The City of CLOVIS

Water Master Plan Update

Phase II

Facilities Plan

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PHASE II - FACILITIES PLAN

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The City of CLOVIS

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INTRODUCTION

The City of Clovis has relied exclusively on groundwater for meeting the water supply needs of the community. Most cities and communities that are located along the floor of the San Joaquin Valley have similarily grown accustomed to this seemingly endless source of water. It is pristine, cool, and refreshingly palatable to the taste. We take for granted that this groundwater supply will be there, forever. Such is not the case. With the easterly development of the City towards the foothills, the aquifer of the valley is left behind. Other means of supply and must be developed to sustain the growth of the community.

This report represents the results of several years of work by the City and its consultants. It has been undertaken to develop a long-term approach to planning for the water development and supply needs for the City into the 21st century. Technical memorandums were prepared over the course of the work and subsequently edited to form the bulk of this document. The planned land uses in the 1993 General Plan are the blueprint upon which this study is based. Several special study areas have also been identified since the acceptance of the General Plan. Specific water system studies for the special study areas have been limited; with the help of staff, general land use designations were made for the areas and subsequently incorporated herein.

Special attention was given to the significantly decreasing groundwater levels in the existing service areas and specific concerns related to water quality. During the course of this study, some ongoing litigation has been resolved and the City now has guidelines and costs related to mitigating DBCP groundwater contamination. The results are incorporated into this study.

Phase I of this Plan Update, which was completed in April, 1995, investigated three alternatives to meet water supply needs at buildout of the General Plan area. These included: 1) total reliance on groundwater and groundwater recharge; 2) large scale use of surface water as the principal supply; and 3) a combination of groundwater, groundwater recharge and surface water (conjunctive use).

It was determined that a conjunctive use program is the most cost effective and implementable alternative to maximize the resources available to the City. This alternative was approved by the City Council in July 1995 and used as the basis for completion of this Phase II Facilities Plan. This strategy includes utilizing both groundwater and treated surface water to provide a secure, drought-resistant water supply. The recommended plan has been structured to be cost-effective and operationally efficient. In addition, it has been developed to be conducive to phased development, which is critical both to community approval and existing operational constraints. The phased development approach allows the City to provide the needed facilities just in time to serve the increasing demands of growth. The rate at which growth occurs will dictate the implementation schedule for construction of new water supply and delivery facilities. It should be noted that the facilities described in this plan are needed not only for growth but some are also needed to reverse the current downward trends of groundwater levels. The aquifer beneath the City is in an overdraft condition and while recharge efforts will continue, it is neither physically nor financially feasible to offset current and future overdraft entirely by groundwater recharge. This study has attempted to identify those facilities required to address current system deficiencies as well as those facilities required for continued growth, and to identify the distinction between the two needs.

ULTIMATE DEMANDS

Based upon the land use designations in the 1993 General Plan, projected water delivery requirements were determined for the study area. At buildout (year 2030 and a corresponding population of 200,000 people) the average demand for City water will be 52,500 acre-feet per year. This represents an average annual per capita use of 250 gallons per capita per day. Two design parameters that most affect the water distribution system are the maximum daily demand and the peak hour demand. At the planning horizon, these two values are 66,000 and 94,000 gallons per minute, respectively; at present, these demands are 22,000 and 32,000 gallons per minute, respectively. Projected growth will roughly triple the peak need for water deliveries. It is planned that new supplies will be brought on line in accordance with increased demands.

