	48" 90 Degrees Elbow	3	EA	4,210.00	12,630
1.6	Compacted Backfill	18,200	CY	9.00	163,800
1.7	Finish Grading	1	LS		5,000
	Subtotal Section 1.0				$366,190
2.0	PUMP STATION				
2.1	Sheet Piling	150	TN	425.00	63,750
2.2	Excavation	5,950	CY	10.00	59,500
2.3	Compacted Granular Fill	5,500	CY	13.00	71,500
CONCRETE					
2.4	Slab on Grade	36	CY	250.00	9,000
2.5	Walls	230	CY	350.00	80,500
2.6	Suspended Slabs	59	CY	450.00	26,550
3.0	METAL	1	LS		15,000
4.0	DOORS				
4.1	Overhead	1	EA	1,200.00	1,200
4.2	Hollow Core Metal	1	EA	400.00	400
5.0	EQUIPMENT				
5.1	Vertical Turbine Pumps	3	EA	126,000	378,000
5.2	Sluice Gates	3	EA	81,000	243,000
5.3	Johnson Screens	1	LS		252,000
6.0	MECHANICAL				
6.1	Steel Pipe	15,000	LB	3.00	45,000
6.2	Misc. Pipe	1,000	LB	3.00	3,000
7.0	CONVEYING SYSTEM				
7.1	10 Ton Travelling Bridge and Crane	1	LS		40,000
8.0	PIPE SUPPORTS	1	LS		2,000
9.0	AIR COMPRESSOR	1	LS		250,000
10.0	ROOFING	790	SF	3.00	2,370
11.0	VALVES				
11.1	24" Butterfly	4	EA	3,100	12,400
11.2	24" Pneumatic Ball	4	EA	20,000	80,000
12.0	FLOWMETERING	1	LS	50,000	50,000
	Subtotal Section 2-12				1,685,170
13.0	HEATING & VENTILATION	5%	PS		84,300
14.0	ELECTRICAL, INSTRUMENTATION AND CONTROLS	25%	PS		421,300
	Total Construction Costs				$ 2,556,960

TABLE B-5
OPINION OF PROBABLE COST
14 MGD
CHANNEL INTAKE/PUMP STATION
EAST BANK

	ITEM	QUANTITY	UNIT	UNIT COST $	TOTAL COST $
1.0	SITE WORK				
1.1	Site Prep, Move-in, etc.	1	LS		25,000
1.2	Access Road	200	LF	28.00	5,600
1.3	Channel Excavation	53,300	CY	2.00	106,600
1.4	Finish Grading	1	LS		5,000
	Subtotal Section 1.0				$142,200
2.0	PUMP STATION				
2.1	Sheet Piling	171	TN	425.00	72,675
2.2	Excavation	8,050	CY	10.00	80,500
2.3	Compacted Granular Fill	7,060	CY	13.00	91,780
	CONCRETE				
2.4	Slab on Grade	65	CY	250.00	16,250
2.5	Walls	357	CY	350.00	124,950
2.6	Suspended Slabs	97	CY	450.00	43,650
3.0	METAL	1	LS		15,000
4.0	DOORS				
4.1	Overhead	1	EA	1,200.00	1,200
4.2	Hollow Core Metal	1	EA	400.00	400
5.0	EQUIPMENT				
5.1	Vertical Turbine Pumps	3	EA	126,000	378,000
5.2	Travelling Water Screens	2	EA	109,000	218,000
5.3	Sluice Gates	3	EA	81,000	243,000
6.0	MECHANICAL				
6.1	Steel Pipe	15,000	LB	3.00	45,000
6.2	Misc. Pipe	1,000	LB	3.00	3,000
7.0	CONVEYING SYSTEM				
7.1	10 Ton Travelling Bridge and Crane	1	LS		40,000
8.0	PIPE SUPPORTS	1	LS		2,000
9.0	AIR COMPRESSOR	1	LS		250,000
10.0	ROOFING	1,400	SF	3.00	4,200
11.0	VALVES				
11.1	24" Butterfly	4	EA	3,100	12,400
11.2	24" Pneumatic Ball	4	EA	20,000	80,000
12.0	FLOWMETERING	1	LS	50,000	50,000
	Subtotal Section 2-12				1,772,005
13.0	HEATING & VENTILATION	5%	PS		88,600
14.0	ELECTRICAL, INSTRUMENTATION AND CONTROLS	25%	PS		443,000
	Total Construction Costs				$ 2,445,805

TABLE B-6
OPINION OF PROBABLE COST
14 MGD
SUBMERGED INTAKE/PUMP STATION
EAST BANK

	ITEM	QUANTITY	UNIT	UNIT COST $	TOTAL COST $
1.0	SITE WORK				
1.1	Site Prep, Move-in, etc.	1	LS		25,000
1.2	Access Road	200	LF	28.00	5,600
1.3	Excavation For Suction Pipe	30,700	CY	2.00	61,400
1.4	48" Suction Piping	1,800	LF	91.00	163,800
1.5	48" 90 Degrees Elbow	3	EA	4,210.00	12,630
1.6	Compacted Backfill	24,400	CY	9.00	219,600
1.7	Finish Grading	1	LS		5,000
	Subtotal Section 1.0				$493,030
2.0	PUMP STATION				
2.1	Sheet Piling	150	TN	425.00	63,750
2.2	Excavation	5,950	CY	10.00	59,500
2.3	Compacted Granular Fill	5,500	CY	13.00	71,500
CONCRETE					
2.4	Slab on Grade	36	CY	250.00	9,000
2.5	Walls	230	CY	350.00	80,500
2.6	Suspended Slabs	59	CY	450.00	26,550
3.0	METAL	1	LS		15,000
4.0	DOORS				
4.1	Overhead	1	EA	1,200.00	1,200
4.2	Hollow Core Metal	1	EA	400.00	400
5.0	EQUIPMENT				
5.1	Vertical Turbine Pumps	3	EA	126,000	378,000
5.2	Sluice Gates	3	EA	81,000	243,000
5.3	Johnson Screens	1	LS		252,000
6.0	MECHANICAL				
6.1	Steel Pipe	15,000	LB	3.00	45,000
6.2	Misc. Pipe	1,000	LB	3.00	3,000
7.0	CONVEYING SYSTEM				
7.1	10 Ton Travelling Bridge and Crane	1	LS		40,000
8.0	PIPE SUPPORTS	1	LS		2,000
9.0	AIR COMPRESSOR	1	LS		250,000
10.0	ROOFING	790	SF	3.00	2,370
11.0	VALVES				
11.1	24" Butterfly	4	EA	3,100	12,400
11.2	24" Pneumatic Ball	4	EA	20,000	80,000
12.0	FLOWMETERING	1	LS	50,000	50,000
	Subtotal Section 2-12				1,685,170
13.0	HEATING & VENTILATION	5%	PS		84,300
14.0	ELECTRICAL, INSTRUMENTATION AND CONTROLS	25%	PS		421,300
	Total Construction Costs				$ 2,683,800

TABLE B-7
OPINION OF PROBABLE COST
14 MGD
ALTERNATIVE ROUTE 1
FINLEY BRANCH
RAW WATER PIPELINE TO WTP

	ITEM	QUANTITY	UNIT	UNIT COST $	TOTAL COST $
1.0	30" Raw Water Line	65,200	LF	70.00	4,564,000
2.0	30" Elbows	18	EA	1,600	28,800
3.0	Air Relief Manholes	5	EA	5,000	25,000
4.0	TUNNELLED CROSSINGS				
4.1	@ Hiway 71 30"	150	LF	625.00	93,750
4.2	@ FM 2285 30"	80	LF	625.00	50,000
4.3	@ St. Louis & Southwestern RR	100	LF	625.00	62,500
5.0	Asphalt Pavement Remove/Replace	450	LF	15.00	6,750
6.0	Misc. Concrete	1,100	CY	175.00	192,500
7.0	Stream Crossings	500	LF	52.00	26,000
8.0	Easement Aquisition	11.29	AC	800.00	9,032
9.0	Tree Clearing & Grubbing	7.5	AC	2,500.00	18,750
10.0	ACCESS ROAD (20' WIDE)				
10.1	Hydrated Lime Ty 4	330	TN	90.00	29,700
10.2	6" Lime Stab. Base	26,670	SY	1.25	33,338
10.3	3 1/2" HMAC Base Course	5,600	TN	40.00	224,000
10.4	1 1/2" HMAC Surface Course	2,401	TN	40.00	96,040
11.0	12 1/2 Ga. Barb Wire Fence Replacement	65.2	MLF	39.00	2,543
	Total Construction Cost				$5,462,703

