Wiley & Wilson
ARCHITECTS ENGINEERS PLANNERS

COMPREHENSIVE WATER AND SEWER STUDY

FOR

TOWN OF APPOMATTOX
AND
APPOMATTOX COUNTY

Comm. No. 90126.00

January 1991

Wiley & Wilson
ARCHITECTS ENGINEERS PLANNERS

January 7, 1991

Mr. Ronald C. Spiggle, Mayor
Town of Appomattox
P. O. Box 705
Appomattox, VA 24522

Mr. Dennis C. Gragg
County Administrator
P. O. Box 863
Appomattox, VA 24522

Re: Comprehensive Water & Sewer Study
W&W Comm. No. 90126.00

Gentlemen:

We are pleased to present herein our Comprehensive Water and Sewer Study. The purpose of this study was to evaluate the existing systems and recommend improvements to meet both present and future needs through the year 2010.

To support the future water needs, a geotechnical study was performed to locate future potential well sites. Also, other potential water sources were evaluated and the existing water system was computer modeled.

A plan was developed for improvements and expansion to the water and sewer systems accompanied by priorities and time frames with rough construction cost estimates.

We trust this information is sufficient to meet your needs and aid in your future planning. If you have any questions, please do not hesitate to contact us. We are available to discuss this report whenever it would suit your schedule. We appreciate the cooperation and support of you and your staff in preparing this report.

Very truly yours,

WILEY & WILSON

Kenneth N. Frye, P.E.

Walter E. Hancock, P.E.
Associate

ACKNOWLEDGMENTS

We wish to acknowledge and express our appreciation for the valuable assistance of the following persons in furnishing maps, drawings, records and other background data necessary for the completion of this Comprehensive Water and Sewer Study for the Town of Appomattox and Appomattox County.

Ronald C. Spiggle, Mayor, Town of Appomattox
Dennis C. Gragg, County Administrator, Appomattox County

David Garrett, Jr., Superintendent of Public Works, Town of Appomattox
Harold Nash, Retired Superintendent of Public Works, Town of Appomattox
Kent Hodges, Chief Wastewater Treatment Operator, Town of Appomattox

Bobbie Mullins, Clerk and Administrative Assistant, Town of Appomattox
Judy Volturo, Secretary, Town of Appomattox

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Appomattox, Virginia

1.0 INTRODUCTION

1.1 Purpose and Scope of Study

Hatcher-Sayre, Inc. was retained by Wiley and Wilson to perform a well location study for a specified area adjacent to and including the town limits of Appomattox, Virginia. The Town of Appomattox is seeking classification as an industrial development community which requires a minimum of 40,000 gpd of water in addition to the existing water supply system. Three wells were specified with a daily discharge of 40,000 gpd each. The total desired yield for any new wells is 80,000 gpd.

Services for this study were provided in accordance with our proposal dated September 18, 1990 and consisted of the following:

1. Reviewing published literature addressing geologic and groundwater conditions in the study area.
2. Contacting area well drillers concerning their knowledge of the local geologic/hydrogeologic conditions and typical well yells. Information for existing area municipal wells was also reviewed.
3. Performance of a fracture trace analysis using aerial photographs and topographic maps of the area. Three (3) well areas consisting of a total of seven (7) well sites were selected based on the fracture trace analysis, literature search, recharge areas and proximity to the existing water distribution system.
4. Performance of a geologic field reconnaissance of the proposed well sites.
5. Preparation of a detailed report presenting the findings

of the study and recommended locations ranked in the order of preference with respect to potential of encountering desirable yields.

This study did not include consultation or attendance of meetings, field staking of locations and services other than those specifically listed in our scope of services.

1.2 Description of Study Area

The approximate study area encompasses the Town of Appomattox, Virginia and surrounding areas in Appomattox County, Virginia as shown on Figure 1. The study area consists of approximately 7.5 square miles, is located in the Piedmont Physiographic Province and is characterized by rolling topography. Areal topography appears to be multi-directional with surface drainage to the north, northwest, west, southwest, south, southeast east, and northeast. Five municipal water supply wells exist in and around the corporate limits of the Town of Appomattox. These wells serve the current needs of the town. Well depths range from 100 to 305 feet and yields range from 45 to 100 gpm.

2.0 LOCAL HYDROGEOLOGIC SETTING

2.1 General Geology

There are several sources of published geologic information for the general area, but no site specific data was available. These sources include the Geologic Map of Virginia, 1963 (a portion of which is included as Figure 2), Geology of the Buckingham Quadrangle, Virginia by Ernest H. Ern, 1968 (15 minute quadrangle located to the north) and the Geology and Mineral Resources of the Lynchburg Quadrangle Virginia by William Randell Brown, 1968 (15 minute quadrangle located to the west). Based upon our review of this literature, the site is in a geologically stable area which is underlain by metamorphosed sedimentary rocks and hornblende gabbro, gneiss and talc of late Precambrian to early Paleozoic geologic age. The contact between these units is gradational. Specific rock descriptions are presented in the legend on Figure 2. The Geologic Map of Virginia shows the presence of a fault trending to the south and southwest of Town of Appomattox (Figure 2).

Local geologic conditions were determined by utilizing well log information from the Virginia Water Control Board, the Town of Appomattox, and Appomattox Well Drilling Company. Two cross sections were generated using the information obtained from the logs and are shown on Figures 3 and 4. Depth to bedrock typically ranges from 60 to 100 feet with the average elevation at 740 feet AMSL. Bedrock elevation appears to decline east of the Town of Appomattox along U.S. Route 460 (elevations 740 - 690). All wells penetrated overburden, described most commonly as a sand and clay and bedrock, described mostly commonly as a metasedimentary rock.

TOWN OF APPOMATTOX
AND
APPOMATTOX COUNTY

COMPREHENSIVE WATER AND SEWER MASTER PLAN

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CHAPTER 1 - SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

SUMMARY

Background

The Town of Appomattox and Appomattox County commissioned Wiley & Wilson to prepare a comprehensive water and sewer master plan on June 12, 1990. A 20-year planning period has been defined for this Comprehensive Study, through the year 2010. The study area is defined and presented in Chapter 2, Figure 2-1. The study area includes all of the Incorporated Town of Appomattox plus approximately a 1/2-mile radius outside of the corporate limits and a strip approximately 2,200 feet in width along the Route 460 corridor from the Town of Appomattox to Spout Spring, Virginia. New industrial sites that are to be served in the Comprehensive Study may be included in the Region 2000 Program and/or the Certified Business Community Certification Program.

Current Water Situation

The existing serviceable population for the study area was estimated to be 2,353 persons for 1990, consisting of 1,703 persons in the Town of Appomattox and 650 persons in the study portion of Appomattox County. Residents within the Town of Appomattox are generally served by a public water system and residents of Appomattox County are generally served by single-family wells. The existing Appomattox water distribution system consists of five operating wells, three storage tanks, a booster pumping station, and water distribution lines installed as early as 1925. The water distribution system operates on a high and low zone, generally divided by the railroad. Water storage for the high zone is provided by a 1,000,000-gallon ground tank and a 100,000-gallon elevated storage tank. Storage for the low zone is provided by a 30,000-gallon elevated tank.

A comprehensive hydrogeologic analysis was performed under the supervisor of Wiley & Wilson for the study area and environs by Hatcher-Sayer, Inc. The report is included as Appendix A. Existing groundwater supply wells range in capacity from 45 to 100 gallons per minute (gpm) with depths ranging from 100 to 305 feet. Generally, water quality is considered good. The average well production is 186,650 gallons per day (gpd) and the average water consumption billed is 169,931 gpd.

An equivalent residential connection (ERC) method of equating residential users with large commercial or industrial users was employed. The total ERC's for the Town are estimated to be 1,262 which consists of 674 residential ERC's and 588 commercial/industrial ERC's. Using estimated 1990 census data and the number of serviced and potential connections in the corporate limits, an average of 2.5 persons per household was calculated. The estimated per capita water usage for the water system is 53.85 gpcd (or 4,095 gallons per month per connection).

To analyze the existing water distribution system, a computer model was developed using the Kentucky Pipe Program. The model was calibrated by flow and pressure tests of selected fire hydrants throughout the water distribution system. The results of the model indicate that the present water distribution system can generally supply sufficient water quantities and pressures for domestic needs. However, in several critical areas, including the High School, downtown, and Middle School, the present water distribution system falls short of providing adequate fire flows. Also, the low zone lacks adequate flows over an extended period of time.

To improve domestic supply/pressure and to provide adequate fire flows in Town, improvements to the water distribution system are recommended and prioritized in order of importance. These five priorities are shown on Figure 3-2 and should be implemented as soon as practical. Chapter 3 presents a detailed description, benefits, and the estimated construction costs for each of the priority improvements.

Current Sewer Situation

Residents within the Town of Appomattox are generally served by a public sanitary wastewater collection and treatment system. Residents in Appomattox County are generally served by individual on-site septic systems. The Town of Appomattox presently operates two wastewater treatment facilities. The Aerated Lagoon Wastewater Treatment Plant is presently permitted for 54,000 gpd, and the Trickling Filter Wastewater Treatment Plant is presently permitted for 170,000 gpd. Each plant is performing in excess of the effluent discharge limits established by their respective VPDES permits. Available capacity at the Aerated Lagoon and Trickling Filter WWTP is estimated to be 11,700 gpd and 26,000 gpd, respectively.

Infiltration/Inflow (I/I) for the combined flows to both plants was analyzed and compared, as a standard of measure, to the SWCB criteria established in the Virginia Revolving Loan Program. The results of the analysis concluded that I/I was nonexcessive when compared to the criteria established by the SWCB.

Future Water Situation

Population projections for the study area are presented in Chapter 4 and the total population for the year 2010 is estimated to be 3,143 persons. Three industrial sites, identified by Town and County officials, will be provided with water and sewer service as shown on Figure 4-1. An overview of the Safe Drinking Water Act amendments, Virginia Waterworks Regulations, and Virginia Pollutant Discharge Elimination System was presented and impacts to future development discussed.

Immediate and long-term water requirements of the study area for the 20-year planning period were developed using existing ERC criteria and future population projections. The total water use for the year 2010 is projected to be 364,579 gpd. Table 4-8 presents a summary of average daily water consumption for specific development periods to the year

2010. Based on a historical ratio of water consumed versus water billed, the projected source supply for the planning period is 400,454 gpd.

Four potential sources of water were evaluated and included the purchase of water from the City of Lynchburg, a river intake, impoundment, and groundwater. Chapter 4 presents a discussion, cost estimate, and present worth analysis, including capital and operation and maintenance costs, for each water source. The groundwater source of supply using wells is the least costly system. This is largely due to the utilization of the existing wells in providing quality water for the future.

The comprehensive hydrogeologic analysis prepared by Hatcher-Sayer, Inc., identified and prioritized three areas of potential groundwater supply. Two or three new wells are proposed in the area identified to have the best potential yield and are shown in Figure 4-2. A layout of the proposed water distribution system expansion is also presented on Figure 4-2. For each development period identified in Table 4-8, an estimated construction cost was developed (in January 1, 1991 dollars). Table 4-11 reflects these estimated costs, and development period, and will serve as the implementation schedule for the expansion of the water distribution system.

Future Sewer Situation

Using the present effluent quantity/load BOD_5, total suspended solids, and ammonia allowed by the present VPDES permit for each wastewater treatment plant, future effluent flows and concentrations were predicted. Design flows for the year 2010 for the Aerated Lagoon and Trickling Filter WWTP are 65,000 gpd and 291,600 gpd, respectively. Table 4-16 presents wastewater flow projections for each development period previously identified.

Figure 4-3 presents a layout of the proposed wastewater collection system expansion for the 20-year planning period; including gravity sewers and interceptors, pumping stations, and force mains. Table 4-21 presents the estimated construction costs (in January 1, 1991 dollars). These development periods will serve as the implementation schedule for the expansion of the wastewater collection system.

Expansion and upgrade of the existing wastewater treatment facilities involve an expansion to accommodate increased design flows and to provide treatment to meet more anticipated stringent discharge limits. The existing Aerated Lagoon WWTP will be able to accept increases in flow with only minor expansion. However, some upgrade is required to alleviate existing problems with the sand filters. The existing Trickling Filter WWTP, on the other hand, will require both expansion and upgrade to accommodate an anticipated 60 percent increase design flow. Table 4-20 presents the estimated construction costs for each plant upgrade.

Funding Sources

In addition to general obligation and revenue bonds, four funding sources are identified that could provide financing for expansion projects in this Study. Funding will depend on availability of funds, project eligibility, and qualification criteria. Chapter 5 presents a general summary of the benefits and criteria for each of the four funding sources.

CONCLUSIONS

1. Fire flow capacity problems in the existing water distribution system were identified, specifically at the High School, downtown, and Middle School areas.

2. Numerous water lines in the Town are 4-inch diameter, resulting in inadequate fire flows due to excessive friction losses.

3. Well pumps are oversized and must be throttled to match well yields.

4. The existing wastewater treatment facilities performance exceeds (better than) the permitted discharge limits. The utilized capacity of the Trickling Filter WWTP is

approaching 85 percent. When the flow reaches 95 percent for 3 consecutive months, the Town is required to prepare a plan of action for expansion.

5. Existing quantities of Infiltration/Inflow in the wastewater collection system, by criteria established by the SWCB, are considered nonexcessive. However, the identification and elimination of every possible I/I source should not be neglected.

6. Future water requirements for the year 2010 are estimated to be 400,454 gpd. Future total wastewater flows for the year 2010 are estimated to be 364,579 gpd.

RECOMMENDATIONS

The following recommendations have been developed from the information and documentation presented in this Master Plan:

1. Plan and implement as soon as practical the construction of the Priority One through Five improvements to the existing water distribution system.

2. Develop a schedule to replace all existing 4-inch water mains with 6-inch or 8-inch mains.

3. As the exiting oversized well pumps wear out or fail, replace with a properly sized pump to eliminate the need for throttling.

4. Drill a test well in the highest prospect area.

5. Proceed with expansion of the existing water system using the development periods identified.

6. Submit a written request to the SWCB for expansion of the Trickling Filter wastewater treatment plant to the ultimate design flows presented in this Master Plan.

7. Vigorously pursue loan/grant funds for qualified projects from funding sources.

8. Consider having discharge from the future Plain Run Branch Pumping Station connected to the gravity lines feeding the Trickling Filter WWTP.

CHAPTER 2 - INTRODUCTION

SCOPE OF REPORT

On June 12, 1990, the Town of Appomattox and Appomattox County authorized Wiley & Wilson to prepare a Comprehensive Water and Sewer Study for the Town and contiguous portions of the County. The scope of the Master Plan included the following:

1. Establish a planning area and identify both existing and potential growth centers that will develop in the planning area throughout the planning period which may require central water and/or sewer systems.

2. Identify potential industrial areas that may develop through the Region 2000 Program or under the Virginia Certified Business Community Certification Program.

3. Project a 20-year population for the planning area using standard planning procedures including trends that have developed in past years and are projected to continue in future years.

4. Master Water Plan

   a. Define nature, location, and problems with existing water system.

   b. Determine sources for new water including purchase of water from the City of Lynchburg and investigate both surface and subsurface water sources.

1) Evaluate purchase of potable water from the City of Lynchburg; and define transmission, pumping, and storage requirements.

2) Evaluate surface waters, either reservoir or stream, and identify general location(s) and size of a future treatment facility.

3) Evaluate the potential for developing future wells with desirable yields through geologic/hydrogeologic conditions.

   a) Perform literature search of geologic and groundwater conditions.

   b) Obtain information on existing wells including contact of area well drillers concerning their knowledge of local geologic/hydrogeologic conditions.

   c) Identify typical well yields in the planning area and perform a fracture trace analysis.

   d) Select potential well sites.

   e) Perform geologic field reconnaissance of the proposed well sites.

f) Rank, in order of preference, the potential well sites.

c. Calculate peak water consumptions for the planning area for the 20-year planning period (through year 2010 demand).

d. Estimate the cost for new water sources, transmission, treatment, storage, and distribution.

e. Determine flow rates, storage requirements, and locations for fire protection based on residential, commercial, and industrial usage.

f. Determine size and routing of pipelines based on flow requirements and necessary residual pressures for normal, peak, and fire protection conditions of service using the Kentucky Pipe Computer Model (KY-PIPE).

5. Master Sewer Plan

a. Define location and problems with existing wastewater collection and treatment systems.

b. Evaluate the existing Aerated Lagoon and Trickling Filter Wastewater Treatment Facilities for hydraulic capacity and treatment capability.

c. Calculate peak wastewater flows for the 20-year planning period (though year 2010 demand) based on population densities of residential areas and equivalent population densities of commercial and industrial areas.

d. Determine size and location of gravity collectors, interceptors, and pumping stations for peak flows.

e. Determine wastewater treatment capacity and degree of treatment necessary to satisfy the Virginia State Water Control Board effluent discharge limits.

f. Develop wastewater treatment upgrade, expansion, or relocation alternatives.

g. Estimate the cost for new wastewater treatment facilities, pumping stations, and collection and transmission systems.

6. Investigate institutional and financing arrangements available to the Town and County.

7. Develop a comprehensive implementation schedule for the study area (Town and contiguous County areas).

8. Prepare a Comprehensive Water and Sewer Study for the previously listed items. Provide detailed conclusions and recommendations for the Town and County of Appomattox.

PLANNING PERIOD

The planning period for this Master Plan is defined as 20 years, from the calendar year 1990 to the year 2010. Projections for the ultimate "build-out" density for a specific subarea of the water and/or sewer system within the planning area was reviewed. The "build-out" density was based on local existing or predicted zoning codes. Based on the ultimate "build-out" density analysis, it is more feasible for the Town to project a 20-year growth rate. This projection involves numerous assumptions and predictions which have dramatic effects on population growth; therefore, specific recommendations for the study area will be based on a 20-year planning period.

STUDY AREA

The Town of Appomattox, centrally located in Appomattox County, is approximately 20 miles east of the City of Lynchburg, Virginia, and 85 miles west of the City of Richmond, Virginia. The Town is the county seat for Appomattox County which lies in the rolling country of the Piedmont Plateau of south central Virginia. Appomattox County was formed from portions of Buckingham, Prince Edward, Charlotte, and Campbell Counties in 1845.

U.S. Route 460 and State Routes 24 and 26 intersect within the Town of Appomattox corporate limits. Route 460, spanning from the east to west side of Appomattox County, is a major east-west route within the state. Both the Town and County of Appomattox are home to a number of historical monuments, structures, and attractions which draw a generous tourist crowd each year. A major tourist attraction in the County is the Appomattox Court House National Historical Park where the Civil War ended.

The Town of Appomattox consists of residential, commercial, and industrial areas. Armstrong Furniture Division is the largest industrial employer within the Town corporate limits, and Courtland Manufacturing is the largest industrial employer in the county portion of the study area. Appomattox County consists of predominantly agricultural areas; however, residential subdivisions are growing on an annual basis. Other manufacturing/business operations existing in the County include general contracting, garment factory, trucking, and a natural gas pumping station.

The study area for the Comprehensive Water and Sewer Study is depicted in Figure 2-1. The study area includes all of the Incorporated Town of Appomattox, plus approximately a 1/2-mile radius outside of the corporate limits and a strip approximately 2,200 feet in width along the Route 460 corridor from the Town of Appomattox to Spout Spring.

REGION 2000

Appomattox County, including the Town of Appomattox, is a participant in a public and private partnership program called Region 2000. Seven localities, Altavista, Amherst County, Appomattox County, City of Bedford, Bedford County, Campbell County, and City of Lynchburg, comprise the over 2,000-square miles of area in the program. Region 2000 is simply a group of municipal governments joined together to present a unified whole to search, promote, and attract both business and industry expansions and relocations into the area. Emphasis is placed on existing labor force, relative cost of labor, available utility services including water and sewer, transportation services available, and tax base. For Appomattox County, available water and sewer services are of vital concern for both present and future industrial needs. This Master Plan will address these aspects and present specific recommendations to insure the implementation of utility services.

CERTIFIED BUSINESS COMMUNITY CERTIFICATION PROGRAM

The Commonwealth of Virginia has developed a Certified Business Community Certification Program to assist communities in attracting industry and economic development to improve job opportunities and capital investments in the State. The Program requirements are rigorous and will require a dedicated effort by the Town and County of Appomattox to become "Certified." The Program is sponsored by the Virginia Department of Economic Development (VDED) through its Office of Community and Business Services.

The Program objectives are as follows:

1. To provide a program of work by which a community can become better prepared for industrial and economic development.

2. To improve the preparedness of the community leadership and to enhance their ability to successfully promote economic development in their communities.

3. To give the Virginia Department of Economic Development a better inventory of communities to bring to the attention of industrial prospects seeking new plant locations.

4. To provide public recognition to those communities making an effort to become better prepared for economic development.

5. To enhance the community spirit within Virginia through involvement by community residents in the several standards of the certification program.

6. To provide benchmarks against which a community can measure its preparedness and readiness for economic development.

To become a "Certified" community, the following standards must be met:

1. Local Economic Development Organization. This involves the establishment of a local economic development organization, a local coordinator, assignments to complete the program, and funds to carry out basic functions.

2. Community Information Requirements. This is a gathering and packaging of community information including marketing book, promotional brochure, community profile, audio-visual presentation emphasizing industrial location in the community, approval of the community brochure by the VDED, community maps, and vocational education program.

3. Quality of Life Requirements. This involves conducting a community assessment, a plan to deal with any deficiencies, a community pride program, and a leisure services program.

4. Local Contact Team. This requires the identification of a prime contact person responsible for local arrangements, formation of a four-member local contact team to meet with potential industrial prospects, preparation of a well-rehearsed and planned community presentation, and designation of a local room to hold meeting.

5. Finance Requirements. This requires the existence of an active industrial development authority, active existence in at least one of the following:

- Industrial Development Corporation
- Certified Development Company (organized under SBA-504)
- Local finance resources committee, and a person knowledgeable in local government finance matters.

6. Local Existing Industry Program. This involves the formation of an active existing industry visitation team, a visitation program to express the community's interest to local industry, and assistance program to facilitate existing industry access to the following:
   - Local resources, technical training, finances, and mutual support programs
   - Local and State governments
   - Other liaison roles

   Public awareness of existing industry and its local contribution, and a means to insure continued coordination of economic development and industry assistance.

7. Site and Building Requirements. This defines the minimum requirements for site availability by community size, complete site and building profiles, a review and update of site and building information on file with VDED on a periodic basis, and knowledge of area construction capabilities.

For the Town and County of Appomattox, with a population of between 1,500 and 15,000, the industrial site requirements under the Program are as follows:

1. Total 20 acres

2. Minimum of two sites, maximum of three sites

3. Minimum site size of 5 acres

4. Property to be controlled by the local development organization

To be evaluated as a "Certified Site," the following criteria must be met:

1. Provide certified boundary survey.

2. Provide preliminary site plan.

3. Insure 60 percent or more of the site meets the minimum slope and flood plain standards.

4. Insure property is zoned for industrial use.

5. Establish price of property in writing.

6. Provide adequate wastewater service from central or package treatment system or septic system.

7. Provide adequate water service with suitable pressure.

Each site is evaluated through a point ranking system. The point system or scorecard is depicted in Table 2-1. The maximum possible points a site can earn is 31 points. To qualify as "Certified," a site must earn 21 points.

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TABLE 2-1
CERTIFIED BUSINESS COMMUNITY
CERTIFICATION PROGRAM

SITE AND BUILDING POINT SYSTEM SCORECARD

A.	Site: (Two Topographic Considerations)		
	Slope: Standard 10%		
	90% or more of site meets standard	=	3 pts.
	80% or more of site meets standard	=	2 pts.
	70% or more of site meets standard	=	1 pt.
	Floodplain: Standard 100-year floodplain		
	90% or more of site out of 100-year floodplain	=	3 pts.
	80% or more of site out of 100-year floodplain	=	2 pts.
	70% or more of site out of 100-year floodplain	=	1 pt.
B.	Zoning	=	1 pt.
C.	Control		
	Actual Ownership	=	4 pts.
	Written Option	=	2 pts.
D.	Price	=	2 pts.
*E.	Wastewater		
	On site with 40,000+ gpd treatment capacity	=	4 pts.
	On site with 30,000+ gpd treatment capacity	=	3 pts.
	On site with 20,000+ gpd treatment capacity	=	2 pts.
	On site with 10,000+ gpd treatment capacity	=	1 pts.
*F.	Water		
	On site with pressure and supply of 40,000+ gpd	=	4 pts.
	On site with pressure and supply of 30,000+ gpd	=	3 pts.
	On site with pressure and supply of 20,000+ gpd	=	2 pts.
	On site with pressure and supply of 10,000+ gpd	=	1 pts.
G.	Site Access: (Two Categories)		
	60 feet of state maintained right-of-way (and)	=	2 pts.
	Site access	=	1 pt.
H.	Electric	=	2 pts.
I.	Site Evaluation Team Judgement (<u>Plus or Minus</u>)	=	5 pts.

CHAPTER 3 - CURRENT SITUATION

EXISTING POPULATION

Historical population data, including preliminary population data from the 1990 census, for the incorporated limits of the Town of Appomattox is presented in Table 3-1. The large increase in population from 1980 census data and the preliminary 1990 census data is attributed in part to a boundary line adjustment agreement between the Town and County in 1988. The 1990 population for the Town of Appomattox is estimated to be 1,703 persons.

Table 3-1 also presents the historical population data and an estimated 1990 population for Appomattox County in its entirety, including the Town of Appomattox. A steady growth rate of the County has been experienced over the past 50 years. The estimated 1990 population for Appomattox County is 13,300 persons.

The estimated serviceable population for the study area depicted in Figure 2-1 is estimated to be 2,353 persons for 1990. This is broken into two components; first, the population of the incorporated Town of Appomattox equalling 1,703 persons, and second, the serviceable population of the remaining study area in Appomattox County which is approximated at 650 persons.

WATER

Private Systems

The residents of the Town of Appomattox are generally served by a public water system and, the residents of Appomattox County are served by either single family private wells or community (multi-family) wells. The areas of Appomattox County that are in the study area (refer to Figure 2-1), for the most part, are served by single-family private wells. However, a 40 plus or minus home subdivision west of the Appomattox corporate limits, known as the Walton Place subdivision, is served by a community well and water distribution system.

TABLE 3-1
HISTORICAL AND EXISTING POPULATION DATA FOR TOWN OF APPOMATTOX AND APPOMATTOX COUNTY[(a)]

Year	Town of Appomattox	Appomattox County
1930	704	8,402
1940	992	9,020
1950	1,094	8,764
1960	1,184	9,148
1970	1,400	9,784
1980	1,378	11,971
1990	1,703[(b)]	13,300[(c)]

(a) Source: U.S. Department of Commerce, Bureau of the Census.

(b) Significant increase in population attributted to increase of Town's corporate limits through a boundary line adjustment agreement. Indicates preliminary 1990 Census data.

(c) Preliminary 1990 census data unavailable.
Source: 1989 Statistical Abstract published by the University of Virginia.

Some of these private wells are subject to high iron concentrations in the groundwater, depending on location. Also, manganese has been a problem in isolated instances. From discussion with the local Virginia Health Department Sanitarian, no wells in the planning area are in a designated health hazard area.

Public System

Background

The Town of Appomattox water system began in 1925 from a privately owned well. Over time, the system was expanded to include a small distribution system and a 30,000-gallon water tank. In 1948, the Town of Appomattox purchased the entire water system. Shortly thereafter, the 100,000-gallon elevated storage tank was built and the distribution system expanded. Some of the original water lines constructed in the late 1920's are still in use. These lines are limited to the Court, Church, and Main Street areas.

In 1976 the water system was again expanded with the construction of a 1 million-gallon ground level storage tank north of Town. As the water system has been expanded over the years, additional wells have been drilled and added to the system to provide potable water to meet the water demands of the people, businesses, and industries in the Town.

Presently, the Town of Appomattox water system includes five subsurface wells, three water storage tanks, and a distribution system consisting of a high and low zone. The Town is generally divided by the railroad; the area north of the railroad consists of the high zone and the area south of the railroad consists of the low zone.

The high zone is supplied by three wells and a booster pumping station to pump excess water from the low to the high zone. Two water storage tanks provide water for both fire storage and operating use in the high zone. Both the 100,000-gallon elevated storage tank and the 1,000,000-gallon ground level storage tank are full at an elevation of 997 feet above mean sea level. The high zone could be called the 997 zone.

The low zone is supplied by two wells. One water storage tank provides water for fire storage and operating use in the low zone. The 30,000-gallon elevated storage tank is full at an elevation of 914 feet above mean sea level. The low zone could be called the 914 zone.

Groundwater Supply

Wiley & Wilson retained the services of Hatcher-Sayre, Inc., to perform a comprehensive hydrogeologic analysis and report for the study area. The analysis included an evaluation of the existing wells and groundwater yields plus locations for future wells to provide additional water for the Town and County. The comprehensive hydrogeologic analysis report from Hatcher-Sayre, Inc., is presented in Appendix A. Refer to this Report to supplement the following information.

As previously noted, water for the Town of Appomattox is supplied through five subsurface wells. Figure 3-1 (see Pocket #1 in back of this report) presents the location of these wells and indicates the existing water distribution system for the Town. Each well consists of the following: submersible well pump, discharge piping and valves, and motor controls, all of which are enclosed in a heated and ventilated above ground pump house. Disinfection facilities do not exist for any of the wells.

Wells range in depth from 100 to 305 feet and yields range from 45 to 100 gpm. Table 3-2 presents a summary of each existing well. Generally, water quality from the existing wells is good. Chemical treatment is not required since iron and manganese levels

TABLE 3-2
TOWN OF APPOMATTOX

SUMMARY OF EXISTING WELLS

Well No.	Well Depth, Feet[(a)]	Pump Setting, Feet[(a)]	Pump Motor, HP	Well Capacity, GPM	Column Diameter, Inches	Pressure Zone
1	100	90	7.5	60	2	Low
5	110	100	7.5	60	2	Low
9	205	155	15.0	90	3	High
15	250	215	15.0	100	3	High
25	305	252	10.0	45	2	High

[(a)] Defined in feet below top of well.

are below levels defined by the Virginia Department of Health Waterworks Regulations. However, several old abandoned wells and a recent speculation well drilled by a local well driller for consideration by the Town are drilled to approximately the same depth of the existing production wells and have experienced high concentrations of iron and manganese.

Table 3-3 presents the monthly average gallons per day of water produced (pumped) and water consumed (billed) from September 1989 through September 1990. Also, the calculated percentage difference of water produced and water consumed is indicated. The average monthly well production over the 13-month period is 186,650 gpd, and the average monthly water consumption over the same 13-month period is 169,931 gpd. Therefore, the calculated difference in production and consumption is approximately 10 percent. This simply means that approximately 10 percent more water was pumped than was billed for the period. This can be attributed to numerous factors including meter reading inaccuracies, old or defective water meters, unmetered water connections, fire department usage, and/or minor water distribution system leaks. However, the 10 percent "loss" is average or better than average for municipal systems.

Distribution System

Figure 3-1, previously presented, indicates the existing water distribution system for the Town of Appomattox. Water lines are constructed of various materials including polyvinyl chloride pipe (PVC), cement lined ductile iron pipe, and unlined cast iron pipe which is the original material used to construct the water system in the late 1920's.

In general, the distribution system is in good condition. This is documented by the Town's maintenance personnel who have performed recent repairs to the system. However, deterioration has occurred in the piping of the original system. The good condition of the distribution is attributed to the selection of high quality, long lasting piping materials, and the lack of chemicals in the groundwater source (predominately large quantities of calcium carbonate).

TABLE 3-3
TOWN OF APPOMATTOX

WELL PRODUCTION AND WATER CONSUMPTION DATA FROM SEPTEMBER 1989 TO SEPTEMBER 1990

Month	Average Monthly Well Production, GPD [a]	Average Monthly Water Consumption, GPD [b]
September 1989	195,390	158,150
October 1989	190,864	169,374
November 1989	178,268	175,150
December 1989	183,792	145,219
January 1990	189,328	178,984
February 1990	163,839 [c]	163,839
March 1990	193,769 [c]	187,532
April 1990	186,704	168,417
May 1990	191,571	166,448
June 1990	191,751	181,860
July 1990	185,646	171,980
August 1990	185,077	177,027
September 1990	190,450	165,120
AVERAGE	186,650	169,931

Percent difference in average water produced per day verses average water consumed per day = [(186,650 - 169,931)/169,931] x 100 = 9.84% [d]

[a] Combined monthly reading for all five production wells.

[b] Total water meter readings at approximate 30-day intervals.

[c] Readings adjusted to indicate production greater than or equal to consumption. Discrepancies attributed to time lapse in readings.

[d] Difference in production verses consumption attributed to meter reading inaccuracies, unmetered water connections, fire department usage, and/or minor system leaks.

Water Use

The Appomattox water system primarily serves the citizens of the Town; however, a small population is presently served outside of the corporate limits. From review of water records and discussions with Town personnel, water distribution service is described as follows:

1. 684 active meters billed on October 1, 1990

2. Approximately 556 residential meters including single and multi-family dwellings.

3. Approximately 128 commercial meters which include businesses, offices, industries, and other nonresidential establishments.

4. 140 connections on system with water only, no public sanitary sewer. This includes a total of 98 connections within the corporate limits consisting of 9 commercial, 1 multi-family (4-unit), and 88 residential connections. Also, this includes a total of 42 connections outside the corporate limits consisting of 3 commercial and 39 residential connections.

5. 5 connections in the corporate limits with only sanitary sewer service, no water service.

6. Approximately 49 units exist in the corporate limits without either public water or sewer service, consisting of 7 commercial and 42 residential units.

7. Total average water use for 13-month period (refer to Table 3-3) is 169,931 gpd or 5,168,735 gallons/month which consists of an average residential water use of 2,760,166 gallons/month (90,745 gpd) and an average commercial water use of 2,408,569 gallons/month (79,186 gpd).

To determine an equitable distribution of water service between residential and commercial users, a system of equivalent residential connections (ERC's) was employed. An average water use per residential connection was calculated and used to determine the quantity of equivalent residential connections for the total commercial water use. Table 3-4 presents the ERC calculations. The total equivalent residential connections (ERC's) for the Town of Appomattox water system is 1,262.

Table 3-5 presents as estimated per capita water use for the residential connections of the water system. To accurately determine the per capita water usage, an assessment of the number of persons per household was performed. Table 3-5 presents calculations to identify an average household in the Town of Appomattox to be 2.5 persons. Using this household data and the water data identified in Table 3-4, a per capita water use was calculated to be 53.85 gallons per capita per day (gpcd).