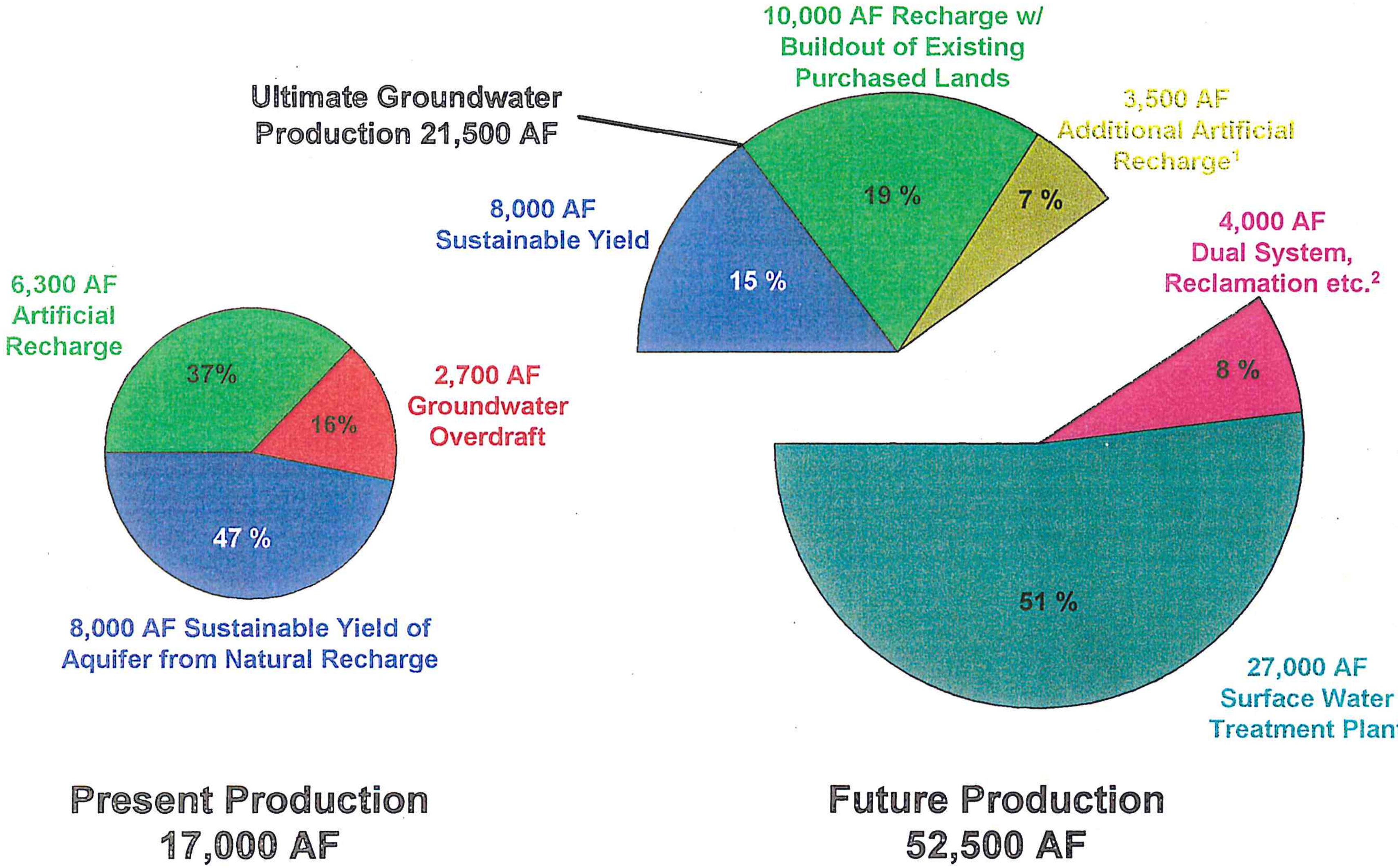
It is envisioned that treated surface water will eventually provide approximately 50 percent of total annual supplies. Groundwater will satisfy 40 percent, and a combination of untreated surface water and/or reclaimed water for outside landscape purposes will satisfy the remainder; this is depicted graphically in **Figure ES-1**. Should reclaimed water or untreated surface supplies not be viable, the treated surface supplies must increase a like amount.

At this time, it is intended that only the planned urban lands are to be served water from the system. However, it is probable that some rural residential properties in close proximity to the system will request service. It may be cost-effective for the City system to serve in-the-house water demands in nearby rural residential lands. Should this be desirable in the future, water demands would be higher and raw water supplies must be adjusted upward.

Seasonal Demands

Seasonal fluctuations in demand will allow the City to optimize surface water delivery so that groundwater resources can be available during extended droughts. To do so, the surface water treatment plant (SWTP) will be base-loaded as shown in **Figure ES- 2** to maximize its water production capabilities. At or near year-round use of treated surface water will allow the City to "bank" groundwater for later use during summer months or protracted drought periods. This would be accomplished by de-activating certain wells during low demand periods. As shown on the figure, the treatment plant will either be shut down during winter canal maintenance, or operate at reduced flows due to decreased seasonal demands. The wells with granular activated carbon treatment must be operated year round to maximize their effectiveness in removing organic contaminants.