TABLE B-8
OPINION OF PROBABLE COST
14 MGD
ALTERNATIVE ROUTE 1A
FINLEY BRANCH/CENTURY DAM
RAW WATER PIPELINE TO WTP

	ITEM	QUANTITY	UNIT	UNIT COST $	TOTAL COST $
1.0	30" Raw Water Line	57,000	LF	70.00	3,990,000
2.0	30" Elbows	9	EA	1,600	14,400
3.0	Air Relief Manholes	5	EA	5,000	25,000
4.0	TUNNELLED CROSSINGS				
4.1	@ Hiway 71 30"	150	LF	625.00	93,750
4.2	@ St. Louis & Southwestern RR	100	LF	625.00	62,500
5.0	Asphalt Pavement Remove/Replace	250	LF	15.00	3,750
6.0	Misc. Concrete	950	CY	175.00	166,250
7.0	Stream Crossings	500	LF	52.00	26,000
8.0	Easement Aquisition	22.13	AC	800.00	17,704
9.0	Tree Clearing & Grubbing	11.2	AC	2,500.00	28,000
10.0	ACCESS ROAD (20' WIDE)				
10.1	Hydrated Lime Ty 4	330	TN	90.00	29,700
10.2	6" Lime Stab. Base	26,670	SY	1.25	33,338
10.3	3 1/2" HMAC Base Course	5,600	TN	40.00	224,000
10.4	1 1/2" HMAC Surface Course	2,401	TN	40.00	96,040
11.0	12 1/2 Ga. Barb Wire Fence Replacement	65.2	MLF	39.00	2,543
12.0	Drilled Piers/Pipe Bridge at Dam	1	LS		1,350,000
13.0	Raw Waterline Across Dam	2,800	LF	35.00	98,000
	Total Construction Cost				$6,260,975

TABLE B-11
OPINION OF PROBABLE COST
14 MGD
ALTERNATIVE ROUTE 2A
HARPER'S HILL/CROSS LAKE SULPHUR SPRINGS
RAW WATER PIPELINE TO WTP

	ITEM	QUANTITY	UNIT	UNIT COST $	TOTAL COST $
1.0	30" Raw Water Line	50,200	LF	80.00	4,016,000
2.0	30" Elbows	31	EA	1,600	49,600
3.0	Air Relief Manholes	3	EA	5,000	15,000
4.0	TUNNELLED CROSSINGS				
4.1	@ Hiway 71 30"	150	LF	625.00	93,750
4.2	@ FM 2285 30"	80	LF	625.00	50,000
4.3	@ St. Louis & Southwestern RR	100	LF	625.00	62,500
5.0	Asphalt Pavement Remove/Replace	500	LF	15.00	7,500
6.0	Misc. Concrete	1,000	CY	175.00	175,000
7.0	Stream Crossings	850	LF	52.00	44,200
8.0	Easement Aquisition	29.02	AC	800.00	23,216
9.0	Tree Clearing & Grubbing	0.62	AC	2,500.00	1,550
10.0	30" Ball Joint Pipe and Dredge Across Lake	7,400	LF	600.00	4,440,000
11.0	12 1/2 Ga. Barb Wire Fence Replacement	63.4	MLF	39.00	2,473
	Subtotal Construction Cost				$8,980,789
12.0	ROAD IMPROVEMENTS				
12.1	New Road North of Hw 71		LS		250,000
12.2	Repair Road South of Hw 71		LS		270,000
	Total Construction Cost w/Road Improvements				$9,500,789

TABLE B-8
OPINION OF PROBABLE COST
14 MGD
ALTERNATIVE ROUTE 1A
FINLEY BRANCH/CENTURY DAM
RAW WATER PIPELINE TO WTP

	ITEM	QUANTITY	UNIT	UNIT COST $	TOTAL COST $
1.0	30" Raw Water Line	57,000	LF	70.00	3,990,000
2.0	30" Elbows	9	EA	1,600	14,400
3.0	Air Relief Manholes	5	EA	5,000	25,000
4.0	TUNNELLED CROSSINGS				
4.1	@ Hiway 71 30"	150	LF	625.00	93,750
4.2	@ St. Louis & Southwestern RR	100	LF	625.00	62,500
5.0	Asphalt Pavement Remove/Replace	250	LF	15.00	3,750
6.0	Misc. Concrete	950	CY	175.00	166,250
7.0	Stream Crossings	500	LF	52.00	26,000
8.0	Easement Aquisition	22.13	AC	800.00	17,704
9.0	Tree Clearing & Grubbing	11.2	AC	2,500.00	28,000
10.0	ACCESS ROAD (20' WIDE)				
10.1	Hydrated Lime Ty 4	330	TN	90.00	29,700
10.2	6" Lime Stab. Base	26,670	SY	1.25	33,338
10.3	3 1/2" HMAC Base Course	5,600	TN	40.00	224,000
10.4	1 1/2" HMAC Surface Course	2,401	TN	40.00	96,040
11.0	12 1/2 Ga. Barb Wire Fence Replacement	65.2	MLF	39.00	2,543
12.0	Drilled Piers/Pipe Bridge at Dam	1	LS		1,350,000
13.0	Raw Waterline Across Dam	2,800	LF	35.00	98,000
	Total Construction Cost				$6,260,975

TABLE B-9
OPINION OF PROBABLE COST
14 MGD
ALTERNATIVE ROUTE 1B
FINLEY BRANCH
RAW WATER PIPELINE TO LAKE SULPHUR SPRINGS

	ITEM	QUANTITY	UNIT	UNIT COST $	TOTAL COST $
1.0	30" Raw Water Line	47,800	LF	70.00	3,346,000
2.0	30" Elbows	8	EA	1,600	12,800
3.0	Air Relief Manholes	4	EA	5,000	20,000
4.0	**TUNNELLED CROSSINGS**				
4.1	@ Hiway 71 30"	150	LF	625.00	93,750
4.2	@ FM 2285 30"	80	LF	625.00	50,000
5.0	Asphalt Pavement Remove/Replace	350	LF	15.00	5,250
6.0	Misc. Concrete	800	CY	175.00	140,000
7.0	Stream Crossings	300	LF	52.00	15,600
8.0	Easement Aquisition	12.30	AC	800.00	9,840
9.0	Tree Clearing & Grubbing	7.8	AC	2,500.00	19,500
10.0	**ACCESS ROAD (20' WIDE)**				
10.1	Hydrated Lime Ty 4	330	TN	90.00	29,700
10.2	6" Lime Stab. Base	26,670	SY	1.25	33,338
10.3	3 1/2" HMAC Base Course	5,600	TN	40.00	224,000
10.4	1 1/2" HMAC Surface Course	2,401	TN	40.00	96,040
11.0	12 1/2 Ga. Barb Wire Fence Replacement	47.8	MLF	39.00	1,864
	Total Construction Cost				$4,097,682

TABLE B-10
OPINION OF PROBABLE COST
14 MGD
ALTERNATIVE ROUTE 2
HARPER'S HILL
RAW WATER PIPELINE TO WTP

	ITEM	QUANTITY	UNIT	UNIT COST $	TOTAL COST $
1.0	30" Raw Water Line	63,400	LF	80.00	5,072,000
2.0	30" Elbows	31	EA	1,600	49,600
3.0	Air Relief Manholes	3	EA	5,000	15,000
4.0	TUNNELLED CROSSINGS				
4.1	@ Hiway 71 30"	150	LF	625.00	93,750
4.2	@ FM 2285 30"	80	LF	625.00	50,000
4.3	@ St. Louis & Southwestern RR	100	LF	625.00	62,500
5.0	Asphalt Pavement Remove/Replace	500	LF	15.00	7,500
6.0	Misc. Concrete	1,000	CY	175.00	175,000
7.0	Stream Crossings	850	LF	52.00	44,200
8.0	Easement Aquisition	29.02	AC	800.00	23,216
9.0	Tree Clearing & Grubbing	0.62	AC	2,500.00	1,550
10.0	12 1/2 Ga. Barb Wire Fence Replacement	63.4	MLF	39.00	2,473
	Subtotal Construction Cost				$5,596,789
11.0	ROAD IMPROVEMENTS				
11.1	New Road North of Hw 71		LS		250,000
11.2	Repair Road South of Hw 71		LS		270,000
	Total Construction Cost w/Road Improvements				$6,116,789

TABLE B-11
OPINION OF PROBABLE COST
14 MGD
ALTERNATIVE ROUTE 2A
HARPER'S HILL/CROSS LAKE SULPHUR SPRINGS
RAW WATER PIPELINE TO WTP