Existing Water Distribution System Analysis

Model Development: To determine capacity of the existing water system, including fire flow availability, a computer model was developed for the water system utilizing the Kentucky Pipe Program. Water lines used in this model were 4 inches in diameter and larger. Water lines smaller than 4 inches were not included due to their negligible ability to transmit fire flows. Refer to Figure 3-1 as required.

TABLE 3-4
TOWN OF APPOMATTOX

WATER CONSUMPTION
AND EQUIVALENT RESIDENTIAL CONNECTIONS (ERC)

User	Connections	Average Water Consumption, Gallons/Month	Average Water Consumption Per Residential Connection, Gallons/Month	Equivalent Residential Connections ERC's
Residential [a]	674	2,760,166	4,095	674
Industrial/ Commercial [b]	128	2,408,569	N/A	588[c]
TOTALS	802	5,168,735	4,095	1,262

[a] Includes 540 single family and 134 multi-family connections.

[b] Industrial/commercial connections only.

[c] Industrial/commercial ERC's calculated by dividing average industrial/commercial water consumption (2,408,569) by average residential water consumption (4,095 gal/mo).

TABLE 3-5
TOWN OF APPOMATTOX

ESTIMATED PER CAPITA WATER USE

CALCULATION OF PERSONS PER HOUSEHOLD

Total Residential connections with water service:	+	674 [a]
Connections with water service outside of corporate limits:	-	39
Units in corporate limits without water or sewer service:	+	42
Units in corporate limits with sanitary sewer service only:	+	5
Total Residential Units in the corporate limits:		682

Number of Persons per Household = 1,703 Persons/682 Household Units
= 2.497 Persons/Household
Use 2.5 Persons/Household[b]

ESTIMATED WATER USE PER CAPITA

Total Average Residential Water Consumption:	2,760,166 gallons/month
Residential Water Connections:	674 connections
Equivalent Residential Population:	1,685 people [c]

Usage = (2,760,166 gallons/month)/(1,685 people)
= (1,638.08 gallons/month/capita)(12 months/year)/(365 days/year)
= 53.85 gallons/capita/day (gpcd)

(a) Includes 540 single and 134 multi-family dwellings.

(b) Assume this calculated persons per household applies to the residential units with water service outside of the corporate limits.

(c) Calculated using 674 connections times 2.5 persons per household.

Input to the model included line sizes, friction factors, pipe lengths, pump curves, depth to water level in the wells, tank sizes and heights, and water usage demands. The model was calibrated using both flow tests and static conditions performed specifically for the Town of Appomattox. Once the program was developed and checked with actual field results, an analysis of the current water system was completed. Appendix B contains additional existing well information, well pump data, and well pump curves that were used in the model.

Model Calibration: To establish accurate friction factors in the water distribution system, flow tests were performed in Appomattox on November 26, 1990. Results from the flow tests are presented in Table 3-6. Calibration of the model basically involves running the model using the flow test data and adjusting the distribution system friction factors, referred to as the Hazen and Williams "C" factor, to closely match these actual field test results.

The relatively new 10-inch water lines on Route 460 west and the 12-inch line serving the 1,000,000-gallon tank have, as expected, low friction factors. Flow test results indicate that areas served from very old pipe, including 4-inch lines on North and South Church Street, Main Street, North Court Street, Lee-Grant Avenue, and Bandana Street, all have high friction factors. Moderate friction factors were experienced in the piping system feeding the high school and lines that feed the area south of Route 460 east.

Demands: Water demands utilized in the model were taken from average, representative water usage as obtained through Town billing records. A demand was given to each point in the model where the line changed size, two pipes intersected, or other points where high demands were expected. These points are referred to as nodes in the model. The total demand for each street was divided among the nodes on the street and multiplied by a correction factor to account for minor water losses in the system.

TABLE 3-6

WATER SYSTEM HYDRANT FLOW TEST DATA USED FOR MODEL CALIBRATION

Site Location On Map[(a)]	Time[(b)]	Hydrant Function	Static Pressure (PSI)	Residual Pressure (PSI)	Pressure/ Flow (PSI/GPM)	Pump System Status (On/Off)
H1	9:35 a.m.	Flowing	71	--	46/1135	Off
H2		Reading	68	50-60	--	
H3	9:55 a.m.	Flowing	57	--	12/580	Off
H4		Reading	49	15-30	--	
H5	10:40 a.m.	Flowing	79	--	46/1135	Off
H6		Reading	65	48	--	
H7	1:25 p.m.	Flowing	40	--	1/100[(c)]	On
H8		Reading	38	18	--	
H9	1:40 p.m.	Flowing	23	--	0/75[(c)]	On
H8		Reading	38	24-25	--	
H10	3:00 p.m.	Flowing	51	--	22/790	On
H11		Reading	59	40-45	--	
H10	3:00 p.m.	Flowing	51	--	6/410	On
H11		Reading	59	55	--	

(a) Refer to Figure 3-2 for location of hydrants tested.

(b) Flow testing occurred on November 26, 1990.

(c) Estimated reading, resulting from low flow reading.

Required Fire Flows: In sparsely populated residential districts, a minimum fire flow of 500 gallons per minute is required. In dense commercial and high value areas, between 1,500 and 3,000 gallons per minute of fire flows are required. Using the existing population of the Town of Appomattox of 1,703 persons, the required fire flow is 1,360 gpm for a duration of 5.4 hours as defined by the National Board of Fire Underwriters and the American Water Works Association. For fire flow analysis, a minimum residual pressure of at least 20 psi was maintained.

Required Storage: Storage requirements for the system should provide a minimum of 2 days of emergency domestic supply plus a fire flow demand of 1,360 gallons per minute for 5.4 hours. With an existing average daily water demand of 169,931 gallons/day and a fire flow requirement of 440,640 gallons, an available storage of 610,571 gallons is required. Since the available storage is 1,130,000 gallons, sufficient storage exists. However, a problem does exist with the present storage system, the area served by the low zone can only use the 30,000-gallon storage tank. This is far below the required amount and it affects approximately one-third of the Town.

Analysis of Present System: The results of the computer model analysis are presented in Appendix C. An interpretation of the analysis and description of problems noted will be discussed in detail. Specific recommendations to upgrade the existing system are presented and prioritized.

The present system can provide enough water for the present domestic needs of the Town. However, adequate fire flows could not be supplied with the present water distribution system to certain areas in Town. As a whole, the high pressure zone could provide adequate fire protection for extended periods of time. However, as noted previously, the low zone cannot provide adequate fire protection for an extended period of time.

Of particular interest in the low zone is the Appomattox County High School. Fire flows for the high school should be in the area of 2,000-3,000 gpm. ISO (Insurance Service Office) calculations would be higher, but the 2,000-3,000 gpm are more reasonably obtainable for Appomattox. The present system can deliver 800 gpm with a residual pressure of 20 psi to the High School. Assuming, at best, that the hydrants could supply 800 gpm of water, the 30,000-gallon tank would last for only 38 minutes. As the tank water level dropped, hydrant flow would decrease. The two wells in the low zone could supply 130 gpm or more; however, this would not be enough water to make a significant difference.

Another area in the low zone where both low fire flows and low pressure exist is Lee-Grant Avenue on the west side of Church Street. As a minimum, residential areas should supply 500 gpm at 20 psi. On the far western end of Lee-Grant Avenue, the last hydrant on the 4-inch line could supply only 15 gpm at 20 psi. Another hydrant on this line, at the end of Wemly Lane, can supply approximately 100 gpm at 20 psi. Pressure problems also exist under normal water usage conditions in the low zone. During periods of maximum water usage in the low zone, little to no water is available to users on the West end of Lee-Grant Avenue.

Areas in the high zone are typically provided with adequate fire protection. These areas are supplied by the 1,000,000-gallon ground storage tank and the 100,000-gallon elevated tank. However, low flows exist in certain areas. The flow to the 6-inch line on Main Street is controlled by two 4-inch lines, one on Court Street and one on Oakleigh Street. A flow of only 640 gpm at 20 psi is available to this high value district in the downtown area. A minimum of 1,500 gpm at 20 psi is required with 3,000 gpm at 20 psi being more suitable. Oakleigh Avenue on the north side of Route 460 also has fire flow problems. A 4-inch line supplying the hydrant can only deliver 325 gpm at 20 psi. This is far below the minimum required flow of 500 gpm at 20 psi.

The Elementary School, supplied by a 12-inch line located on Route 460, is provided with adequate fire protection. This 12-inch line turns onto Patricia Ann Lane, parallels Route 460, until reconnecting with Route 460 east of the Middle School. This scenario leaves the Middle School with fire protection from only the 4-inch line on North Church Street. Fire flows for the Middle School should also be at least 2,000-3,000 gpm. Again, this may not be the ultimate required by ISO, but is a reasonable flow obtainable by this system. The existing 4-inch line can only supply 560 gpm.

The shopping centers on Route 460 and the Plaza have adequate fire protection. This is attributed to 10- and 12-inch lines parallelling Route 460 in that area which supplies these commercial users.

Recommended Water Distribution System Improvements: The following water distribution system improvements are recommended now to insure adequate water supplies in the event of a fire. These improvements have been prioritized in the order of need and should be added in the Capital Improvement process in 1991. The improvements are shown in Figure 3-2 and described as follows:

Priority One Improvements: Extended fire protection for the low zone has the highest priority. This can be accomplished by a direct connection to the high zone and the large storage tanks. To supply a fire flow volume to the low zone, a 10-inch line is proposed, connecting the existing 10-inch line on the eastern end of Harrell Street and extending south to an 8-inch line in Lee-Grant Avenue as shown on Figure 3-2.

A pressure reducing valve (PRV) is required in this line. The PRV will normally be closed but will open when the 30,000-gallon tank drops to a point below the level at which well pumps No. 1 and No. 5 operate. In the event of a fire or large system demand, the 30,000-gallon tank water level will fall to the point where the PRV will open.

This will allow water from the high zone to flow to the low zone. Additionally, because two 4-inch lines would feed the 10-inch PRV line, another line is needed. A 10-inch line should be extended from the corner of Oakleigh Avenue and Harrell Street to the 10-inch line on Route 460. This extension will increase fire protection to the Middle School and to the downtown area.

To serve the Middle School will require the addition of a short line and hydrant off Oakleigh Avenue to the back of the school.

With the addition of these new lines, the water system model predicted the following water availability increases to occur:

	Existing (gpm at 20 psi)	Immediate Improvements (gpm at 20 psi)
Middle School	560	1,800
Downtown Area	640	980
High School	800	800

With these improvements, the High School does not receive any additional available flow from that currently received; however, fire fighting capabilities will be extended from 38 minutes to nearly 24 hours. The entire low zone will be provided with increased fire fighting capabilities above the existing conditions.

The 1972 Water and Sewer Report, prepared by Hankins and Anderson Consulting Engineers, recommended the Town use only one pressure zone to eliminate the storage problems in the low zone. Based on the computer analysis, observed problems using a single pressure zone include high pressure (greater than 100 psi) in areas of the low zone below elevation 765, replacement of well pump No. 5, and removal of the 30,000-gallon tank and the booster pumping station. For these reasons, the low zone is recommended to remain in service and a PRV valve installed to connect the high and low zones.

As described earlier, pressure and fire flow problems exist on the west end of Lee-Grant Avenue. To improve the pressure by nearly 38 psi without creating excessive pressures, the valve at the intersection of Ferguson Street and Lee-Grant Avenue should be opened. This will allow the high zone to extend as far as, but not including, Sunnydale Avenue. The high zone should be valved off between Sunnydale Avenue and South Court Street. Fire flows on the far western end of Lee-Grant Avenue will increase from 15 gpm at 20 psi to 500 gpm at 20 psi. Flows in the vicinity of Lee-Grant and South Court will also be increased, but to a lesser extent due to the friction loss of the existing 4-inch line.

The estimated construction cost for the Priority One Improvements, including Contractor Overhead and Profit, Contingency, and Engineering, Legal, and related costs, in 1991 dollars, is as follows:

Item	Location	Estimated Cost
10-inch Line	Oakleigh Avenue (Harrell to Route 460)	$ 48,600
10-inch Line	Railroad Crossing and PRV Vault	124,700
	TOTAL	$173,300

Priority Two Improvements: These improvements will increase fire flows in areas where the previously defined Priority One Improvements have helped. Bottlenecks are formed by small diameter lines in the vicinity of the High School. By increasing the existing 6-inch line on the south end of Hunter Street to 8 inches, flows to the High School will be increased from 800 gpm at 20 psi to 1,325 gpm at 20 psi. This increase represents a significant water availability for both the High School and the nursing home on Evergreen Avenue.

To provide more flow to the downtown area, a new 10-inch line is proposed to extend from the existing 10-inch line on Church Street. This line will run parallel to Main Street in an alley north of Main Street, and tie back into the 6-inch line on Main Street, and continue west to Court Street, to replace the existing 4-inch line to the end of Main Street. (Refer to Figure 3-2.) Flow capacity in the downtown area is expected to increase from 980 gpm at 20 psi to 2,250 gpm at 20 psi.

The estimated construction cost for Priority Two Improvements, including Contractor Overhead and Profit, Contingency, Engineering, Administrative, and related costs, in 1991 dollars is as follows:

Item	Location	Estimated Cost
8-inch Line	Hunter Street	$ 26,000
10-inch Line	Main Street	58,700
	TOTAL	$84,700

Priority Three Improvements: The recommended Priority Three, Four, and Five Improvements will provide water for fire protection to residential zones that do not have access to loops in the system. Also, more water will be provided to the Main Street area in one case.

To increase flows to hydrants on Lee-Grant Avenue from 100 gpm at 20 psi (worst case) to 500 plus gpm at 20 psi will require an 8-inch line. This line should extend from Church Street to the intersection of Ferguson Street and Lee-Grant Avenue.

The estimated construction cost for Priority Three Improvements, including Contractor Overhead and Profit, Contingency, Engineering, Administrative, and related costs, in 1991 dollars, is as follows:

Item	Location	Estimated Cost
8-inch Line	Lee-Grant Avenue (West)	$207,000

Priority Four Improvements: To increase flows to residential areas along Virginia Avenue and Morton Lane, a 6-inch water line connection is required between the 6-inch line in Morton Lane to the 10-inch line in Ferguson Street. This connection will double flows to the area. Currently, a hydrant on Virginia Avenue will deliver 550 gpm at 20 psi; however, with the proposed extension, the hydrant will deliver 1,000 gpm at 20 psi. Implementation of this connection will provide high return (water yields) for minimum cost.

Also, to increase flows to the Main Street area, to the Middle School, and to the South Court Street area, an 8-inch line is proposed to replace the existing 4-inch line on South Court Street between Main Street and Morton Lane. This addition, along with the Virginia Avenue connection, will increase flows to the downtown area from 2,250 gpm at 20 psi to 3,000 gpm at 20 psi and to the Middle School from 1,800 gpm at 20 psi to 2,000 gpm at 20 psi. The water distribution system in these areas will benefit from increased reliability since the system will be looped. This will allow either the Priority One Improvement of the 10-inch line on Oakleigh Avenue to be shut down or the Priority Four Improvement of the 8-inch line on Court Street to be shut down while continuing to supply water to the downtown area, the Middle School, and, most importantly, the entire low zone for fire protection.

The estimated construction cost for Priority Four Improvements including Contractor Overhead and Profit, Contingency, Engineering, Administrative, and related costs, in 1991 dollars, is as follows:

Item	Location	Estimated Cost
6-inch Line	Virginia Avenue Connection	$ 5,000
8-inch Line	South Court Street	45,100
	TOTAL	$50,100

Priority Five Improvements: Priority Five Improvements include the addition of an 8-inch water line for fire protection on Oakleigh Avenue, north of Route 460. This area presently is served by a single 4-inch dead end line. Only 325 gpm at 20 psi are available for this area. An 8-inch line is needed to provide adequate fire protection and to provide water to meet anticipated growth in this area.

The estimated construction cost for Priority Five Improvements including contractor Overhead and Profit, Contingency, Engineering, Administrative, and related costs, in 1991 dollars, is as follows:

Item	Location	Estimated Cost
8-inch Line	Oakleigh Avenue	$79,400

General Improvements: For fire protection purposes, a minimum water line size of 6 inches is generally regarded as the smallest line that should be used. However, 8-inch lines are recommended when practical. Many areas in Town use old 4-inch lines for fire protection. These should be replaced with larger lines over time. Priority Improvements One through Five were singled out as the most pressing problems that exist in the Town.

Well and Booster Pumping Station Improvements: The existing well pumps and the booster pumps are larger than required; i.e., they have a capacity greater than required. Each well pump discharge line includes a valve that is used to throttle the well pump. Typically, wells are not designed to require a throttling valve; instead, they should be sized to operate on a specific curve that closely matches the specific system requirements without the need for an artificial source of friction loss (i.e., a throttling valve).

Because of the increased size of the pumps, purchase, replacement, and operation of each pump is expensive. Approximately 10 percent more electricity is used than is actually needed, as estimated on an annual basis. On wells No. 1, No. 5, and No. 25, and the booster pumps, gate valves are used to throttle the well pump to control the amount of water that is pumped. Gate valves should not be used for throttling as they are designed for on/off service. Gate valves used for throttling are subjected to severe erosion of both the disc and seat as a result of high velocities and are subjected to damage from seat vibration which will eventually prevent tight shutoff. Butterfly or ball valves are used for both on/off and throttling service. Therefore, when these gate valves wear out, they should be replaced with either butterfly or ball valves. On wells No. 9 and No. 15, butterfly valves are used for throttling. When each of the existing pumps require replacement due to age, equipment failure, motor damage, or the like, they should be replaced with the correct size pump which will not require throttling.

SEWER

<u>Private Systems</u>

The residents of the Town of Appomattox are generally served by a public wastewater collection, treatment, and disposal system. However, some residents in Town use single family on-site septic systems. The residents of Appomattox County, on the other hand, rely on single family on-site septic systems for wastewater treatment and disposal.

The local Virginia Health Department Sanitarian has identified one public health hazard that exists in the planning area. The problem is an unaerated lagoon west of the corporate limits along Route 460 across from the History Junction Shopping Center and Super 8 Motel. The lagoon presently serves a small motel and trailer park on the south side of Route 460. The lagoon has no discharge point; however, during periods of heavy rain, untreated and/or partially treated wastewater spills onto the surrounding ground. Although sanitary sewer service is in the general vicinity, a substantial capital investment is required to provide service under Route 460 to connect to the Town's wastewater collection system. The Health Department is pursuing means to assist the people served by the lagoon with a solution to the health hazard.

Many on-site systems in the Appomattox vicinity were installed prior to the adoption of 1982 stringent standards for septic systems issued by the Health Department. When these old systems fail, replacement is often difficult and extremely expensive due to percolation requirements dictated by the Health Department. An on-site system that experiences problems and is within a reasonable distance to public sanitary sewer service cannot receive a permit for repair or replacement of the existing septic system. Instead, according to the local Health Department Sanitarian, the home owner will be required to connect to the public sanitary sewer system.

Public System

Collection System

The Town of Appomattox wastewater collection system is generally divided by Route 460. The majority of the collection system north of Route 460 flows to an aerated lagoon wastewater treatment plant, and the collection system south of Route 460 flows to a trickling filter wastewater treatment plant. Figure 3-3 identifies the present wastewater collection system and the location of both wastewater treatment plants.

The initial wastewater collection system for the Town of Appomattox was constructed of nonreinforced concrete pipe in the 1930's. This system was designed with adequate slopes to provide both the capacity and velocity necessary to ensure good performance. In later years, the collection system was expanded by the addition of several thousand linear feet of extra strength vitrified clay pipe. More recently, the collection system has been expanded using a bell and spigot polyvinyl chloride (PVC) pipe. Ductile iron pipe has been used in major highway crossings and in areas of severe traffic impact.

These older collection system materials have experienced problems over the years, including material deterioration, infiltration resulting from broken pipe and leaking joints, line blockage, and settlement. To alleviate these problems in the nonreinforced concrete and vitrified clay pipe systems, the Town of Appomattox has been and is continuing to replace these pipelines with PVC piping.

In addition to the gravity collection system, the Town of Appomattox owns, operates, and maintains three wastewater pumping stations and associated force mains. Refer to Figure 3-3 for these specific locations. Also, four privately owned and operated pumping stations and force main serve locations in the Town of Appomattox. These are not shown in Figure 3-3 since they are of small capacity, are a minute portion of the total collection system, and are not maintained by the Town.

The Town of Appomattox wastewater collection system is strictly a sanitary wastewater system which receives no stormwater flow from combined sewers. In the 1970's, an extensive Infiltration/Inflow (I/I) study was conducted. The majority of the roof leaders and basement drains were eliminated through an I/I abatement program. As previously noted, the Town is actively removing sources of infiltration in old or broken piping systems. Although I/I problems still exist in the wastewater collection system, the Town's wastewater operational staff has noted that flow records over the last year indicate that quantities of I/I appear to have been reduced.

Wastewater Treatment Facilities

Both the Town of Appomattox Aerated Lagoon and Trickling Filter Wastewater Treatment Plants are authorized to discharge treated wastewater under Permit Numbers VA 0020257-001 and VA 0020249-001, respectively, by the Virginia Pollutant Discharge Elimination System (VPDES) and Virginia State Water Control Law. Each permit is issued and administered by the Virginia State Water Control Board (SWCB). The receiving stream for the Aerated Lagoon WWTP is near the headwaters of the South Fork of the Appomattox River, and the receiving stream for the Trickling Filter WWTP is Caldwells Creek which is a tributary to Falling River. The discharge limits for each treatment plant, including the monthly average design flow rate, is presented in Table 3-7.

The VPDES permits for each wastewater facility indicates that a plan of action for ensuring continued compliance with the terms of the permit be submitted to the Regional Office of the State Water Control Board, when the monthly average influent flow to the treatment plant reaches 95 percent of the design capacity authorized by the permit for each

TABLE 3-7
TOWN OF APPOMATTOX

EFFLUENT DISCHARGE LIMITS FOR TRICKLING FILTER AND AERATED LAGOON WASTEWATER TREATMENT PLANTS

	DISCHARGE LIMITS			
	Trickling Filter WWTP		Aerated Lagoon WWTP	
Parameter	Monthly Average	Monthly Maximum	Monthly Average	Monthly Maximum
Flow, mgd	0.1700	--	0.0540	--
pH	6.00	9.00	6.00	9.00
BOD_5: Year Round, mg/l	30.00	45.00	--	--
BOD_5: April - Sept., mg/l	--	--	21.00	31.50
BOD_5: Oct. - March, mg/l	--	--	30.00	45.00
Total Suspended Solids, mg/l	30.00	45.00	60.00	90.00
Instantaneous Chlorine Residual, mg/l	--	Non-Detectable	--	Non-Detectable
Ammonia: April - Sept., mg/l	10.00	15.00	6.00	9.00

month of any 3 consecutive month period. This plan of action must be received at the Regional Office no later than 90 days from the third consecutive month for which the flow reached 95 percent of the design capacity. The plan will include the necessary steps and a prompt schedule of implementation for controlling any current or reasonably anticipated problem resulting from high influent flows. Failure to timely submit an adequate plan is deemed a violation of the permit. This criteria will apply to the evaluation of existing capacity in each plant.

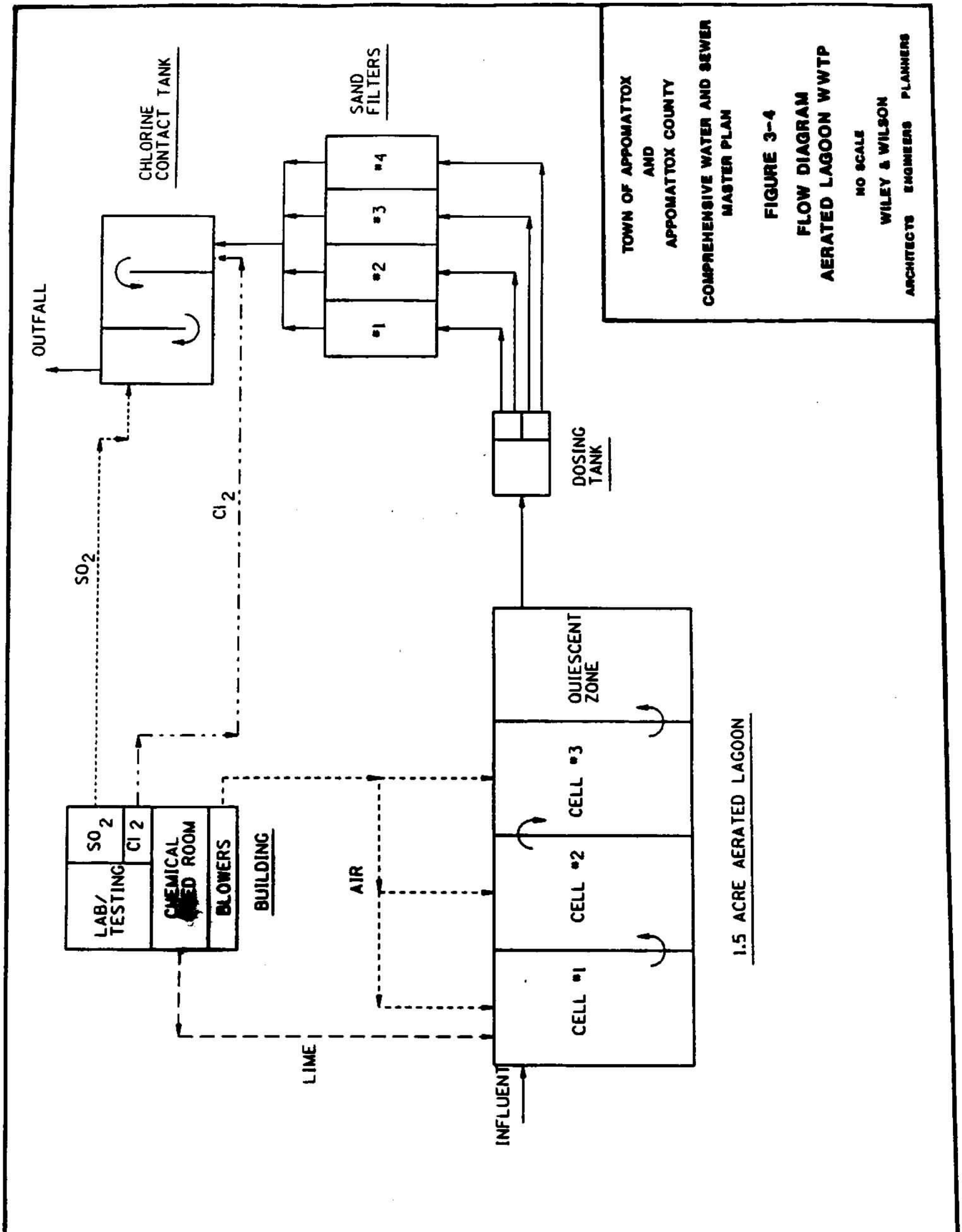
Aerated Lagoon WWTP: The Aerated Lagoon WWTP has a VPDES permit rated capacity of 0.054 mgd (54,000 gpd). The plant was originally constructed in 1964 as a 2.1-acre waste stabilization pond and an upgrade was completed in 1988 to reflect the present day facility. Figure 3-4 depicts the flow diagram for this treatment plant. The plant consists of a 1.5-acre aerated lagoon with three cells and an integral quiescent or clarification zone. Treated wastewater then enters a dosing siphon to provide head and flow distribution to one or more of four slow rate, gravity sand filters for effluent polishing. Prior to discharge, all wastewater is disinfected in the chlorine contact tank, then dechlorinated to remove all residual chlorine, and the flow is measured and recorded. Support facilities for the aerated lagoon plant include lime feed system for pH adjustment in the lagoon, chlorination system, dechlorination system, blowers, and laboratory/testing facilities.

The advantages of this specific treatment scheme include the following:

1. Operational simplicity.

2. No return or waste sludge requirements from the clarification zone.

3. No backwash requirements of the gravity sand filters.


TOWN OF APPOMATTOX
AND
APPOMATTOX COUNTY
COMPREHENSIVE WATER AND SEWER
MASTER PLAN
FIGURE 3-4
FLOW DIAGRAM
AERATED LAGOON WWTP
NO SCALE
WILEY & WILSON
ARCHITECTS ENGINEERS PLANNERS
CHLORINE CONTACT TANK
SAND FILTERS
#1
#2
#3
#4
OUTFALL
DOSING TANK
SO_2
Cl_2
QUIESCENT ZONE
CELL #3
CELL #2
CELL #1
1.5 ACRE AERATED LAGOON
LAB/ TESTING
CHEMICAL
ED ROOM
BLOWERS
BUILDING
AIR
LIME
INFLUENT

4. Ability of large aerated lagoon to dampen heavy intermittent surge flows and to dilute strong influent wastewater.

Disadvantages and observed operational difficulties of the present aerated lagoon treatment scheme include the following:

1. Maintenance responsibilities associated with manual sludge removal in the clarification zone.

2. Maintenance responsibilities associated with removal and replacement of sand in the filters after plugging occurs.

3. Uneven distribution of flow over entire sand filter surface.

4. Tremendous land requirements due to large size of unit processes.

Table 3-8 presents the average daily effluent flow and rainfall data for the aerated lagoon treatment plant from July 1989 through September 1990. The average monthly flow is 0.0319 mgd (31,900 gpd), and the average monthly rainfall is 3.66 inches for the 15-month period. Subtracting this average flow of 0.0319 mgd from the permitted flow of 0.0540 mgd, then an available plant capacity of 0.0228 mgd (22,800 gpd) could be assumed. However, during high groundwater periods with significant rainfall events (e.g., January through May), flows average about 0.041 mgd. The effective available plant capacity is therefore calculated to be 0.0117 mgd (11,700 gpd). To insure that a permit violation would not occur during a consecutive 3-month period, the available Aerated Lagoon WWTP capacity is considered to be 11,700 gpd. Of the permitted discharge of 54,000 gpd, 78.3 percent (42,300 gpd) is used, and 21.7 percent (11,700 gpd) could be considered as available.

TABLE 3-8
TOWN OF APPOMATTOX

AERATED LAGOON WASTEWATER TREATMENT PLANT
AVERAGE DAILY FLOW

Month/Year	Average Monthly Effluent Flow Per Day, mgd [a]	Total Monthly Rainfall, Inches
7/89	0.026	4.20
8/89	0.019	2.10
9/89	0.050	7.70
10/89	0.035	3.55
11/89	0.027	3.20
12/89	0.019	1.36 [b]
1/90	0.041	3.75
2/90	0.036	3.45
3/90	0.042	5.30
4/90	0.043	3.10
5/90	0.043	5.70
6/90	0.025	2.05
7/90	0.025	4.05
8/90	0.025	3.90
9/90	0.022	1.50
AVERAGE	0.0319 mgd	3.66 inches

[a] Permitted average monthly effluent flow is 0.054 mgd.

[b] A total rainfall calculated using snowfall of 11.5 inches, converted to 0.96 inch of rain, plus 0.4 inch of rainfall.

Performance of the Aerated Lagoon WWTP for the 15-month period of July 1989 through September 1990 is presented in Table 3-9. Average monthly BOD_5, TSS, and ammonia effluent concentrations for the period were 10.60 mg/l, 16.78 mg/l, and 1.41 mg/l, respectively. No monthly average effluent parameters were violated through the 15-month period. This reflects proper and responsible operation and maintenance of the plant by the operating staff to insure discharge permit compliance.

Trickling Filter WWTP: The Trickling Filter WWTP has a VPDES permit rated capacity of 0.17 mgd (170,000 gpd). The plant was constructed in the mid 1930's as a combination Imhoff tank-trickling filter plant and an upgrade was completed in 1988 to reflect the present day facility. Figure 3-5 depicts the flow diagram for this treatment plant. Flow first enters the plant through a gravity system where grit is removed in an aerated stilling area. This area also provides a pool of wastewater for lifting by two influent wastewater screw pumps. After the screw pumps, the wastewater flows by gravity through a fine bar screen with 1/4-inch openings or alternately through a manually cleaned bypass bar rack. Influent flow is then measured using a Parshall flume. Forward flow then enters dual rectangular primary clarifiers. Operating personnel must use valves to remove both sludge and scum to the aerobic digester.

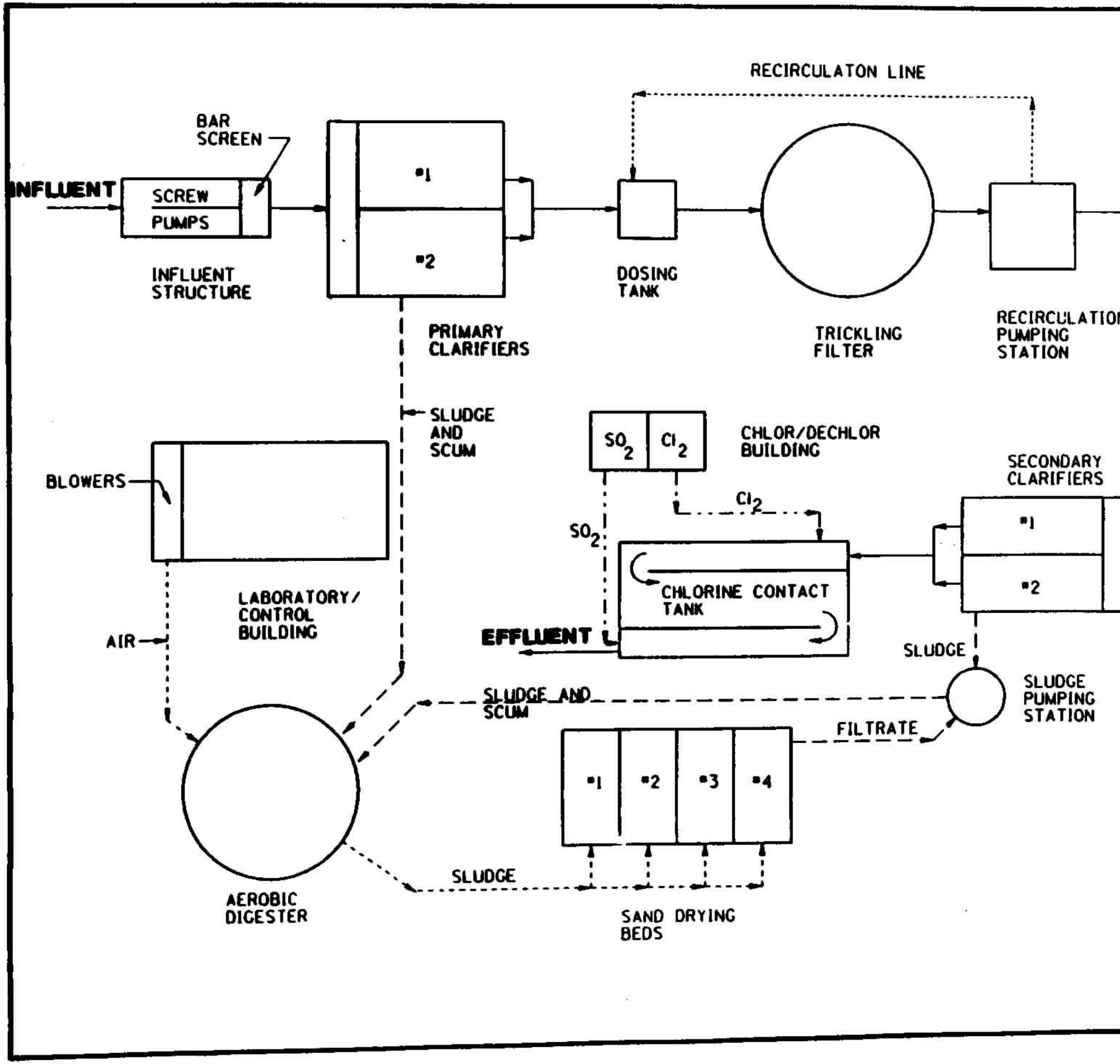
After the primary clarifiers, flow enters a dosing chamber which feeds a rock trickling filter. The trickling filter is the heart of the biological treatment system and provides for BOD_5 and ammonia removal. A recirculation pumping station equipped with two submersible pumps recirculates a percentage of flow to the dosing chamber for a second pass through the trickling filter. Typically, 100 percent of the flow is recirculated to allow for polishing of BOD_5 removal and, more importantly, for ammonia removal. After the recirculation pumping station, flow enters dual secondary clarifiers for removal of suspended solids. Sludge is wasted to the aerobic digester.