Figure ES-1. WATER MASTER PLAN ANNUAL WATER SUPPLY SUMMARY



1. Approximately equal to one additional Marion site
2. Equivalent to 200 Letterman Park Systems

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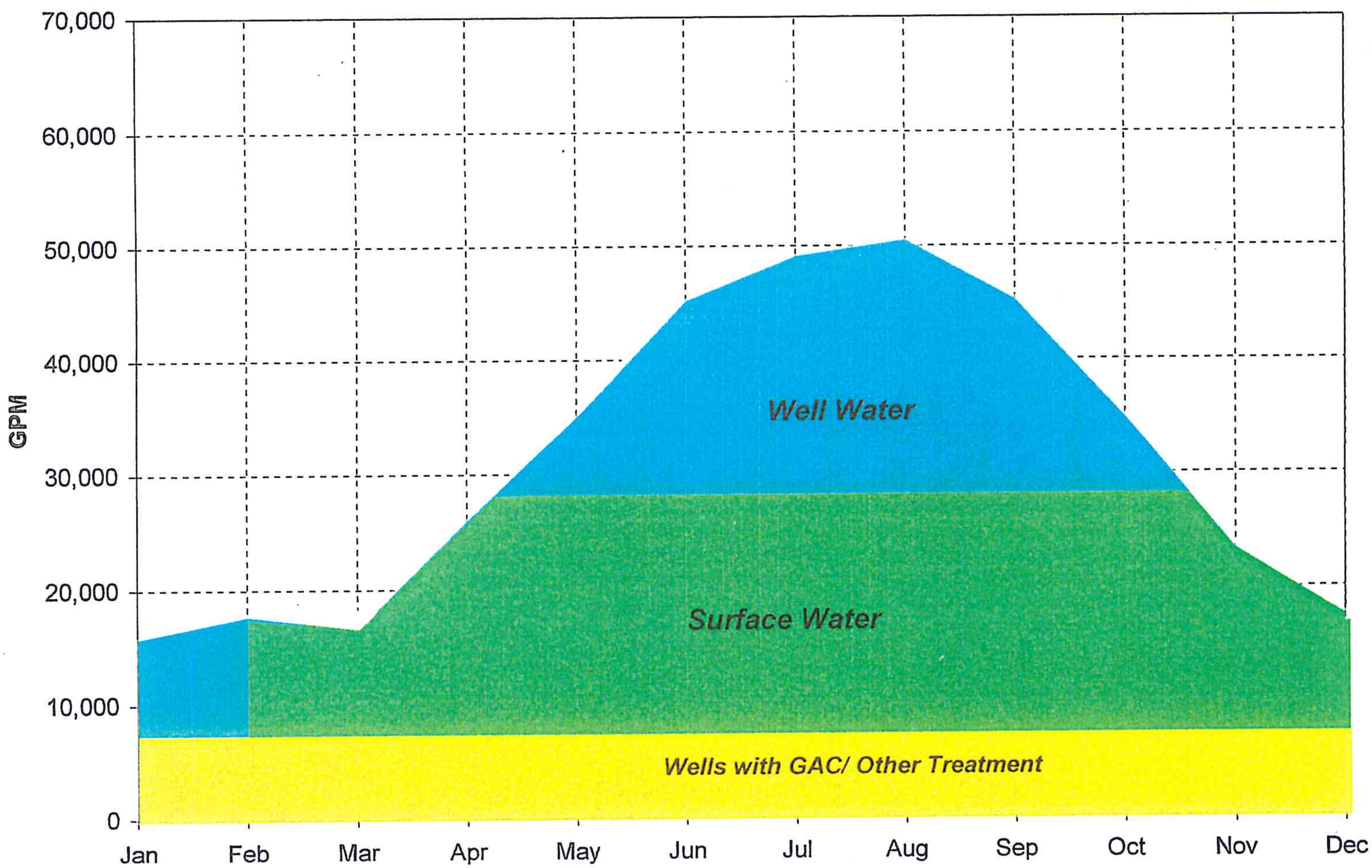
ES-2. Annual Monthly Water Demand and Supply At Buildout (Year 2030)



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GROUNDWATER

Groundwater will remain a significant portion of the ultimate supply. An increase of 20 percent over existing production will be needed under buildout conditions, mainly from wells west of Clovis Avenue. To allow this continued development of groundwater, annual recharge volume will have to be increased to match the production of new wells. Purchase and siting of approximately 160 acres of additional recharge basins after the completion of the Marion & Alluvial site will be necessary. Since suitable sites for recharge are limited and subject to near-term development, the City should move quickly to secure the necessary acreage.

Some basic information that should be kept in mind to understand groundwater conditions in the area:

- The aquifer is thickest under the southwest portion of the City, generally south of Herndon and west of Clovis Avenues. To the north and east of Clovis, the aquifer becomes substantially thinner and bedrock becomes shallow, with a resulting reduction in water production capacity.