	ITEM	QUANTITY	UNIT	UNIT COST $	TOTAL COST $
1.0	30" Raw Water Line	50,200	LF	80.00	4,016,000
2.0	30" Elbows	31	EA	1,600	49,600
3.0	Air Relief Manholes	3	EA	5,000	15,000
4.0	TUNNELLED CROSSINGS				
4.1	@ Hiway 71 30"	150	LF	625.00	93,750
4.2	@ FM 2285 30"	80	LF	625.00	50,000
4.3	@ St. Louis & Southwestern RR	100	LF	625.00	62,500
5.0	Asphalt Pavement Remove/Replace	500	LF	15.00	7,500
6.0	Misc. Concrete	1,000	CY	175.00	175,000
7.0	Stream Crossings	850	LF	52.00	44,200
8.0	Easement Aquisition	29.02	AC	800.00	23,216
9.0	Tree Clearing & Grubbing	0.62	AC	2,500.00	1,550
10.0	30" Ball Joint Pipe and Dredge Across Lake	7,400	LF	600.00	4,440,000
11.0	12 1/2 Ga. Barb Wire Fence Replacement	63.4	MLF	39.00	2,473
	Subtotal Construction Cost				$8,980,789
12.0	ROAD IMPROVEMENTS				
12.1	New Road North of Hw 71		LS		250,000
12.2	Repair Road South of Hw 71		LS		270,000
	Total Construction Cost w/Road Improvements				$9,500,789

TABLE B-12
OPINION OF PROBABLE COST
14 MGD
ALTERNATIVE ROUTE ROUTE 2B
HARPER'S HILL
RAW WATER PIPELINE TO LAKE SULPHUR SPRINGS

	ITEM	QUANTITY	UNIT	UNIT COST $	TOTAL COST $
1.0	30" Raw Water Line	46,000	LF	80.00	3,680,000
2.0	30" Elbows	21	EA	1,600	33,600
3.0	Air Relief Manholes	2	EA	5,000	10,000
4.0	TUNNELLED CROSSINGS				
4.1	@ Hiway 71 30"	150	LF	625.00	93,750
4.2	@ FM 2285 30"	80	LF	625.00	50,000
5.0	Asphalt Pavement Remove/Replace	350	LF	15.00	5,250
6.0	Misc. Concrete	1,000	CY	175.00	175,000
7.0	Stream Crossings	725	LF	52.00	37,700
8.0	Easement Aquisition	30.03	AC	800.00	24,024
9.0	Tree Clearing & Grubbing	0.92	AC	2,500.00	2,300
10.0	12 1/2 Ga. Barb Wire Fence Replacement	46.0	MLF	39.00	1,794
	Subtotal Construction Cost				$4,113,418
11.0	ROAD IMPROVEMENTS				
11.1	New Road North of Hw 71		LS		250,000
11.2	Repair Road South of Hw 71		LS		270,000
	Total Construction Cost w/Road Improvements				$4,633,418

TABLE B-14
OPINION OF PROBABLE COST
26 MGD
CHANNEL INTAKE/PUMP STATION
FINLEY BRANCH

	ITEM	QUANTITY	UNIT	UNIT COST $	TOTAL COST $
1.0	SITE WORK				
1.1	Site Prep. Move-in etc.	1	LS		25,000
1.2	Access Road	300	LF	28.00	8,400
1.3	Channel Excavation	95,650	CY	2.00	191,300
1.4	Site Fill	2,300	CY	15.35	35,305
1.5	Finish Grading	1	LS		5,000
	Subtotal Section 1.0				$265,005
2.0	PUMP STATION				
2.1	Sheet Piling	180	TN	425.00	76,500
2.2	Excavation	9,200	CY	10.00	92,000
2.3	Compacted Granular Fill	7,700	CY	13.00	100,100
	CONCRETE				
2.4	Slab on Grade	96	CY	250.00	24,000
2.5	Walls	441	CY	350.00	154,350
2.6	Suspended Slabs	156	CY	450.00	70,200
3.0	METAL	1	LS		15,000
4.0	DOORS				
4.1	Overhead	1	EA	1,200.00	1,200
4.2	Hollow Core Metal	1	EA	400.00	400
5.0	EQUIPMENT				
5.1	Vertical Turbine Pumps	4	EA	129,000	516,000
5.2	Travelling Water Screens	3	EA	109,000	327,000
5.3	Sluice Gates	3	EA	81,000	243,000
6.0	MECHANICAL				
6.1	Steel Pipe	15,000	LB	3.00	45,000
6.2	Misc. Pipe	1,000	LB	3.00	3,000
7.0	CONVEYING SYSTEM				
7.1	10 Ton Travelling Bridge and Crane	1	LS		40,000
8.0	PIPE SUPPORTS	1	LS		2,000
9.0	AIR COMPRESSOR	1	LS		250,000
10.0	ROOFING	2,300	SF	3.00	6,900
11.0	VALVES				
11.1	24" Butterfly	4	EA	3,100	12,400
11.2	24" Pneumatic Ball	4	EA	20,000	80,000
12.0	FLOWMETERING	1	LS		50,000
	Subtotal Section 2-12				2,109,050
13.0	HEATING & VENTILATION	5%	PS		105,450
14.0	ELECTRICAL, INSTRUMENTATION AND CONTROLS	25%	PS		527,300
	Total Construction Costs				$ 3,006,805

TABLE B-13
OPINION OF PROBABLE COST
14 MGD
ALTERNATIVE ROUTE 3
EAST BANK
RAW WATER PIPELINE TO WTP

	ITEM	QUANTITY	UNIT	UNIT COST $	TOTAL COST $
1.0	30" Raw Water Line	73,000	LF	70.00	5,110,000
2.0	30" Elbows	16	EA	1,600	25,600
3.0	Air Relief Manholes	6	EA	5,000	30,000
4.0	<u>TUNNELLED CROSSINGS</u>				
4.1	@ Hiway 71 30"	150	LF	625.00	93,750
4.2	@ FM 2285 30"	80	LF	625.00	50,000
4.3	@ St. Louis & Southwestern RR	100	LF	625.00	62,500
5.0	Asphalt Pavement Remove/Replace	530	LF	15.00	7,950
6.0	Misc. Concrete	1,100	CY	175.00	192,500
7.0	Stream Crossings	500	LF	52.00	26,000
8.0	12 1/2 Ga. Barb Wire Fence Replacement	73.0	MLF	39.00	2,847
	Total Construction Cost				$5,601,147

TABLE B-14
OPINION OF PROBABLE COST
26 MGD
CHANNEL INTAKE/PUMP STATION
FINLEY BRANCH

	ITEM	QUANTITY	UNIT	UNIT COST $	TOTAL COST $
1.0	SITE WORK				
1.1	Site Prep. Move-in etc.	1	LS		25,000
1.2	Access Road	300	LF	28.00	8,400
1.3	Channel Excavation	95,650	CY	2.00	191,300
1.4	Site Fill	2,300	CY	15.35	35,305
1.5	Finish Grading	1	LS		5,000
	Subtotal Section 1.0				$265,005
2.0	PUMP STATION				
2.1	Sheet Piling	180	TN	425.00	76,500
2.2	Excavation	9,200	CY	10.00	92,000
2.3	Compacted Granular Fill	7,700	CY	13.00	100,100
CONCRETE					
2.4	Slab on Grade	96	CY	250.00	24,000
2.5	Walls	441	CY	350.00	154,350
2.6	Suspended Slabs	156	CY	450.00	70,200
3.0	METAL	1	LS		15,000
4.0	DOORS				
4.1	Overhead	1	EA	1,200.00	1,200
4.2	Hollow Core Metal	1	EA	400.00	400
5.0	EQUIPMENT				
5.1	Vertical Turbine Pumps	4	EA	129,000	516,000
5.2	Travelling Water Screens	3	EA	109,000	327,000
5.3	Sluice Gates	3	EA	81,000	243,000
6.0	MECHANICAL				
6.1	Steel Pipe	15,000	LB	3.00	45,000
6.2	Misc. Pipe	1,000	LB	3.00	3,000
7.0	CONVEYING SYSTEM				
7.1	10 Ton Travelling Bridge and Crane	1	LS		40,000
8.0	PIPE SUPPORTS	1	LS		2,000
9.0	AIR COMPRESSOR	1	LS		250,000
10.0	ROOFING	2,300	SF	3.00	6,900
11.0	VALVES				
11.1	24" Butterfly	4	EA	3,100	12,400
11.2	24" Pneumatic Ball	4	EA	20,000	80,000
12.0	FLOWMETERING	1	LS		50,000
	Subtotal Section 2-12				2,109,050
13.0	HEATING & VENTILATION	5%	PS		105,450
14.0	ELECTRICAL, INSTRUMENTATION AND CONTROLS	25%	PS		527,300
	Total Construction Costs				$ 3,006,805