TABLE 3-9
TOWN OF APPOMATTOX

AERATED LAGOON WASTEWATER TREATMENT PLANT
AVERAGE MONTHLY EFFLUENT CONCENTRATIONS

Month/Year	BOD_5, mg/l	TSS, mg/l	Ammonia, mg/l [a]
7/89	6.10	5.80	0.50
8/89	3.90	3.20	0.70
9/89	6.90	10.50	0.50
10/89	20.40	35.60	--
11/89	13.20	23.00	--
12/89	12.40	19.10	--
1/90	19.10	14.00	--
2/90	17.03	42.60	--
3/90	11.30	32.90	--
4/90	6.20	4.40	5.40
5/90	10.40	12.20	3.20
6/90	12.40	15.10	0.50
7/90	5.50	6.70	0.90
8/90	4.20	7.70	0.50
9/90	9.90	18.90	0.50
AVERAGE	10.60	16.78	1.41
VPDES Limits	21 (Apr - Sept) 30 (Oct - Mar)	60.0	6.0

[a] Ammonia discharge limits apply for April through September.



TOWN OF APPOMATTOX
AND
APPOMATTOX COUNTY

COMPREHENSIVE WATER AND SEWER
MASTER PLAN

FIGURE 3-5

FLOW DIAGRAM
TRICKLING FILTER WWTP

NO SCALE

WILEY & WILSON
ARCHITECTS ENGINEERS PLANNERS

Following the secondary clarifiers, all wastewater is disinfected in the chlorine contact tanks, then dechlorinated to remove all residual chlorine prior to discharge, and the flow measured and recorded. The gravity outfall line travels approximately 1.5 miles prior to discharge to Caldwells Creek below Caldwells Pond. This discharge point, because of the increased drainage area, has an impact on the effluent discharge limits issued by the SWCB.

The sludge handling facilities include aerobic sludge digestion and four uncovered sand drying beds. The aerobic digester was created during the recent upgrade through renovation of the existing Imhoff Tank. From discussions with operating personnel, problems are ongoing in the digester. A dissolved oxygen concentration of between 0 and 0.4 mg/l is typical and is a result of limited air supply. This concentration should be between 1.0 and 2.0 mg/l. Support facilities for the trickling filter plant include a chlorination system; dechlorination system; and laboratory/control building housing blowers, lab, lime, soda ash, and polymer chemical feed facilities.

The advantages of this specific treatment scheme include the following:

1. Minimal operation attention for process operation.

2. No sludge recirculation system required from secondary clarifiers to biological treatment system.

3. Simple sludge digestion and disposal system.

4. Low sludge production from trickling filter process.

Disadvantages and observed operational difficulties of the present trickling filter treatment scheme include the following:

1. Insufficient downstream drop at the influent Parshall flume resulting in inaccurate flow data caused by backwater flow conditions.

2. Insufficient primary clarifier weir length and surface overflow rates under peak flow conditions.

3. Single trickling filter biological treatment unit.

4. Inability of trickling filter to absorb daily diurnal flows, heavy intermittent surge flows, or shock waste strength load.

5. Improper dissolved oxygen level in the aerobic digester.

6. Inability to dewater sludge during wet or poor weather conditions.

Table 3-10 presents the average daily effluent flow and rainfall data for the trickling filter treatment plant from July 1989 through September 1990. The average monthly flow was 0.128 mgd (128,000 gpd), and the average monthly rainfall was 3.68 inches for the 15-month period. Subtracting this average flow of 0.128 mgd from the permitted flow of 0.170 mgd, then an available plant capacity of 0.042 mgd (42,000 gpd) could be predicted.

Review of this 15-month period of flow data indicates that flow increases significantly during late winter, early spring of the year. This increase in flow is attributed to infiltration during this high groundwater period. The Town will begin construction of a sewer rehabilitation project in late 1990 or early 1991 to eliminate infiltration problems in substantial areas of the collection system associated with the Trickling Filter WWTP.

TABLE 3-10
TOWN OF APPOMATTOX

TRICKLING FILTER WASTEWATER TREATMENT PLANT
AVERAGE DAILY FLOW

Month/Year	Average Monthly Effluent Flow Per Day, mgd [a]	Total Monthly Rainfall, Inches
7/89	0.094	4.10
8/89	0.105	2.10
9/89	0.126	7.70
10/89	0.139	3.55
11/89	0.138	3.20
12/89	0.127	1.36 [b]
1/90	0.176	3.75
2/90	0.164	3.45
3/90	0.155	5.30
4/90	0.161	3.25
5/90	0.141	5.75
6/90	0.113	2.05
7/90	0.093	4.25
8/90	0.093	3.90
9/90	0.090	1.50
AVERAGE	0.128	3.68

(a) Permitted average monthly influent flow is 0.170 mgd.

(b) Total rainfall calculated using snowfall of 11.5 inches, converted to 0.96 inch of rain, plus 0.4 inch of rainfall.

Averaging wastewater flows during the high groundwater period of January through May yields daily flows per month of 0.159 mgd (159,000 gpd). This is an increase of 31,000 gpd over the 15-month average flow of 0.128 mgd (128,000 gpd). Assuming that the rehabilitation project will remove approximately one-half of this increased flow, or 15,000 gpd, then the plant flow can be calculated to be 144,000 gpd.

A flow rate of 144,000 gpd will insure that a permit violation would not occur during a consecutive 3-month period. Thus, the available trickling filter plant capacity is calculated to be 26,000 gpd. Of the permitted discharge of 170,000 gpd, 84.7 percent (144,000 gpd) is used and 15.3 percent (26,000 gpd) could be considered as available.

Performance of the Trickling Filter WWTP for the 15-month period of July 1989 through September 1990 is presented in Table 3-11. Average monthly BOD_5, TSS, and ammonia effluent concentrations for the period were 7.32 mg/l, 12.40 mg/l, and 0.77 mg/l, respectively. No monthly average effluent parameters were violated through the 15-month period. The plant performed as well as a tertiary, advanced wastewater facility without the need for any chemical addition. Again, this can be attributed to a responsible, skilled, and motivated operation and maintenance staff.

Infiltration/Inflow

Infiltration is defined as water that enters a wastewater collection system from the ground during periods of saturated soil and high groundwater. Causes of infiltration include defective pipes, pipe joints, and manholes. Inflow is defined as water that enters a wastewater collection through sources such as roof leaders, basement and area drains, manhole covers, cross connections between storm and sanitary sewer systems, catch basins, and cooling towers. Most inflow occurs during and immediately following a precipitation event.

TABLE 3-11
TOWN OF APPOMATTOX

TRICKLING FILTER WASTEWATER TREATMENT PLANT
AVERAGE MONTHLY EFFLUENT CONCENTRATIONS

Month/Year	BOD_5, mg/l	TSS, mg/l	Ammonia, mg/l [(a)]
7/89	8.00	8.98	0.65
8/89	5.20	7.90	0.63
9/89	6.60	7.80	0.74
10/89	7.70	8.60	--
11/89	8.30	16.10	--
12/89	8.90	14.30	--
1/90	9.60	19.10	--
2/90	8.50	15.20	--
3/90	7.20	13.60	--
4/90	8.00	17.90	1.00
5/90	8.60	20.20	0.80
6/90	7.20	15.10	0.90
7/90	6.10	8.20	0.90
8/90	5.40	6.10	0.70
9/90	4.50	6.90	0.60
AVERAGE	7.32	12.40	0.77
VPDES Limits	30.0	30.0	10.0

(a) **Ammonia discharge limits apply for April through September.**

Infiltration/Inflow (I/I) sources directly impact wastewater treatment facilities by reducing the capacity of the collection system and by reducing the ability for effective wastewater treatment. High flows induced by I/I sources could result in:

1. Treatment plant bypasses or overflows of untreated wastewater to surface waters.

2. Treatment process detention time reductions and subsequent VPDES violations.

3. Treatment process washout and reduction of treatment efficiency.

4. Treatment operating cost increases resulting from increased pumping, chemicals, labor, and associated operating and maintenance costs.

The SWCB has developed criteria for determining excessive I/I for all candidates who are seeking funds through the Virginia Revolving Loan Program. For a standard of measure, the SWCB criteria will be employed to define the relative I/I situation for the Town of Appomattox. Regardless of the excessive or nonexcessive ranking for I/I, the Town of Appomattox should continue to pursue methods for removal of undesirable groundwater and surface or stormwater entering the sanitary sewer system. The SWCB's definition of excessive I/I which is used for all Virginia Revolving Loan projects is that quantity of I/I which can be more cost effectively handled through elimination when compared to the cost of conveyance and treatment.

The I/I analysis will be based on the combined flows of both wastewater treatment plants for the 15-month period previously presented in Tables 3-8 and 3-10. Information that was analyzed for determining the quantity of I/I that exists in the sewer system included the following:

1. Recent flow data from each treatment facility.

2. Weather information for the time period corresponding to the flow data.

3. Existing service population.

4. Existing sewer system age, routing, materials of construction, and known physical condition.

To be considered as nonexcessive, the I/I analysis must positively conclude both of the following criteria:

1. For periods of high groundwater, the flow rate per capita must not exceed 125 gallons per capita per day (gpcd) based on the highest 7- through 14-day average.

2. For a single precipitation event, the total daily flow must not exceed 275 gpcd.

If both criteria are not satisfied, then further I/I analysis is required to determine if I/I is nonexcessive based on the cost of treatment and conveyance versus elimination of I/I. Otherwise, I/I is considered to be excessive.

Dry Weather Flow

To determine dry weather flow, a period of no significant rainfall was selected from months that historically experience low groundwater periods such as June through October. From review of daily flow records and rainfall, the period of September 16 through 29, 1990 was selected as a dry weather flow period. Table 3-12 summarizes the dry weather flows for each plant on a daily basis. The adjusted, average combined dry weather flow rate is 0.112 mgd (112,085 gpd).

To insure that a true dry weather, low groundwater period was selected, an analysis of the water records for users that receive sanitary sewer service was performed. Comparison of this analysis to historic flow data will provide a check for dry weather flows. The existing sanitary sewer user breakdown for both the Aerated Lagoon and Trickling Filter WWTP's are as follows:

1. Residential, consisting of 434 single family units and 114 multi-family units, totalling 548 connections.

2. Commercial, totalling 116 connections

The water use for these connections is defined in Table 3-13. The total equivalent residential connections, for both residential and commercial users is 1,079. Using the previously calculated 2.5 persons per household, the equivalent population connected to the wastewater treatment plants is 2,698 persons. The total estimated wastewater flow is 117,727 gpd.

TABLE 3-12
TOWN OF APPOMATTOX

AERATED LAGOON AND TRICKLING FILTER WASTEWATER TREATMENT PLANTS

DRY WEATHER FLOW
SEPTEMBER 16 - 29, 1990[a]

1. ACTUAL FLOW DATA

Date	Aerated Lagoon WWTP, MGD	Trickling Filter WWTP, MGD	Total Both Plants, MGD
9/16/90	0.020	0.081	0.101
9/17/90	0.012	0.078	0.090
9/18/90	0.025	0.097	0.122
9/19/90	0.027	0.098	0.125
9/20/90	0.018	0.092	0.110
9/21/90	0.024	0.093	0.117
9/22/90	0.025	0.097	0.122
9/23/90	0.031	0.088	0.119
9/24/90	0.023	0.071	0.094
9/25/90	0.016	0.093	0.109
9/26/90	0.017	0.090	0.107
9/27/90	0.018	0.086	0.104
9/28/90	0.018	0.085	0.103
9/29/90	0.020	0.077	0.097
AVERAGE	0.021	0.088	0.109
PERMIT	0.054	0.170	0.224

2. ADJUSTED FLOW DATA

Adjust the wastewater flow data to reflect water usage for the dry weather period to insure equivalent comparison. For the 13-month period, average water usage is 169,931 gpd and September 1990 average water usage is 165,120 gpd (refer to Table 3-3). This is a 2.83 percent reduction in water use. Thus, to accurately determine the impact of September 1990 wastewater flows, a wastewater flow increase of 2.83 percent is required.

Adjusted Flow	=	109,000 + 2.83%
	=	109,000 + 3,085
	=	112,085 gpd

[a] Low groundwater period with insignificant rainfall.

TABLE 3-13
TOWN OF APPOMATTOX

DRY WEATHER FLOW ANALYSIS
USING SEPTEMBER WATER CONSUMPTION DATA

User	Actual Connections	Equivalent Connections	Equivalent Population[a]	Water Consumption GPD	Percent Water Use Entering Wastewater Plant[b]	Wastewater Flow, GPD
Residential	548	548	1,370	71,706 [c]	0.75	53,780
Commercial	116	531 [b]	1,328	69,508 [c]	0.92	63,947
TOTAL	664	1,079	2,698	141,214		117,727

(a) Calculated using 2.5 persons/connection.

(b) Estimated percentage of water consumed per user that is returned to the wastewater treatment plant(s).

(c) Calculated using 134.6 gpcd and 1989 adjusted in Town commercial water user flow of 2,175,669 gallons/month.

(d) Calculated using 5,022,400 gallons water used for month of September 1990 divided by a total equivalent water system population of 3,155 which equals 52.34 gpcd. This per capita use of 52.34 gpcd is multiplied by the equivalent population to equal the water consumption.

To calculate and compare per capita usage, the equivalent population of 2,698 people will be divided into the per day wastewater flows as follows:

1. Historic flow data: gpcd = (112,085 gpd)/(2,698 people) = 41.54 gpcd

2. Calculated flow data: gpcd = (117,727 gpd)/(2,698 people) = 43.63 gpcd

The percentage of error in the historic verses the calculated flow data is 5 percent. This low and reasonable difference is acceptable due to meter inaccuracies as well as assumption inaccuracies. Thus, the dry weather flow for both wastewater treatment plants is 112,085 gpd or 41.54 gpcd.

<u>Wet Weather Flow</u>

Seasonal periods of the year, namely spring, can be associated with high groundwater tables. To determine the wet weather flow, the period of April 18 through April 28, 1990 was analyzed. Total rainfall through the period was nominal and totalled 0.5 inches. Table 3-14 summarizes the wet weather flows for each plant on a daily basis. The adjusted, average combined wet weather flow rate is 0.167 mgd (167,477 gpd).

The per capita usage is calculated using the adjusted wet weather flow and the equivalent population of 2,698 people as follows:

GPCD = (167,477 gpd)/(2,698 people) = 62.07 gpcd

<u>Inflow</u>

To determine flow during a heavy rainfall event, or inflow to the system, a heavy rain event was experienced on May 30, 1990.

TABLE 3-14
TOWN OF APPOMATTOX

AERATED LAGOON AND TRICKLING FILTER WASTEWATER TREATMENT PLANTS

WET WEATHER FLOW
APRIL 18 - 28, 1990 [a]

1. ACTUAL FLOW DATA

Date	Aerated Lagoon WWTP, MGD	Trickling Filter WWTP, MGD	Total Both Plants, MGD
4/18/90	0.027	0.150	0.177
4/19/90	0.022	0.146	0.168
4/20/90	0.023	0.132	0.155
4/21/90	0.025	0.140	0.165
4/22/90	0.041	0.152	0.193
4/23/90	0.030	0.124	0.154
4/24/90	0.057	0.138	0.195
4/25/90	0.028	0.138	0.166
4/26/90	0.024	0.129	0.153
4/27/90	0.022	0.118	0.140
4/28/90	0.028	0.134	0.162
AVERAGE	0.030	0.136	0.166
PERMIT	0.054	0.170	0.224

2. ADJUSTED FLOW DATA

Adjust the wastewater flow data to reflect water usage for the dry weather period to insure equivalent comparison. For the 13-month period, average water usage is 169,931 gpd and April 1990 average water usage is 168,417 gpd (refer to Table 3-3). This is a 0.89 percent reduction in water use. Thus, to accurately determine the impact of April 1990 wastewater flows, a wastewater flow increase of 0.89 percent is required.

Adjusted flow	=	166,000 + 0.89%
	=	166,000 + 1,477
	=	167,477 gpd

[a] Assumed high groundwater period with maximum rainfall of 0.5 inch on April 22, 1990.

The combined flow to the wastewater treatment plants was 439,000 gpd, with the aerated lagoon plant flow being 176,000 gpd and the trickling filter plant flow being 263,000 gpd. Using the equivalent population of 2,698 people, the per capita usage is as follows:

GPCD = (439,000 gpd)/(2,698 people) = 162.71 gpcd

Table 3-15 presents the quantity of I/I and a summary of the calculated per capita flows during dry weather, wet weather, and heavy rainfall events. Also, a comparison to the SWCB Virginia Revolving Loan Program I/I certification is presented. The conclusions of this comparison are as follows:

1. Both treatment plants experience I/I. During heavy rainfall events, inflow is appreciable.

2. Amount of I/I is "nonexcessive" when compared to the criteria established by the SWCB.

3. Although the I/I is "nonexcessive," this is not a substitute for an ongoing I/I abatement program.

TABLE 3-15
TOWN OF APPOMATTOX

I/I SUMMARY

1. **I/I QUANTITY**

A.	Wet Weather Flow	=	167,477 gpd
	Dry Weather Flow	=	- 112,085 gpd
	Infiltration	=	55,392 gpd
B.	Flow Rain Event	=	439,000 gpd
	Dry Weather Flow	=	- 112,085 gpd
	Infiltration/Inflow	=	326,915 gpd

2. **PER CAPITA I/I COMPARISON**

Flow Event	Actual or Experienced Usage, GPCD	SWCB Allowable Nonexcessive Usage, GPCD	Conclusion
Dry Weather	41.54	--	Nonexcessive
Wet Weather	62.07	125	Nonexcessive
During Heavy Rainfall	162.71	275	Nonexcessive

CHAPTER 4 - FUTURE SITUATION

POPULATION PROJECTION

To develop a 20-year population projection for the study area, the following sources were reviewed or contacted:

1. Central Virginia Planning District Commission

2. Virginia Statistical Abstract, 1989 Edition, Center for Public Service, University of Virginia, Charlottesville (formerly, the Tayloe Murphy Institute)

3. Virginia Employment Commission, Commonwealth of Virginia, April 1990

4. Appomattox Comprehensive Plan, prepared by the Appomattox Planning Commission with the assistance of the Central Virginia Planning District Commission, adopted 1988.

The available existing population projection information for the incorporated Town of Appomattox is from the 1989 Virginia Statistical Abstract and is as follows:

Year 1980	1,345	
Year 1982	1,378	(Growth per year = 1.219%)
Year 1984	1,424	(Growth per year = 1.655%)
Year 1986	1,470	(Growth per year = 1.602%)

Since the 1990 preliminary census data for the Town is available and presented in Chapter 3, these projections are not construed to be valid. From discussions with Town officials, the population of Appomattox is not expected to grow appreciably over the 20-year planning period. This is supported by existing land development, lack of large developable tracts of land, and corporate limit boundaries. Therefore, a growth rate of 1 percent per year is assumed to apply over the next 20 years beginning with the existing 1990 population of 1,703 people. Table 4-1 presents the population projections for the Town of Appomattox. Total population for the year 2010 is estimated to be 2,078 people.

Both the 1989 Virginia Statistical Abstract and 1990 Virginia Employment Commission have population projections for Appomattox County. Table 4-2 presents and compares the population projections from each source. These population projections reflect the entire County, including the Town, developing subdivisions, and rural areas. Since the water and sewer utilities provided by the Town of Appomattox are expected to be expanded into the portions of Appomattox County in the study area, it is safe to infer that growth will center around such services. This growth may exceed the average predicted County growth by a large margin, such as four times the average growth percentage. Therefore, the population projection for the portion of the study area located in Appomattox County will be based on an average annual growth rate of 2.5 percent per year. Table 4-3 summarizes the 20-year serviceable population for the portion of Appomattox County in the study area.

As reflected in Table 4-4, the total study area population for the year 2010 is 3,143 persons, consisting of 2,078 persons in the Town of Appomattox and 1,065 persons in the Appomattox County portion of the study area.

TABLE 4-1
TOWN OF APPOMATTOX

POPULATION PROJECTIONS
TO YEAR 2010

Year	Percent Growth Per Year	Population[a]
1990	1.0	1,703
1995	1.0	1,790
2000	1.0	1,881
2005	1.0	1,977
2010	1.0	2,078

(a) From 1990 Preliminary Census Data, includes recent annexed area population.

TABLE 4-3

RESIDENTIAL POPULATION PROJECTIONS TO YEAR 2010
PORTIONS OF APPOMATTOX COUNTY IN STUDY AREA

Year	Percent Growth Per Year	Population
1990	2.5	650[a]
1995	2.5	735
2000	2.5	832
2005	2.5	941
2010	--	1,065

[a] Refer to initial serviceable population presented in Chapter 3.

TABLE 4-4

RESIDENTIAL POPULATION PROJECTIONS TO YEAR 2010
STUDY AREA (TOWN AND COUNTY)

Year	Percent Growth Per Year	Population
1990	1.42	2,353
1995	1.45	2,525
2000	1.47	2,713
2005	1.50	2,918
2010	--	3,143

INDUSTRIAL SITES

Three potential industrial sites for the study area were identified during meetings with both Town and County officials. Specific criteria for water and sewer service for each site was identified. This criteria was developed to generally parallel the Virginia Certified Business Community Certification Program guidelines. Figure 4-1 indicates a general location of the sites. The water/sewer service requirements have been defined for each site as follows:

Site 1:	Water/Sewer =	20,000 gpd
Site 2:	Water/Sewer =	40,000 gpd
Site 3:	Water/Sewer =	20,000 gpd

REGULATIONS

Safe Drinking Water Act (SDWA)

The Safe Drinking Water Act was amended in 1986 and identifies 83 contaminants that the Environmental Protection Agency (EPA) will be required to regulate. An overview of the regulation of the 83 contaminants is presented in Appendix D. Of most significant impact to the Town of Appomattox is the Disinfection - Disinfection Byproduct (D-DBP) Rule which will set mandatory disinfection regulations for all public water supplies using groundwater sources. Additionally, monitoring or treatment of some or all of the 83 contaminants will be required.

EPA has not formally issued a final deadline for disinfection of all public water supplies using groundwater sources. However, from discussions with the Virginia Department of Health (VDH), EPA issued a "Strawman Rule" in April 1990 indicating disinfection of community water systems compliance by December 29, 1995. A "Strawman Rule" is defined as EPA's intentions that will be followed by a final adopted ruling either with or without amendments. All states will be required to adopt EPA's final rule. Another significant impact of the disinfection requirement is the kill of bacteria, viruses, and cysts. This, in some cases, may require that an adequate contact time be allowed before the water is consumed. Specific impact to the Town and County are unknown and will largely depend on how the final regulations are adopted by the VDH.

Virginia Waterworks Regulations

The Virginia Waterworks Regulations for potable water define two categories of chemical and physical water quality as follows:

1. Primary Contaminants - Substances involving public health

2. Secondary Contaminants - Substances involving aesthetics, staining from water, and taste/odor

Primary contaminants include both organic chemicals such as arsenic, lead, and mercury, and inorganic chemicals such as chlorinated hydrocarbons and trihalomethanes. The regulations define the primary maximum contaminant level for each substance in mg/l. Other primary contaminants include turbidity and radiological quality. Primary contaminants are regulated and enforced by VDH.

Secondary contaminants include inorganic chemicals such as chloride, iron, manganese, sodium, and zinc. Other secondary contaminants include color, odor, and pH. The regulations define the secondary maximum contaminant level for each substance in mg/l. Secondary contaminants are included in the regulations and recommended to be followed for aesthetic and taste/odor concerns. However, VDH has no legal power at the present time to enforce these secondary contaminant levels.

The most prevalent contaminants of concern for groundwater sources in the study area are iron and manganese. Secondary maximum contaminant levels for these contaminants are 0.3 mg/l and 0.05 mg/l, respectively. Although these contaminants are not enforced by the VDH, treatment for removal of excess contaminant concentrations is recommended to insure that aesthetic considerations, such as staining effects, taste, and odor impacts, are satisfied. This will minimize customer complaints and negative comments.

Virginia Pollutant Discharge Elimination System (VPDES)

As indicated in Chapter 3, both of the existing Town of Appomattox wastewater treatment facilities are authorized to discharge wastewater under VPDES permits. These permits are issued, regulated, and enforced by the Virginia State Water Control Board (SWCB). To increase the discharge capacity of the existing treatment works, a new VPDES permit must be issued through the SWCB. Such a permit modification would include a justification for increased flow and a public notice period to allow for public comment regarding changes in discharge conditions.

Discharge limits are set by the SWCB using various criteria relating to the point of discharge. Examples of the discharge criteria could include the assimilative capacity of the receiving waters (water quality limits); protection of receiving waters from excessive effluent concentrations; scenic river, recreational area, or shellfish area protection; and protection of receiving waters from enriched nutrients in the effluent. The discharge points for both existing wastewater treatment plants have been identified in Chapter 3.

The SWCB has indicated that receiving waters for both treatment plants are water quality limited. This, in turn, identifies a maximum quantity (i.e., pounds or kilograms per day) of BOD_5, total suspended solids, and ammonia that can be discharged. From discussions with the Roanoke office of the SWCB, the maximum discharge quantity of each effluent constituent can be assumed to be equal to the present permitted discharge limits. The limits are as follows:

Monthly Average Discharge Parameter	Aerated Lagoon WWTP	Trickling Filter WWTP
BOD_5, kg/d	4.3	19.3
TSS, kg/d	12.3	19.3
Ammonia, kg/d*	1.2	6.4

* Seasonal limits, April - September.

These effluent limits will be used to evaluate and identify treatment requirements. Using the monthly average discharge quantities, effluent concentrations can be calculated once an average monthly discharge flow is know.

To define the exact discharge limits for either facility, the Town of Appomattox must officially request a VPDES permit modification from the SWCB. The request will identify the revised design flow rate. Using this information, the SWCB will officially act on the request and define the effluent concentrations and quantities for each discharge parameter.

WATER

Future Water Needs

Chapter 3 defined the existing Equivalent Residential Connection (ERC) method of equating residential and commercial/industrial water users and indicated an existing water use of 4,095 gallons per month per ERC. Both of these criteria, in combination with the population projections, will be applied to predict the future water needs of the study area for the next 20 years. Residential water use will strictly rely on projected population. Commercial and industrial water use is somewhat more complicated and hard to predict.

The officials of both the Town of Appomattox and Appomattox County have been consulted to insure that all future needs are defined as accurately as possible.

Tables 4-5, 4-6, and 4-7 summarize the existing and future water requirements for the study area. Water requirements are broken down into three areas: Table 4-5, Existing 1990 Connections; Table 4-6, New Connections to Year 2000; and Table 4-7, New Connections - Year 2000-2010. The total water for consumption for each of the three time periods is 169,931 gpd, 99,564 gpd, and 93,584 gpd, respectively. The total water use for the year 2000 is projected to be 269,495 gpd, and for the year 2010 the projection is 363,079 gpd.

Flow Rates

To predict water supply needs, both the average daily, peak hour, and fire flow rates must be defined. Expansion of the existing system will take place over a period of time. Water demands and flow rates for proposed industrial sites will depend on industrial growth in the area. Furthermore, commercial growth along Route 460 West has been estimated; however, as utility service becomes more available, growth could exceed predictions. To insure accurate pipe sizing and system demand, a peak hour flow rate of three times the average daily flow will be utilized and compared with the required fire flow rates.

Table 4-8 presents a yearly summary of both average daily water consumption and peak hourly flow rates for the study area. Expansion of the existing water distribution system is represented on the basis of a 2 or more year development period. The predicted expansion is based on several assumptions, including full development of each new industrial site and expansion of the existing water system to existing residential and commercial establishments in the study area.

TABLE 4-5

WATER REQUIREMENTS FOR STUDY AREA

EXISTING 1990 CONNECTIONS

Location	Type	Water Use, GPD[a]
Town of Appomattox	635 Existing Residential Connections	85,490
	588 Existing Commercial ERC's[b]	79,163
Portion of Appomattox County in Study Area	39 Existing Residential Connections	5,251
Adjustment	--	27 [c]
	TOTAL	169,931 gpd[c]

(a) Calculated using 134.63 gpd (4,095 gallons/month) per ERC.

(b) Includes In- and Out-Town commercial ERC's.

(c) Adjusted to reflect water consumption of Table 3-3.

TABLE 4-6

FUTURE WATER REQUIREMENTS FOR STUDY AREA

NEW CONNECTIONS TO YEAR 2000

Location	Type	Water Use, GPD[a]
Town of Appomattox	50 Potential In-Town Residential Connections	6,732
	61 Potential In-Town Commerical ERC's	8,212
	New Industry	20,000 [c]
Portion of Appomattox County in Study Area	260 Serviceable Residential Connections	35,004
	Existing Industry[b]	6,000
	New Industry	20,000 [c]
	New Commercial, One Site: 26.9 ERC's[d]	3,616
TOTAL		99,564 gpd

(a) Calculated using 134.63 gpd (4,095 gallons/month) per ERC, unless otherwise noted.

(b) Represents Courtland Manufacturing on Route 460 West. Water use estimate obtained from Town of Appomattox officials.

(c) Refer to future industrial site discussion.

(d) Represents proposed restaurant. Water use estimate obtained from Town of Appomattox officials.

TABLE 4-7

FUTURE WATER REQUIREMENTS FOR STUDY AREA

NEW CONNECTIONS - YEAR 2000 - 2010

Location	Population	Type	Water Use GPD[a]
Town of Appomattox	375	150 Future Residential Connections	20,195
	--	39 Future Minor Commercial ERC's[b]	5,251
		New Industry	20,000 [c]
Portion of Appomattox County in Study Area	415	166 Future Residential Connections	22,349
	--	43 Future Minor commercial ERC's[b]	5,789
		New Industry	20,000 [c]
	TOTAL		93,584 gpd

[a] Calculated using 134.63 gpd (4,095 gallons/month) per ERC.

[b] Minor Commercial ERC calculated to be 26 percent of future residential connections based on present relationship of residential to minor ERC's.

[c] Refer to future industrial site discussion.

TABLE 4-8

SUMMARY OF AVERAGE DAILY WATER CONSUMPTION AND PEAK HOURLY FLOW RATES FOR THE STUDY AREA

Development Period	Item	Average Daily Flow, GPD	Peak Hour Flow, GPM
1990	Existing Residential Connections	90,768	189
	Existing Commercial Connections	79,163	165
	TOTAL[a]	169,931	354
1991-1993	100% Potential In-Town Residential Connections	6,732	14
	100% Potential In-town Commercial Connections	8,212	17
	New Industrial Site No. 1	20,000	42
	New Commercial Site	3,616	8
	TOTAL	208,491	435
1993-1995	34% Potential Out-town Serviceable Residential Connections	11,902	25
	New Industrial Site No. 2 (50%)	20,000	42
	TOTAL	240,393	502
1995-1997	33% Potential Out-Town Serviceable Residential Connections	11,551	24
	Existing Industry Connection	6,000	13
	TOTAL	257,944	539
1997-1999	33% Potential Out-Town Serviceable Residential Connections	11,551	24
	TOTAL	269,495	563
2000	New Industrial Site No. 3	20,000	42
	33% Study Area Growth Projections	17,683	36
	TOTAL	307,178	641
2005	Industrial Site No. 2 (50%)	20,000	42
	33% Study Area Growth Projections	17,683	36
	TOTAL	344,861	719
2010	34% Study Area Growth Projections	18,218	38
	TOTAL	363,079	757

(a) Includes adjustments.

Utilizing the water demands from Table 4-8 for the present and future years, actual water quantity requirements have been estimated. Table 4-9 presents the water quantities required for each development period defined in Table 4-8. Water quantity requirements reflect the historical 9.84 percent difference in water supplied to water billed. Although the water distribution system will age over the planning period, the 9.84 percent of unbilled water is assumed to remain unchanged. This low difference is attributed to the relative small size of the distribution system and the Town's historical record of good maintenance.

POTENTIAL WATER SOURCES

Having identified water demands for the planning period, potential water sources must be investigated to meet future needs. Three practical water supply sources have been identified for the study area as follows:

1. Purchase water from the City of Lynchburg
2. Surface water
3. Groundwater

Purchase Water From the City of Lynchburg

The City of Lynchburg presently sells water to surrounding localities including Bedford and Campbell Counties. The City of Lynchburg is interested in selling water to any of its adjacent or nearby communities. Since Campbell County is in the process of phasing out purchase of water from the City of Lynchburg, an available supply will exist within the next 1 to 2 years.

TABLE 4-9

SUMMARY OF AVERAGE DAILY WATER SOURCE SUPPLY FOR THE STUDY AREA

Development Period	Average Daily Water Consumption, GPD	Percent Increase of Consumption	Average Daily Water Source Supply, GPD
1990	169,931	9.84	186,652
1991-1993	208,491	9.84	229,007
1993-1995	240,393	9.84	264,048
1995-1997	257,944	9.84	283,326
1997-1999	269,495	9.84	296,013
2000	307,178	9.84	337,404
2005	344,861	9.84	378,795
2010	363,079	9.84	398,806

At the present time, the water distribution system for the City of Lynchburg extends to the Campbell County/City boundary line. A 12-inch line presently serves both residential and commercial users in the outskirts of the City limits. This line is fed by the "1005 Pressure Zone" from the storage tanks on Candler's Mountain with the tank overflow at elevation 1,005. The City of Lynchburg has indicated that an upcoming Capital Improvement Project (CIP) will provide a 16-inch water line in this Route 460/501 corridor from Candler's Mountain to the Campbell County line.