- Planned growth areas are less favorable for groundwater recharge or well development than in the existing City of Clovis.

- Surface and subsurface geologic conditions favorable for intentional recharge are limited. The areas most favorable for intentional recharge activities are along Dry Creek and other stream channels.

- Groundwater quality varies widely over the study area; the most favorable areas for groundwater quality lie west of Clovis and South of Bullard Avenue. DBCP contamination is the constraint North of Bullard Avenue.

Since groundwater will remain a major source of water for the City, Clovis must assume a more active role in monitoring, recharging, and managing this valuable resource.

SURFACE WATER

Rights to existing and future surface water quantities will not be sufficient to meet future annual requirements at buildout of the General Plan Area. By the end of the planning period, the City will need to have between 5,000 and 14,000 acre-feet per year of additional surface water available to meet the projected total annual demand of 52,500 acre-feet. The range in additional quantities needed results from several factors which could influence the actual additional amount needed.

The factor most affecting the additional need is the location of the planned growth. About 40 percent of the new growth is included in the Northeast Village, which land area has entitlement to little surface water. Current surface water supplies to this area represents about 9 percent of that needed to meet the water needs of the proposed Village. The high end of the range (14,000 acre-feet) represents the additional amount of surface water that would be needed above the amount currently available.

The additional quantity of surface water needed at buildout could be reduced to the low end of the range (5,000 acre feet) from actions taken by the City. These include the potential direct utilization of reclaimed water for irrigation of large landscaped areas if this source became available from satellite water reclamation facilities. Alternatively, reclaimed water from satellite facilities or from the Regional Wastewater Treatment Plant could be exchanged for surface water from the Fresno Irrigation District or others. This supply, while developed regionally, would be of specific benefit for water short areas like the future Northeast Village because in general, the other growth areas are located where surface water supplies are adequate.

Even though the additional supplies will not be needed until about 2020, the City should immediately begin investigating potential sources or suppliers of surface water. This action is even more important after the recently enacted Federal legislation which allows federally developed supplies to be transferred for use outside their historic boundaries, and requires Federal agencies to find additional supplies for environmental purposes. The marketplace has been set, and supplies will become more limited which will cause the price to rise with time.

FUTURE SYSTEM DESCRIPTION

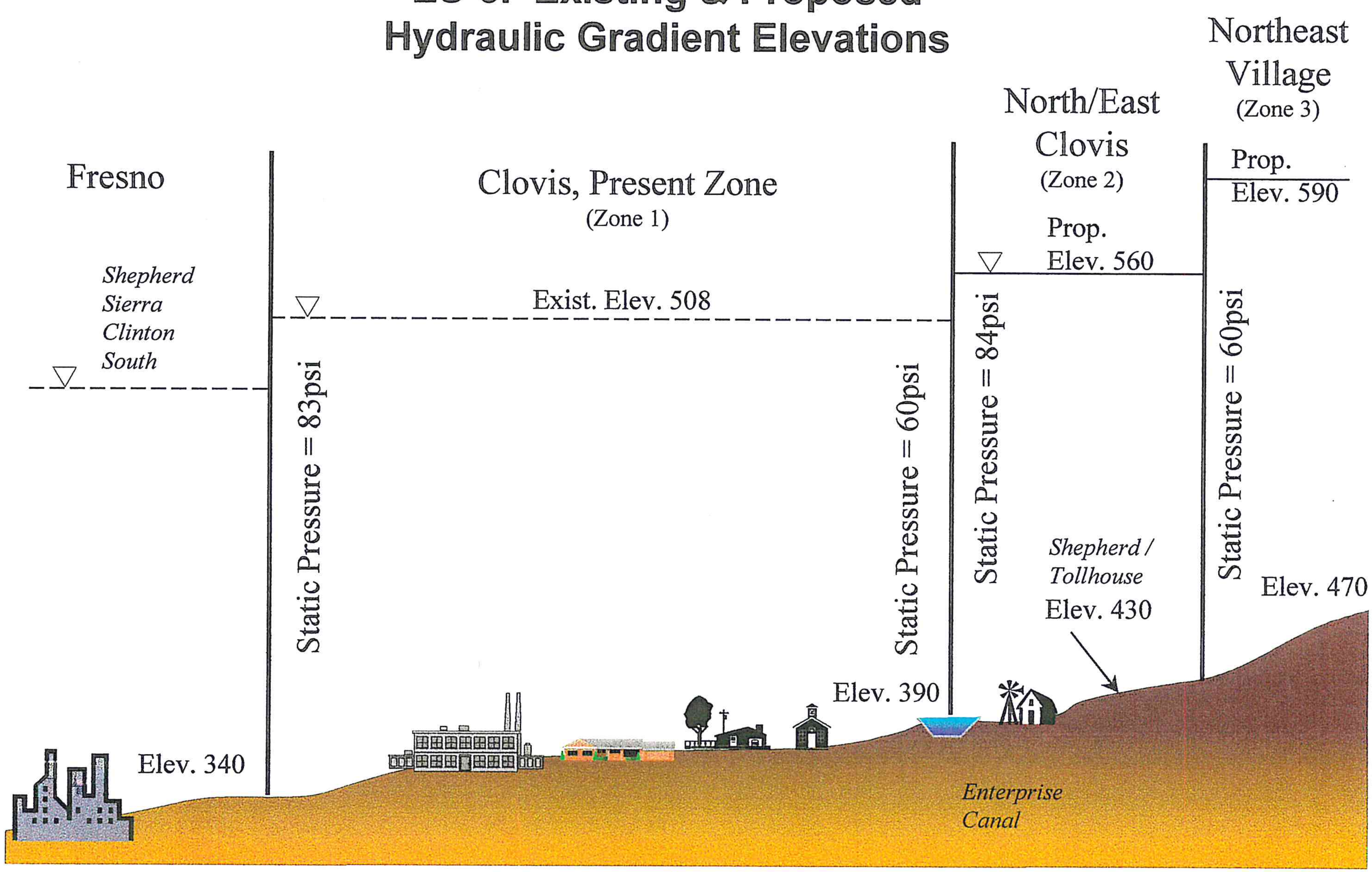
Analysis of the preceding water demands and supplies presents an overall picture of the future water supply and distribution system. The inclusion of a surface water treatment facility requires some changes in the present mode of operation of the distribution system. The following is a summary description of the future system.