TABLE B-15
OPINION OF PROBABLE COST
26 MGD
SUBMERGED INTAKE/PUMP STATION
FINLEY BRANCH

	ITEM	QUANTITY	UNIT	UNIT COST $	TOTAL COST $
1.0	SITE WORK				
1.1	Site Prep. Move-in etc.	1	LS		25,000
1.2	Access Road	300	LF	28.00	8,400
1.3	Excavation For Suction Pipe	58,900	CY	2.00	117,800
1.4	66" Suction Piping	2,000	LF	149.00	298,000
1.5	66" 90 Degrees Elbow	3	EA	9,085.00	27,255
1.6	Compacted Backfill	46,800	CY	9.00	421,200
1.7	Finish Grading	1	LS		5,000
	Subtotal Section 1.0				$902,655
2.0	PUMP STATION				
2.1	Sheet Piling	157	TN	425.00	66,725
2.2	Excavation	6,650	CY	10.00	66,500
2.3	Compacted Granular Fill	6,000	CY	13.00	78,000
	CONCRETE				
2.4	Slab on Grade	47	CY	250.00	11,750
2.5	Walls	275	CY	350.00	96,250
2.6	Suspended Slabs	73	CY	450.00	32,850
3.0	METAL	1	LS		15,000
4.0	DOORS				
4.1	Overhead	1	EA	1,200.00	1,200
4.2	Hollow Core Metal	1	EA	400.00	400
5.0	EQUIPMENT				
5.1	Vertical Turbine Pumps	4	EA	129,000	516,000
5.2	Sluice Gates	3	EA	81,000	243,000
5.3	Johnson Screens	1	LS	252,000	252,000
6.0	MECHANICAL				
6.1	Steel Pipe	15,000	LB	3.00	45,000
6.2	Misc. Pipe	1,000	LB	3.00	3,000
7.0	CONVEYING SYSTEM				
7.1	10 Ton Travelling Bridge and Crane	1	LS		40,000
8.0	PIPE SUPPORTS	1	LS		2,000
9.0	AIR COMPRESSOR	1	LS		250,000
10.0	ROOFING	990	SF	3.00	2,970
11.0	VALVES				
11.1	24" Butterfly	4	EA	3,100	12,400
11.2	24" Pneumatic Ball	4	EA	20,000	80,000
12.0	FLOWMETERING	1	LS		50,000
	Subtotal Section 2-12				1,865,045
13.0	HEATING & VENTILATION	5%	PS		93,300
14.0	ELECTRICAL, INSTRUMENTATION AND CONTROLS	25%	PS		466,300
	Total Construction Costs				$ 3,327,300

TABLE B-16
OPINION OF PROBABLE COST
26 MGD
CHANNEL INTAKE/PUMP STATION
HARPER'S HILL

	ITEM	QUANTITY	UNIT	UNIT COST $	TOTAL COST $
1.0	SITE WORK				
1.1	Site Prep, Move-in, etc.	1	LS		25,000
1.2	Access Road	600	LF	28.00	16,800
1.3	Channel Excavation	42,400	CY	2.00	84,800
1.4	Finish Grading	1	LS		5,000
	Subtotal Section 1.0				$131,600
2.0	PUMP STATION				
2.1	Sheet Piling	180	TN	425.00	76,500
2.2	Excavation	9,200	CY	10.00	92,000
2.3	Compacted Granular Fill	7,700	CY	13.00	100,100
	CONCRETE				
2.4	Slab on Grade	96	CY	250.00	24,000
2.5	Walls	441	CY	350.00	154,350
2.6	Suspended Slabs	156	CY	450.00	70,200
3.0	METAL	1	LS		15,000
4.0	DOORS				
4.1	Overhead	1	EA	1,200.00	1,200
4.2	Hollow Core Metal	1	EA	400.00	400
5.0	EQUIPMENT				
5.1	Vertical Turbine Pumps	4	EA	129,000	516,000
5.2	Travelling Water Screens	3	EA	109,000	327,000
5.3	Sluice Gates	3	EA	81,000	243,000
6.0	MECHANICAL				
6.1	Steel Pipe	15,000	LB	3.00	45,000
6.2	Misc. Pipe	1,000	LB	3.00	3,000
7.0	CONVEYING SYSTEM				
7.1	10 Ton Travelling Bridge and Crane	1	LS		40,000
8.0	PIPE SUPPORTS	1	LS		2,000
9.0	AIR COMPRESSOR	1	LS		250,000
10.0	ROOFING	2,300	SF	3.00	6,900
11.0	VALVES				
11.1	24" Butterfly	4	EA	3,100	12,400
11.2	24" Pneumatic Ball	4	EA	20,000	80,000
12.0	FLOWMETERING	1	LS		50,000
	Subtotal Section 2-12				2,109,050
13.0	HEATING & VENTILATION	5%	PS		105,450
14.0	ELECTRICAL, INSTRUMENTATION AND CONTROLS	25%	PS		527,300
	Total Construction Costs				$ 2,873,400

TABLE B-17
OPINION OF PROBABLE COST
26 MGD
SUBMERGED INTAKE/PUMP STATION
HARPER'S HILL

	ITEM	QUANTITY	UNIT	UNIT COST $	TOTAL COST $
1.0	SITE WORK				
1.1	Site Prep, Move-in, etc.	1	LS		25,000
1.2	Access Road	600	LF	28.00	16,800
1.3	Excavation For Suction Pipe	27,800	CY	2.00	55,600
1.4	66" Suction Piping	960	LF	149.00	143,040
1.5	66" 90 Degrees Elbow	3	EA	9,085.00	27,255
1.6	Compacted Backfill	18,200	CY	9.00	163,800
1.7	Finish Grading	1	LS		5,000
	Subtotal Section 1.0				$513,245
2.0	PUMP STATION				
2.1	Sheet Piling	157	TN	425.00	66,725
2.2	Excavation	6,650	CY	10.00	66,500
2.3	Compacted Granular Fill	6,000	CY	13.00	78,000
	CONCRETE				
2.4	Slab on Grade	47	CY	250.00	11,750
2.5	Walls	275	CY	350.00	96,250
2.6	Suspended Slabs	73	CY	450.00	32,850
3.0	METAL	1	LS		15,000
4.0	DOORS				
4.1	Overhead	1	EA	1,200.00	1,200
4.2	Hollow Core Metal	1	EA	400.00	400
5.0	EQUIPMENT				
5.1	Vertical Turbine Pumps	4	EA	129,000	516,000
5.2	Sluice Gates	3	EA	81,000	243,000
5.3	Johnson Screens	1	LS	252,000	252,000
6.0	MECHANICAL				
6.1	Steel Pipe	15,000	LB	3.00	45,000
6.2	Misc. Pipe	1,000	LB	3.00	3,000
7.0	CONVEYING SYSTEM				
7.1	10 Ton Travelling Bridge and Crane	1	LS		40,000
8.0	PIPE SUPPORTS	1	LS		2,000
9.0	AIR COMPRESSOR	1	LS		250,000
10.0	ROOFING	990	SF	3.00	2,970
11.0	VALVES				
11.1	24" Butterfly	4	EA	3,100	12,400
11.2	24" Pneumatic Ball	4	EA	20,000	80,000
12.0	FLOWMETERING	1	LS		50,000
	Subtotal Section 2-12				1,865,045
13.0	HEATING & VENTILATION	5%	PS		93,300
14.0	ELECTRICAL, INSTRUMENTATION AND CONTROLS	25%	PS		466,300
	Total Construction Costs				$ 2,937,890

TABLE B-18
OPINION OF PROBABLE COST
26 MGD
CHANNEL INTAKE/PUMP STATION
EAST BANK

	ITEM	QUANTITY	UNIT	UNIT COST $	TOTAL COST $
1.0	SITE WORK				
1.1	Site Prep, Move-in, etc.	1	LS		25,000
1.2	Access Road	200	LF	28.00	5,600
1.3	Channel Excavation	53,300	CY	2.00	106,600
1.4	Finish Grading	1	LS		5,000
	Subtotal Section 1.0				$142,200
2.0	PUMP STATION				
2.1	Sheet Piling	180	TN	425.00	76,500
2.2	Excavation	9,200	CY	10.00	92,000
2.3	Compacted Granular Fill	7,700	CY	13.00	100,100
CONCRETE					
2.4	Slab on Grade	96	CY	250.00	24,000
2.5	Walls	466	CY	350.00	163,100
2.6	Suspended Slabs	156	CY	450.00	70,200
3.0	METAL	1	LS		15,000
4.0	DOORS				
4.1	Overhead	1	EA	1,200.00	1,200
4.2	Hollow Core Metal	1	EA	400.00	400
5.0	EQUIPMENT				
5.1	Vertical Turbine Pumps	4	EA	129,000	516,000
5.2	Travelling Water Screens	3	EA	109,000	327,000
5.3	Sluice Gates	3	EA	81,000	243,000
6.0	MECHANICAL				
6.1	Steel Pipe	15,000	LB	3.00	45,000
6.2	Misc. Pipe	1,000	LB	3.00	3,000
7.0	CONVEYING SYSTEM				
7.1	10 Ton Travelling Bridge and Crane	1	LS		40,000
8.0	PIPE SUPPORTS	1	LS		2,000
9.0	AIR COMPRESSOR	1	LS		250,000
10.0	ROOFING	2,300	SF	3.00	6,900
11.0	VALVES				
11.1	24" Butterfly	4	EA	3,100	12,400
11.2	24" Pneumatic Ball	4	EA	20,000	80,000
12.0	FLOWMETERING	1	LS		50,000
	Subtotal Section 2-12				2,117,800
13.0	HEATING & VENTILATION	5%	PS		105,900
14.0	ELECTRICAL, INSTRUMENTATION AND CONTROLS	25%	PS		529,500
	Total Construction Costs				$ 2,895,400