The cost of water purchased from the City of Lynchburg would be computed using the rates charged for water service within the City times a cost multiplier. For example, a contractual arrangement between Lynchburg and Bedford County specifies a cost multiplier of 2.35. City of Lynchburg personnel have indicated that this cost factor is variable and negotiable on a case-to-case basis. However, for the sake of this comparison, the 2.35 factor will be used.

Table 4-10 presents the water rates for the City of Lynchburg and for the users in the Appomattox study area. Also, Table 4-10 provides a comparison of these rates with the present water rates. For a typical single family connection (1 ERC) using an estimated 4,095 gallons/month of water, the monthly water bill would be $6.96 plus markup compared to the present bill of $6.08. This represents a cost increase of 14.5 percent for water only and excludes the debt service and annual operation and maintenance cost for the transmission and pumping systems.

The water transmission system from Lynchburg to Appomattox could be sized to deliver adequate water for average daily flow conditions considering economics and adequate velocities while minimizing pipe friction losses. Two booster pumping stations would be required to insure adequate pressure delivery.

TABLE 4-10

WATER RATES FOR WATER PURCHASE FROM CITY OF LYNCHBURG AND COMPARISON TO EXISTING 1990 RATES

1. MONTHLY WATER RATES IF PURCHASED FROM CITY OF LYNCHBURG

Usage Category in 1,000 Gal.	Water Rates City of Lynchburg in \$/1,000 Gal.	Cost Factor of Areas Outside of City Boundaries	Adjusted Water Rates For Study Area in \$/1,000 Gal.
0 - 22.44 (0 - 30)	1.47 (1.10)	2.35	3.45 (2.59)
23.19 - 523.60 (31 - 700)	0.90 (0.67)	2.35	2.12 (1.57)
Greater than 523.60 (Greater than 700.0)	0.70 (0.52)	2.35	1.65 (1.22)

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2. EXISTING 1990 MONTHLY WATER RATES FOR TOWN OF APPOMATTOX

Usage (Gallons)	Water Rates
0 - 2,000	\$3.25 (Flat Fee)
Greater than 2,000	\$1.35/1,000 Gallons

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3. COMPARISON OF EXISTING 1990 AND WATER PURCHASE RATES

Usage (1 ERC: 2.5 Persons per Home at 134.63 gpcd) = 4,095 gallons/month

Existing Water Rate = (\$3.25) + (4,095 - 2,000) (\$1.35/1,000) = \$6.08/month

Purchase Water Rate = (\$1.70/1,000) (4,095) = \$6.96/month

The water transmission line would follow Route 460 East for approximately 14 miles. Cost estimates will be presented later in this Section. Other cost considerations would include possible upgrading of the water system within the City of Lynchburg; however, these costs would likely be shared by both contracting parties. For the sake of this analysis, it will be assumed that the City of Lynchburg system is or will be of suitable size at the time of connection. Thus, no costs will be included to upgrade the City's water transmission system.

Surface Water Sources

No practical surface water source exists within the boundaries of the study area. However, two practical surface water sources can be identified in the general vicinity. These include a potential raw water reservoir east of Appomattox and the James River to the north. Both of these sources would require complete development and project commitment from beginning to end. This commitment includes source development, treatment, pumping, and transmission. Raw surface water quality for each source is an important consideration. The SDWA Amendments, previously discussed, revolve around finished water quality. Obviously, the better water quality entering a treatment works, the less treatment and cost incurred.

Raw Water Reservoir

To develop a raw water reservoir for domestic use on a stream of reasonable proximity to the Town of Appomattox would require a site with a minimum drainage area of 3-4 square miles. This could provide a yield of 1 mgd or more with a reservoir volume of about 300 acre-feet (approximately 25 surface acres).

A larger drainage area would require a smaller reservoir and vice versa. Streams such as the Appomattox River or some of its tributaries would be good potential candidates for a raw water reservoir site. Some of the Appomattox River tributaries to the north have a somewhat protected area for water quality since a major portion of their drainage areas are within the State forest lands.

Development of a raw water reservoir on the outskirts of the Appomattox - Buckingham State Forest, approximately 6 miles northeast of the Town corporate limits, would require a dam, reservoir, water treatment facility, pumping, and transmission facilities. The transmission main would run from the reservoir site to the large main at the Historical Park by the most expeditious route. The approximate length of the transmission main would be 4.0 miles plus or minus and the water level for the reservoir could be around elevation 630. Water quality is expected to be good.

James River

The James River runs along the northern boundary of Appomattox County, approximately 12.5 miles north of the Town corporate limits at Bent Creek. A water intake system, pumping and transmission facilities and water treatment plant would be required. The transmission main would run south for 12.5 miles following Route 26 to the 1-million gallon tank. The water level of the James River at Bent Creek is estimated to be elevation 410. Due to upstream industrialization along the James River, water quality is expected to be fair to poor. Although the process to treat surface water from the James River would be equal to that of a reservoir, the operation and maintenance costs would be greater due to the increased cost of chemicals and sludge disposal.

Groundwater Sources

As noted in Chapter 3, the Town uses groundwater for their present source of potable water supply and Figure 3-1 identified the existing well locations. Also, Chapter 3 indicated that a hydrogeologic analysis (Appendix A) has been prepared as a part of this study by the consulting firm of Hatcher-Sayer, Inc. Refer to Appendix A for specific information on future wells, including recommended locations, ranked in order of predicted yield and water quality.

Three areas were identified to provide an additional water supply for the study area. Within these three areas, nine well sites were selected. Based on existing well data within the general study area and vicinity, the well yields could be predicted to range between 70 and 150 gpm. Water quality relating to iron and manganese concentrations for the nine well sites is unknown; however, several points of consideration are as follows:

1. Conversations with Mr. Fred Jones, Appomattox Well Drilling Company, indicate that water quality in all three areas could be poor; i.e., the presence of iron and manganese could be higher than the standards set by the Virginia Department of Health.

2. There are no known high production wells within Area 1; thus, no historic data can be reviewed or compared.

3. Drilling depth seems to impact the water quality.

4. Statistically, wells in and around the study area produce good quality water; i.e., above the Virginia Department of Health standards for iron and manganese.

For the sake of this analysis, water quality is assumed to be good. Thus, no physical/chemical treatment of the water to remove iron and manganese will be included in any cost analysis. However, chlorination of the water supply, including adequate contact time, will be addressed for future wells.

To compare the cost of continuing to use groundwater as the source of potable water for the study area, a yield of 100 gpm will be assumed for all future wells. Therefore, all cost estimates will be developed using this 100 gpm yield. Using the 20-year water production needs of 398,806 gpd previously presented in Table 4-9, the quantity of future wells was estimated to be two wells. This is based on a total existing well capacity of 355 gpm plus an average production of 100 gpm per well, pumped for 12 hours per day. If the wells were pumped for only 10 hours per day the total well yield would need to be 616 gpm, and three new wells would be required. The two new wells would be located in Area 1 as presented in Figure 7 of the Hatcher-Sayer Report. Each well would be drilled in a location to minimize their influence on each other. The transmission main from each well would be routed from the new site due east until intersecting with Route 26. Then, the main would follow the most direct route to tie into the 12-inch main feeding the 1-million gallon tank.

Cost Estimates

Before proceeding with the water distribution system layout and analysis for the planning period, the viable water source should be defined. Assuming the water source can provide sufficient water quantities, capital costs and operation and maintenance (O&M) costs were developed for each water source.

Capital and O&M costs were estimated using historical construction cost data, F. W. Means construction cost data, and past O&M cost data from typical operating facilities. All costs are in January 1, 1991 dollars. Operation and maintenance costs were estimated based on an average daily design water production of 398,806 gpd. No costs are included for water system billing and administrative supplies. Replacement and salvage value costs for facilities have not been included. The actual useful life for many facilities (e.g., pipe lines and concrete structures) extends beyond the planning period while the useful life of equipment (e.g., pumps and water treatment equipment) equals the 20-year planning period. All estimates include Contractor Overhead and Profit, Contingency, Engineering, Surveying, Legal, and related costs. Land cost is estimated to be $5,000 per acre.

Table 4-11 presents the capital and O&M costs for the water supply sources through the year 2010. Water supplied from groundwater using wells has the lowest capital and O&M costs, $515,200 and $35,700, respectively.

Present Worth Analysis

A present worth analysis was performed on each of the water supply sources to compare and choose the most cost effective alternative for the planning area. A discount rate of 8-7/8 percent was used to develop the total present worth. This discount rate was used to calculate the present worth of annual operation and maintenance costs. Salvage values for buildings, pipes, concrete structures, equipment, and land were not evaluated in the present worth analysis. Table 4-12 presents the 20-year present worth analysis for each water supply source. Water supplied from groundwater using wells has the total present worth, including capital costs plus annual operating expenses of $844,000.

Fire Flow Requirements

Fire flow requirements are based on two parameters, rate of flow and duration of fire. Residential fire flow requirements are 500 gpm. However, fire flow requirements for the principal business or high-value district for the study area was based on population projections for various years in the planning period identified in Tables 4-1 and 4-3. Corresponding fire flows for the principal business district are presented in Table 4-13, based on American Water Works Association and National Board of Fire Underwriters recommendations. The distribution system should sustain a minimum system pressure of 20 psi during a peak day flow for all fire flow conditions.

TABLE 4-11

CAPITAL AND ANNUAL O&M COSTS FOR WATER SUPPLY SOURCES THROUGH YEAR 2010 (IN JANUARY 1991 DOLLARS)

Water Supply Alternative	Capital Costs	O&M Costs[(a)]
Purchase of Water From City of Lynchburg	$8,245,000 [(b)]	$400,000 [(c)]
Surface Water Using Raw Water Reservoir	6,885,000 [(d)]	134,100
Surface Water Using James River	8,777,000 [(e)]	163,800
Groundwater Using Wells	515,200 [(f)]	35,700

(a) Includes all applicable labor, power, chemical, maintenance, sludge disposal, repair, and material costs.

(b) Includes 14 miles of water line, two booster pumping stations, land, and all support systems and appurtenances.

(c) Includes annual cost for water purchase.

(d) Includes dam, reservoir, water treatment plant, booster pumping station, land, 4 plus or minus miles of water line, and all support systems and appurtenances.

(e) Includes river intake facility, water treatment plant, booster pumping station, land, 12.5 miles of water line, and all support systems and appurtenances.

(f) Includes two production wells with well houses (space included for future addition of chlorination equipment), land, 0.89 mile of water line, and all support systems and appurtenances.

TABLE 4-12

PRESENT WORTH ANALYSIS OF WATER SUPPLY SOURCES

Item	Purchase of Water from City of Lynchburg	Raw Water Reservoir	James River	Groundwater Using Wells
Capital Cost[(a)]	$8,245,000	$6,885,000	$8,777,000	$515,200
O&M Cost[(a)]	400,000	134,100	163,800	35,700
Present Worth O&M[(b)]	3,684,160	1,235,100	1,508,700	328,800
TOTAL PRESENT WORTH[(c)]	$11,929,160	$8,120,100	$10,285,700	$844,000

(a) Total costs presented in Table 4-10; January 1, 1991 dollars.

(b) Present worth factor at 8-7/8 percent interest for 20 years equals 9.2104.

(c) Total Present Worth equals (Capital Cost) plus (Present Worth O&M Costs).

TABLE 4-13

FIRE FLOW REQUIREMENTS FOR PRINCIPAL BUSINESS DISTRICTS IN THE STUDY AREA

Year	Population[a]	Recommended Fire Flow, GPM	Recommended Duration, Hour
1990	2,353	1,580	6.35
1995	2,525	1,630	6.50
2000	2,713	1,675	6.70
2005	2,918	1,725	6.90
2010	3,143	1,790	7.20

(a) Populations from Town of Appomattox and portions of Appomattox County in study area, refer to Tables 4-1 and 4-3, respectively.

System Layout, Analysis, and Storage Requirements

Figure 4-2 presents the future water distribution system for the study area through the year 2010. Two new water supply wells are proposed in the area predicted to have the highest yields. Groundwater will be pumped from the two wells to the 1-million gallon storage tank and existing water distribution system. Water distribution system expansions in residential areas were sized without the computer model. Main lines and lines with the greatest potential for expansion were selected to be 8 inches. Other dead-end or low volume lines are 6 inches. For these residential areas, a fire flow of 500 gpm to 1,000 gpm is adequate.

Industrial Sites No. 1 and No. 3 are located within the Appomattox corporate limits and are on the high zone. Each site is adjacent to existing large diameter water mains which provide daily peak and fire flow requirements. The computer model of the existing distribution system indicates that 3,500 gpm at 20 psi is available for either area.

To analyze future development along the Route 460 corridor, the computer model for the existing water distribution system was utilized. Before beginning the analysis, the model was revised to reflect the recommendations for upgrading the existing water distribution system as defined in Chapter 3. Using water systems demands for the planning period, previously indicated in Table 4-9, line sizes were selected. The line from the History Junction Shopping Center to Industrial Site No. 2 will be 12 inches and will provide up to 1,500 gpm at 20 psi.

The line from Industrial Site No. 2, continuing west on Route 460 to Spout Spring, was sized to be 12 inches in diameter. However, only 800 gpm at 20 psi was predicted to be available in Spout Spring. Required minimum flow is estimated to be 1,000 gpm. Therefore, to provide maximum system performance and to provide adequate fire protection, a water storage tank is required. Due to the elevation of the Spout Spring community, a separate "higher" pressure zone and pumping station is required. Thus, the line from Industrial Site No. 2 to the new tank will be 8 inches. The tank capacity is estimated to be 250,000 gallons. Location of the tank and pumping station is shown in Figure 4-2.

COST ESTIMATES AND IMPLEMENTATION SCHEDULE

Construction cost estimates for expansion of the water distribution system were based on historical construction cost data and F. W. Means construction cost data. All costs are in January 1, 1991 dollars. All estimates include Contractor Overhead and Profit, Contingency, Engineering, Surveying, Legal, and related costs. Table 4-14 presents the estimated construction costs of the water distribution system expansion for the development periods identified in Table 4-8. The implementation of each development period is predicted to be required before the end of that specific period. Thus, Table 4-14 further serves as the implementation schedule.

SEWER

Future Wastewater Needs

Chapter 3 describes the existing conditions for the wastewater collection and treatment systems. Future wastewater needs are predicted for the serviceable population of the study area for the next 20 years. Using the ERC method, the dry weather wastewater flow rate (gpcd), the estimated I/I, and the projected 20-year water needs have been previously defined.

Consistent with the predicted water supply needs, expansion of the existing wastewater collection system will take place over a period of time. Total water use for the year 2010 is projected to be 363,079 gpd.

TABLE 4-14

CONSTRUCTION COST ESTIMATES AND IMPLEMENTATION SCHEDULE FOR WATER DISTRIBUTION SYSTEM (IN JANUARY 1, 1991 DOLLARS)

Development Period	Item	Estimated Construction Cost
1991-1993[a]	100%In-Town Residential and Commercial Connections, New Industrial Site No. 1, New Commercial Site	$ 733,600
1993-1995	34% Potential Out-Town Serviceable Residential Connections, New Industrial Site No. 2	$1,941,100
1995-1997	33% Potential Out-town Serviceable Residential Connections, Existing Industrial Connection	$1,450,700
1997-1999	33% Potential Out-Town Serviceable Residential Connections[b]	$1,780,000
2000	New Industrial Site No. 3, 33% Study Area Growth Projections	$28,100
2005	33% Study Area Growth Projections	--[c]
2010	33% Study Area Growth Projections	--[c]

[a] Assumes upgrade of existing water distribution system, Priorities 1-5 in Chapter 3 complete.

[b] Includes Pumping Station and Water Storage Tank at Spout Spring.

[c] Water distribution system assumed to be in place and single connection required by user.

Wastewater Treatment Plant Design Flows

As previously discussed, the VPDES permit for each plant defines effluent discharge quantities (in kilograms per day) for BOD_5, total suspended solids, and ammonia (April through September). To define flows to each plant for the 20-year planning period, the total combined wastewater flow for the serviceable area was estimated. Table 4-15, Part 1 presents the total design flow for the serviceable population in the planning area for the year 2010 to be 355,100 gpd. This consists of approximately 299,700 gpd residential, commercial, and industrial flow plus 55,400 gpd of infiltration. The quantity of infiltration defined for the planning period is equal to that presently existing. However, the quantity of future I/I is predicted to be less than presently exists due to continuing I/I rehabilitation efforts by the Town.

In addition, Part 2 of Table 4-15 summarizes the estimated effluent concentrations and calculated flows to each wastewater treatment plant for the year 2010. Consideration was given to eliminating the Aerated Lagoon Plant in favor of the Trickling Filter Plant (TFP) only. However, this scenario proved to force the discharge concentration below a reasonable level for the TFP. Thus, design flow for each plant was calculated by equating BOD_5 discharge concentrations. Design flow for the Aerated Lagoon WWTP is calculated to be 64,700 gpd, and design flow to the Trickling Filter WWTP is calculated to 290,400 gpd.

Using these calculated design flows for the year 2010, the required wastewater treatment plant expansions are summarized as follows:

TABLE 4-15

WASTEWATER TREATMENT PLANT DESIGN FLOWS TO YEAR 2010

1. TOTAL COMBINED WASTEWATER FLOW TO YEAR 2010

Type	Water Use, gpd[a]	ERC[b]	Population Per ERC	Wastewater Production in GPCD[c]	Infiltration GPD	Design Wastewater Flow, GPD[d]
Residential	175,048	1,300	2.5	41.54	55,392	190,397
Commercial	102,031	758	2.5	41.54	--	78,718
Industrial	86,000	650	--	--	--	86,000
Total	363,079	2,708			55,392	355,115

Use Q (Design) = 355,100 gpd

2. SUMMARY OF CALCULATED WASTEWATER FLOWS AND ESTIMATED EFFLUENT CONCENTRATIONS FOR AERATED LAGOON AND TRICKLING FILTER WWTP'S.

Item	Aerated Lagoon WWTP	Trickling Filter WWTP
Flow, gpd[e]	64,700	290,400
BOD_5, mg/l	17.5	17.5
Total Suspended Solids, mg/l	50.1	17.5
Ammonia (April-September), mg/l	4.9	5.8

(a) Refer to Tables 4-5, 4-6, and 4-7.
(b) Number of ERC's calculated using water consumption of 134.63 gpd.
(c) Dry weather wastewater production calculated in Chapter 3.
(d) Calculated using ERC's times population times wastewater production plus infiltration.
(f) Calculated by assigning BOD_5 concentrations for each plant and maximum effluent quantities.

Flow Condition	Aerated Lagoon WWTP, gpd	Trickling Filter WWTP, gpd
Future Design, Year 2010	64,700	290,400
Present Design, Year 1990	54,000	170,000
Minimum Required Expansion	10,700 gpd	120,400 gpd

Table 4-16 prevents a summary of daily wastewater flows for the study area based on the development periods established previously for expansion of the water distribution system. The present day wastewater to water flow factor of 0.7714 (calculated from 103.85 gpd of wastewater per ERC divided by 134.63 gpd of water per ERC) was used to predict these wastewater flows.

For preliminary hydraulic design of the gravity collection system, pumping station/force mains, and wastewater treatment facilities, a peaking factor of 3.0 was used. This allows the collection, pumping, and treatment systems to adequately handle combined diurnal and I/I flows during heavy rainfall events, with no system surcharges, backups, or overflows.

Wastewater Collection System Layout and Analysis

Proposed wastewater collection system components, including gravity collector sewers, gravity interceptors, pumping stations, and force mains, must be designed and constructed in accordance with the Virginia Department of Health Sewerage Regulations. Pertinent criteria for these items are as follows:

TABLE 4-16

SUMMARY OF DAILY WASTEWATER FLOWS FOR THE STUDY AREA FOR TIME PERIODS

Development Period	Average Daily Residential and Commercial Water Consumption GPD[a]	Average Daily New Industrial Water Consumption GPD[b]	I/I, GPD	Average Daily Wastewater Flows, GPD[c]
1990	169,931	--	55,392	
1991-1993	188,491	20,000	55,392	220,800
1993-1995	200,393	40,000	55,392	250,000
1995-1997	211,944	46,000	55,392	264,900
1997-1999	223,495	46,000	55,392	273,800
2000	241,178	66,000	55,392	307,400
2005	258,861	86,000	55,392	341,100
2010	277,079	86,000	55,392	355,100

(a) Refer to Table 4-6.

(b) Assumes 100 percent of water consumed returned as wastewater.

(c) Calculated using average daily water consumption times wastewater to water flow factor of 0.7714 plus I/I, then rounded to the nearest 100 gpd.

1. Gravity collector sewers and interceptors:

Pipe Size	Minimum Slope Feet/100 Feet	Maximum Manhole Spacing, Feet
8 inches	0.40	400
10 inches	0.28	400
12 inches	0.22	400
15 inches	0.15	400
18 inches	0.12	500

2. Pumping Stations: Minimum of two pumps required; capable of passing a 2-inch sphere or grinder pumps required.

3. Force Mains: Minimum 4 inches in diameter (except 2 inches for grinder pumps); velocity, 2 feet per second (minimum) to 8 feet per second (maximum).

Table 4-17 presents the rate of flow and the estimated ERC's that can be served by the sewer sizes presented. Using estimated flows, piping sizes were selected. Generally, 8-inch gravity sewers adequately serve much of the area; however, main trunk sewers or interceptors are larger in diameter.

Figure 4-3 presents the expansion of the wastewater collection system including gravity lines and wastewater pumping stations. System expansion will occur to meet the development needs of areas. Therefore, the areas delineated for construction in a specific development period are for preliminary and subject to change based on specific need. The wastewater collection system expansion is described in the following paragraphs. Pumping station

TABLE 4-17

MAXIMUM FLOW RATES AND ERC's FOR GRAVITY COLLECTION SYSTEM

Pipe Size, Inches	Minimum Slope Percent	Flow Rate, GPD[(a)] at Minimum Slope	Flow Rate Per ERC, GPD[(b)]	ERC's
8	0.4	493,920	311.55	1,585
10	0.28	748,800	311.55	2,403
12	0.22	1,080,000	311.55	3,467
15	0.15	1,617,120	311.55	5,191
18	0.12	2.351,520	311.55	7,548

(a) Calculated using Mannings Equation with a coefficient of friction ("n" factor) of 0.013 and velocity of 2 fps at full flow.

(b) Calculated using 103.85 gpd/ERC times peaking factor of 3.

numbering begins with No. 4 to account for the three existing Town owned, operated, and maintained pumping stations.

1991-1993 Expansion

The 1991-1993 expansion provides wastewater service to all existing residential and commercial users within the corporate limits, especially in the recently annexed area. The existing 12-inch interceptor from the intersection of Morris Avenue and Dogwood Street to the Trickling Filter WWTP will require replacement with a 12-inch or 15-inch line depending on the available capacity at the existing slope. This interceptor was repaired around 1988 to minimize infiltration; however, the effective internal diameter of the line was reduced to 10 inches or less.

Also, this expansion period would provide wastewater service to Industrial Site No. 1, including the adjacent residential and commercial connections. This area would be served by the Plain Run Branch Pumping Station. This pumping station and force main is presently in the design stages to serve the Appomattox Court House National Historical Park. The National Park Service will then abandon their existing on-site wastewater lagoon. The Plain Run Branch Pumping Station, based on the future expansions documented in this Study, should be sized to accommodate the National Park Service, Industrial Site No. 1, residential and commercial connections along Route 24, the Meadowlark subdivision, and eventually Industrial Site No. 3.

The force main from the Plain Run Branch Pumping Station, as presently proposed, will empty into the gravity collection system serving the Aerated Lagoon WWTP; refer to Figure 4-3. However, strong consideration should be given to rerouting this discharge to the Trickling Filter WWTP collection system. The Plain Run Branch Pumping Station will discharge in excess of 40,000 gpd at design capacity. This will exceed the flow potentials of the expanded Aerated Lagoon as will be justified during the discussion for WWTP expansions.

1993-1995 Expansion

The 1993-1995 expansion provides service to Industrial Site No. 2 and the existing manufacturing facility adjacent to Route 460 and Route 613 (Police Tower Road) and adjacent residential and commercial development. Based on topography in this area, gravity sewers along Route 460 ultimately terminate at a proposed Pumping Station No. 4 probably in the vicinity of Route 460 and the Moose Lodge Road. Residential and commercial users north of the Moose Lodge/Route 460 intersection could be served with gravity sewers which would terminate at Pumping Station No. 5; refer to Figure 4-3 as required. Pumping Station No. 4 would be designed to accommodate the estimated initial flow from this area and to accommodate future expansion west of the intersection of Route 460 and Route 613 (Police Tower Road). The force main is sized to accommodate both proposed and future flows. Discharge would be to the gravity collection system near Armstrong Furniture which is served by the Trickling Filter WWTP. The existing 8-inch gravity system receiving this discharge is predicted to be approaching capacity during peak flow periods and will require replacement before adding an area west of Pumping Station No. 4 service area.

Also, as part of this expansion of the wastewater collection system, new service could be provided in the eastern end of the study area along Route 460. With the proposed Route 460 bypass and the provision of utility service, this area will eventually grow. The expected flow from this area is predicted to be in the range of the Aerated lagoon WWTP upgraded capacity and, therefore, will go to the Aerated Lagoon WWTP. Pumping Station No. 6, located in the vicinity of the intersections of the proposed Bypass and existing Route 460, will serve the area.

1995-1997 Expansion

The 1995-1997 expansion provides service along Routes 691, 719, and 641 including all subdivisions and developments. Two separate Pumping Stations, Nos. 7 and 8, are required. Pumping Station No. 7 would act as a lift station to serve a smaller percentage of residential users, and Pumping Station No. 8 would pump directly to the Trickling Filter WWTP.

Also, this expansion provides service east of the Appomattox High School and along Route 635. This area can be served by extending the existing gravity sewer system. Flow will go to the Trickling Filter WWTP.

1997-1999 Expansion

The 1997-1999 expansion provides service along Route 460 and Route 689 to serve Spout Spring and vicinity. A gravity collection system and Pumping Station No. 9 serves a residential area between Routes 703 and 708. The force main empties into the collection system serving Pumping Station No. 10. The remaining area consists of a gravity sewer network feeding Pumping Station No. 10 approximately 1,000 feet west of Route 703. However, as a detailed design is initiated, a second pumping station may be necessary along either Route 460 or Route 689. The force main from Pumping Station No. 10 would supply Pumping Station No. 4 for subsequent pumping to the Trickling Filter WWTP wastewater collection system. Due to the increased flow in the existing system, the existing 8-inch collection system on Route 460 near Armstrong Furniture must be upgraded to a 12-inch gravity line. This will insure an adequate size for existing and future use to the year 2010 and beyond; refer to Figure 4-3.

2000, 2005, and 2010 Expansions

The 2000 expansion provides the addition of a gravity collector extension to serve Industrial Site No. 3. As previously discussed, this flow would be discharged to Plain Run Branch Pumping Station. Also, the 2000 expansion provides for connection to the existing wastewater collection system resulting in growth within the study area.

The 2005 and 2010 expansions also provide for wastewater connections to the wastewater collection system that are necessary from growth within the study area. No sewer extensions or pumping stations are included in this evaluation due to insufficient information. As the system and area grows, reevaluation will be necessary.

Pumping Stations

Each of the expansion areas previously described require pumping stations and force mains. From an efficient operation and cost effective standpoint, submersible pumping stations are recommended. These utilize two (or more if required) submersible pumps in a precast concrete manhole with external valve vault. Examples of the system proposed include the Town's pumping stations for both the History Junction and Shoppes of Appomattox Shopping Centers. However, design of Pumping Station No. 4 must allow for future expansion.

For future expansion, pump replacement or the addition of one or more pumps is a viable solution. Due to the unknown time frame between the original pumping station installation and the requirement for an upgrade, pump replacement option will be used to estimate costs. For simple level control operation of the pumps, float switches could be used. However, due to the distance of the stations from a manned facility, a telemetry system such as automatic telephone dialers are necessary to maintain system reliability.

The Sewerage Regulations state that the objective of reliability is to prevent the discharge of raw or partially treated sewage to any waters and to protect public health by preventing backup of sewage and subsequent discharge to basements, streets, and other public and private property. Also, provisions for continuous operability of pumping stations is generally required. Since each pumping station will receive flow from connections that are on a public, continuous water supply, the Virginia Department of Health will likely assign a Class I reliability. This requires pump operation at all times. Therefore, a standby generator is included for each municipal pumping station proposed except for Pumping Stations No. 5 and 9.

Force Mains

The force main from each wastewater pumping station will be pressure type polyvinyl chloride (PVC), AWWA C900, push-on joint, with cast or ductile iron fittings. However, in paved, heavy traffic areas, major road crossings, railroad crossings, and stream crossings, coated and lined ductile iron pipe will be used. Also, a steel casing pipe will be included for all major road and all railroad crossings. Minimum cover for all force mains should be 3 feet. Bends should be minimized and restrained or anchored. Air/vacuum relief valves will be included at the necessary high points to relieve air locking (air relief) and to allow air to enter to prevent pipe collapse (vacuum relief).

Wastewater Treatment Facilities

Table 4-18 presents a flow summary and expansion schedule for each treatment plant for the year 2010, including the available capacity identified in Chapter 3 and the wastewater treatment plant flows previously discussed in this Chapter. Expansion of the Aerated Lagoon Plant will depend on growth in the defined service area; however, completion of the single system expansion should be completed prior to the year 1995. Expansion of the Trickling Filter WWTP would consist of two phases. Phase One should be completed in

TABLE 4-18

WASTEWATER FLOW SUMMARY AND WWTP EXPANSION SCHEDULE FOR THE YEAR 2010

1. WASTEWATER FLOW SUMMARY

Plant	1990 Design Flow, GPD	Estimated 1990 Available Capacity, GPD	Required 2010 Expansion Capacity, GPD
Aerated Lagoon	54,000	11,700	10,700
Trickling Filter	170,000	26,000	120,400

2. WWTP EXPANSION SCEHDULE

Plant	Expansion One Capacity, GPD	Total Plant Flow, GPD	Year Required	Expansion Two Capacity, GPD	Total Plant Flow, GPD	Year Required
Aerated Lagoon[a]	10,700	64,700	1995	--	--	--
Trickling Filter[b]	60,200	230,200	1992	60,200	290,400	1997 or beyond

(a) One plant expansion is adequate and would be most cost effective for the Aerated Lagoon WWTP.

(b) A two-phase plant expansion is deemed to be most cost effective for the Trickling Filter WWTP. The two-phase expansion allows maximum flexibility and conserves capital investment in the event the predicted growth rate exceeds actual growth conditions.

1992 to allow for immediate residential, commercial, and industrial growth (specifically Industrial Site No. 1). The completion of Phase Two is dependent on growth in the defined areas. However, expansion could be needed before the end of 1997.

Aerated Lagoon WWTP

The single phase expansion of the Aerated Lagoon WWTP involves a design flow increase to 64,700 gpd and a decrease in effluent discharge concentrations predicted to be 17.5 mg/l BOD_5, 50.1 mg/l TSS, and 4.9 mg/l Ammonia (April - September). These decreased effluent concentrations will require increased treatment responsibilities. Table 4-19 presents plant upgrade criteria for each unit process at the Aerated Lagoon WWTP. No modifications are proposed for the aerated lagoon and quiescent zone size. However, additional diffusers to double the existing aeration capacity will be added proportionally in each of the three cells. One additional blower is required and would be interconnected with the two existing blowers. Normal operation will utilize two blowers operating and one standby. A separate air feed line would provide air to the new diffusers to avoid replacement of the existing air line and to maximize flexibility.

Upgrade of the existing sand filters is recommended to include hydraulically operated rotary distribution arms, similar to distributors used for trickling filters. Conversation with the Danville office of the Virginia Department of Health (VDH) confirmed the previously noted problem of flow distribution over the entire surface of each sand filter. This issue has not been pursued by VDH since the existing plant is operating well within the permitted limits. The existing dosing siphons will need to be analyzed in greater detail during design to insure minimal headloss for operation of the distributors. No renovations to the dosing siphons are expected.

TABLE 4-19

UPGRADE CRITERIA FOR AERATED LAGOON WWTP WITH DESIGN FLOW OF 64,700 GPD

Unit/Process	Regulation Requirements	Existing Conditions at 54,000 GPD	Upgrade Conditions at 64,700 GPD
Flow:			
Average Daily	--	54,000 gpd	64,700 gpd
Maximum Daily	--	162,000 gpd	194,100 gpd
Aerated Lagoon:			
Detention Time	20 days plus 10 percent volume increase for sludge storage	54 days	45 days
Aeration	2 pounds O_2/pound BOD_5 Applied	108 pounds/day	260 pounds/day
Blowers	--	2 at 105 scfm (1 operating, 1 standby)	3 at 105 scfm (2 operating, 1 standby)
Final Clarification (Quiescent zone integral with lagoon)	1.5 hour detention time	1 day	0.91 day
Sand Filters:			
Flooded Design	2.3 gal/sf/day	(Total A = 23,100 sf) 2.34 gal/sf/day	--
Rotary Distributor Design	3.5 gal/sf/day	--	(Total A = 23,100 sf) 2.80 gal/sf/day
Chlorine Contact Tank:			
Detention Time	60 minutes (for unmanned facilities)	145 minutes	119 minutes
Chlorination/ Dechlorination Feed Equipment	6 mg/l Residual at maximum daily flow	50 pounds/day capacity	50 pounds/day capacity

No upgrade or expansion will be required for the chlorine contact tank, chlorination system, dechlorination system, chemical feed systems, or building area. Each system will meet the requirements for operation at the new design flow of 64,700 gpd. These modifications require no additional land. Therefore, the existing site is adequate through the planning period.

This upgrade does not address the fact that the Sewerage Regulations require multiple lagoons. Based on the long detention time proposed in the expansion, the Town can likely receive a waiver on the dual unit requirement. Additionally, based on the expansion timing (i.e., end of 1995), it is unsure how this or any facility would be affected by the Health Department Regulations. Therefore, no capacity upgrade or provision for dual lagoons has been included in this expansion. Due to operation simplicity, standby power is not considered applicable.

Operation and maintenance considerations for the upgraded plant include an increase cost for running two blowers plus increased chlorine and sulfur dioxide chemical costs. The plant will require no increase in staff or manpower.