Pressure Zones

Lengthy discussions of the present system with operating staff and analyses of present operations has revealed several important facts. Although the system is operating satisfactorily at present, low pressures are sometimes experienced in the northern and eastern portions of the system. The existing hydraulic grade line (HGL) has been increased over the years as growth has occurred to the east and northeast. A hydraulic grade line (HGL) is a graphic representation of the pressure existing in a water system. If one were to cut a hole in a pipe and insert a tube, the HGL is represented by the height, to which water in the pipe would rise in the tube. The land in Clovis and its environs rises toward the east and northeast. As the City has grown in this direction, so has the water system. Most of the water production is in the central and south westerly (lowest) part of the City. The net result is that to maintain adequate pressure in the east and northeast, pressures (the HGL) must be higher in the southwest. The HGL has been increased above the top of the elevated storage tanks, making them ineffective as peaking storage in their present configuration. In addition, the increased HGL has caused leaks in the older portion of the distribution system.

A further impact of the higher pressure levels is that energy requirements (and associated costs) for pumping have increased due to the increased pressures in the system. From inspection of **Figure ES-3**, it is clear that continued increase of the hydraulic grade line is not an adequate solution to providing service to the newer parts of the City, in predominantly higher elevation areas.

ES-3. Existing & Proposed Hydraulic Gradient Elevations



To resolve many of these issues, the future system is planned to have three pressure zones as shown on **Figure ES- 3**. This figure shows schematically the design hydraulic grades for the separate zones. Although separate pressure zones will provide better overall pressure throughout the City, the existence of zone boundaries complicates the transfer of water between zones. The recommended plan includes additional storage tanks and pumping facilities to accommodate interzonal transfers of water.

Reduction of demand for treated water

To reduce treated water requirements, several large water demands presently supplied with potable water may instead be supplied with alternate sources of water. This does not decrease the amount of water needed by the City only whether the water needs to be pumped from the aquifer or processed through the surface water treatment plant. Possible sources include water from the irrigation canals and treated effluent from a wastewater reclamation facility; either source could be delivered through the same facilities. There are many irrigation canals and laterals that cross the City; all are capable of supplying water for outside landscaping purposes. Areas that appear to be favorable for this type of service are:

- Letterman Park (presently supplied with canal water)
- Clovis school sites in close proximity to canals (Reagan, Buchanan complexes)
- Rural residential properties north of Nees Avenue and adjacent to Enterprise Canal
- Greenway/ beltways along transportation corridors and trails

Both the Clovis Wastewater Master Plan and a Regional Satellite Wastewater Study are underway; implementation of a water reclamation facility resulting from either of these planning scenarios would result in reclaimed water for such uses, or present an opportunity of exchange for canal water.