TABLE B-19
OPINION OF PROBABLE COST
26 MGD
SUBMERGED INTAKE/PUMP STATION
EAST BANK

	ITEM	QUANTITY	UNIT	UNIT COST $	TOTAL COST $
1.0	SITE WORK				
1.1	Site Prep, Move-in, etc.	1	LS		25,000
1.2	Access Road	200	LF	28.00	5,600
1.3	Excavation For Suction Pipe	30,700	CY	2.00	61,400
1.4	66" Suction Piping	1,800	LF	149.00	268,200
1.5	66" 90 Degrees Elbow	3	EA	9,085.00	27,255
1.6	Compacted Backfill	24,400	CY	9.00	219,600
1.7	Finish Grading	1	LS		5,000
	Subtotal Section 1.0				$612,055
2.0	PUMP STATION				
2.1	Sheet Piling	157	TN	425.00	66,725
2.2	Excavation	6,650	CY	10.00	66,500
2.3	Compacted Granular Fill	6,000	CY	13.00	78,000
	CONCRETE				
2.4	Slab on Grade	47	CY	250.00	11,750
2.5	Walls	275	CY	350.00	96,250
2.6	Suspended Slabs	73	CY	450.00	32,850
3.0	METAL	1	LS		15,000
4.0	DOORS				
4.1	Overhead	1	EA	1,200.00	1,200
4.2	Hollow Core Metal	1	EA	400.00	400
5.0	EQUIPMENT				
5.1	Vertical Turbine Pumps	4	EA	129,000	516,000
5.2	Sluice Gates	3	EA	8,100	243,000
5.3	Johnson Screens	1	LS		252,000
6.0	MECHANICAL				
6.1	Steel Pipe	15,000	LB	3.00	45,000
6.2	Misc. Pipe	1,000	LB	3.00	3,000
7.0	CONVEYING SYSTEM				
7.1	10 Ton Travelling Bridge and Crane	1	LS		40,000
8.0	PIPE SUPPORTS	1	LS		2,000
9.0	AIR COMPRESSOR	1	LS		250,000
10.0	ROOFING	990	SF	3.00	2,970
11.0	VALVES				
11.1	24" Butterfly	4	EA	3,100	12,400
11.2	24" Pneumatic Ball	4	EA	20,000	80,000
12.0	FLOWMETERING	1	LS		50,000
	Subtotal Section 2-12				1,865,045
13.0	HEATING & VENTILATION	5%	PS		93,300
14.0	ELECTRICAL, INSTRUMENTATION AND CONTROLS	25%	PS		466,300
	Total Construction Costs				$ 3,036,700

TABLE B-20
OPINION OF PROBABLE COST
26 MGD
ALTERNATIVE ROUTE 1
FINLEY BRANCH
RAW WATER PIPELINE TO WTP

	ITEM	QUANTITY	UNIT	UNIT COST $	TOTAL COST $
1.0	36" Raw Water Line	65,200	LF	100.00	6,520,000
2.0	36" Elbows	18	EA	2,000	36,000
3.0	Air Relief Manholes	5	EA	5,000	25,000
4.0	TUNNELLED CROSSINGS				
4.1	@ Hiway 71 36"	150	LF	730.00	109,500
4.2	@ FM 2285 36"	80	LF	730.00	58,400
4.3	@ St. Louis & Southwestern RR	100	LF	730.00	73,000
5.0	Asphalt Pavement Remove/Replace	450	LF	15.00	6,750
6.0	Misc. Concrete	1,100	CY	175.00	192,500
7.0	Stream Crossings	500	LF	64.00	32,000
8.0	Easement Aquisition	11.29	AC	800.00	9,032
9.0	Tree Clearing & Grubbing	7.5	AC	2,500.00	18,750
10.0	ACCESS ROAD (20' WIDE)				
10.1	Hydrated Lime Ty 4	330	TN	90.00	29,700
10.2	6" Lime Stab. Base	26,670	SY	1.25	33,338
10.3	3 1/2" HMAC Base Course	5,600	TN	40.00	224,000
10.4	1 1/2" HMAC Surface Course	2,401	TN	40.00	96,040
11.0	12 1/2 Ga. Barb Wire Fence Replacement	65.2	MLF	39.00	2,543
	Total Construction Cost				$7,466,553

TABLE B-21
OPINION OF PROBABLE COST
26 MGD
ALTERNATIVE ROUTE 2
HARPER'S HILL
RAW WATER PIPELINE TO WTP

	ITEM	QUANTITY	UNIT	UNIT COST $	TOTAL COST $
1.0	36" Raw Water Line	63,400	LF	110.00	6,974,000
2.0	36" Elbows	31	EA	2,000	62,000
3.0	Air Relief Manholes	3	EA	5,000	15,000
4.0	TUNNELLED CROSSINGS				
4.1	@ Hiway 71 36"	150	LF	730.00	109.500
4.2	@ FM 2285 36"	80	LF	730.00	58,400
4.3	@ St. Louis & Southwestern RR	100	LF	730.00	73,000
5.0	Asphalt Pavement Remove/Replace	500	LF	15.00	7,500
6.0	Misc. Concrete	1,000	CY	175.00	175,000
7.0	Stream Crossings	850	LF	64.00	54,400
8.0	Easement Aquisition	29.02	AC	800.00	23,216
9.0	Tree Clearing & Grubbing	0.62	AC	2,500.00	1,550
10.0	12 1/2 Ga. Barb Wire Fence Replacement	63.4	MLF	39.00	2,473
	Subtotal Construction Cost				$7,556,039
11.0	ROAD IMPROVEMENTS				
11.1	New Road Construction North of Hiway 71		LS		250,000
11.2	Road Repair South of Hiway 71		LS		270,000
	Total Construction w/Road Improvements				$8,076,039

TABLE B-22
OPINION OF PROBABLE COST
26 MGD
ALTERNATIVE ROUTE 3
EAST BANK
RAW WATER PIPELINE TO WTP

	ITEM	QUANTITY	UNIT	UNIT COST $	TOTAL COST $
1.0	36" Raw Water Line	73,000	LF	100.00	7,300,000
2.0	36" Elbows	16	EA	2,000	32,000
3.0	Air Relief Manholes	6	EA	5,000	30,000
4.0	TUNNELLED CROSSINGS				
4.1	@ Hiway 71 36"	150	LF	730.00	109,500
4.2	@ FM 2285 36"	80	LF	730.00	58,400
4.3	@ St. Louis & Southwestern RR	100	LF	730.00	73,000
5.0	Asphalt Pavement Remove/Replace	530	LF	15.00	7,950
6.0	Misc. Concrete	1,100	CY	175.00	192,500
7.0	Stream Crossings	500	LF	64.00	32,000
8.0	12 1/2 Ga. Barb Wire Fence Replacement	73.0	MLF	39.00	2,847
	Total Construction Cost				$7,838,197

TABLE B-23
OPINION OF PROBABLE COST
OZONE FACILITIES

	ITEM	QUANTITY	UNIT	UNIT COST $	TOTAL COST $
1.0	SITE WORK	1	LS		250,000
2.0	OZONE BUILDING AND BASINS	1	LS		600.000
3.0	OZONE GENERATORS	1	LS		210,000
4.0	AIR PREPARATION	1	LS		190,000
5.0	DISSOLUTION/DESTRUCT UNITS	1	LS		375,000
6.0	PIPING AND VALVES	1	LS		220,000
7.0	SYSTEM ENGINEERING AND PERFORMANCE GUARANTEES	1	LS		405,000
8.0	PILOT PLANT TESTING	1	LS		40,000
	Total Cost				$2,290,000

Note: Prices include Engineering, Legal, and Administrative fees and an allowance for contingencies.