Trickling Filter WWTP

The two-phased expansion of the Trickling Filter WWTP involves an ultimate design flow increase to 290,400 gpd and a decrease in effluent concentrations to 17.5 mg/l BOD_5 and TSS and 5.8 mg/l Ammonia (April-September). This increase in flow and decrease in effluent discharge concentrations will require both expansion and increased treatment responsibilities. Table 4-20 presents plant upgrade criteria for each unit process for each phase of the Trickling Filter WWTP. An additional 5 to 10 acres of land is required for

TABLE 4-20

UPGRADE CRITERIA FOR TRICKLING FILTER WWTP WITH ULTIMATE DESIGN FLOW OF 290,400 GPD

Unit/ Process	Regulation Requirements	Existing Conditions at 170,000 GPD	Phase One Upgrade Conditions at 230,200 GPD	Phase Two Upgrade Conditions at 290,400 GPD
Flow:				
Average Daily	--	170,000 gpd	230,200 gpd	290,400 gpd
Maximum Daily	--	510,000 gpd	690,600 gpd	871,200 gpd
Peak Hydraulic	--	--	920,800 gpd	1,161,600 gpd
Screw Pumps:				
Capacity	As Required	350 gpm	800 gpm	800 gpm
Operating/ Standby	1/1	1/1	2/1	2/1
Bar Screen:				
Mechanical	As Required	1 Operating	2 Operating	2 Operating
Manual	As Required	1 Standby	1 Standby	1 Standby
Primary Clarifiers:				
Surface Overflow Rate at Peak Daily Flow with One Train Out of Service (two clarifiers per train)	2,500 gpd/sf	2,470 gpd/sf	1,727 gpd/sf	2,178 gpd/sf
Operating Units	2	2	3	4
Dosing Siphon:				
Operating/Standby	1/1	1/1	1/1	2/1
Maximum Operating Capacity	As Required	650 gpm	650 gpm	1,300 gpm

(Table continued on page 4-47.)

TABLE 4-20 (Continued)

UPGRADE CRITERIA FOR TRICKLING FILTER WWTP
WITH ULTIMATE DESIGN FLOW OF 290,400 GPD

Unit/ Process	Regulation Requirements	Existing Conditions at 170,000 GPD	Phase One Upgrade Conditions at 230,200 GPD	Phase Two Upgrade Conditions at 290,400 GPD
Trickling Filter:				
Operating Units	1 (min.)	1	2	2
Hydaulic Loading Rate per Unit at Average Flow Condition (with 100 Percent Recycle flow)	2-4 mgd/ac	4.02 mgd/ac	2.72 mgd/ac	3.44 mgd/ac
Secondary Clarifiers:				
Surface Overflow Rate at Peak Daily Flow with one train out of service	1,200 gpd/sf	1,200 gpd/sf	1,114 gpd/sf	1,055 gpd/sf
Operating Units	2	2	3	4
Recycle Pumping Station:				
Operating/Standby	1/1	1/2	2/1	2/1
Required Recycle Pumping Capacity	As Required	110 gpm	160 gpm	203 gpm
Sand Filter Feed Pumping Station:				
Operating/Standby	1/1	--	2/1	2/1
Capacity (Maximum)	As Required	--	640 gpm	810 gpm

(Table continued on 4-48.)

TABLE 4-20 (Continued)

UPGRADE CRITERIA FOR TRICKLING FILTER WWTP WITH ULTIMATE DESIGN FLOW OF 290,400 GPD

Unit/ Process	Regulation Requirements	Existing Conditions at 170,000 GPD	Phase One Upgrade Conditions at 230,200 GPD	Phase Two Upgrade Conditions at 290,400 GPD
Sand Filters:				
Filter Area	As Required	--	150 sf	150 sf
Filter Loading Rate at Maximum Daily Flow	4 gpm/sf	--	3.2 gpm/sf	4 gpm/sf
Chlorine Contact:				
Tank Volume	As Required	6,300 gal.	12,600 gal.	12,600 gal.
Detention Time at Maximum Daily Flow	20 minutes	18 minutes	26 minutes	21 minutes
Aerobic Digester				
Operating Units	2	2	3	3
Detention Time	20 days	20 days	20 days	20 days
Sand Drying Beds				
Area	As Required	3,600 sf	4,900 sf	6,200 sf
Loading	15 lb/sf/yr	15 lb/sf/yr	15 lb/sf/yr	15 lb/sf/yr

total expansion which includes a 300-foot buffer area along the perimeter of the site. The VDH may approve a reduction in this required buffer depending upon various physical factors which would reduce the total area requirement.

Phase One Expansion

The Phase One expansion will provide effective wastewater treatment for an average daily flow of 230,200 gpd. The maximum daily flow was calculated using a 3.0 peaking factor, and the peak hydraulic flow was calculated using a 4.0 peaking factor. The limited capacity of the existing screw pumps will require the addition of one screw pump so that two units can operate with one acting as a standby. A slight change in operating speed will increase the design flow for each screw to 400 gpm. This modification will easily accommodate both expansion phases.

For expanding the bar screens, an additional channel is required to provide two operating screens with one manually cleaned bypass screen. The pipe from the headworks structure must be upgraded to accommodate future flows. This will eliminate present backup conditions that result from undersized piping. The addition of one primary clarifier adjacent to the existing units will accommodate the Phase One expansion.

No modifications are required for the trickling filter dosing siphon for the Phase One expansion. However, the existing trickling filter will be overloaded under average design flow conditions plus a 100 percent recycle flow. Therefore, a new trickling filter is required to parallel the existing filter. Minor modifications to the dosing chamber will insure equal flow splitting. This new trickling filter will also accommodate the Phase Two expansion. The addition of one secondary clarifier adjacent to the existing units will accommodate the Phase One expansion.

The existing recycle pumping station must be modified to add pumps for feeding the required gravity sand filtration system. The recycle pumps are of sufficient capacity to provide 100 percent recycle flow to the trickling filter for both expansions. The sand filter feed pumps are of submersible type, sized for peak hydraulic flow predicted for the Phase One expansion. The gravity sand filters will provide effluent polishing for BOD_5, TSS, and Ammonia discharge limits compliance. A successful filtration system that is recommended for Appomattox is the upflow, continuously backwashing, dynamic filter. A modular system utilizing three 50-square foot filters will provide the necessary filter area while maximizing operational flexibility.

Doubling of the chlorine contact tank capacity will accommodate the Phase One and Two expansions. This is proposed for the Phase One expansion to minimize complexity in equal flow distribution. A complete renovation of the effluent flow measurement system will be required for the Phase One expansion including V-notch weir replacement and instrumentation upgrade. The capacity of the chlorination and dechlorination feed equipment must be upgraded for the Phase One expansion.

For expansion of the sludge handling system to accommodate the Phase One expansion, the waste sludge pumping station, aerobic digester, blowers, and sludge drying beds require expansion. Refer to Table 4-20 for expansion details. The existing administration and chemical feed facilities are adequate to accommodate the Phase One expansion.

Phase Two Expansion

The Phase Two expansion will provide effective wastewater treatment for an average daily flow of 290,400 gpd. The maximum daily flow was calculated using a 3.0 peaking factor, and the peak hydraulic flow was calculated using a 4.0 peaking factor. From a practical and economic standpoint, the Phase One expansion included increased capacity in many areas that will serve the Phase Two expansion. The unit processes that can be economically phased for this expansion are indicated in the previously presented Table 4-20. These include the addition of a primary clarifier, expansion of the trickling filter dosing siphon, upgrade of the sand filter feed pumps, and addition of sand drying beds.

COST ESTIMATES AND IMPLEMENTATION SCHEDULE

Construction cost estimates for expansion of the wastewater collection system and wastewater treatment facilities were based on historical construction cost data and F. W. Means construction cost data. All costs are in January 1, 1991 dollars. All estimates include Contractor Overhead and profit, Contingency, Engineering, Surveying, Legal, and related costs. Table 4-21 presents the estimated construction costs for the expansion of the wastewater collection and treatment systems for each development period previously identified. The implementation of each development period is predicted to be required before the end of that specific period. Thus, Table 4-21 further serves as the implementation schedule.

TABLE 4-21

CONSTRUCTION COST ESTIMATES AND IMPLEMENTATION SCHEDULE FOR WASTEWATER COLLECTION AND TREATMENT SYSTEM (IN JANUARY, 1991 DOLLARS)

1. WASTEWATER COLLECTION SYSTEM

Development Period	Item	Estimated Construction Cost
1991-1993[a]	100 Percent In-Town Residential and Commercial Connections, New Industrial Site No. 1, New Commercial Site	$ 915,000
1993-1995	34% Potential Out-Town Serviceable Residential Connections, New Industrial Site No. 2	1,842,300
1995-1997	33% Potential Out-Town Serviceable Residential Connections, Existing Industrial Connection	1,675,300
1997-1999	33% Potential Out-Town Serviceable Residential Connections	2,105,500
2000	New Industrial Site No. 3, 33% Study Area Growth Projections	-- [b]
2005	33% Study Area Growth Projections	-- [b]
2010	33% Study Area Growth Projections	-- [b]

2. WASTEWATER TREATMENT FACILITIES

Predicted Construction Completion Date	WWTP	Expansion Phase	Estimated Construction Cost
1992	Trickling Filter	1	$1,421,000
1995	Aerated Lagoon	1	$ 303,000
1997	Trickling Filter	2	$ 370,200

[a] Assumes all wastewater collection and pumping facilities for the Historical Park Service are complete and in place.

[b] Wastewater collection system assumed to be in place and single connection required by user.

CHAPTER 5 - FUNDING SOURCES

FARMERS HOME ADMINISTRATION

The Farmers Home Administration (FmHA) provides financial assistance to local governments. Operated by the United States Department of Agriculture and administered through the Secretary of Agriculture, the FmHA operates on the State level through districts. Appomattox County, all inclusive, falls under District V and is administered by an office in Farmville, Virginia. However, Appomattox County is further served on a local level by an office located in the Triangle Shopping Center. The staff of the local office includes a county and assistant county supervisor, program technician, and office clerk.

FmHA loans and grants are based on the median household income of the local area (Appomattox) as compared to the total State median household income. For 1990, the State median household income is $14,225. FmHA loans and grants are determined using a tiered schedule as follows:

1. Median household income of $12,600 - $14,225: Grant up to 55 percent of eligible cost and loan at 6 percent interest rate.

2. Median household income below $12,600 (considered to be poverty level): Grant up to 75 percent of eligible cost and loan at 5 percent interest rate.

Loan terms can be up to 40 years; they come as packages of loan plus grant. Thus, a grant cannot be obtained without a loan for the remainder of the project. Both water and wastewater systems are eligible for FmHA funding.

COMMUNITY DEVELOPMENT BLOCK GRANT

The United States Department of Housing and Community Development (DHCD) assumes responsibility for the Community Development Block Grant funds which are made available through the U.S. Department of Housing and Urban Development. The Commonwealth of Virginia participates in this program which is administered through the Virginia Community Development Block Grant (CDBG) program. This program provides funding to local governments to address their particular community development needs.

The Virginia CDBG program has adopted the following goal:

> "Improve the economic and physical environment in Virginia's communities and neighborhoods, benefiting persons of low- and moderate-income; thus preventing and eliminating slums and blight, and meeting urgent community development needs posing a serious and immediate threat to the health, safety, and welfare of Virginia citizens."

This goal is centered around DHCD law requiring that any activity funded under the CDBG program must meet one of three national objectives:

1. Provide benefit to low-to-moderate-income (LMI) persons.

2. Aid in the prevention or elimination of slums and blight.

3. Address an urgent community development need.

For the Appomattox study area, the objective most likely to receive funding is a project which will benefit LMI persons.

Low- and moderate-income persons are defined as families or individuals whose family income is less than 80 percent of the county or town median family income for like size families or 80 percent of the median income of the entire nonmetropolitan area of the State, whichever is higher. Projects providing primary benefit to LMI persons must demonstrate the following to be eligible:

1. 51 percent or more of project beneficiaries are LMI residents, or

2. Project serves an area where 51 percent or more of the residents are LMI, or

3. 51 percent or more of the jobs created are held by or available to LMI residents.

Although annual funding is limited due to the quantity of applicants, both water and wastewater systems are eligible for CDBG grants.

VIRGINIA WATER PROJECT, INC.

Virginia Water Project, Inc., headquartered in Roanoke, Virginia, works with local governments and similar bodies to provide assistance to rural low income communities and individuals to obtain adequate, affordable, and safe water/sanitation services. The following services are available at no cost to localities of 10,000 persons and under:

1. Technical assistance such as organizational and general training workshops, selecting and working with engineers, water conservation education, and emergency procedures.

2. Facilitation assistance such as action to keep projects moving ahead, planning and implementing water and wastewater systems, and communications between government, engineers, and community.

3. Groundwater program such as public education on groundwater principles and assistance in developing community groundwater protection plans.

Funding through the Virginia Water Project, Inc., is as follows:

1. Grants
   a. Funds for preliminary engineering studies, soil tests, and community wells.
   b. Emergency funds (maximum $400) to low income families for water and sewer hook-on fees, wells, and/or septic tank.

2. Loans to local governments of low income communities with populations no greater than 10,000.

   a. Amounts from $1,000 - $100,000 at a sliding interest scale, depending on ability to repay. Maximum interest rate 7 percent.

   b. Uses include system upgrades, predevelopment, and/or new development.

VIRGINIA REVOLVING LOAN FUND

Federal and State funding for wastewater collection and treatment facilities evolved from a 75 percent grant from the United States Environmental Protection Agency (EPA) to today's revolving loan program. The Virginia Revolving Loan Fund (VRLF) is administered by the Virginia State Water Control Board, with the Virginia Resources Authority serving as the financial manager. The VRLF was established as a renewing source of low interest loan funding for political subdivisions for wastewater system improvements to publicly-owned facilities.

The State annually prepares a priority lost of VRLF projects. No project can receive financial assistance unless it is on the priority list. VRLF criteria used to develop priority lists for assisting local governments include the evaluation and ranking of project type, the locality's fiscal stress, the project's readiness to proceed, previous compliance history, severity of pollution and effects on water usage, and other economic and environmental factors. The SWCB determines project eligibility if design allows only for reasonable and necessary expansion, upgrade, extension, replacement, repair, rehabilitation, and/or additions made to public wastewater treatment facilities.

The SWCB evaluates and ranks all potential loan projects (including potential refinancing efforts) in accordance with the following criteria:

1. Project Needs

    a. National Municipal Policy (NMP) or Water Quality Standards Compliance

    b. Elimination of Public Health Concerns

c. Growth-Related Needs

Projects needed for NMP and Water Quality Standard Compliance will receive the highest priority; new systems needed for economic reasons will receive the lowest priority.

2. Severity of Pollution Factor: Projects which affect water quality in high priority water bodies will receive higher priority.

3. Fiscal Stress Index: This is the relative financial need of the local government and is based on the Composite Stress Index which centers around several economic indicators including revenue capacity, tax effort, change in revenue capacity, change in tax effort, and a poverty index.

Once a locality satisfies eligibility criteria including all priority list approvals, a loan value is approved and an interest rate is assigned by the SWCB. Presently, interest rates vary from 0 percent to 6-1/2 percent. The maximum interest rate is calculated to be 1 percent below current bond issue rate and is evaluated every 6 months. All loans have a 20-year payback period. A Preliminary Engineering Report (PER) is the first step required in the VRLF process, followed by project design, approval, bidding, and construction.

WELL LOCATION STUDY

Appomattox Water Supply
Appomattox, Virginia

Prepared for

WILEY & WILSON
Lynchburg, Virginia

Prepared by

HATCHER-SAYRE, INC.
Richmond, Virginia

Job No. 0410-001

December, 1990

2.2 Groundwater

Throughout the Piedmont Physiographic Province of Virginia, groundwater occurs within two basic horizons, in the soils or "overburden materials" overlying the bedrock and also in the fractures present within the bedrock. These two water bearing horizons are typically termed the Water Table Aquifer and the Bedrock Aquifer, respectively. In many instances the Water Table and the shallow Bedrock Aquifers are hydraulically interconnected and behave as a single aquifer under water table to slightly artesian conditions. With increasing depth in the Bedrock Aquifer, the likelihood of hydraulic interconnection with the Water Table Aquifer decreases.

Groundwater flow in the Water Table Aquifer usually conforms to the slope of the ground surface, but in a subdued manner. Groundwater gradients are typically much less than those of the ground surface. Flow in the Bedrock Aquifer is controlled by the frequency and orientation of the bedrock fractures which provide a secondary permeability to the bedrock. Since groundwater is confined essentially to the fractures, it is possible to drill dry wells in this system as the result of not penetrating any water bearing fractures.

Recharge to the Water Table Aquifer is by infiltration of precipitation and runoff through the overlying soils. The underlying Bedrock Aquifer is recharged slowly by the vertical migration of infiltrating waters through the overburden and into the bedrock fractures. More rapid recharge occurs where fractured bedrock is exposed in stream beds, drainageways or surface water bodies such as ponds and lakes.

Based on the review of groundwater conditions in the area, the majority of the local wells appear to be constructed into a confined (artesian) groundwater system.

3.0 **LOCAL WELL CONDITIONS**

3.1 Conversations With Area Well Drillers

We attempted to contact seven (7) area drillers concerning their knowledge of local hydrogeologic conditions. Area drillers were selected using the Lynchburg, Virginia Yellow Pages. Of these seven drilling companies, we were able to contact two; Appomattox Well Drilling Company and Falwell Corporation. Of the remaining five companies, Cregar Well and Pump Company and Rice Well Drilling and Pump Installation were no longer in business, we were unable to contact Randy J. Hunt Well Drilling and Pump Service, and Johnson's Well Drilling and Bedford Well Drilling said that they did not have experience in the area.

We met with Mr. Fred Jones of Appomattox Well Drilling Company on October 25, 1990 concerning his knowledge of local hydrogeologic conditions. Mr. Jones told us that there were two good areas for water (quantity and quality) in and near the Town of Appomattox. These were identified as an area adjacent to State Route 703 near its intersection with U.S. Route 460 (west of town) and an area just east of Mr. Jones' office/shop headquarters located on State Route 24 (See Figure 5). Mr. Jones also told us that there were two areas which generally had high yield wells with elevated concentrations of iron and manganese. One area was roughly identified as being bounded by State Route 708 to the west, U.S. Route 460 to the south and State Route 26 to the east. The other area was identified as being south of a line formed by State Route 641. State Route 641 was also identified by Mr. Jones as being an approximate boundary between hard rock to the north and soft (shale) rock to the south (See Figure 5 for approximate locations). Wells completed in the soft rock are generally 110 feet deep or less. No other specific trends could be identified from information provided by Mr. Jones for the remainder of the area. He did state that he normally drills wells to a depth of 300 feet below the ground surface. The yield of wells are generally not increased by drilling below this depth according to Mr. Jones, therefore he does not drill below this depth unless requested.

We also contacted Falwell Corporation concerning their knowledge of local hydrogeologic conditions in the Appomattox area. Falwell Corporation told us that wells that they have drilled in the area generally range from 100 to 300 feet in depth. They also told us that lower capacity wells are encountered in what was described as "hard blue rock". Falwell pointed out that they had recently drilled a well for the new Appomattox High School and had encountered a water bearing zone at 100 feet in shale. Appomattox High School is located south of State Route 641. According to Falwell, water quality in wells they had drilled in the Appomattox area was generally good with no bacteria, low acidity and pH's ranging from 6.3 to 6.8. They did however point out that mineral content (iron and manganese) normally ran a little high in their wells.

3.2 Conversations With Town and County Officials

We met with Appomattox town and county officials on October 25, 1990 concerning their knowledge of existing municipal water supply wells. Those present at the meeting included the Mayor, the County Administrator, the Superintendent of Public Works and the retired Superintendent of Public Works. During the course of our conversation we were able to ascertain that the municipal water supply wells had a history of elevated levels of iron and manganese that exceeded state groundwater criteria. It was pointed out by the retired Superintendent of Public Works that the well drilled by Appomattox Well Drilling Company also had elevated levels of iron and manganese. This was attributed to the depth of the well; 300 feet. According to the retired Superintendent of Public Works, who had 37 years of experience prior to retirement, wells drilled deeper than 225 to 250 feet are likely to encounter poor quality water. Additionally, he pointed out that the well yield in town well TW-15 is directly affected by town well TW-9. Furthermore the yield in well TW-15 is dependent on the amount of water in well TW-9; if the water level in well TW-9 is low, the yield in well TW-15 is low and if the water level in well TW-9 is high, the yield in well TW-15 is high.

Well information, both physical and water quality, for town wells TW-9, TW-15 and TW-25 were also provided during the meeting. Additionally, limited information concerning town wells TW-1 and TW-5 were provided Wiley and Wilson. A summary of the physical information for the town wells (depth to bedrock, pump intake, total depth, depth at which groundwater was encountered, static water level) is included in Table 1 and shown on Figure 4. Groundwater quality is discussed in Section 3.4.

3.3 Virginia Water Control Board (VWCB), West Regional Office, Roanoke, Virginia

We reviewed water well records for the County of Appomattox at the VWCB on October 23, 1990. Seventeen (17) wells were identified in the near vicinity of and in the study area, nine (9) of which were utilized to create geologic cross sections (Figure 4) or to assess general groundwater quality in the area.

3.4 Groundwater Quality

Based upon information collected and on conversations with individuals familiar with groundwater quality in the area, groundwater quality appears to be satisfactory with the exception of elevated levels of iron and manganese. Iron and manganese concentrations from eleven (11) area wells were compared with Virginia groundwater criteria for the Piedmont and Blue Ridge Physiographic Provinces. This information is summarized in Table 2. Based on the results, 41% of the wells tested for iron were higher than the Virginia groundwater criteria. Manganese results showed that 24% of the wells were higher and 23% were equal to the groundwater criteria. Therefore, statistically it appears that

TABLE 1

Area Well Completion Data

Well	Depth to Bedrock (ft)	Depth to Pump Intake (ft)	Total Well Depth (ft)	Depth Groundwater First Encountered	Static Water Level (Elev.) Unpumped	Yield, gpm after drawdown test
F.J[a,b]	90	275	305	272	18	142
TW-1[b,c]	--	90	100	---	--	60
TW-5[b]	--	100	110	---	--	60
TW-9[b]	100	155	205	155	--	90
TW-15[b]	70	216	250	---	85	100
TW-25[b]	60	250	305	80	12	45
105-28[d]	90	---	205	40	--	--
105-48[b]	75	290	305	---	--	5
105-80[b]	69	---	155	---	20	--
105-100	72	---	142	---	20	30
105-101	88	---	155	---	--	5
105-111[b]	80	---	125	---	110	12
105-114[b]	100	---	280	---	--	20
105-115	66	---	245	---	45	10
105-118[b]	90	---	165	---	35	6

Notes:
a. F.J. - Fred Jones Well
b. Well used for geologic cross-section
c. TW - Town Well, TW-1
d. VWCB designation, 105-28

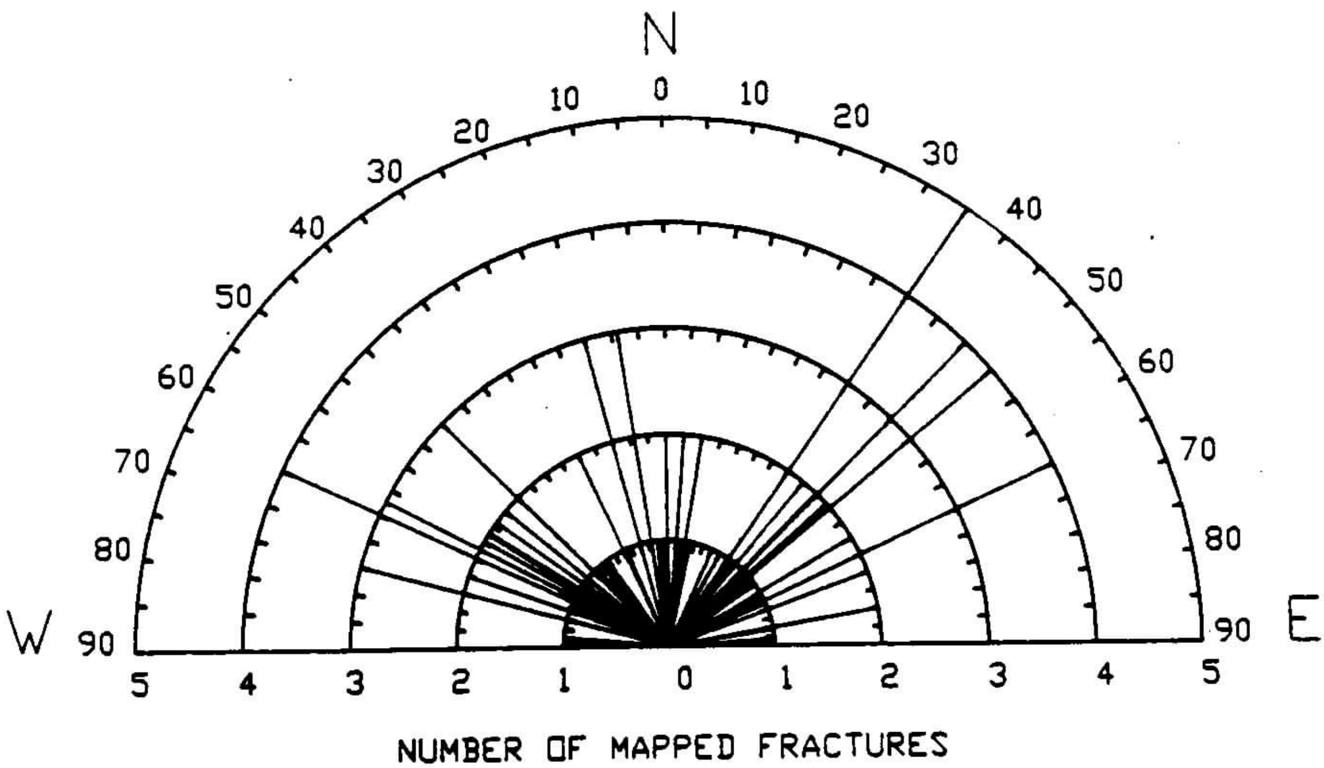
TABLE 2

Iron and Manganese Concentrations

Well	Iron (mg/l)	Manganese (mg/l)
Virginia Groundwater Criteria	0.3	0.05
F.J.	0.7	0.07
TW-9	0.37	0.02
TW-15	0.43-0.62	0.04-0.05
TW-25	0.16	0.03
105-28	0.21	0.05
105-80	0.01-0.21	0.01
105-100	1.1	0.02
105-101	0.06	0.19
105-111	0.24	0.06
105-115	0.02	0.01
105-118	0.08-0.4	0.05-0.15

elevated levels of iron and manganese may not be as problematic as indicated by the individuals contacted concerning the groundwater quality. Data used for this comparison were based on the most recent results for each well.

Groundwater standards have not been established for select parameters in the Piedmont and Blue Ridge Physiographic Provinces because natural groundwater quality can vary greatly from area to area. Groundwater criteria guidelines were discussed with the Virginia Department of Health - Danville Office. It was pointed out that although compliance with the criteria was not enforceable, it was recommended by the Health Department. Specifically, for the Town of Appomattox, maximum iron and manganese levels equivalent to the groundwater criteria were recommended in the past to prevent staining and bitter tasting water and the town has reportedly complied with these recommendations. Therefore, if the Town of Appomattox complies with the recommended groundwater criteria for future water well supplies, staining and bitter tasting water may be avoided.



JOB #: 0410-001	FIGURE 6 FRACTURE TRACE FREQUENCY DIAGRAM WELL LOCATION STUDY WILEY & WILSON APPOMATTOX, VIRGINIA
DATE: DEC. 1990	
SCALE: NONE	
DRAWN BY: D. SAENZ	

HATCHER-SAYRE, INC.

5.0 RECOMMENDED WELL LOCATIONS

5.1 Proposed Sites for Drilling

Based on the information discussed herein, nine (9) well locations have been selected, two of which fall within the study area boundaries and seven (7) which are located outside the study area boundaries. All nine locations are located north of U.S. Route 460 in the "hard rock" area. U.S. Route 460 appears to be located on a surface water drainage divide; the James River drains the area north of U.S. 460 and the Falling River/Roanoke River drains the area south of U.S. Route 460. No locations were selected south of U.S. Route 460 because fracture traces mapped were generally in close proximity to State Route 641 which is considered locally to be the dividing line between "hard" and "soft" rock. As discussed earlier, wells completed in the "soft rock" are generally of poorer quality than those completed in the "hard rock".

The proposed sites are ranked numerically in the order of preference with Area 1 being the preferred location. Locations are based upon potentially favorable hydrogeologic conditions, the fracture trace analysis and proximity to the existing water supply system. At the request of Wiley and Wilson, proposed well locations were confined to the close proximity of the Appomattox town limits in order to reduce potential costs associated with transporting the water to the town. Selected well locations are shown on **Figure** 7.

Area 1

Area 1 is comprised of seven (7) proposed well locations (Sites A - G) and is located 4500 feet northwest of the intersections of State Routes 24 and 26. Area 1 contains the greatest concentration of fractures and has the best potential for recharge from the surface based on the fracture frequency. Though, there are no wells in the immediate area recorded with the Virginia

Water Control Board, based on the major fracture traces, well yields are expected to be higher in this area. All seven proposed locations within Area 1 are within 3500 feet of the town's 1,000,000 gallon water tank. There is the potential for encountering elevated levels of iron and manganese in this area based on conversations with individuals familiar with the area. However by limiting the depth of the proposed wells, the potential for elevated levels of iron and manganese might possibly be reduced.

Proposed Sites A - G which are also ranked alphabetically in the order of preference with Site A being the preferred location, are all located at the intersection of at least two (2) fracture traces. Sites A, and C - G are also located adjacent to intermittent and/or perennial streams which may enhance the potential for surface water recharge. Locations of Sites A - G are shown on Figure 7.

Area 2

Area 2 is comprised of one (1) well site located approximately 2600 feet west of the intersection of State Route 24 and U.S. Route 460. This area is located at the intersection of two major fractures and is located adjacent to Phelps Branch, an intermittent stream which might enhance the potential for surface recharge to the water bearing unit. Area 2 is located along the same fracture trends as Sites C and F. It is also located approximately 600 feet north of U.S. Route 460 along which water lines for the Town of Appomattox are reportedly located.

It is anticipated that elevated levels of iron and manganese will be encountered, however limiting the depth of the proposed well may reduce the potential for elevated concentrations of these parameters.

Area 3

In conjunction with the Areas 1 and 2, Area 3 is the proposed well site with the lowest priority, due to its location with respect to town wells TW-9 and TW-15 and the well drilled by Appomattox Well Drilling Company. This area is located approximately 2000 feet north of the intersection of State Route 131 and U.S. Route 460 and 600 feet east of State Route 24. The location was selected based upon the intersection of two major fracture traces and upon the yield of the well installed by Appomattox Well Drilling Company (142 gpm). Town wells TW-9 and TW-15 are located within a 1200 foot radius of Area 3. Additionally, the well drilled by Appomattox Well Drilling Company is located approximately 300 feet north-northeast of Area 3. Based on the proximity of the two town wells to the proposed location, their noted hydraulic interconnection and the proximity to the well drilled by Appomattox Well Drilling Company, interference may ultimately occur. Although pump testing would be required to determine the severity of such well interference, it is possible that pumping rates and/or periods could be regulated in such a manner as to provide a net increase in overall groundwater withdrawal in this general area. A potential deficiency of this location could be elevated levels of iron and manganese, however limiting the depth of the proposed well may reduce the potential for elevated concentrations of these parameters.

Based on the selection of well sites at the intersection of two or more fracture traces, it is anticipated that a combination of three proposed well sites may sufficiently meet the desired yield of 80,000 gpd so that the Town of Appomattox can be classified as an industrial development community. However because proposed well sites in Areas 1 - 3 are located along the same fractures or in close proximity to existing supply wells, the potential exists for hydraulic interconnection between well sites. Therefore, careful consideration should be given to well spacing in this area and operational schedules to minimize possible interference between wells, while maximizing overall yields.

5.2 Limitations of Drilling Fracture Traces

When attempting to successfully locate wells in rock formations similar to those in the study area, it is imperative that the wells be drilled within the limits of the mapped fracture traces. Wells offset as little as 50 feet from the desired locations can result in much lower yields and even "dry holes". This is usually not a problem when the water bearing fractures extend vertically into the bedrock. However, when the fractures are inclined or dip at a low angle into the bedrock (i.e. 45 degrees from horizontal), it is possible to drill a mapped fracture trend and not penetrate the water bearing portion of the fracture. This condition is illustrated in Figure 8.

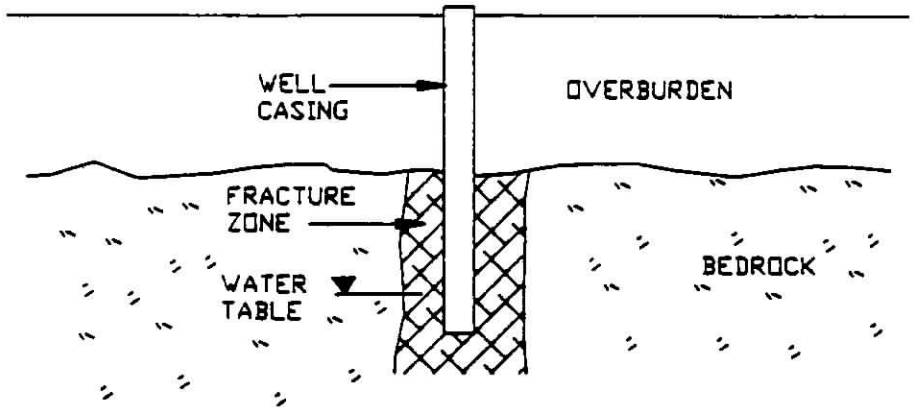
5.3 Well Considerations

When drilling water supply wells, there are a number of factors which should be considered including the following items:

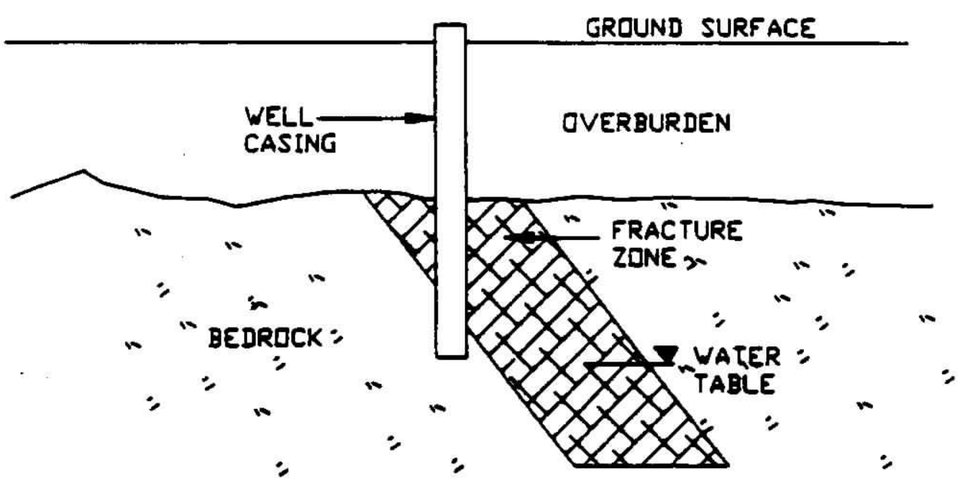
a. Wells should be drilled by a reputable well drilling contractor with experience in the area. It is advisable to request well construction bids prior to entering into a contract.

b. Wells should be drilled initially as test wells in order to minimize costs, should undesirable yields or water quality be encountered. Most well drilling contractors drill a minimum 6 inch diameter well, which is generally acceptable for the well yields normally encountered. However, if Class II B construction is required by the Health Department, the bore hole will have to be reamed to a larger diameter in order to set and grout the outer casing.

c. All well sites must be approved by the Virginia Department of Health prior to drilling.