ENERGY

This study did not directly review or evaluate energy issues. Currently, the State of California is embarking on a new era of free enterprise and energy deregulation. This is fostering opportunities to save money by shifting providers, changing to alternative fuels, and changing and manipulating rate schedules and pumping patterns. One basic fact remains-- the future load profile of the City water system will closely resemble the current load profile, whereby:

- Pumpage of water will occur to meet demand,
- The energy load profile has peak daily pumping in the early morning and late evening hours, thus water temporarily stored in tanks will continue to be repumped for later use, and
- More than 50 percent of electricity demand is in the months of May, June, July, and August.

Two energy cost reduction opportunities will occur when the recommended facilities are in operation. The first is the significant load which will be established when the surface water treatment plant is constructed. There may be an opportunity to construct a substation and buy power directly at transmission line voltage, with a significant savings in rates. A second opportunity exists in the design and layout of booster pump stations. Since there is a planned transfer of water from zone 1 to 2, there may be an opportunity at certain times to utilize a booster pump rather than discharge to a tank and repumping to the higher zone. Final design of these facilities should explore the required capital costs and energy cost savings potential, keeping in mind that interzonal transfers will be intermittent throughout the year.

In addition to the above potential kilowatt savings, it is likely that the City can reduce energy costs by proper selection of service tariffs and rates. It is recommended that the City continue to review the rates for each existing service. Significant savings may be realized by placing wells on the best energy rate tariffs and monthly reviewing all energy bills.

RECOMMENDED FACILITIES

Figure ES-4 is a map showing the planned facilities to meet build-out conditions of the General Plan. Major features consist of an ultimate 30 MGD surface water treatment plant located adjacent to the Enterprise Canal. The proposed location is near the canal somewhere between Shaw and Bullard Avenues. Other features include 160 acres of additional recharge basins, the addition of 27 new wells, and 75.3 miles of conveyance and distribution piping varying in size from 12 to 36 inches; 3 million gallons of new storage is also planned.

COSTS

As documented later in this report, project costs for the proposed water supply facilities are estimated at approximately $92 million. These costs are broken down over the various phases as shown below.

Table ES-1
Estimate of Capital Costs for
Supply Facilities

Phase Period	Water Supply Facilities	Est. Cost (million)
Near Term	1 well	$ 0.5
To 2005	Surface Water Treatment Plant (5MGD) 40 Ac. recharge basin	14.7
To 2010	2 wells Letterman Park Pump Station Alternate System (1000 AF) 1 million gallon storage tank and booster station	4.6
To 2020	9 wells Surface Water Treatment Plant Expansion (15 MGD) Ashlan booster (2500 gpm) 60 ac. recharge basin	32.5
To 2030	15 wells Surface Water Treatment Plant Expansion(10 MGD) 2 million gallon storage 60 acres recharge basin	39.0
	TOTAL	91.3