Appendix C

Pumping Cost Analyses

SULPHUR SPRINGS, TX
COOPER RES. WATER STUDY PN 14719100
RAW WATER PUMPING COST ANALYSIS
alternative 1
FINLEY BRANCH TO WTP 30-inch pipe 14mgd

K= 5 LENGTH 65200
C= 130 static head= 62
DIA.= 30

YEAR	AAD mgd	PIPE Dia.	PIPE Area ft^2	VEL. ft/s	FITTING LOSS ft.	FRICTION LOSS ft	STATIC HEAD ft	TOTAL HEAD ft	PUMPING COST $/year
1990	5.5	30	4.90	1.73	0.23	23.04	62	85.27	61414
1995	6.9	30	4.90	2.17	0.37	35.05	62	97.42	88025
2000	7.8	30	4.90	2.45	0.47	43.97	62	106.44	108720
2005	8.3	30	4.90	2.61	0.53	49.33	62	111.86	121580
2010	8.5	30	4.90	2.67	0.56	51.55	62	114.11	127014
2015	9.1	30	4.90	2.86	0.64	58.48	62	121.12	144334
2020	9.8	30	4.90	3.08	0.74	67.08	62	129.82	166601
2025	10.4	30	4.90	3.27	0.83	74.87	62	137.7	187533
2030	11.3	30	4.90	3.56	0.98	87.29	62	150.27	222362
2035	12.4	30	4.90	3.90	1.19	103.66	62	166.85	270931
2040	14	30	4.90	4.41	1.51	129.76	62	193.27	354326

PRESENT WORTH ANALYSIS FOR PUMPING COSTS

ANNUAL PUMPING COST $/YEAR	ANNUAL PUMPING COST PW $	ANNUAL GRAD. PUMP COST $/YEAR	ANNUAL GRAD. PUMP COST PW $	TOTAL PRESENT WORTH $
61414	664483	5858	653833	$ 1318316

SULPHUR SPRINGS, TX
COOPER RES. WATER STUDY   PN 14719100
RAW WATER PUMPING COST ANALYSIS
alternative 1A
FINLEY BRANCH TO WTP   30-inch pipe   14mgd
ACROSS CENTURY DAM

K=	6	LENGTH	57000
C=	130	static head=	62
DIA.=	30		

YEAR	AAD mgd	PIPE Dia.	PIPE Area ft^2	VEL. ft/s	FITTING LOSS ft.	FRICTION LOSS ft	STATIC HEAD ft	TOTAL HEAD ft	PUMPING COST $/year
1990	5.5	30	4.90	1.73	0.28	20.14	62	82.42	59362
1995	6.9	30	4.90	2.17	0.44	30.64	62	93.08	84104
2000	7.8	30	4.90	2.45	0.56	38.44	62	101	103164
2005	8.3	30	4.90	2.61	0.64	43.12	62	105.76	114950
2010	8.5	30	4.90	2.67	0.67	45.07	62	107.74	119924
2015	9.1	30	4.90	2.86	0.77	51.13	62	113.9	135730
2020	9.8	30	4.90	3.08	0.89	58.64	62	121.53	155963
2025	10.4	30	4.90	3.27	1	65.45	62	128.45	174936
2030	11.3	30	4.90	3.56	1.18	76.32	62	139.5	206425
2035	12.4	30	4.90	3.90	1.42	90.63	62	154.05	250146
2040	14	30	4.90	4.41	1.81	113.44	62	177.25	324956

PRESENT WORTH ANALYSIS FOR PUMPING COSTS

ANNUAL PUMPING COST $/YEAR	ANNUAL PUMPING COST PW $	ANNUAL GRAD. PUMP COST $/YEAR	ANNUAL GRAD. PUMP COST PW $	TOTAL PRESENT WORTH $
59362	642281	5312	592892	$ 1235173

SULPHUR SPRINGS, TX
COOPER RES. WATER STUDY   PN 14719100
RAW WATER PUMPING COST ANALYSIS
alternative 2
HARPERS HILL TO WTP   30-inch pipe   14mgd

K= 5   LENGTH 63400
C= 130   static head= 62
DIA.= 30

YEAR	AAD mgd	PIPE Dia.	PIPE Area ft^2	VEL. ft/s	FITTING LOSS ft.	FRICTION LOSS ft	STATIC HEAD ft	TOTAL HEAD ft	PUMPING COST $/year
1990	5.5	30	4.90	1.73	0.23	22.4	62	84.63	60953
1995	6.9	30	4.90	2.17	0.37	34.08	62	96.45	87149
2000	7.8	30	4.90	2.45	0.47	42.76	62	105.23	107484
2005	8.3	30	4.90	2.61	0.53	47.97	62	110.5	120102
2010	8.5	30	4.90	2.67	0.56	50.13	62	112.69	125434
2015	9.1	30	4.90	2.86	0.64	56.87	62	119.51	142415
2020	9.8	30	4.90	3.08	0.74	65.22	62	127.96	164214
2025	10.4	30	4.90	3.27	0.83	72.8	62	135.63	184714
2030	11.3	30	4.90	3.56	0.98	84.88	62	147.86	218796
2035	12.4	30	4.90	3.90	1.19	100.8	62	163.99	266287
2040	14	30	4.90	4.41	1.51	126.17	62	189.68	347744

PRESENT WORTH ANALYSIS FOR PUMPING COSTS

ANNUAL PUMPING COST $/YEAR	ANNUAL PUMPING COST PW $	ANNUAL GRAD. PUMP COST $/YEAR	ANNUAL GRAD. PUMP COST PW $	TOTAL PRESENT WORTH $
60953	659495	5736	640217	$ 1299712

SULPHUR SPRINGS, TX
COOPER RES. WATER STUDY PN 14719100
RAW WATER PUMPING COST ANALYSIS
alternative 2A
HARPERS HILL TO WTP 30-inch pipe 14mgd
ACROSS LAKE SS
K= 6 LENGTH 57600
C= 130 static head= 62
DIA.= 30

YEAR	AAD mgd	PIPE Dia.	PIPE Area ft^2	VEL. ft/s	FITTING LOSS ft.	FRICTION LOSS ft	STATIC HEAD ft	TOTAL HEAD ft	PUMPING COST $/year
1990	5.5	30	4.90	1.73	0.28	20.35	62	82.63	59513
1995	6.9	30	4.90	2.17	0.44	30.96	62	93.4	84393
2000	7.8	30	4.90	2.45	0.56	38.85	62	101.41	103582
2005	8.3	30	4.90	2.61	0.64	43.58	62	106.22	115450
2010	8.5	30	4.90	2.67	0.67	45.54	62	108.21	120447
2015	9.1	30	4.90	2.86	0.77	51.66	62	114.43	136362
2020	9.8	30	4.90	3.08	0.89	59.26	62	122.15	156758
2025	10.4	30	4.90	3.27	1	66.14	62	129.14	175875
2030	11.3	30	4.90	3.56	1.18	77.12	62	140.3	207609
2035	12.4	30	4.90	3.90	1.42	91.58	62	155	251689
2040	14	30	4.90	4.41	1.81	114.63	62	178.44	327138

PRESENT WORTH ANALYSIS FOR PUMPING COSTS

ANNUAL PUMPING COST $/YEAR	ANNUAL PUMPING COST PW $	ANNUAL GRAD. PUMP COST $/YEAR	ANNUAL GRAD. PUMP COST PW $	TOTAL PRESENT WORTH $
59513	643915	5353	597468	$ 1241383

SULPHUR SPRINGS, TX
COOPER RES. WATER STUDY PN 14719100
RAW WATER PUMPING COST ANALYSIS
alternative 3
EASTBANK TO WTP 30-inch pipe 14mgd

K= 5 LENGTH 73000
C= 130 static head= 62
DIA.= 30

YEAR	AAD mgd	PIPE Dia.	PIPE Area ft^2	VEL. ft/s	FITTING LOSS ft.	FRICTION LOSS ft	STATIC HEAD ft	TOTAL HEAD ft	PUMPING COST $/year
1990	5.5	30	4.90	1.73	0.23	25.8	62	88.03	63402
1995	6.9	30	4.90	2.17	0.37	39.24	62	101.61	91811
2000	7.8	30	4.90	2.45	0.47	49.23	62	111.7	114093
2005	8.3	30	4.90	2.61	0.53	55.23	62	117.76	127993
2010	8.5	30	4.90	2.67	0.56	57.72	62	120.28	133882
2015	9.1	30	4.90	2.86	0.64	65.48	62	128.12	152675
2020	9.8	30	4.90	3.08	0.74	75.1	62	137.84	176894
2025	10.4	30	4.90	3.27	0.83	83.83	62	146.66	199736
2030	11.3	30	4.90	3.56	0.98	97.74	62	160.72	237826
2035	12.4	30	4.90	3.90	1.19	116.06	62	179.25	291066
2040	14	30	4.90	4.41	1.51	145.28	62	208.79	382779