VERTICAL FRACTURE ZONE



DIPPING FRACTURE ZONE

JOB #: 04 10-001	**FIGURE 8** **SCHEMATIC OF FRACTURE ORIENTATION VS. WELL YIELDS** **WELL LOCATION STUDY** **WILEY & WILSON** **APPOMATOX, VIRGINIA**	HATCHER-SAYRE, INC.
DATE: DEC. 1990		
SCALE: NONE		
DRAWN BY: D. SAENZ		

d. A hydrogeologic consultant should be either on site during drilling or available for consultation with respect to the conditions encountered and well termination depths. Based upon the local hydrogeologic setting, the most favorable well yields at the site are anticipated at depths no deeper than 225 to 250 feet. If favorable quantities of water are not encountered at these depths, deeper drilling may increase the potential of developing the desired yields, however, water quality may decrease.

e. Once favorable yields are encountered, a pump test should be performed in accordance with Department of Health requirements. A minimum 48 hour test is required and it is advised to extend the pumping test to a longer time period if drawdowns are not approaching equilibrium. This provides assurance that an extensive, saturated fracture network has been encountered rather than an isolated pocket of water.

f. Water quality tests must be performed in accordance with Department of Health regulations. Such samples are typically obtained during pump testing.

g. Design of an efficient well includes the selection of a properly sized pump and setting of pump bowls or intake in the well.

6.0 LIMITATIONS

This report has been prepared for the exclusive use of Wiley & Wilson, or their agents for specific application to the Town of Appomattox, Virginia. This report has been prepared in accordance with generally accepted hydrogeologic practices. No other warranty, either express or implied, is made. Our conclusions and recommendations are based on information provided to us, the review of published data and our site observations. We cannot be responsible for information provided by others. The conclusions and recommendations presented have taken into consideration of possible variations in hydrogeologic conditions, however, unanticipated variations may still exist. In performing this well location study no guarantees with respect to well yields or water quality are made. However, scientific technology acceptable to the groundwater industry and hydrogeologic profession has been used in selecting potentially favorable locations for developing satisfactory well yields and water quality.

HATCHER-SAYRE, INC.

James W. Patterson, P.G.
Project Hydrogeologist

Stephen G. Werner, P.G.
V.P., Hydrogeologic Services

December 14, 1990

0410-1.rpt/sdb

OWRATE IS EXPRESSED IN GPM AND PRESSURE IN PSIG

A SUMMARY OF THE ORIGINAL DATA FOLLOWS

PIPE NO.	NODE NOS.		LENGTH (FEET)	DIAMETER (INCHES)	ROUGHNESS	MINOR LOSS K	FIXED GRADE
2	2	4	1460.0	6.0	110.0	.00	
4	4	6	736.0	4.0	70.0	.00	
6	6	36	1084.0	4.0	60.0	.00	
8	6	8	821.0	4.0	70.0	.00	
9	8	9	325.0	8.0	110.0	.00	
10	9	10	930.0	8.0	110.0	.00	
11	9	18	1540.0	6.0	110.0	.00	
12	8	12	336.0	6.0	70.0	.00	
13	0	86	70.0	8.0	110.0	.00	995.00
14	12	14	105.0	4.0	110.0	.00	
16	16	18	368.0	6.0	110.0	.00	
18	18	22	631.0	6.0	110.0	.00	
20	16	20	631.0	6.0	110.0	.00	
21	20	22	380.0	6.0	110.0	.00	
22	24	324	2.0	.9	120.0	.00	
23	22	23	420.0	6.0	110.0	.00	
24	24	28	452.0	4.0	110.0	.00	
25	20	24	60.0	4.0	110.0	.00	
26	28	30	736.0	4.0	110.0	.00	
28	12	30	127.0	8.0	70.0	.00	
30	30	34	463.0	8.0	70.0	.00	
32	334	34	2.0	.6	105.0	.00	
34	34	36	210.0	8.0	70.0	.00	
36	34	0	30.0	8.0	110.0	.00	908.00
38	36	38	420.0	10.0	110.0	.00	
40	38	340	100.0	4.0	110.0	.00	
LINE 40 PUMP DATA (HEAD-FLOW): 134.0 .0 120.0 85.0 53.0 165.0							
42	48	42	955.0	4.0	70.0	.00	
44	48	44	180.0	4.0	70.0	.00	
46	36	44	485.0	4.0	70.0	.00	
48	44	46	200.0	4.0	70.0	.00	
50	48	50	400.0	4.0	70.0	.00	
51	51	54	5.0	4.0	110.0	.00	
LINE 51 IS CLOSED							
52	50	52	350.0	4.0	70.0	.00	
54	450	51	1800.0	4.0	70.0	.00	
55	57	55	125.0	8.0	110.0	.00	
56	54	56	925.0	8.0	110.0	.00	
57	54	55	760.0	10.0	110.0	.00	
58	458	60	3200.0	10.0	130.0	.00	
60	0	60	8800.0	12.0	130.0	.00	995.00
61	66	82	215.0	8.0	110.0	.00	
62	60	62	2000.0	12.0	130.0	.00	
63	65	67	8015.0	12.0	110.0	.00	
64	65	62	2285.0	12.0	110.0	.00	
65	79	83	1450.0	6.0	110.0	.00	
66	62	63	580.0	12.0	110.0	.00	
67	63	69	160.0	10.0	110.0	.00	
68	68	69	1300.0	4.0	110.0	.00	
69	66	69	1470.0	12.0	110.0	.00	
70	68	71	285.0	10.0	110.0	.00	

71	72	57	760.0	10.0	110.0		.00		
72	66	68	368.0	10.0	110.0		.00		
73	71	87	930.0	4.0	110.0		.00		
74	72	74	630.0	8.0	110.0		.00		
75	71	72	865.0	10.0	110.0		.00		
76	70	76	670.0	4.0	110.0		.00		
77	78	87	415.0	4.0	110.0		.00		
78	76	78	680.0	4.0	110.0		.00		
79	82	83	250.0	10.0	110.0		.00		
80	70	78	610.0	4.0	110.0		.00		
81	82	86	1935.0	4.0	110.0		.00		
82	82	84	500.0	6.0	110.0		.00		
83	82	64	1470.0	4.0	110.0		.00		
84	80	84	336.0	4.0	60.0		.00		
85	83	85	670.0	10.0	110.0		.00		
86	80	68	760.0	4.0	110.0		.00		
87	85	108	340.0	10.0	110.0		.00		
88	87	89	510.0	4.0	110.0		.00		
89	76	90	800.0	6.0	110.0		.00		
90	80	89	550.0	4.0	60.0		.00		
91	90	92	170.0	4.0	60.0		.00		
92	89	88	150.0	4.0	60.0		.00		
93	92	94	870.0	4.0	60.0		.00		
94	88	90	240.0	4.0	60.0		.00		
96	94	96	442.0	6.0	60.0		.00		
97	40	96	160.0	10.0	110.0		.00		
98	98	96	420.0	4.0	110.0		.00		
100	98	100	500.0	4.0	60.0		.00		
101	100	102	673.0	4.0	110.0		.00		
102	96	100	860.0	10.0	110.0		.00		
104	102	104	1000.0	10.0	110.0		.00		
106	104	106	150.0	10.0	110.0		.00		
107	106	108	260.0	10.0	110.0		.00		
108	107	109	835.0	8.0	110.0		.00		
109	107	108	85.0	10.0	110.0		.00		
110	110	112	370.0	8.0	110.0		.00		
111	85	86	85.0	10.0	110.0		.00		
112	112	65	1800.0	8.0	110.0		.00		
113	109	309	2.0	.8	105.0		.00		
114	102	116	1110.0	4.0	110.0		.00		
115	109	110	230.0	8.0	110.0		.00		
116	0	379	215.0	3.0	110.0		.00	652.00	
LINE 116	PUMP	DATA	(HEAD-FLOW):	450.0	.0	410.0	60.0	190.0	190.0
118	102	122	790.0	8.0	110.0		.00		
120	122	118	294.0	6.0	110.0		.00		
122	118	120	330.0	4.0	110.0		.00		
124	120	124	630.0	6.0	110.0		.00		
126	124	122	660.0	8.0	110.0		.00		
128	124	126	350.0	8.0	110.0		.00		
130	126	128	315.0	8.0	110.0		.00		
132	128	130	1285.0	6.0	110.0		.00		
134	130	330	2.0	.5	110.0		.00		
136	128	130	1325.0	6.0	110.0		.00		
313	0	309	155.0	3.0	100.0		.00	672.00	
LINE 313	PUMP	DATA	(HEAD-FLOW):	450.0	.0	410.0	60.0	190.0	190.0
316	79	379	2.0	2.0	120.0		.00		
322	0	324	100.0	2.0	110.0		.00	735.00	
LINE 322	PUMP	DATA	(HEAD-FLOW):	310.0	.0	290.0	26.0	120.0	80.0
332	0	334	90.0	2.0	100.0		.00	783.00	
LINE 332	PUMP	DATA	(HEAD-FLOW):	520.0	.0	310.0	60.0	155.0	100.0
334	0	330	252.0	2.0	100.0		.00	571.00	
LINE 334	PUMP	DATA	(HEAD-FLOW):	640.0	.0	600.0	26.0	260.0	80.
450	450	50	400.0	4.0	70.0		.00		
458	58	458	1000.0	10.0	130.0		.00		
340	340	40	2.0	1.0	65.0		.00		

JUNCTION NUMBER	DEMAND	ELEVATION	CONNECTING PIPES				
2	.00	755.00	2				
4	.00	790.00	2	4			
6	.16	809.00	4	6	8		
8	1.11	772.00	8	9	12		
9	4.07	770.00	9	10	11		
10	1.17	790.00	10				
12	.15	780.00	12	14	28		
14	.00	780.00	14				
16	1.05	790.00	16	20			
18	1.34	800.00	11	16	18		
20	.64	796.00	20	21	25		
22	.00	820.00	18	21	23		
23	.00	830.00	23				
24	.80	796.00	22	24	25		
28	1.91	798.00	24	26			
30	1.41	799.00	26	28	30		
34	1.29	822.00	30	32	34	36	
36	1.42	825.00	6	34	38	46	
38	.16	825.00	38	40			
40	1.99	825.00	97	340			
42	1.58	820.00	42				
44	.27	835.00	44	46	48		
46	1.95	836.00	48				
48	1.52	825.00	42	44	50		
50	2.01	828.00	50	52	450		
51	.10	859.00	51	54			
52	2.87	830.00	52				
54	.72	859.00	51	56	57		
55	.62	850.00	55	57			
56	.57	860.00	56				
57	.62	850.00	55	71			
58	11.03	830.00	458				
60	1.19	877.00	58	60	62		
62	2.57	856.00	62	64	66		
63	2.12	882.00	66	67			
64	2.65	883.00	83				
65	2.82	825.00	63	64	112		
66	2.41	875.00	61	69	72		
67	2.82	760.00	63				
68	2.00	881.00	68	70	72	86	
69	2.12	882.00	67	68	69		
70	.26	865.00	76	80			
71	1.15	874.00	70	73	75		
72	.62	880.00	71	74	75		
74	.00	863.00	74				
76	2.48	872.00	76	78	89		
78	.76	848.00	77	78	80		
79	.00	840.00	65	316			
80	1.09	872.00	84	86	90		
82	3.29	870.00	61	79	81	82	83
83	.62	865.00	65	79	85		
84	.78	878.00	82	84			
85	.62	866.00	85	87	111		
86	.00	866.00	13	81	111		
87	1.04	848.00	73	77	88		
88	2.91	867.00	92	94			
89	1.83	868.00	88	90	92		
90	3.07	862.00	89	91	94		
92	1.09	861.00	91	93			
94	1.66	838.00	93	96			
96	3.22	838.00	96	97	98	102	
98	3.91	833.00	98	100			
100	1.48	830.00	100	101	102		
102	4.09	828.00	101	104	114	118	

104	2.64	852.00	104	106	
106	2.61	857.00	106	107	
107	.12	865.00	108	109	
108	.73	865.00	87	107	109
109	.12	815.00	108	113	115
110	2.20	830.00	110	115	
112	1.29	842.00	110	112	
116	2.30	840.00	114		
118	3.21	809.00	120	122	
120	.08	802.00	122	124	
122	2.12	821.00	118	120	126
124	3.33	822.00	124	126	128
126	2.27	832.00	128	130	
128	3.30	840.00	130	132	136
130	.64	800.00	132	134	136
309	.00	.00	113	313	
324	.00	.00	22	322	
330	.00	.00	134	334	
334	.00	.00	32	332	
340	.00	.00	40	340	
379	.00	.00	116	316	
450	.00	826.00	54	450	
458	.00	837.00	58	458	

OUTPUT SELECTION: ALL RESULTS ARE OUTPUT EACH PERIOD

AN EPS WILL BE CARRIED OUT FOR 2.000 HOURS USING A PERIOD OF 1.000 HOURS

THE SYSTEM CONTAINS 3 VARIABLE HEAD TANKS - TANK DATA IS SUMMARIZED BELOW

TANK NO.	CONNECTING PIPE	MAXIMUM ELEVATION	MINIMUM ELEVATION	TANK DIAMETER
1	60	997.00	959.00	64.00
2	36	911.00	881.00	13.00
3	13	997.00	967.00	24.00

- STATUS OF PIPE 332 IS CHANGED BY GRADES = 909.0AND 902.0 AT NODE 34
- STATUS OF PIPE 322 IS CHANGED BY GRADES = 909.0AND 902.0 AT NODE 34
- STATUS OF PIPE 40 IS CHANGED BY GRADES = 996.0AND 992.5 AT NODE 86
- STATUS OF PIPE 116 IS CHANGED BY GRADES = 996.0AND 992.5 AT NODE 86
- STATUS OF PIPE 313 IS CHANGED BY GRADES = 996.0AND 992.5 AT NODE 86
- STATUS OF PIPE 334 IS CHANGED BY GRADES = 996.0AND 992.5 AT NODE 86

THIS SYSTEM HAS 109 PIPES WITH 87 JUNCTIONS , 15 LOOPS AND 8 FGNS

THE RESULTS ARE OBTAINED AFTER 7 TRIALS WITH AN ACCURACY = .00356

PERIOD NO. = 0 -- TIME FROM INITIATION OF EPS = .0000 HOURS

PIPE NO.	NODE NOS.		FLOWRATE	HEAD LOSS	PUMP HEAD	MINOR LOSS	VELOCITY	HL/1000
2	2	4	.00	.00	.00	.00	.00	.00
4	4	6	.00	.00	.00	.00	.00	.00

6	6	36	5.92	.18	.00	.00	.15	.17
8	6	8	-6.08	-.11	.00	.00	-.16	-.13
9	8	9	-31.08	-.01	.00	.00	-.20	-.04
10	9	10	1.17	.00	.00	.00	.01	.00
11	9	18	-36.32	-.34	.00	.00	-.41	-.22
12	8	12	23.89	.08	.00	.00	.27	.23
13	0	86	-166.53	-.06	.00	.00	-1.06	-.90
14	12	14	.00	.00	.00	.00	.00	.00
16	16	18	18.56	.02	.00	.00	.21	.06
18	18	22	-19.10	-.04	.00	.00	-.22	-.07
20	16	20	-19.61	-.04	.00	.00	-.22	-.07
21	20	22	19.10	.03	.00	.00	.22	.07
22	24	324	-61.18	-13.23	.00	.00	-34.59	-6617.14
23	22	23	.00	.00	.00	.00	.00	.00
24	24	28	21.03	.26	.00	.00	.54	.57
25	20	24	-39.35	-.11	.00	.00	-1.00	-1.82
26	28	30	19.12	.35	.00	.00	.49	.48
28	12	30	23.74	.01	.00	.00	.15	.06
30	30	34	41.45	.07	.00	.00	.26	.16
32	334	34	71.57	123.56	.00	.00	81.20	61781.69
34	34	36	87.29	.13	.00	.00	.56	.63
36	34	0	24.44	.00	.00	.00	.16	.03
38	36	38	81.49	.03	.00	.00	.33	.08
40	38	340	81.33	.70	121.54	.00	2.08	6.98
42	48	42	1.58	.01	.00	.00	.04	.01
44	48	44	-8.08	-.04	.00	.00	-.21	-.22
46	36	44	10.30	.17	.00	.00	.26	.35
48	44	46	1.95	.00	.00	.00	.05	.02
50	48	50	4.98	.04	.00	.00	.13	.09
LINE	51 IS	CLOSED						
52	50	52	2.87	.01	.00	.00	.07	.03
54	450	51	.10	.00	.00	.00	.00	.00
55	57	55	1.91	.00	.00	.00	.01	.00
56	54	56	.57	.00	.00	.00	.00	.00
57	54	55	-1.29	.00	.00	.00	-.01	.00
58	458	60	-11.03	.00	.00	.00	-.05	.00
60	0	60	-41.37	-.06	.00	.00	-.12	-.01
61	66	82	-34.66	-.01	.00	.00	-.22	-.05
62	60	62	-53.59	-.02	.00	.00	-.15	-.01
63	65	67	2.82	.00	.00	.00	.01	.00
64	65	62	27.43	.01	.00	.00	.08	.00
65	79	83	102.16	2.14	.00	.00	1.16	1.48
66	62	63	-28.73	.00	.00	.00	-.08	.00
67	63	69	-30.85	.00	.00	.00	-.13	-.01
68	68	69	1.86	.01	.00	.00	.05	.01
69	66	69	31.11	.01	.00	.00	.09	.01
70	68	71	-.18	.00	.00	.00	.00	.00
71	72	57	2.53	.00	.00	.00	.01	.00
72	66	68	1.14	.00	.00	.00	.00	.00
73	71	87	-4.48	-.03	.00	.00	-.11	-.03
74	72	74	.00	.00	.00	.00	.00	.00
75	71	72	3.15	.00	.00	.00	.01	.00
76	70	76	-3.12	-.01	.00	.00	-.08	-.02
77	78	87	6.31	.03	.00	.00	.16	.06
78	76	78	4.20	.02	.00	.00	.11	.03
79	82	83	-46.00	-.01	.00	.00	-.19	-.03
80	70	78	2.86	.01	.00	.00	.07	.01
81	82	86	3.71	.04	.00	.00	.09	.02
82	82	84	1.70	.00	.00	.00	.02	.00
83	82	64	2.65	.02	.00	.00	.07	.01
84	80	84	-.92	.00	.00	.00	-.02	-.01
85	83	85	55.53	.03	.00	.00	.23	.04
86	80	68	2.54	.01	.00	.00	.06	.01
87	85	108	-107.91	-.05	.00	.00	-.44	-.14
88	87	89	.78	.00	.00	.00	.02	.00
89	76	90	-9.81	-.02	.00	.00	-.11	-.02

90	80	89	-2.71	-.02	.00	.00	-.07	-.04
91	90	92	-19.54	-.26	.00	.00	-.50	-1.53
92	89	88	-3.76	-.01	.00	.00	-.10	-.07
93	92	94	-20.63	-1.47	.00	.00	-.53	-1.69
94	88	90	-6.67	-.05	.00	.00	-.17	-.21
96	94	96	-22.29	-.12	.00	.00	-.25	-.27
97	40	96	79.34	.01	.00	.00	.32	.08
98	98	96	-5.44	-.02	.00	.00	-.14	-.05
100	98	100	1.53	.01	.00	.00	.04	.01
101	100	102	48.44	1.80	.00	.00	1.24	2.67
102	96	100	48.38	.03	.00	.00	.20	.03
104	102	104	68.15	.06	.00	.00	.28	.06
106	104	106	65.51	.01	.00	.00	.27	.05
107	106	108	62.90	.01	.00	.00	.26	.05
108	107	109	-45.86	-.07	.00	.00	-.29	-.08
109	107	108	45.74	.00	.00	.00	.19	.03
110	110	112	34.36	.02	.00	.00	.22	.05
111	85	86	162.82	.02	.00	.00	.67	.29
112	112	65	33.07	.08	.00	.00	.21	.05
113	109	309	-82.53	-54.28	.00	.00	-59.93	-27138.33
114	102	116	2.30	.01	.00	.00	.06	.01
115	109	110	36.56	.01	.00	.00	.23	.05
116	0	379	102.16	9.29	355.08	.00	4.64	43.22
118	102	122	-26.11	-.02	.00	.00	-.17	-.03
120	122	118	-2.14	.00	.00	.00	-.02	.00
122	118	120	-5.35	-.01	.00	.00	-.14	-.05
124	120	124	-5.43	.00	.00	.00	-.06	-.01
126	124	122	26.09	.02	.00	.00	.17	.03
128	124	126	-34.85	-.02	.00	.00	-.22	-.05
130	126	128	-37.12	-.02	.00	.00	-.24	-.06
132	128	130	-20.38	-.10	.00	.00	-.23	-.07
134	130	330	-41.06	-98.43	.00	.00	-67.08	-49215.31
136	128	130	-20.04	-.10	.00	.00	-.23	-.07
313	0	309	82.53	5.38	382.87	.00	3.75	34.74
316	79	379	-102.16	-.53	.00	.00	-10.43	-265.01
322	0	324	61.18	12.05	198.97	.00	6.25	120.48
332	0	334	71.57	17.30	265.86	.00	7.31	192.17
334	0	330	41.06	17.30	540.12	.00	4.19	68.66
450	450	50	-.10	.00	.00	.00	.00	.00
458	58	458	-11.03	.00	.00	.00	-.05	.00
340	340	40	81.33	31.63	.00	.00	33.22	15814.29

JUNCTION NUMBER	DEMAND	GRADE LINE	ELEVATION	PRESSURE
2	.00	908.05	755.00	66.32
4	.00	908.05	790.00	51.16
6	.16	908.05	809.00	42.92
8	1.11	908.16	772.00	59.00
9	4.07	908.17	770.00	59.87
10	1.17	908.17	790.00	51.21
12	.15	908.08	780.00	55.50
14	.00	908.08	780.00	55.50
16	1.05	908.53	790.00	51.36
18	1.34	908.51	800.00	47.02
20	.64	908.57	796.00	48.78
22	.00	908.55	820.00	38.37
23	.00	908.55	830.00	34.04
24	.80	908.68	796.00	48.83
28	1.91	908.43	798.00	47.85
30	1.41	908.07	799.00	47.27
34	1.29	908.00	822.00	37.27
36	1.42	907.87	825.00	35.91
38	.16	907.83	825.00	35.90
40	1.99	997.05	825.00	74.56
42	1.58	907.65	820.00	37.98
44	.27	907.70	835.00	31.50

46	1.95	907.70	836.00	31.07
48	1.52	907.66	825.00	35.82
50	2.01	907.62	828.00	34.50
51	.10	907.62	859.00	21.07
52	2.87	907.61	830.00	33.63
54	.72	995.10	859.00	58.98
55	.62	995.10	850.00	62.88
56	.57	995.10	860.00	58.54
57	.62	995.10	850.00	62.88
58	11.03	995.06	830.00	71.52
60	1.19	995.06	877.00	51.16
62	2.57	995.08	856.00	60.27
63	2.12	995.09	882.00	49.00
64	2.65	995.09	883.00	48.57
65	2.82	995.09	825.00	73.71
66	2.41	995.10	875.00	52.04
67	2.82	995.09	760.00	101.87
68	2.00	995.10	881.00	49.44
69	2.12	995.09	882.00	49.01
70	.26	995.16	865.00	56.40
71	1.15	995.10	874.00	52.48
72	.62	995.10	880.00	49.88
74	.00	995.10	863.00	57.24
76	2.48	995.17	872.00	53.37
78	.76	995.15	848.00	63.77
79	.00	997.26	840.00	68.14
80	1.09	995.11	872.00	53.35
82	3.29	995.11	870.00	54.21
83	.62	995.11	865.00	56.38
84	.78	995.11	878.00	50.75
85	.62	995.09	866.00	55.94
86	.00	995.06	866.00	55.93
87	1.04	995.13	848.00	63.76
88	2.91	995.14	867.00	55.53
89	1.83	995.13	868.00	55.09
90	3.07	995.19	862.00	57.71
92	1.09	995.45	861.00	58.26
94	1.66	996.92	838.00	68.86
96	3.22	997.04	838.00	68.92
98	3.91	997.02	833.00	71.07
100	1.48	997.01	830.00	72.37
102	4.09	995.21	828.00	72.46
104	2.64	995.16	852.00	62.03
106	2.61	995.15	857.00	59.86
107	.12	995.14	865.00	56.39
108	.73	995.13	865.00	56.39
109	.12	995.21	815.00	78.09
110	2.20	995.19	830.00	71.58
112	1.29	995.17	842.00	66.38
116	2.30	995.20	840.00	67.25
118	3.21	995.24	809.00	80.70
120	.08	995.25	802.00	83.74
122	2.12	995.24	821.00	75.50
124	3.33	995.26	822.00	75.08
126	2.27	995.27	832.00	70.75
128	3.30	995.29	840.00	67.29
130	.64	995.39	800.00	84.67
309	.00	1049.48		
324	.00	921.92		
330	.00	1093.82		
334	.00	1031.56		
340	.00	1028.68		
379	.00	997.79		
450	.00	907.62	826.00	35.37
458	.00	995.06	837.00	68.49

THE NET SYSTEM DEMAND = 126.16

SUMMARY OF INFLOWS(+) AND OUTFLOWS(-) FROM FIXED GRADE NODES

PIPE NUMBER	FLOWRATE
13	-166.53
36	-24.44
60	-41.37
116	102.16
313	82.53
322	61.18
332	71.57
334	41.06

THE NET FLOW INTO THE SYSTEM FROM FIXED GRADE NODES = 358.50
THE NET FLOW OUT OF THE SYSTEM INTO FIXED GRADE NODES = -232.34

TANK STATUS REPORT

TANK NO.	CONN. PIPE	NET FLOW	ADJ. HGL	WATER SURFACE ELE.	PROJECTED W.S.E.
1	60	41.37	995.06	995.00	995.07
2	36	24.44	908.00	908.00	909.00
3	13	166.53	995.06	995.00	997.00

THE RESULTS ARE OBTAINED AFTER 4 TRIALS WITH AN ACCURACY = .00442

PERIOD NO. = 0 -- TIME FROM INITIATION OF EPS = .6773 HOURS

PIPE NO.	NODE NOS.		FLOWRATE	HEAD LOSS	PUMP HEAD	MINOR LOSS	VELOCITY	HL/1000
2	2	4	.00	.00	.00	.00	.00	.00
4	4	6	.00	.00	.00	.00	.00	.00
6	6	36	5.89	.18	.00	.00	.15	.17
8	6	8	-6.05	-.11	.00	.00	-.15	-.13
9	8	9	-30.94	-.01	.00	.00	-.20	-.04
10	9	10	1.17	.00	.00	.00	.01	.00
11	9	18	-36.18	-.33	.00	.00	-.41	-.22
12	8	12	23.78	.08	.00	.00	.27	.23
13	0	86	-4.07	.00	.00	.00	-.03	.00
14	12	14	.00	.00	.00	.00	.00	.00
16	16	18	18.48	.02	.00	.00	.21	.06
18	18	22	-19.03	-.04	.00	.00	-.22	-.07
20	16	20	-19.53	-.04	.00	.00	-.22	-.07
21	20	22	19.03	.02	.00	.00	.22	.07
22	24	324	-60.95	-13.14	.00	.00	-34.46	-6571.84
23	22	23	.00	.00	.00	.00	.00	.00
24	24	28	20.95	.26	.00	.00	.53	.57
25	20	24	-39.21	-.11	.00	.00	-1.00	-1.81
26	28	30	19.04	.35	.00	.00	.49	.47
28	12	30	23.63	.01	.00	.00	.15	.06
30	30	34	41.26	.07	.00	.00	.26	.16
32	334	34	71.43	123.14	.00	.00	81.05	61568.21
34	34	36	86.63	.13	.00	.00	.55	.62
36	34	0	24.77	.00	.00	.00	.16	.03
38	36	38	80.80	.03	.00	.00	.33	.08
40	38	340	80.64	.69	121.82	.00	2.06	6.87
42	48	42	1.58	.01	.00	.00	.04	.01
44	48	44	-8.08	-.04	.00	.00	-.21	-.22
46	36	44	10.30	.17	.00	.00	.26	.35
48	44	46	1.95	.00	.00	.00	.05	.02
50	48	50	4.98	.04	.00	.00	.13	.09
LINE 51 IS CLOSED								
52	50	52	2.87	.01	.00	.00	.07	.03
54	450	51	.10	.00	.00	.00	.00	.00

55	57	55	1.91	.00	.00	.00	.01	.00
56	54	56	.57	.00	.00	.00	.00	.00
57	54	55	-1.29	.00	.00	.00	-.01	.00
58	458	60	-11.03	.00	.00	.00	-.05	.00
60	0	60	-201.18	-1.15	.00	.00	-.57	-.13
61	66	82	-148.39	-.16	.00	.00	-.95	-.73
62	60	62	-213.40	-.29	.00	.00	-.61	-.15
63	65	67	2.82	.00	.00	.00	.01	.00
64	65	62	66.36	.05	.00	.00	.19	.02
65	79	83	101.08	2.10	.00	.00	1.15	1.45
66	62	63	-149.62	-.06	.00	.00	-.42	-.10
67	63	69	-151.74	-.04	.00	.00	-.62	-.26
68	68	69	8.63	.14	.00	.00	.22	.11
69	66	69	145.22	.14	.00	.00	.41	.10
70	68	71	-2.47	.00	.00	.00	-.01	.00
71	72	57	2.53	.00	.00	.00	.01	.00
72	66	68	.76	.00	.00	.00	.00	.00
73	71	87	-6.77	-.06	.00	.00	-.17	-.07
74	72	74	.00	.00	.00	.00	.00	.00
75	71	72	3.15	.00	.00	.00	.01	.00
76	70	76	-3.27	-.01	.00	.00	-.08	-.02
77	78	87	6.68	.03	.00	.00	.17	.07
78	76	78	4.43	.02	.00	.00	.11	.03
79	82	83	-156.98	-.07	.00	.00	-.64	-.27
80	70	78	3.01	.01	.00	.00	.08	.02
81	82	86	-5.61	-.10	.00	.00	-.14	-.05
82	82	84	8.25	.01	.00	.00	.09	.01
83	82	64	2.65	.02	.00	.00	.07	.01
84	80	84	-7.47	-.09	.00	.00	-.19	-.26
85	83	85	-56.52	-.03	.00	.00	-.23	-.04
86	80	68	7.41	.06	.00	.00	.19	.08
87	85	108	-66.82	-.02	.00	.00	-.27	-.06
88	87	89	-1.12	.00	.00	.00	-.03	.00
89	76	90	-10.18	-.02	.00	.00	-.12	-.02
90	80	89	-1.02	.00	.00	.00	-.03	-.01
91	90	92	-20.14	-.27	.00	.00	-.51	-1.62
92	89	88	-3.98	-.01	.00	.00	-.10	-.08
93	92	94	-21.23	-1.55	.00	.00	-.54	-1.78
94	88	90	-6.89	-.05	.00	.00	-.18	-.22
96	94	96	-22.89	-.13	.00	.00	-.26	-.28
97	40	96	78.65	.01	.00	.00	.32	.08
98	98	96	-5.36	-.02	.00	.00	-.14	-.05
100	98	100	1.45	.01	.00	.00	.04	.01
101	100	102	47.15	1.71	.00	.00	1.20	2.54
102	96	100	47.17	.03	.00	.00	.19	.03
104	102	104	66.68	.06	.00	.00	.27	.06
106	104	106	64.04	.01	.00	.00	.26	.05
107	106	108	61.43	.01	.00	.00	.25	.05
108	107	109	-6.24	.00	.00	.00	-.04	.00
109	107	108	6.12	.00	.00	.00	.03	.00
110	110	112	73.29	.07	.00	.00	.47	.20
111	85	86	9.68	.00	.00	.00	.04	.00
112	112	65	72.00	.34	.00	.00	.46	.19
113	109	309	-81.85	-53.45	.00	.00	-59.44	-26722.87
114	102	116	2.30	.01	.00	.00	.06	.01
115	109	110	75.49	.05	.00	.00	.48	.21
116	0	379	101.08	9.11	356.70	.00	4.59	42.38
118	102	122	-25.92	-.02	.00	.00	-.17	-.03
120	122	118	-2.09	.00	.00	.00	-.02	.00
122	118	120	-5.30	-.01	.00	.00	-.14	-.04
124	120	124	-5.38	.00	.00	.00	-.06	-.01
126	124	122	25.95	.02	.00	.00	.17	.03
128	124	126	-34.66	-.02	.00	.00	-.22	-.05
130	126	128	-36.93	-.02	.00	.00	-.24	-.06
132	128	130	-20.28	-.10	.00	.00	-.23	-.07
134	130	330	-40.87	-97.60	.00	.00	-66.78	-48801.70

136	128	130	-19.95	-.10	.00	.00	-.23	-.07
313	0	309	81.85	5.30	383.77	.00	3.71	34.21
316	79	379	-101.08	-.52	.00	.00	-10.32	-259.84
322	0	324	60.95	11.97	199.79	.00	6.22	119.65
332	0	334	71.43	17.24	266.37	.00	7.29	191.50
334	0	330	40.87	17.16	541.03	.00	4.17	68.09
450	450	50	-.10	.00	.00	.00	.00	.00
458	58	458	-11.03	.00	.00	.00	-.05	.00
340	340	40	80.64	31.13	.00	.00	32.94	15564.77