Figure ES-4
City of CLOVIS
WATER FACILITIES
1998 - 2030

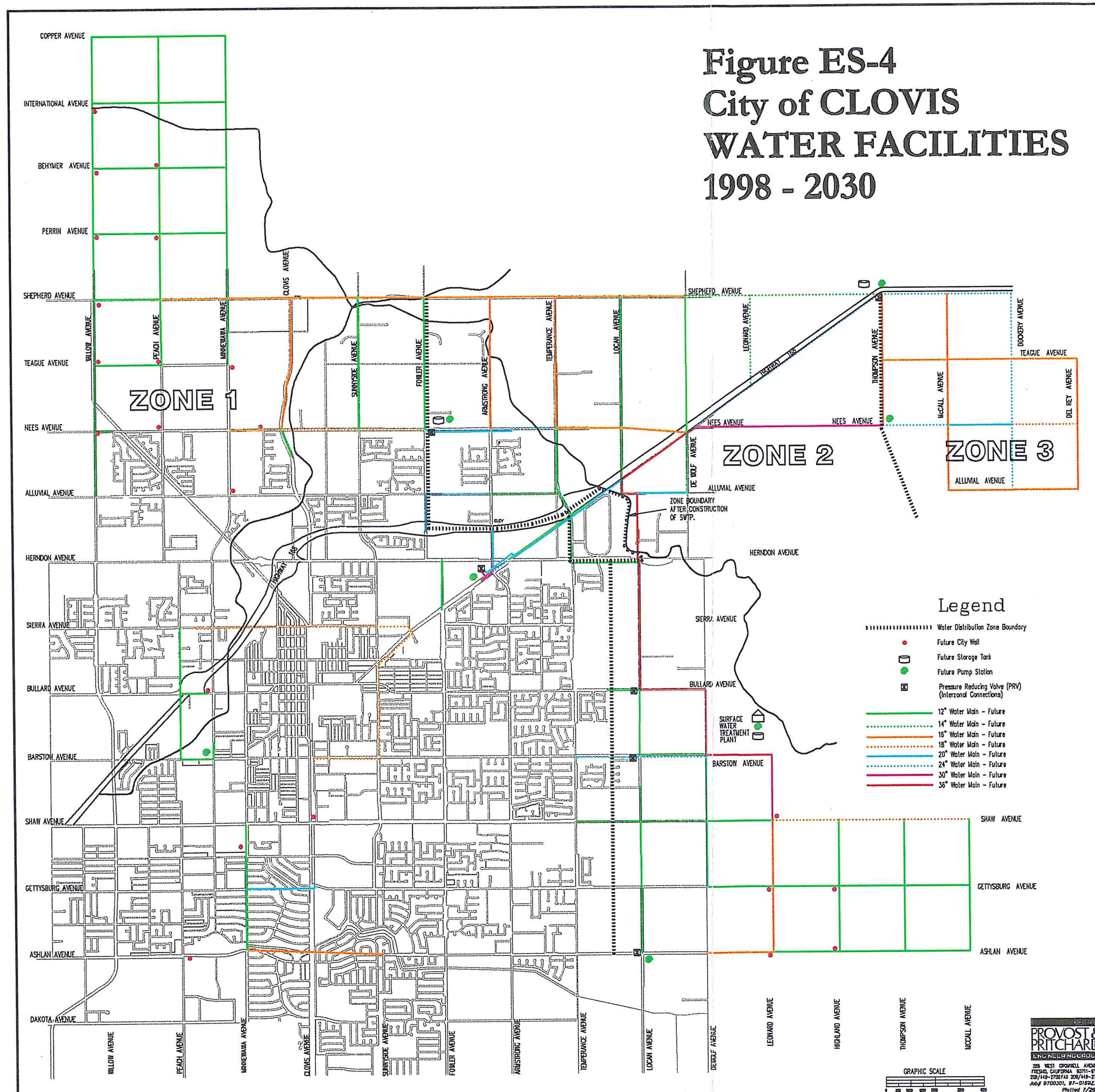


Table ES-2
Estimate of miles of Distribution Piping

Year	Piping Size (in)							
	12	14	16	18	20	24	30	36
2000	3.3		3.0	1.8		3.0		
2005	7.8		2.5	0.5		2.0		0.5
2010	8.5	1.0	4.3	0.8				
2020	14.8		2.3					3.4
2030	3.0	5.0	1.0	1.0	0.5	4.0	1.3	
TOTAL	**37.4**	**6.0**	**13.1**	**4.1**	**0.5**	**9.0**	**1.3**	**3.9**

It should be noted that significant capital expenses occur in the early years. The most significant overall expense is for providing treatment to the raw water supply either through a treatment plant or intentional recharge facilities. Also, the phases are based on estimated development activity. If the rate of activity increases the improvements required to serve the development would need to be expedited. Conversely, if development activity subsides, construction of water supply improvements needed to serve development may be able to be delayed.

RECOMMENDED ACTIONS

The City is currently pursuing many of the following activities that are consistent with this plan:

1. The most significant of the on-going activities is the purchase and development of properties for intentional recharge to help offset the current groundwater overdraft. Since suitable sites for recharge are limited and subject to near-term development, the City should move quickly to secure the necessary acreage.
2. Negotiation of a water supply and delivery contract with Fresno Irrigation District.
3. Pursuing funding and corrective actions for conveyance of municipal water supplies in FID Enterprise Canal.
4. Initiating pilot studies evaluating micro-filtration process for surface water treatment plant to determine effectiveness for compliance with water quality requirements.
5. Pursuing low-interest funding for water supply development through State of California.
6. Preparing energy evaluations of the water supply system.
7. Measure water levels semi-annually to evaluate groundwater level changes and mitigation of overdraft.
8. Developing a water level monitoring program using existing wells in the western portion of City.
9. Developing groundwater quality monitoring programs for the well field and intentional recharge programs.
10. Initiating discussions with FMFCD and Corps of Engineers on the possibility of using Big Dry Creek Reservoir for intentional recharge.
11. Initiating designs for a booster station to pump water from Zone 1 to Zone 2.
12. Performing pump tests on existing wells to verify unit costs and well efficiencies and provide information to assist in establishing a replacement fund.
13. Drilling test wells in the northwest to verify soil conditions. Drilling test holes with zonal sampling in a manner to test for DBCP contamination.
14. Continuing to meet with the City of Fresno regarding assignment of a portion of Fresno's CVP contract water to Clovis.