PRESENT WORTH ANALYSIS FOR PUMPING COSTS

ANNUAL PUMPING COST $/YEAR	ANNUAL PUMPING COST PW $	ANNUAL GRAD. PUMP COST $/YEAR	ANNUAL GRAD. PUMP COST PW $	TOTAL PRESENT WORTH $
63402	685993	6388	712989	$ 1398982

SULPHUR SPRINGS, TX
COOPER RES. WATER STUDY   PN 14719100
RAW WATER PUMPING COST ANALYSIS
alternative 4
FINLEY BRANCH TO LAKE SS 30-inch pipe   14mgd

K= 5   LENGTH 47800
C= 130   static head= 62
DIA.= 30

YEAR	AAD mgd	PIPE Dia.	PIPE Area ft^2	VEL. ft/s	FITTING LOSS ft.	FRICTION LOSS ft	STATIC HEAD ft	TOTAL HEAD ft	PUMPING COST $/year
1990	5.5	30	4.90	1.73	0.23	16.89	62	79.12	56985
1995	6.9	30	4.90	2.17	0.37	25.69	62	88.06	79568
2000	7.8	30	4.90	2.45	0.47	32.24	62	94.71	96739
2005	8.3	30	4.90	2.61	0.53	36.16	62	98.69	107266
2010	8.5	30	4.90	2.67	0.56	37.79	62	100.35	111698
2015	9.1	30	4.90	2.86	0.64	42.87	62	105.51	125732
2020	9.8	30	4.90	3.08	0.74	49.17	62	111.91	143617
2025	10.4	30	4.90	3.27	0.83	54.89	62	117.72	160322
2030	11.3	30	4.90	3.56	0.98	64	62	126.98	187899
2035	12.4	30	4.90	3.90	1.19	76	62	139.19	226017
2040	14	30	4.90	4.41	1.51	95.13	62	158.64	290838

PRESENT WORTH ANALYSIS FOR PUMPING COSTS

ANNUAL PUMPING COST $/YEAR	ANNUAL PUMPING COST PW $	ANNUAL GRAD. PUMP COST $/YEAR	ANNUAL GRAD. PUMP COST PW $	TOTAL PRESENT WORTH $
56985	616562	4677	522018	$ 1138580

SULPHUR SPRINGS, TX
COOPER RES. WATER STUDY   PN 14719100
RAW WATER PUMPING COST ANALYSIS
alternative 5
HARPERS HILL TO LAKE SS   30-inch pipe   14mgd

K= 5   LENGTH 46000
C= 130   static head= 62
DIA.= 30

YEAR	AAD mgd	PIPE Dia.	PIPE Area ft^2	VEL. ft/s	FITTING LOSS ft.	FRICTION LOSS ft	STATIC HEAD ft	TOTAL HEAD ft	PUMPING COST $/year
1990	5.5	30	4.90	1.73	0.23	16.25	62	78.48	56524
1995	6.9	30	4.90	2.17	0.37	24.73	62	87.1	78701
2000	7.8	30	4.90	2.45	0.47	31.02	62	93.49	95493
2005	8.3	30	4.90	2.61	0.53	34.8	62	97.33	105788
2010	8.5	30	4.90	2.67	0.56	36.37	62	98.93	110118
2015	9.1	30	4.90	2.86	0.64	41.26	62	103.9	123813
2020	9.8	30	4.90	3.08	0.74	47.32	62	110.06	141243
2025	10.4	30	4.90	3.27	0.83	52.82	62	115.65	157503
2030	11.3	30	4.90	3.56	0.98	61.59	62	124.57	184333
2035	12.4	30	4.90	3.90	1.19	73.14	62	136.33	221373
2040	14	30	4.90	4.41	1.51	91.55	62	155.06	284275

PRESENT WORTH ANALYSIS FOR PUMPING COSTS

ANNUAL PUMPING COST $/YEAR	ANNUAL PUMPING COST PW $	ANNUAL GRAD. PUMP COST $/YEAR	ANNUAL GRAD. PUMP COST PW $	TOTAL PRESENT WORTH $
56524	611574	4555	508401	$ 1119975

SULPHUR SPRINGS, TX
COOPER RES. WATER STUDY   PN 14719100
RAW WATER PUMPING COST ANALYSIS
alternative 7
HARPERS HILL TO WTP   36-inch pipe   26mgd

K= 5   LENGTH 63400
C= 130   static head= 62
DIA.= 36

YEAR	MDD mgd	PIPE Dia.	PIPE Area ft^2	VEL. ft/s	FITTING LOSS ft.	FRICTION LOSS ft	STATIC HEAD ft	TOTAL HEAD ft	PUMPING COST $/year
1990	10	36	7.06	2.18	0.37	27.86	62	90.23	118158
1995	12.5	36	7.06	2.73	0.58	42.1	62	104.68	171350
2000	14.3	36	7.06	3.13	0.76	54	62	116.76	218646
2005	15.1	36	7.06	3.30	0.85	59.72	62	122.57	242366
2010	15.8	36	7.06	3.45	0.93	64.94	62	127.87	264567
2015	17	36	7.06	3.72	1.07	74.36	62	137.43	305943
2020	18.3	36	7.06	4.00	1.25	85.22	62	148.47	355795
2025	19.8	36	7.06	4.33	1.46	98.59	62	162.05	420170
2030	21	36	7.06	4.59	1.64	109.93	62	173.57	477314
2035	23.7	36	7.06	5.18	2.09	137.5	62	201.59	625645
2040	26	36	7.06	5.69	2.51	163.2	62	227.71	775293

PRESENT WORTH ANALYSIS FOR PUMPING COSTS

ANNUAL PUMPING COST $/YEAR	ANNUAL PUMPING COST PW $	ANNUAL GRAD. PUMP COST $/YEAR	ANNUAL GRAD. PUMP COST PW $	TOTAL PRESENT WORTH $
118158	1278438	13143	1466940	$ 2745378

SULPHUR SPRINGS, TX
COOPER RES. WATER STUDY PN 14719100

RAW WATER PUMPING COST ANALYSIS
alternative 8
EASTBANK TO WTP 36-inch pipe 26mgd

K= 5 LENGTH 73000
C= 130 static head= 62
DIA.= 36

YEAR	MDD mgd	PIPE Dia.	PIPE Area ft^2	VEL. ft/s	FITTING LOSS ft.	FRICTION LOSS ft	STATIC HEAD ft	TOTAL HEAD ft	PUMPING COST $/year
1990	10	36	7.06	2.18	0.37	32.08	62	94.45	123684
1995	12.5	36	7.06	2.73	0.58	48.48	62	111.06	181793
2000	14.3	36	7.06	3.13	0.76	62.18	62	124.94	233964
2005	15.1	36	7.06	3.30	0.85	68.76	62	131.61	260241
2010	15.8	36	7.06	3.45	0.93	74.78	62	137.71	284927
2015	17	36	7.06	3.72	1.07	85.62	62	148.69	331010
2020	18.3	36	7.06	4.00	1.25	98.13	62	161.38	386733
2025	19.8	36	7.06	4.33	1.46	113.52	62	176.98	458881
2030	21	36	7.06	4.59	1.64	126.58	62	190.22	523102
2035	23.7	36	7.06	5.18	2.09	158.32	62	222.41	690261
2040	26	36	7.06	5.69	2.51	187.91	62	252.42	859424

PRESENT WORTH ANALYSIS FOR PUMPING COSTS

ANNUAL PUMPING COST $/YEAR	ANNUAL PUMPING COST PW $	ANNUAL GRAD. PUMP COST $/YEAR	ANNUAL GRAD. PUMP COST PW $	TOTAL PRESENT WORTH $
123684	1338228	14715	1642396	$ 2980624

SULPHUR SPRINGS, TX
COOPER RES. WATER STUDY PN 14719100
RAW WATER PIPELINE ANALYSIS
Existing pump station & RWL @ Lake Sulphur Springs
14" & 18" lead up to 20" (all DIP)