JUNCTION NUMBER	DEMAND	GRADE LINE	ELEVATION	PRESSURE
2	.00	909.05	755.00	66.76
4	.00	909.05	790.00	51.59
6	.16	909.05	809.00	43.36
8	1.11	909.16	772.00	59.44
9	4.07	909.17	770.00	60.31
10	1.17	909.17	790.00	51.64
12	.15	909.08	780.00	55.93
14	.00	909.08	780.00	55.93
16	1.05	909.53	790.00	51.79
18	1.34	909.50	800.00	47.45
20	.64	909.57	796.00	49.21
22	.00	909.55	820.00	38.80
23	.00	909.55	830.00	34.47
24	.80	909.68	796.00	49.26
28	1.91	909.42	798.00	48.28
30	1.41	909.07	799.00	47.70
34	1.29	909.00	822.00	37.70
36	1.42	908.87	825.00	36.34
38	.16	908.84	825.00	36.33
40	1.99	998.84	825.00	75.33
42	1.58	908.65	820.00	38.41
44	.27	908.70	835.00	31.94
46	1.95	908.70	836.00	31.50
48	1.52	908.66	825.00	36.25
50	2.01	908.62	828.00	34.94
51	.10	908.62	859.00	21.50
52	2.87	908.61	830.00	34.07
54	.72	996.75	859.00	59.69
55	.62	996.75	850.00	63.59
56	.57	996.75	860.00	59.26
57	.62	996.75	850.00	63.59
58	11.03	996.21	830.00	72.02
60	1.19	996.22	877.00	51.66
62	2.57	996.51	856.00	60.89
63	2.12	996.56	882.00	49.64
64	2.65	996.89	883.00	49.35
65	2.82	996.56	825.00	74.34
66	2.41	996.75	875.00	52.76
67	2.82	996.56	760.00	102.51
68	2.00	996.75	881.00	50.16
69	2.12	996.61	882.00	49.66
70	.26	996.85	865.00	57.14
71	1.15	996.75	874.00	53.19
72	.62	996.75	880.00	50.59
74	.00	996.75	863.00	57.96
76	2.48	996.86	872.00	54.11
78	.76	996.84	848.00	64.50
79	.00	999.07	840.00	68.93
80	1.09	996.81	872.00	54.08
82	3.29	996.90	870.00	54.99
83	.62	996.97	865.00	57.19
84	.78	996.90	878.00	51.52
85	.62	997.00	866.00	56.77
86	.00	997.00	866.00	56.77

87	1.04	996.81	848.00	64.49
88	2.91	996.83	867.00	56.26
89	1.83	996.81	868.00	55.82
90	3.07	996.88	862.00	58.45
92	1.09	997.15	861.00	59.00
94	1.66	998.71	838.00	69.64
96	3.22	998.83	838.00	69.69
98	3.91	998.81	833.00	71.85
100	1.48	998.81	830.00	73.15
102	4.09	997.09	828.00	73.27
104	2.64	997.04	852.00	62.85
106	2.61	997.03	857.00	60.68
107	.12	997.02	865.00	57.21
108	.73	997.02	865.00	57.21
109	.12	997.02	815.00	78.88
110	2.20	996.97	830.00	72.35
112	1.29	996.90	842.00	67.12
116	2.30	997.08	840.00	68.07
118	3.21	997.12	809.00	81.52
120	.08	997.13	802.00	84.56
122	2.12	997.12	821.00	76.32
124	3.33	997.14	822.00	75.89
126	2.27	997.15	832.00	71.57
128	3.30	997.17	840.00	68.11
130	.64	997.27	800.00	85.48
309	.00	1050.47		
324	.00	922.82		
330	.00	1094.87		
334	.00	1032.14		
340	.00	1029.97		
379	.00	999.59		
450	.00	908.62	826.00	35.80
458	.00	996.21	837.00	68.99

THE NET SYSTEM DEMAND = 126.16

SUMMARY OF INFLOWS(+) AND OUTFLOWS(-) FROM FIXED GRADE NODES

PIPE NUMBER	FLOWRATE
13	-4.07
36	-24.77
60	-201.18
116	101.08
313	81.85
322	60.95
332	71.43
334	40.87

THE NET FLOW INTO THE SYSTEM FROM FIXED GRADE NODES = 356.18
THE NET FLOW OUT OF THE SYSTEM INTO FIXED GRADE NODES = -230.02

GRADE = 909.0 AT JUN. 34 SWITCH FOR PIPE 332- NEXT SWITCHING VALUE = 902.0

GRADE = 909.0 AT JUN. 34 SWITCH FOR PIPE 322- NEXT SWITCHING VALUE = 902.0

GRADE = 997.0 AT JUN. 86 SWITCH FOR PIPE 40- NEXT SWITCHING VALUE = 992.5

GRADE = 997.0 AT JUN. 86 SWITCH FOR PIPE 116- NEXT SWITCHING VALUE = 992.5

GRADE = 997.0 AT JUN. 86 SWITCH FOR PIPE 313- NEXT SWITCHING VALUE = 992.5

GRADE = 997.0 AT JUN. 86 SWITCH FOR PIPE 334- NEXT SWITCHING VALUE = 992.5

THE RESULTS ARE OBTAINED AFTER 4 TRIALS WITH AN ACCURACY = .00033

PERIOD NO. = 0 -- TIME FROM INITIATION OF EPS = .6773 HOURS

PIPE NO.	NODE NOS.		FLOWRATE	HEAD LOSS	PUMP HEAD	MINOR LOSS	VELOCITY	HL/1000
2	2	4	.00	.00	.00	.00	.00	.00
4	4	6	.00	.00	.00	.00	.00	.00
6	6	36	-1.26	-.01	.00	.00	-.03	-.01
8	6	8	1.10	.00	.00	.00	.03	.01
9	8	9	7.79	.00	.00	.00	.05	.00
10	9	10	1.17	.00	.00	.00	.01	.00
11	9	18	2.55	.00	.00	.00	.03	.00
12	8	12	-7.80	-.01	.00	.00	-.09	-.03
13	0	86	274.83	.16	.00	.00	1.75	2.28
14	12	14	.00	.00	.00	.00	.00	.00
16	16	18	-.80	.00	.00	.00	-.01	.00
18	18	22	.41	.00	.00	.00	.00	.00
20	16	20	-.25	.00	.00	.00	.00	.00
21	20	22	-.41	.00	.00	.00	.00	.00
22	24	324	.00	.00	.00	.00	.00	.00
23	22	23	.00	.00	.00	.00	.00	.00
24	24	28	-1.28	.00	.00	.00	-.03	.00
25	20	24	-.48	.00	.00	.00	-.01	.00
26	28	30	-3.19	-.01	.00	.00	-.08	-.02
28	12	30	-7.95	.00	.00	.00	-.05	-.01
30	30	34	-12.55	-.01	.00	.00	-.08	-.02
32	334	34	.00	.00	.00	.00	.00	.00
34	34	36	13.14	.00	.00	.00	.08	.02
36	34	0	-26.98	.00	.00	.00	-.17	-.03
38	36	38	.16	.00	.00	.00	.00	.00
LINE	40 IS	CLOSED						
42	48	42	1.58	.01	.00	.00	.04	.01
44	48	44	-8.08	-.04	.00	.00	-.21	-.22
46	36	44	10.30	.17	.00	.00	.26	.35
48	44	46	1.95	.00	.00	.00	.05	.02
50	48	50	4.98	.04	.00	.00	.13	.09
LINE	51 IS	CLOSED						
52	50	52	2.87	.01	.00	.00	.07	.03
54	450	51	.10	.00	.00	.00	.00	.00
55	57	55	1.91	.00	.00	.00	.01	.00
56	54	56	.57	.00	.00	.00	.00	.00
57	54	55	-1.29	.00	.00	.00	-.01	.00
58	458	60	-11.03	.00	.00	.00	-.05	.00
60	0	60	-175.65	-.89	.00	.00	-.50	-.10
61	66	82	-147.14	-.15	.00	.00	-.94	-.72
62	60	62	-187.87	-.23	.00	.00	-.53	-.11
63	65	67	2.82	.00	.00	.00	.01	.00
64	65	62	57.15	.04	.00	.00	.16	.02
65	79	83	.00	.00	.00	.00	.00	.00
66	62	63	-133.28	-.05	.00	.00	-.38	-.08
67	63	69	-135.40	-.03	.00	.00	-.55	-.21
68	68	69	7.67	.11	.00	.00	.20	.09
69	66	69	129.85	.12	.00	.00	.37	.08
70	68	71	8.98	.00	.00	.00	.04	.00
71	72	57	2.53	.00	.00	.00	.01	.00
72	66	68	14.89	.00	.00	.00	.06	.00
73	71	87	4.68	.03	.00	.00	.12	.04
74	72	74	.00	.00	.00	.00	.00	.00
75	71	72	3.15	.00	.00	.00	.01	.00
76	70	76	.74	.00	.00	.00	.02	.00
77	78	87	-3.00	-.01	.00	.00	-.08	-.02
78	76	78	-1.23	.00	.00	.00	-.03	.00
79	82	83	-152.65	-.06	.00	.00	-.62	-.26
80	70	78	-1.00	.00	.00	.00	-.03	.00
81	82	86	-10.41	-.30	.00	.00	-.27	-.15
82	82	84	9.98	.01	.00	.00	.11	.02

83	82	64	2.65	.02	.00	.00	.07	.01
84	80	84	-9.20	-.13	.00	.00	-.23	-.38
85	83	85	-153.27	-.17	.00	.00	-.63	-.26
86	80	68	3.77	.02	.00	.00	.10	.02
87	85	108	110.53	.05	.00	.00	.45	.14
88	87	89	.65	.00	.00	.00	.02	.00
89	76	90	-.50	.00	.00	.00	-.01	.00
90	80	89	4.33	.05	.00	.00	.11	.09
91	90	92	-3.33	-.01	.00	.00	-.09	-.06
92	89	88	3.15	.01	.00	.00	.08	.05
93	92	94	-4.42	-.08	.00	.00	-.11	-.10
94	88	90	.24	.00	.00	.00	.01	.00
96	94	96	-6.08	-.01	.00	.00	-.07	-.02
97	40	96	-1.99	.00	.00	.00	-.01	.00
98	98	96	-2.38	.00	.00	.00	-.06	-.01
100	98	100	-1.53	-.01	.00	.00	-.04	-.01
101	100	102	-16.68	-.25	.00	.00	-.43	-.37
102	96	100	-13.68	.00	.00	.00	-.06	.00
104	102	104	-38.02	-.02	.00	.00	-.16	-.02
106	104	106	-40.66	.00	.00	.00	-.17	-.02
107	106	108	-43.27	-.01	.00	.00	-.18	-.03
108	107	109	66.40	.14	.00	.00	.42	.16
109	107	108	-66.52	.00	.00	.00	-.27	-.06
110	110	112	64.08	.06	.00	.00	.41	.15
111	85	86	-264.42	-.06	.00	.00	-1.08	-.71
112	112	65	62.79	.27	.00	.00	.40	.15
113	109	309	.00	.00	.00	.00	.00	.00
114	102	116	2.30	.01	.00	.00	.06	.01
115	109	110	66.28	.04	.00	.00	.42	.16
LINE	116 IS	CLOSED						
118	102	122	14.95	.01	.00	.00	.10	.01
120	122	118	4.38	.00	.00	.00	.05	.00
122	118	120	1.17	.00	.00	.00	.03	.00
124	120	124	1.09	.00	.00	.00	.01	.00
126	124	122	-8.45	.00	.00	.00	-.05	.00
128	124	126	6.21	.00	.00	.00	.04	.00
130	126	128	3.94	.00	.00	.00	.03	.00
132	128	130	.32	.00	.00	.00	.00	.00
134	130	330	.00	.00	.00	.00	.00	.00
136	128	130	.32	.00	.00	.00	.00	.00
LINE	313 IS	CLOSED						
316	79	379	.00	.00	.00	.00	.00	.00
LINE	322 IS	CLOSED						
LINE	332 IS	CLOSED						
LINE	334 IS	CLOSED						
450	450	50	-.10	.00	.00	.00	.00	.00
458	58	458	-11.03	.00	.00	.00	-.05	.00
340	340	40	.00	.00	.00	.00	.00	.00

JUNCTION NUMBER	DEMAND	GRADE LINE	ELEVATION	PRESSURE
2	.00	908.98	755.00	66.73
4	.00	908.98	790.00	51.56
6	.16	908.98	809.00	43.33
8	1.11	908.98	772.00	59.36
9	4.07	908.98	770.00	60.22
10	1.17	908.98	790.00	51.56
12	.15	908.99	780.00	55.90
14	.00	908.99	780.00	55.90
16	1.05	908.98	790.00	51.56
18	1.34	908.98	800.00	47.22
20	.64	908.98	796.00	48.96
22	.00	908.98	820.00	38.56
23	.00	908.98	830.00	34.22
24	.80	908.98	796.00	48.96
28	1.91	908.98	798.00	48.09

30	1.41	908.99	799.00	47.66
34	1.29	909.00	822.00	37.70
36	1.42	909.00	825.00	36.40
38	.16	909.00	825.00	36.40
40	1.99	996.45	825.00	74.29
42	1.58	908.77	820.00	38.47
44	.27	908.82	835.00	31.99
46	1.95	908.82	836.00	31.56
48	1.52	908.78	825.00	36.31
50	2.01	908.75	828.00	34.99
51	.10	908.75	859.00	21.56
52	2.87	908.74	830.00	34.12
54	.72	996.38	859.00	59.53
55	.62	996.38	850.00	63.43
56	.57	996.38	860.00	59.10
57	.62	996.38	850.00	63.43
58	11.03	995.95	830.00	71.91
60	1.19	995.96	877.00	51.55
62	2.57	996.19	856.00	60.75
63	2.12	996.24	882.00	49.50
64	2.65	996.52	883.00	49.19
65	2.82	996.23	825.00	74.20
66	2.41	996.39	875.00	52.60
67	2.82	996.23	760.00	102.37
68	2.00	996.39	881.00	50.00
69	2.12	996.27	882.00	49.52
70	.26	996.34	865.00	56.92
71	1.15	996.39	874.00	53.03
72	.62	996.38	880.00	50.43
74	.00	996.38	863.00	57.80
76	2.48	996.34	872.00	53.88
78	.76	996.35	848.00	64.28
79	.00	996.61	840.00	67.86
80	1.09	996.40	872.00	53.91
82	3.29	996.54	870.00	54.83
83	.62	996.61	865.00	57.03
84	.78	996.53	878.00	51.36
85	.62	996.78	866.00	56.67
86	.00	996.84	866.00	56.70
87	1.04	996.35	848.00	64.29
88	2.91	996.34	867.00	56.05
89	1.83	996.35	868.00	55.62
90	3.07	996.34	862.00	58.22
92	1.09	996.35	861.00	58.65
94	1.66	996.44	838.00	68.66
96	3.22	996.45	838.00	68.66
98	3.91	996.45	833.00	70.83
100	1.48	996.45	830.00	72.13
102	4.09	996.70	828.00	73.10
104	2.64	996.72	852.00	62.71
106	2.61	996.72	857.00	60.55
107	.12	996.73	865.00	57.08
108	.73	996.73	865.00	57.08
109	.12	996.59	815.00	78.69
110	2.20	996.55	830.00	72.17
112	1.29	996.50	842.00	66.95
116	2.30	996.69	840.00	67.90
118	3.21	996.69	809.00	81.33
120	.08	996.69	802.00	84.37
122	2.12	996.69	821.00	76.13
124	3.33	996.69	822.00	75.70
126	2.27	996.69	832.00	71.37
128	3.30	996.69	840.00	67.90
130	.64	996.69	800.00	85.23
309	.00	996.59		
324	.00	908.98		

330	.00	996.69		
334	.00	909.00		
340	.00	996.45		
379	.00	996.61		
450	.00	908.75	826.00	35.86
458	.00	995.96	837.00	68.88

THE NET SYSTEM DEMAND = 126.16

SUMMARY OF INFLOWS(+) AND OUTFLOWS(-) FROM FIXED GRADE NODES

PIPE NUMBER	FLOWRATE
13	274.83
36	26.98
60	-175.65

THE NET FLOW INTO THE SYSTEM FROM FIXED GRADE NODES = 301.81
THE NET FLOW OUT OF THE SYSTEM INTO FIXED GRADE NODES = -175.65

TANK STATUS REPORT

TANK NO.	CONN. PIPE	NET FLOW	ADJ. HGL	WATER SURFACE ELE.	PROJECTED W.S.E.
1	60	175.65	995.96	995.07	995.21
2	36	-26.98	909.00	909.00	908.47
3	13	-274.83	996.84	997.00	995.43

THE RESULTS ARE OBTAINED AFTER 4 TRIALS WITH AN ACCURACY = .00488

PERIOD NO. = 1 -- TIME FROM INITIATION OF EPS = 1.0000 HOURS

PIPE NO.	NODE NOS.		FLOWRATE	HEAD LOSS	PUMP HEAD	MINOR LOSS	VELOCITY	HL/1000
2	2	4	.00	.00	.00	.00	.00	.00
4	4	6	.00	.00	.00	.00	.00	.00
6	6	36	-1.26	-.01	.00	.00	-.03	-.01
8	6	8	1.10	.00	.00	.00	.03	.01
9	8	9	7.79	.00	.00	.00	.05	.00
10	9	10	1.17	.00	.00	.00	.01	.00
11	9	18	2.55	.00	.00	.00	.03	.00
12	8	12	-7.80	-.01	.00	.00	-.09	-.03
13	0	86	134.57	.04	.00	.00	.86	.61
14	12	14	.00	.00	.00	.00	.00	.00
16	16	18	-.80	.00	.00	.00	-.01	.00
18	18	22	.41	.00	.00	.00	.00	.00
20	16	20	-.25	.00	.00	.00	.00	.00
21	20	22	-.41	.00	.00	.00	.00	.00
22	24	324	.00	.00	.00	.00	.00	.00
23	22	23	.00	.00	.00	.00	.00	.00
24	24	28	-1.28	.00	.00	.00	-.03	.00
25	20	24	-.48	.00	.00	.00	-.01	.00
26	28	30	-3.19	-.01	.00	.00	-.08	-.02
28	12	30	-7.95	.00	.00	.00	-.05	-.01
30	30	34	-12.55	-.01	.00	.00	-.08	-.02
32	334	34	.00	.00	.00	.00	.00	.00
34	34	36	13.14	.00	.00	.00	.08	.02
36	34	0	-26.98	.00	.00	.00	-.17	-.03
38	36	38	.16	.00	.00	.00	.00	.00
LINE 40 IS CLOSED								
42	48	42	1.58	.01	.00	.00	.04	.01
44	48	44	-8.08	-.04	.00	.00	-.21	-.22
46	36	44	10.30	.17	.00	.00	.26	.35
48	44	46	1.95	.00	.00	.00	.05	.02
50	48	50	4.98	.04	.00	.00	.13	.09
LINE 51 IS CLOSED								

52	50	52	2.87	.01	.00	.00	.07	.03
54	450	51	.10	.00	.00	.00	.00	.00
55	57	55	1.91	.00	.00	.00	.01	.00
56	54	56	.57	.00	.00	.00	.00	.00
57	54	55	-1.29	.00	.00	.00	-.01	.00
58	458	60	-11.03	.00	.00	.00	-.05	.00
60	0	60	-35.39	-.05	.00	.00	-.10	-.01
61	66	82	-57.59	-.03	.00	.00	-.37	-.13
62	60	62	-47.61	-.02	.00	.00	-.14	-.01
63	65	67	2.82	.00	.00	.00	.01	.00
64	65	62	15.49	.00	.00	.00	.04	.00
65	79	83	.00	.00	.00	.00	.00	.00
66	62	63	-34.69	.00	.00	.00	-.10	-.01
67	63	69	-36.81	.00	.00	.00	-.15	-.02
68	68	69	1.99	.01	.00	.00	.05	.01
69	66	69	36.94	.01	.00	.00	.10	.01
70	68	71	12.07	.00	.00	.00	.05	.00
71	72	57	2.53	.00	.00	.00	.01	.00
72	66	68	18.24	.00	.00	.00	.07	.01
73	71	87	7.77	.08	.00	.00	.20	.09
74	72	74	.00	.00	.00	.00	.00	.00
75	71	72	3.15	.00	.00	.00	.01	.00
76	70	76	1.71	.00	.00	.00	.04	.01
77	78	87	-5.31	-.02	.00	.00	-.14	-.04
78	76	78	-2.58	-.01	.00	.00	-.07	-.01
79	82	83	-64.16	-.01	.00	.00	-.26	-.05
80	70	78	-1.97	.00	.00	.00	-.05	-.01
81	82	86	-4.55	-.06	.00	.00	-.12	-.03
82	82	84	5.18	.00	.00	.00	.06	.01
83	82	64	2.65	.02	.00	.00	.07	.01
84	80	84	-4.40	-.03	.00	.00	-.11	-.10
85	83	85	-64.78	-.04	.00	.00	-.26	-.05
86	80	68	-2.18	-.01	.00	.00	-.06	-.01
87	85	108	64.63	.02	.00	.00	.26	.05
88	87	89	1.42	.00	.00	.00	.04	.00
89	76	90	1.81	.00	.00	.00	.02	.00
90	80	89	5.49	.08	.00	.00	.14	.15
91	90	92	.90	.00	.00	.00	.02	.01
92	89	88	5.08	.02	.00	.00	.13	.13
93	92	94	-.19	.00	.00	.00	.00	.00
94	88	90	2.17	.01	.00	.00	.06	.03
96	94	96	-1.85	.00	.00	.00	-.02	.00
97	40	96	-1.99	.00	.00	.00	-.01	.00
98	98	96	-2.49	.00	.00	.00	-.06	-.01
100	98	100	-1.42	-.01	.00	.00	-.04	-.01
101	100	102	-12.45	-.15	.00	.00	-.32	-.22
102	96	100	-9.55	.00	.00	.00	-.04	.00
104	102	104	-33.79	-.02	.00	.00	-.14	-.02
106	104	106	-36.43	.00	.00	.00	-.15	-.02
107	106	108	-39.04	-.01	.00	.00	-.16	-.02
108	107	109	24.74	.02	.00	.00	.16	.03
109	107	108	-24.86	.00	.00	.00	-.10	-.01
110	110	112	22.42	.01	.00	.00	.14	.02
111	85	86	-130.02	-.02	.00	.00	-.53	-.19
112	112	65	21.13	.04	.00	.00	.13	.02
113	109	309	.00	.00	.00	.00	.00	.00
114	102	116	2.30	.01	.00	.00	.06	.01
115	109	110	24.62	.01	.00	.00	.16	.03
LINE 116 IS CLOSED								
118	102	122	14.95	.01	.00	.00	.10	.01
120	122	118	4.38	.00	.00	.00	.05	.00
122	118	120	1.17	.00	.00	.00	.03	.00
124	120	124	1.09	.00	.00	.00	.01	.00
126	124	122	-8.45	.00	.00	.00	-.05	.00
128	124	126	6.21	.00	.00	.00	.04	.00
130	126	128	3.94	.00	.00	.00	.03	.00

132	128	130	.32	.00	.00	.00	.00	.00
134	130	330	.00	.00	.00	.00	.00	.00
136	128	130	.32	.00	.00	.00	.00	.00
LINE 313 IS CLOSED								
316	79	379	.00	.00	.00	.00	.00	.00
LINE 322 IS CLOSED								
LINE 332 IS CLOSED								
LINE 334 IS CLOSED								
450	450	50	-.10	.00	.00	.00	.00	.00
458	58	458	-11.03	.00	.00	.00	-.05	.00
340	340	40	.00	.00	.00	.00	.00	.00

JUNCTION NUMBER	DEMAND	GRADE LINE	ELEVATION	PRESSURE
2	.00	908.46	755.00	66.50
4	.00	908.46	790.00	51.33
6	.16	908.46	809.00	43.10
8	1.11	908.45	772.00	59.13
9	4.07	908.45	770.00	60.00
10	1.17	908.45	790.00	51.33
12	.15	908.46	780.00	55.67
14	.00	908.46	780.00	55.67
16	1.05	908.45	790.00	51.33
18	1.34	908.45	800.00	47.00
20	.64	908.45	796.00	48.73
22	.00	908.45	820.00	38.33
23	.00	908.45	830.00	34.00
24	.80	908.45	796.00	48.73
28	1.91	908.45	798.00	47.86
30	1.41	908.47	799.00	47.43
34	1.29	908.47	822.00	37.47
36	1.42	908.47	825.00	36.17
38	.16	908.47	825.00	36.17
40	1.99	995.18	825.00	73.74
42	1.58	908.25	820.00	38.24
44	.27	908.30	835.00	31.76
46	1.95	908.30	836.00	31.33
48	1.52	908.26	825.00	36.08
50	2.01	908.22	828.00	34.76
51	.10	908.22	859.00	21.33
52	2.87	908.21	830.00	33.89
54	.72	995.29	859.00	59.06
55	.62	995.29	850.00	62.96
56	.57	995.29	860.00	58.63
57	.62	995.29	850.00	62.96
58	11.03	995.25	830.00	71.61
60	1.19	995.26	877.00	51.24
62	2.57	995.28	856.00	60.35
63	2.12	995.28	882.00	49.09
64	2.65	995.30	883.00	48.66
65	2.82	995.28	825.00	73.79
66	2.41	995.29	875.00	52.13
67	2.82	995.28	760.00	101.95
68	2.00	995.29	881.00	49.53
69	2.12	995.28	882.00	49.09
70	.26	995.18	865.00	56.41
71	1.15	995.29	874.00	52.56
72	.62	995.29	880.00	49.96
74	.00	995.29	863.00	57.33
76	2.48	995.18	872.00	53.38
78	.76	995.19	848.00	63.78
79	.00	995.33	840.00	67.31
80	1.09	995.28	872.00	53.42
82	3.29	995.32	870.00	54.31
83	.62	995.33	865.00	56.48
84	.78	995.32	878.00	50.84

85	.62	995.37	866.00	56.06
86	.00	995.39	866.00	56.07
87	1.04	995.21	848.00	63.79
88	2.91	995.19	867.00	55.55
89	1.83	995.20	868.00	55.12
90	3.07	995.18	862.00	57.71
92	1.09	995.18	861.00	58.14
94	1.66	995.18	838.00	68.11
96	3.22	995.18	838.00	68.11
98	3.91	995.18	833.00	70.28
100	1.48	995.18	830.00	71.58
102	4.09	995.33	828.00	72.51
104	2.64	995.34	852.00	62.12
106	2.61	995.35	857.00	59.95
107	.12	995.35	865.00	56.49
108	.73	995.35	865.00	56.49
109	.12	995.33	815.00	78.14
110	2.20	995.32	830.00	71.64
112	1.29	995.31	842.00	66.44
116	2.30	995.32	840.00	67.30
118	3.21	995.32	809.00	80.74
120	.08	995.32	802.00	83.77
122	2.12	995.32	821.00	75.54
124	3.33	995.32	822.00	75.10
126	2.27	995.32	832.00	70.77
128	3.30	995.32	840.00	67.30
130	.64	995.32	800.00	84.64
309	.00	995.33		
324	.00	908.45		
330	.00	995.32		
334	.00	908.47		
340	.00	995.18		
379	.00	995.33		
450	.00	908.22	826.00	35.63
458	.00	995.25	837.00	68.58

THE NET SYSTEM DEMAND = 126.16

SUMMARY OF INFLOWS(+) AND OUTFLOWS(-) FROM FIXED GRADE NODES

PIPE NUMBER	FLOWRATE
13	134.57
36	26.98
60	-35.39

THE NET FLOW INTO THE SYSTEM FROM FIXED GRADE NODES = 161.55
THE NET FLOW OUT OF THE SYSTEM INTO FIXED GRADE NODES = -35.39

TANK STATUS REPORT

TANK NO.	CONN. PIPE	NET FLOW	ADJ. HGL	WATER SURFACE ELE.	PROJECTED W.S.E.
1	60	35.39	995.26	995.21	995.30
2	36	-26.98	908.47	908.47	906.84
3	13	-134.57	995.39	995.43	993.04

THE RESULTS ARE OBTAINED AFTER 5 TRIALS WITH AN ACCURACY = .00002

RIOD NO. = 2 -- TIME FROM INITIATION OF EPS = 2.0000 HOURS

PIPE NO.	NODE NOS.		FLOWRATE	HEAD LOSS	PUMP HEAD	MINOR LOSS	VELOCITY	HL/1000
2	2	4	.00	.00	.00	.00	.00	.00
4	4	6	.00	.00	.00	.00	.00	.00
6	6	36	-1.26	-.01	.00	.00	-.03	-.01

8	6	8	1.10	.00	.00	.00	.03	.01
9	8	9	7.79	.00	.00	.00	.05	.00
10	9	10	1.17	.00	.00	.00	.01	.00
11	9	18	2.55	.00	.00	.00	.03	.00
12	8	12	-7.80	-.01	.00	.00	-.09	-.03
13	0	86	-129.78	-.04	.00	.00	-.83	-.57
14	12	14	.00	.00	.00	.00	.00	.00
16	16	18	-.80	.00	.00	.00	-.01	.00
18	18	22	.41	.00	.00	.00	.00	.00
20	16	20	-.25	.00	.00	.00	.00	.00
21	20	22	-.41	.00	.00	.00	.00	.00
22	24	324	.00	.00	.00	.00	.00	.00
23	22	23	.00	.00	.00	.00	.00	.00
24	24	28	-1.28	.00	.00	.00	-.03	.00
25	20	24	-.48	.00	.00	.00	-.01	.00
26	28	30	-3.19	-.01	.00	.00	-.08	-.02
28	12	30	-7.95	.00	.00	.00	-.05	-.01
30	30	34	-12.55	-.01	.00	.00	-.08	-.02
32	334	34	.00	.00	.00	.00	.00	.00
34	34	36	13.14	.00	.00	.00	.08	.02
36	34	0	-26.98	.00	.00	.00	-.17	-.03
38	36	38	.16	.00	.00	.00	.00	.00
LINE 40 IS CLOSED								
42	48	42	1.58	.01	.00	.00	.04	.01
44	48	44	-8.08	-.04	.00	.00	-.21	-.22
46	36	44	10.30	.17	.00	.00	.26	.35
48	44	46	1.95	.00	.00	.00	.05	.02
50	48	50	4.98	.04	.00	.00	.13	.09
LINE 51 IS CLOSED								
52	50	52	2.87	.01	.00	.00	.07	.03
54	450	51	.10	.00	.00	.00	.00	.00
55	57	55	1.91	.00	.00	.00	.01	.00
56	54	56	.57	.00	.00	.00	.00	.00
57	54	55	-1.29	.00	.00	.00	-.01	.00
58	458	60	-11.03	.00	.00	.00	-.05	.00
60	0	60	228.96	1.46	.00	.00	.65	.17
61	66	82	117.15	.10	.00	.00	.75	.47
62	60	62	216.74	.30	.00	.00	.61	.15
63	65	67	2.82	.00	.00	.00	.01	.00
64	65	62	-63.68	-.05	.00	.00	-.18	-.02
65	79	83	.00	.00	.00	.00	.00	.00
66	62	63	150.49	.06	.00	.00	.43	.10
67	63	69	148.37	.04	.00	.00	.61	.25
68	68	69	-8.27	-.13	.00	.00	-.21	-.10
69	66	69	-137.98	-.13	.00	.00	-.39	-.09
70	68	71	15.68	.00	.00	.00	.06	.00
71	72	57	2.53	.00	.00	.00	.01	.00
72	66	68	18.42	.00	.00	.00	.08	.01
73	71	87	11.38	.17	.00	.00	.29	.18
74	72	74	.00	.00	.00	.00	.00	.00
75	71	72	3.15	.00	.00	.00	.01	.00
76	70	76	2.65	.01	.00	.00	.07	.01
77	78	87	-7.58	-.04	.00	.00	-.19	-.09
78	76	78	-3.91	-.02	.00	.00	-.10	-.03
79	82	83	105.83	.03	.00	.00	.43	.13
80	70	78	-2.91	-.01	.00	.00	-.07	-.01
81	82	86	6.75	.13	.00	.00	.17	.07
82	82	84	-1.37	.00	.00	.00	-.02	.00
83	82	64	2.65	.02	.00	.00	.07	.01
84	80	84	2.15	.01	.00	.00	.05	.03
85	83	85	105.21	.09	.00	.00	.43	.13
86	80	68	-9.01	-.09	.00	.00	-.23	-.12
87	85	108	-18.44	.00	.00	.00	-.08	-.01
88	87	89	2.76	.01	.00	.00	.07	.01
89	76	90	4.08	.00	.00	.00	.05	.00
90	80	89	5.77	.09	.00	.00	.15	.16

91	90	92	4.80	.02	.00	.00	.12	.11
92	89	88	6.70	.03	.00	.00	.17	.21
93	92	94	3.71	.06	.00	.00	.09	.07
94	88	90	3.79	.02	.00	.00	.10	.07
96	94	96	2.05	.00	.00	.00	.02	.00
97	40	96	-1.99	.00	.00	.00	-.01	.00
98	98	96	-2.57	.00	.00	.00	-.07	-.01
100	98	100	-1.34	-.01	.00	.00	-.03	-.01
101	100	102	-8.55	-.07	.00	.00	-.22	-.11
102	96	100	-5.73	.00	.00	.00	-.02	.00
104	102	104	-29.89	-.01	.00	.00	-.12	-.01
106	104	106	-32.53	.00	.00	.00	-.13	-.01
107	106	108	-35.14	.00	.00	.00	-.14	-.02
108	107	109	-54.43	-.09	.00	.00	-.35	-.11
109	107	108	54.31	.00	.00	.00	.22	.04
110	110	112	-56.75	-.05	.00	.00	-.36	-.12
111	85	86	123.03	.01	.00	.00	.50	.17
112	112	65	-58.04	-.23	.00	.00	-.37	-.13
113	109	309	.00	.00	.00	.00	.00	.00
114	102	116	2.30	.01	.00	.00	.06	.01
115	109	110	-54.55	-.03	.00	.00	-.35	-.11
LINE	116 IS	CLOSED						
118	102	122	14.95	.01	.00	.00	.10	.01
120	122	118	4.38	.00	.00	.00	.05	.00
122	118	120	1.17	.00	.00	.00	.03	.00
124	120	124	1.09	.00	.00	.00	.01	.00
126	124	122	-8.45	.00	.00	.00	-.05	.00
128	124	126	6.21	.00	.00	.00	.04	.00
130	126	128	3.94	.00	.00	.00	.03	.00
132	128	130	.32	.00	.00	.00	.00	.00
134	130	330	.00	.00	.00	.00	.00	.00
136	128	130	.32	.00	.00	.00	.00	.00
NE	313 IS	CLOSED						
316	79	379	.00	.00	.00	.00	.00	.00
LINE	322 IS	CLOSED						
LINE	332 IS	CLOSED						
LINE	334 IS	CLOSED						
450	450	50	-.10	.00	.00	.00	.00	.00
458	58	458	-11.03	.00	.00	.00	-.05	.00
340	340	40	.00	.00	.00	.00	.00	.00