C= 140 static head= 12

YEAR	AAD mgd	14" VEL. ft/s	18" VEL. ft/s	20" VEL. ft/s	FITTING LOSS ft.	FRICTION LOSS ft	STATIC HEAD ft	TOTAL HEAD ft	PUMPING COST $/year
1990	5.5	3.48	2.70	3.89	0.6	21.84	12	34.44	24805
1995	6.9	4.37	3.39	4.89	0.94	33.27	12	46.21	41754
2000	7.8	4.94	3.84	5.53	1.2	41.73	12	54.93	56107
2005	8.3	5.26	4.08	5.88	1.36	46.86	12	60.22	65453
2010	8.5	5.38	4.18	6.02	1.43	48.89	12	62.32	69368
2015	9.1	5.76	4.48	6.45	1.64	55.47	12	69.11	82356
2020	9.8	6.21	4.82	6.94	1.9	63.71	12	77.61	99599
2025	10.4	6.59	5.12	7.37	2.14	71.11	12	85.25	116102
2030	11.3	7.16	5.56	8.01	2.53	82.9	12	97.43	144172
2035	12.4	7.85	6.10	8.79	3.04	98.34	12	113.38	184106
2040	14	8.86	6.89	9.92	3.88	123.04	12	138.92	254685

PRESENT WORTH ANALYSIS FOR PUMPING COSTS

ANNUAL PUMPING COST $/YEAR	ANNUAL PUMP COST PRESENT WORTH $	ANNUAL GRAD. PUMPING COST $/YEAR	ANNUAL GRAD. PUMPCOST PRESENT WORTH $	TOTAL PRESENT WORTH $
24805	268383	4598	513200	781583

City of Sulphur Springs (Black + Veatch)
contract - Work Description

1. It shall be the responsibility of the City to establish formal and direct liaison with the entities listed in Exhibit 2 of Attachment A (the grant application), official representatives of Hopkins County, the Sulphur River Municipal Water District, and community leaders in the proposed service area for the purpose of coordinating the work of the planning project and to acquire available data pertinent to the planning effort. Planning shall be coordinated with all existing water supply studies and activities for the purpose of providing information for the proposed project. As the organizing entity, the City has the responsibility to solicit comments from the general public as to the content of the planning project.

2. The project will produce a feasibility-level plan for a regional water system for Hopkins County. The project will consist of the following tasks:

   Task I. Conduct Initial Scoping and Coordination

   A. Hold initial meetings to discuss and review project with City of Sulphur Springs staff and other participants.

   B. Present final planning outline to City Council.

   C. Conduct meetings with appropriate state and federal officials.

   Task II. Review Existing Information

   A. Compile available information on population growth, past and future water use, surface water source availability and quality for Lake Sulphur Springs and Copper Reservoir, existing treatment plant facilities and processes, and distribution facilities.

   B. Review existing information to determine deficiencies.

Task III. Collect or Develop Supplemental Data

A. Collect data necessary to supplement existing information on surface water sources; intake locations; raw water pipeline routings; treatment plant facilities, processes, and practices; and wholesale customer distribution facilities.

B. Develop supplemental data on population growth and projected water requirements if needed.

Task IV. Prepare a Water Conservation Plan

A. Prepare a draft water conservation plan according to the Board's guidelines that emphasizes the efficient use of water resources. Submit plan for Board review, and modify plan to reflect Board comments.

B. Incorporate water savings identified in water conservation plan in demand projections compiled in Task II and developed in Task III and in facility alternatives developed in Task IX.

Task V. Evaluate Existing Facilities

A. Evaluate existing water supply facilities for potential treatment impacts resulting from passage of the Safe Drinking Water Act Amendments of 1986, future treatment of larger hydraulic flows, Texas Department of Health standards and requirements, and operational reliability and economy.

B. Present findings to City and obtain and review comments.

Task VI. Develop Raw Water Pipeline Routings

A. Develop alternative routings.

B. Visit routes as necessary.

Task VII. Develop Treatment Process Alternatives

Task VIII. Evaluate Alternative Routings and Treatment Processes

A. Develop alternatives.

B. Present alternatives to City Council for review and comment.

Task IX. Develop Facility Alternatives and Staging Schedules

A. Prepare alternatives for raw water storage and delivery system, upgrading existing facilities, new facilities required for larger hydraulic flows, a combination of new facilities and upgrading existing facilities, and new facilities required for users.

Task X. Develop Preliminary Costs for Facility Alternatives

Task XI. Recommend Preliminary Approach

A. Evaluate facility alternatives.

B. Provide preliminary recommendations to City.

C. Receive City's review comments.

Task XII. Develop Recommended Plan

A. Prepare plan of recommended improvements.

B. Develop implementation schedule.

Task XIII. Prepare Final Report

A. Prepare draft report.

B. Submit draft report to the City and the Board.

Prepare final report based on City and Board review comments.

II. PROJECT SCHEDULE AND REPORTS

The City has until April 20, 1988, to execute this Contract and to provide written evidence acceptable to the Executive Administrator that the City has available its 50-percent ($35,000) matching grant share. The Board's approval

City of Sulphur Springs
Utilities Department

Information about Water, Wastewater, Distribution & Collection,Parks, and Streets, Water (ext. 268 & 269) E-mail Assistant Dir. of Utilities, Robert LeeWastewater (ext. 265 & 266) Streets (ext. 261 & 264)

Water Sources

The City of Sulphur Springs has two sources of raw water; Lake Sulphur Springs just one and one half miles northwest ofthe city; and Cooper Lake twelve miles to the north. Water is pumped from pump stations located at each surface impoundment to the water treatment plant. The primary source of raw water is through the newly constructed twelve milewater line from Cooper Lake.

Water Treatment

Water Treatment is accomplished at the city's 7 MGD plant locatedon Highway 19 on the west side of town. The Sulphur Springs Water Treatment Plant was built in 1966 and expanded in 1980 and 1985. It is a conventional clarification plant consisting of two 1.5 MGD modules, one 1.0 MGD module, and one 3.0 MGD module. Alum is used as a flocculationaid before the water is clarified and filtered. Taste and Odor Controland Disinfection is accomplished through the use of activated carbon(summermonths if needed), Chlorine Dioxide, Chorine, and Chloramines. Finishedwater is stored in two underground clearwell with a total capacity of 3 MG.

Water and Sewer Distribution

Water Distribution is accomplished through 127 miles of water mainswhich distribute over a billion gallons of water per year. Fire Flow ismaintained by a network of looped 12 inch mains and three elevated towersholding 1,500,000 gallons. Fire Hydrants and water meters are also maintainedby the distribution department.

Rural Water Supply Corporations

Rural water systems in Hopkins County purchase treated water from thecity. Demand from the rural water systems comprises 1/3 of the total watertreated. Rural WSC's supplied by the city include:

Name	Direction From City
North Hopkins WSC	North and Northeast
Gafford Chapel WSC	Northwest
Brashear WSC	West
Pleasant Hill WSC	South
Shady Grove WSC	Southwest
Martin Springs WSC	Southeast
Brinker WSC	East

NOAA Weather Station

The water plant also serves as an official NOAA weather station collectingtemperature, rainfall, evaporation, and wind data.

Water Demand History (million gallons-MG)

Year	Raw Water	Treated Water	Rural Water Sold
1980	1334	1108	n/a
1981	1298	1107	n/a
1982	1250	1152	n/a
1983	1478	1220	n/a
1984	1440	1213	n/a
1985	1425	1200	297
1986	1281	1054	338
1987	1151	1057	321
1988	1261	1156	294
1989	1204	1083	280
1990	1251	1208	326
1991	1250	1178	283
1992	1263	1181	206
1993	1326	1312	318
1994	1359	1226	299
1995	1568	1279	323
1996	1447	1223	311

Wastewater

Sewer Collection

Wastewater is collected for treatment through 112 miles of sewermains and six lift stations. Our sewer collection system is maintained by the Water and Sewer Distribution Department.

Wastewater Treatment

Treatment is accomplished at one 5.4 MGD wastewater facility with athree MG equalization basin located on the east side of town. The facilityis a complete-mix, mechanically aerated, activated sludge plant with primaryclarifiers and tertiary filters. Sludge is aerobically digested and furtherstabilized in sludge storage basins before being dewatered in the beltpress. Treated wastewater is disinfected with chlorine, then dechlorinatedbefore discharging to Rock Creek. The city presently meets biomonitoringrequirements and some of the strictest discharge limits in Texas.

Biosolids are beneficially reused at registered land application sitesin Hopkins County.

Wastewater Flows History (Avg. MGD)

Cooper Reservoir
Water Supply Study
Contract #8-483-611

The following maps are not attached to this report. They are located in the official file and may be copied upon request.

Map 1 – Study Area Hopkins County – Project No. 14719.100, Figure 1

Map 2 – Water distribution System-Figure 2

Map 3-Water Distribution System, Project No. 14719.100- Figure 3

Map 4-Raw Water Intake/Pump Station and Pipeline Alternatives-Figure 4, Project No. 14719.100

Please contact Research and Planning Fund Grants Management Division at (512) 463-7926 for copies.