JUNCTION NUMBER	DEMAND	GRADE LINE	ELEVATION	PRESSURE
2	.00	906.83	755.00	65.79
4	.00	906.83	790.00	50.63
6	.16	906.83	809.00	42.39
8	1.11	906.82	772.00	58.42
9	4.07	906.82	770.00	59.29
10	1.17	906.82	790.00	50.62
12	.15	906.83	780.00	54.96
14	.00	906.83	780.00	54.96
16	1.05	906.82	790.00	50.62
18	1.34	906.82	800.00	46.29
20	.64	906.82	796.00	48.02
22	.00	906.82	820.00	37.62
23	.00	906.82	830.00	33.29
24	.80	906.82	796.00	48.02
28	1.91	906.82	798.00	47.16
30	1.41	906.83	799.00	46.73
34	1.29	906.84	822.00	36.77
36	1.42	906.84	825.00	35.46
38	.16	906.84	825.00	35.46
40	1.99	993.01	825.00	72.80
42	1.58	906.62	820.00	37.53
44	.27	906.67	835.00	31.06
46	1.95	906.67	836.00	30.62

48	1.52	906.63	825.00	35.37
50	2.01	906.59	828.00	34.06
51	.10	906.59	859.00	20.62
52	2.87	906.58	830.00	33.18
54	.72	993.31	859.00	58.20
55	.62	993.31	850.00	62.10
56	.57	993.31	860.00	57.77
57	.62	993.31	850.00	62.10
58	11.03	993.84	830.00	71.00
60	1.19	993.84	877.00	50.63
62	2.57	993.55	856.00	59.60
63	2.12	993.49	882.00	48.31
64	2.65	993.20	883.00	47.75
65	2.82	993.50	825.00	73.02
66	2.41	993.32	875.00	51.27
67	2.82	993.50	760.00	101.18
68	2.00	993.31	881.00	48.67
69	2.12	993.45	882.00	48.29
70	.26	993.10	865.00	55.51
71	1.15	993.31	874.00	51.70
72	.62	993.31	880.00	49.10
74	.00	993.31	863.00	56.47
76	2.48	993.09	872.00	52.47
78	.76	993.11	848.00	62.88
79	.00	993.18	840.00	66.38
80	1.09	993.22	872.00	52.53
82	3.29	993.22	870.00	53.39
83	.62	993.18	865.00	55.55
84	.78	993.22	878.00	49.93
85	.62	993.10	866.00	55.07
86	.00	993.08	866.00	55.07
87	1.04	993.14	848.00	62.90
88	2.91	993.11	867.00	54.65
89	1.83	993.14	868.00	54.23
90	3.07	993.09	862.00	56.80
92	1.09	993.07	861.00	57.23
94	1.66	993.01	838.00	67.17
96	3.22	993.01	838.00	67.17
98	3.91	993.00	833.00	69.33
100	1.48	993.01	830.00	70.64
102	4.09	993.08	828.00	71.53
104	2.64	993.09	852.00	61.14
106	2.61	993.09	857.00	58.97
107	.12	993.10	865.00	55.51
108	.73	993.10	865.00	55.51
109	.12	993.20	815.00	77.22
110	2.20	993.22	830.00	70.73
112	1.29	993.27	842.00	65.55
116	2.30	993.07	840.00	66.33
118	3.21	993.07	809.00	79.76
120	.08	993.07	802.00	82.80
122	2.12	993.07	821.00	74.56
124	3.33	993.07	822.00	74.13
126	2.27	993.07	832.00	69.80
128	3.30	993.07	840.00	66.33
130	.64	993.07	800.00	83.66
309	.00	993.20		
324	.00	906.82		
330	.00	993.07		
334	.00	906.84		
340	.00	993.01		
379	.00	993.18		
450	.00	906.59	826.00	34.92
458	.00	993.84	837.00	67.96

THE NET SYSTEM DEMAND = 126.16

SUMMARY OF INFLOWS(+) AND OUTFLOWS(-) FROM FIXED GRADE NODES

PIPE NUMBER	FLOWRATE
13	-129.78
36	26.98
60	228.96

THE NET FLOW INTO THE SYSTEM FROM FIXED GRADE NODES = 255.94
THE NET FLOW OUT OF THE SYSTEM INTO FIXED GRADE NODES = -129.78

TANK STATUS REPORT

TANK NO.	CONN. PIPE	NET FLOW	ADJ. HGL	WATER SURFACE ELE.	PROJECTED W.S.E.
1	60	-228.96	993.84	995.30	994.73
2	36	-26.98	906.84	906.84	905.21
3	13	129.78	993.08	993.04	995.34

Town of Appomattox
Well and Booster Pump Station Data

WELLDATA.WQ1

	Est. Ground Elev. at Well	Bottom of Well Elevation	Pump Elevation	Static Water Level Elevation (See note 1)	Pumping Water Level Elevation (See note 1)	Flow Rate (gpm)	Throttling Valve Type	Estimated Head Loss Thru Valve (ft) (See note 2)
Well #1	822	722	732	796	783	70	Gate	125
Well #5	796	686	696	751	735	62	Gate	13
Well #9	815	610	660	706	672	83	Butterfly	54
Well #15	840	590	625	685	652	105	Butterfly	0
Well #25	800	495	548	691	571	42	Gate	98
Booster Pump	--------	--------	--------	--------	--------	80	Gate	32

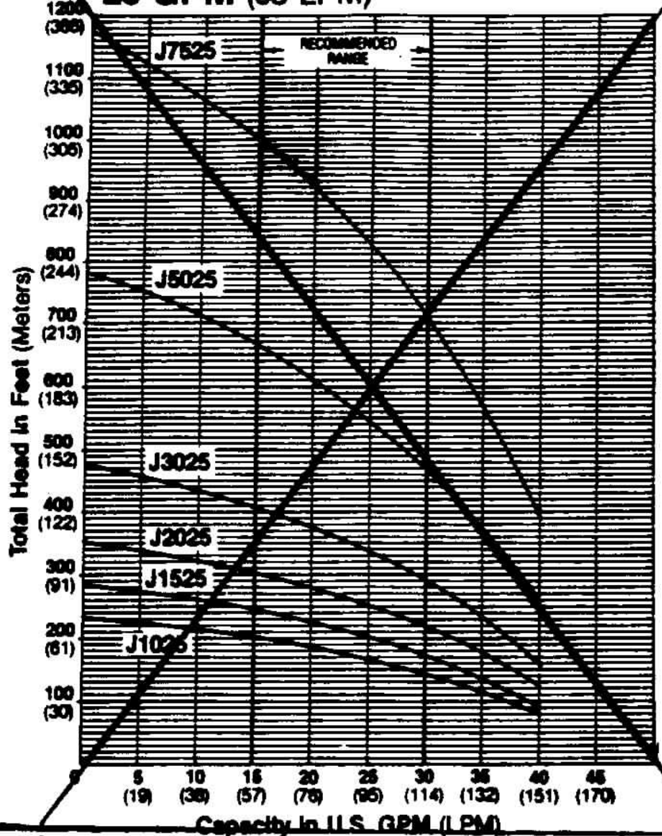
Note 1: Based on tests done in Town June 25, 1990-August 7, 1990

Note 2: Estimated using KYPIPE modeling program

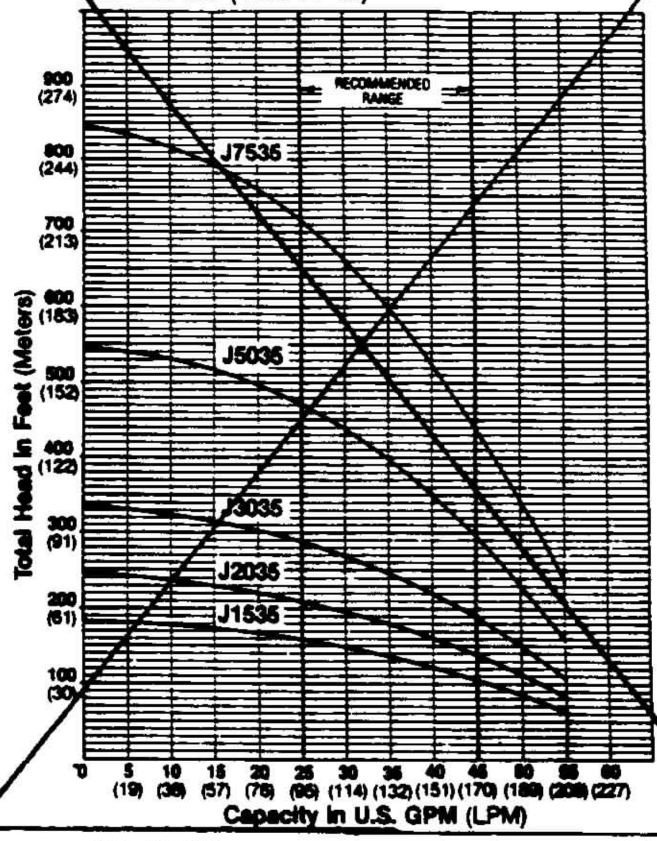
140/12

SJ PERFORMANCE CURVES

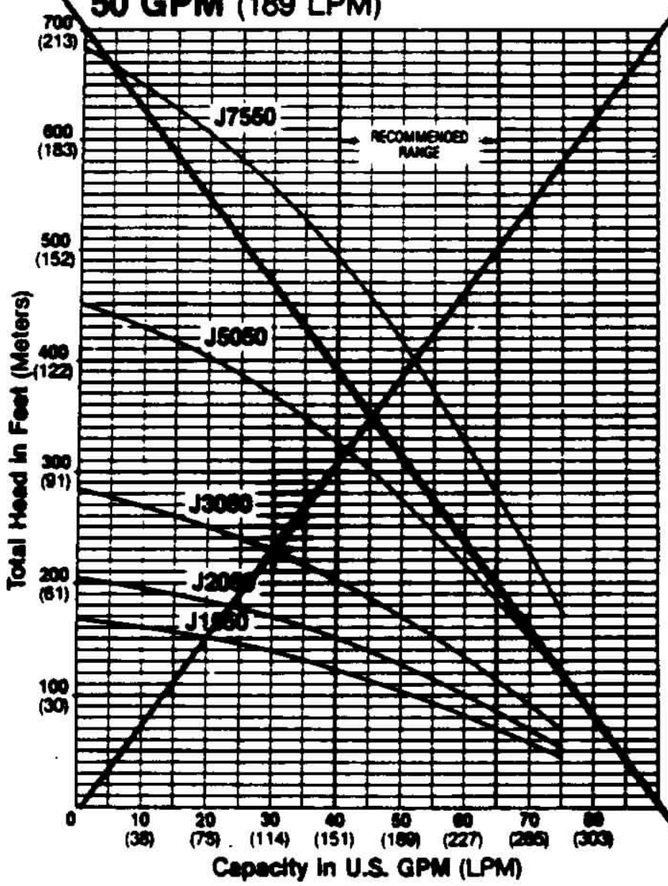
25 GPM (95 LPM)



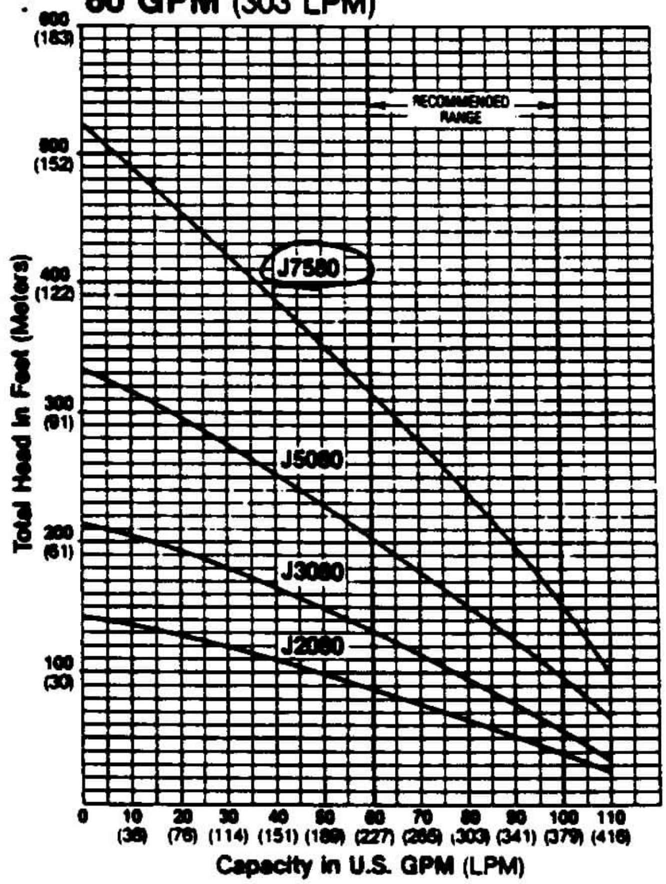
35 GPM (132 LPM)



50 GPM (189 LPM)



80 GPM (303 LPM)



WELL #1 J7580

Myers®
F E Myers, A Pentair Company, 1101 Myers Parkway, Ashland, Ohio 44805-1923 • 419/289-1144 • FAX 419/289-6658, TLX 98-7443

WELL #5 5HP-LB

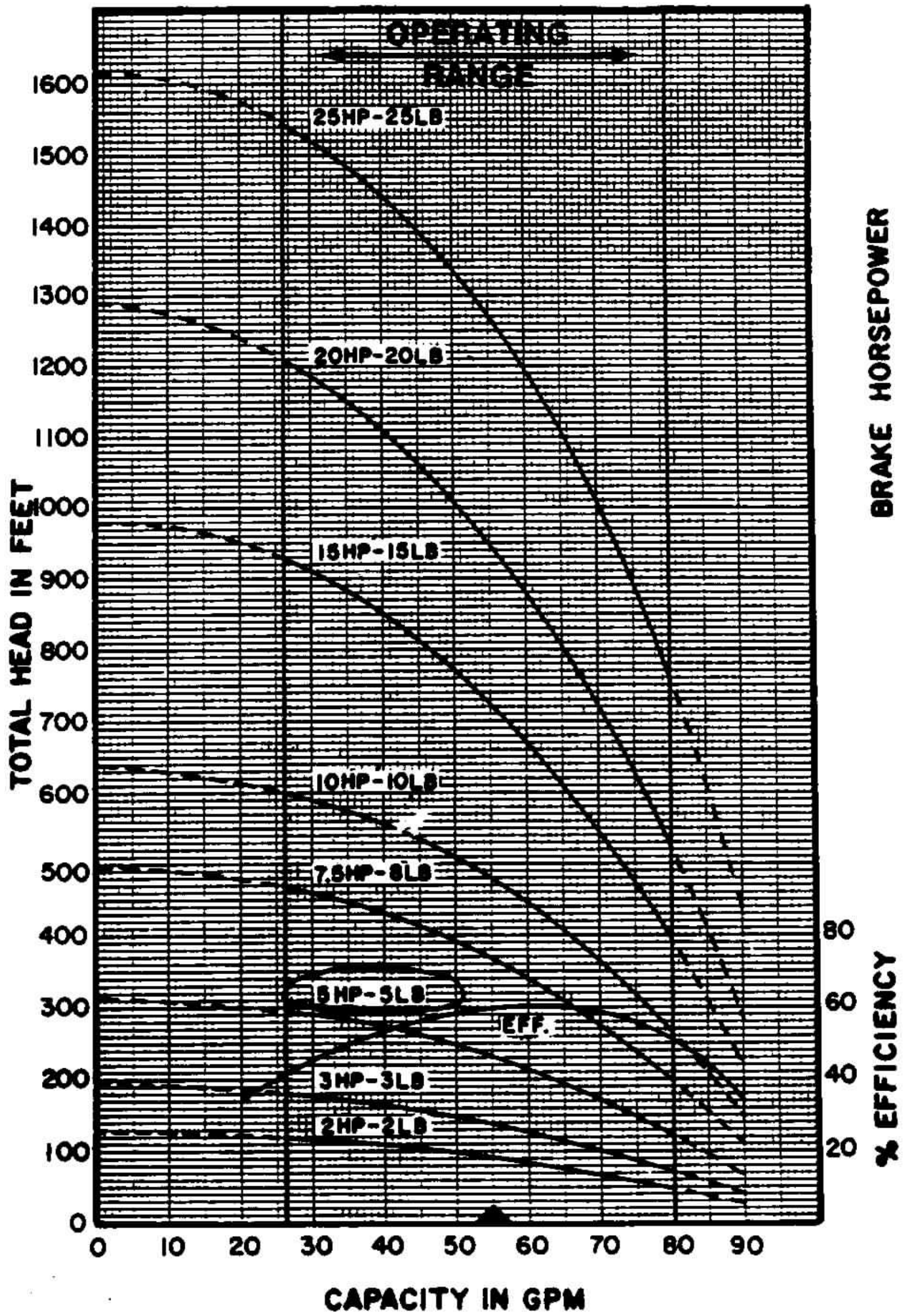
RED JACKET PUMPS
A Marley Pump Company

BIG-FLO®

PAGE: 2-7
EFFECTIVE: SEPTEMBER 1, 1987

55 GPM SERIES "LB" PUMPS

PERFORMANCE CURVES



▲ RATED FLOW

Note: Continuous operation outside operating range will void warranty.

MODEL LB 55 GPM — RPM 3450
STAGE "LB" SERIES

GUARANTEED AS MINIMUM PERFORMANCE ONLY IF CERTIFIED

MINIMUM WELL SIZE 6" I.D.

WELL #9 15HP-6HB

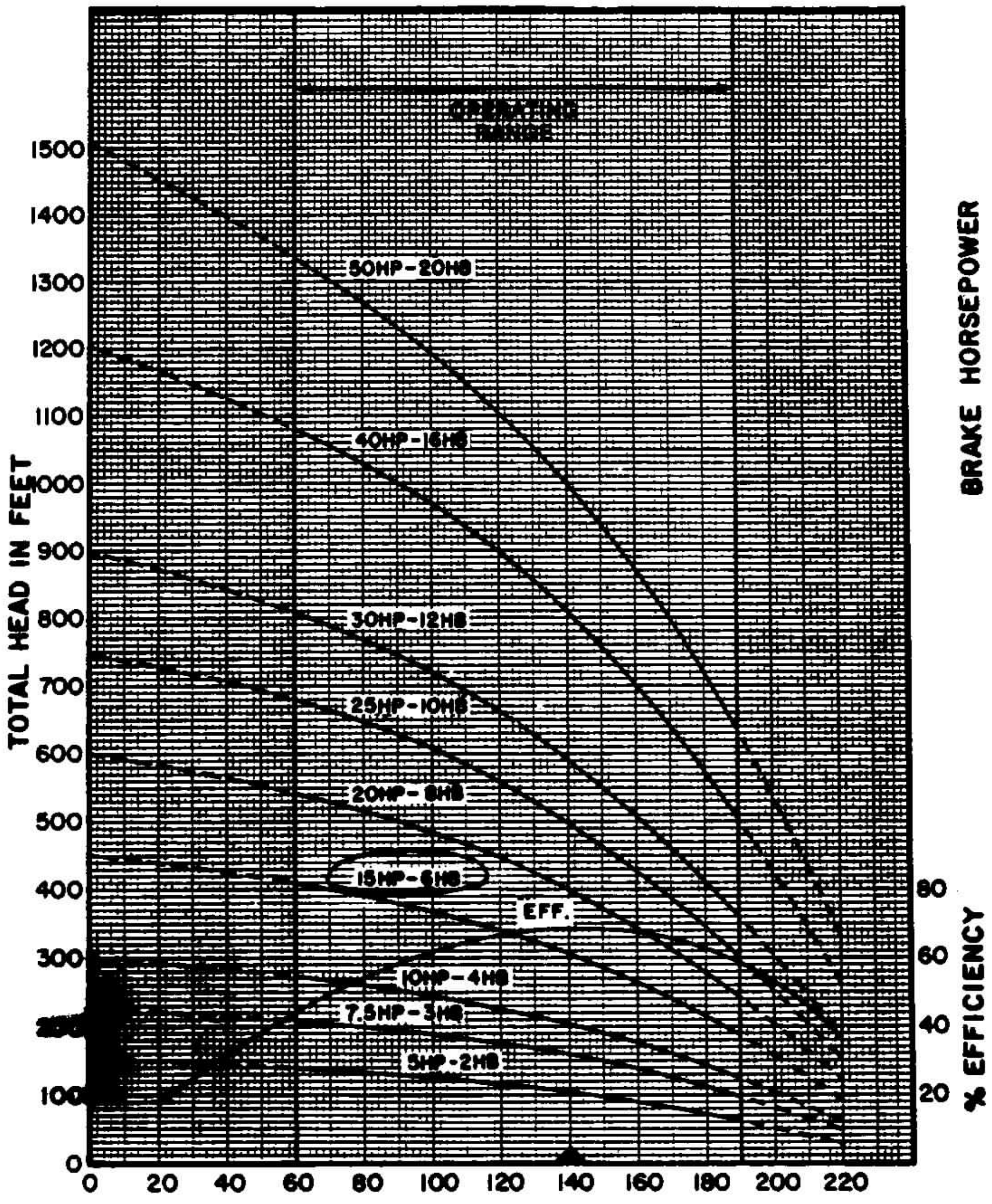
RED JACKET PUMPS
A Marley Pump Company

BIG-FLO®

EFFECTIVE: SEPTEMBER 1, 1987

140 GPM SERIES "HB" PUMPS

PERFORMANCE CURVES



▲ RATED FLOW

Note: Continuous operation outside operating range will void warranty.

MODEL HB 140 GPM — RPM 3450
STAGE "HB" SERIES

GUARANTEED AS MINIMUM PERFORMANCE ONLY IF CERTIFIED

MINIMUM WELL SIZE 6" I.D.

WELL #15 15HP - 6HB

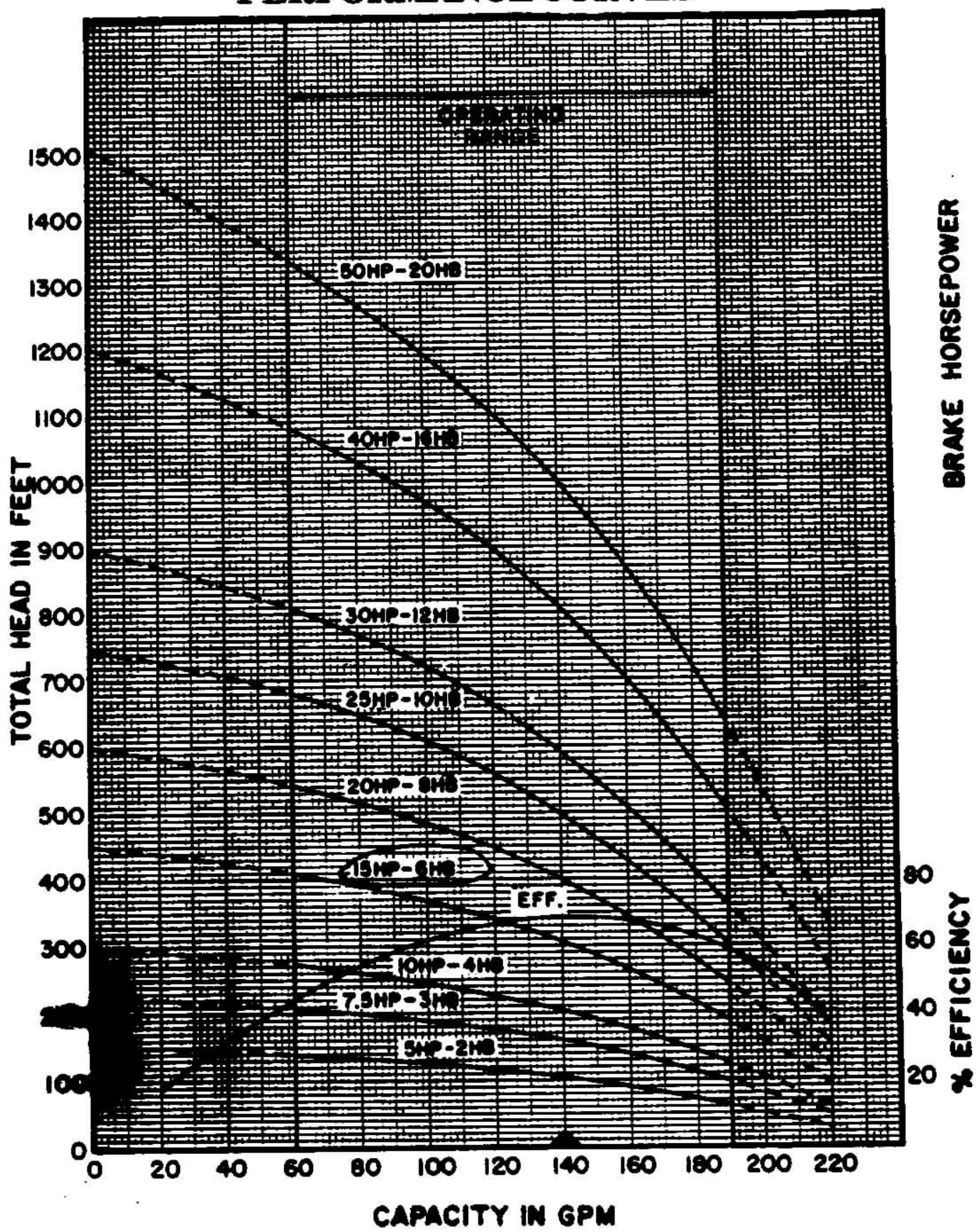
RED JACKET PUMPS
A Marley Pump Company

BIG-FLO®

PAGE: 2-11
EFFECTIVE: SEPTEMBER 1, 1987

140 GPM SERIES "HB" PUMPS

PERFORMANCE CURVES



▲ RATED FLOW

Note: Continuous operation outside operating range will void warranty.

MODEL HB 140 GPM — RPM 3450
STAGE "HB" SERIES

GUARANTEED AS MINIMUM PERFORMANCE ONLY IF CERTIFIED

MINIMUM WELL SIZE 6" I.D.

WELL #25 10HP-10LB

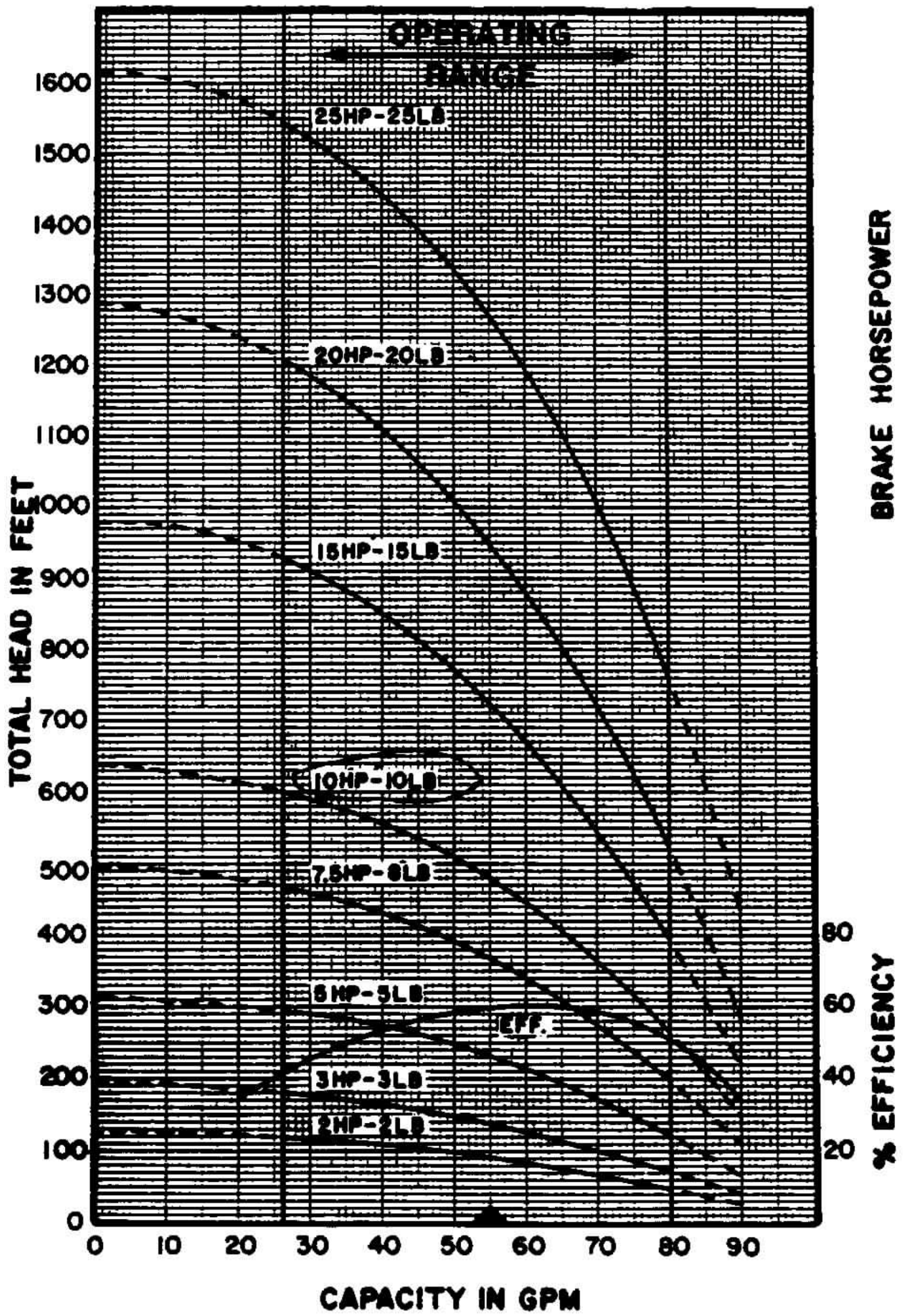
RED JACKET PUMPS
A Marley Pump Company

BIG-FLO®

PAGE: 2-7
EFFECTIVE: SEPTEMBER 1, 1987

55 GPM SERIES "LB" PUMPS

PERFORMANCE CURVES



▲ **RATED FLOW**

Note: Continuous operation outside operating range will void warranty.

MODEL LB 55 GPM — RPM 3450
STAGE "LB" SERIES

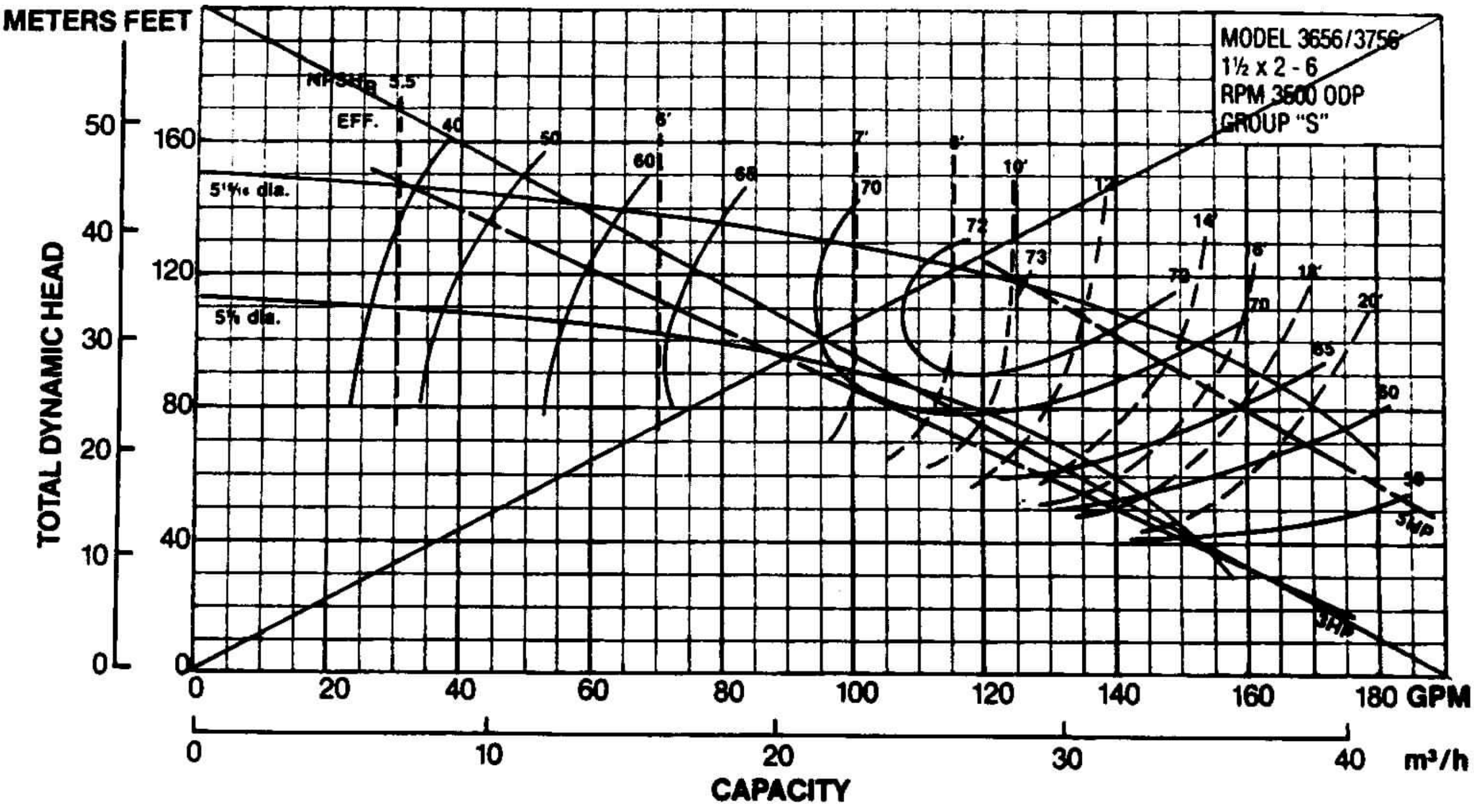
GUARANTEED AS MINIMUM PERFORMANCE ONLY IF CERTIFIED

MINIMUM WELL SIZE 6" I.D.

BOOSTER PUMP
MODEL 3656/3756
IMP. DIA. 5 5/8"

Performance Curves

"S" Group Pumps



G&L