THE CITY OF CLOVIS
WATER MASTER PLAN UPDATE
PHASE II - FACILITIES PLAN

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PART 1 - INTRODUCTION

Under a previous contract to conduct Phase I of the City's Water Master Plan Update, Provost & Pritchard Engineering Group, Inc. (P&P) performed a review of total supplies available to the City of Clovis, and of total projected consumption of potable water. The recommendations of that Phase I Study, completed in April 1995, included additional recharge facilities to supplement the present well supplies and also indications that a treated surface water supply may become necessary within five to ten years. P&P subsequently contracted with the City to perform Phase II of the Master Plan Update for the water supply and distribution system. This phase of planning is to determine future water infrastructure needs and identify facilities to economically satisfy those needs. Recommendations for facility staging is to be provided, along with budget level estimates of present day costs. The resulting plan will allow the City to schedule improvements and expansions to the system in an organized and economical manner and to be fully prepared to accommodate anticipated growth.

Phase II builds on the findings of the Phase I report and satisfies the following goals:

- Determine the need for a surface water supply for the City, both in terms of schedule and quantity.
- Evaluate alternative sources of supply, locations, and related capital improvements required.
- Review the effects of recommendations from a parallel wastewater master plan, specifically the water supply impacts of possible effluent reuse.
- Develop a model of the present and future distribution system for future use.
- Determine the adequacy of the City's present distribution and storage system, specifically with regard to future extensions and added consumption.
- Recommend facilities to supplement water production, distribution, and storage.

The results of our work are described in a series of individual technical memoranda, each of which discusses one or two key elements of the overall Plan Update. The memoranda were issued as work progressed and revealed our findings as quickly as possible to solicit input and direction from the City. Near the conclusion of our work the various technical memoranda were consolidated to form the basis of the Phase II Master Plan Update.

This Technical Memo No. 1 presents a summary of the first step in the Phase II Master Plan Update: definition of present and projected future consumption. The data contained herein will be used in future efforts as input data for a water system computer model to be used for evaluating existing facilities and planning for the future. A discussion of our methodology and findings is presented below.

1.1 Background

The basis for all work in this task was data provided by the Clovis Public Utilities and Finance and Development Departments. Following is a brief description of data that was obtained and used for analysis.

1.2 Pumping Records

Water production records for the last 12 years were obtained in the form of monthly gross output for each well. When compiled, the records provide a good indication of total use within the system for each month. The records also show the changes in operation and production for each well over the period of record.

In addition to the monthly production records, daily production records for the summer of 1996 were provided. These records provided critical information used to identify the peak day use under current operating conditions.

1.3 Meter Route Records

A second reliable source of information used was the computerized information developed during billing of individual customers for water. Billing records are updated bimonthly and it is important to realize that the billing records are not directly additive. This is true because each individual billing route is canvassed bimonthly, but not all routes are canvassed within the same week of the bimonthly cycle. Some allowance for redistribution of usage must be made to the billing records to synchronize the billing records with production records.

These records are also of value in distributing water consumption among different land uses. Each water customer's account is broken down by land use, as well as time increment. The distribution of meter routes (meter routes are grouped within each quarter section) also facilitated direct comparison with annual use values predicted by land use coefficients.

1.4 SCADA Records

The City has installed and put into service a Supervisory Control and Data Acquisition System (SCADA). It is understood that with the move to the new corporation yard a new system will be utilized. When fully operational, this system will provide pumpage, system pressure, and other information throughout the system every five minutes and will be an invaluable source of production/ storage/ consumption information. Due to archiving problems, a full year of operational data was not available from the existing system. Working with representatives of the City and Tesco, we obtained complete records from the SCADA system for the month of February 1997. From this data we were able to identify hourly, daily, and weekly flow patterns for a base line period. Our hope is that this database will continue to be built and will serve in the future as a convenient and precise source of data to update and calibrate the models developed under this study.

1.5 Land Use

Water usage varies with many factors; time of year/climate, time of day, emergency needs such as fire, and land use. In order to determine and project future water consumption, it is necessary to evaluate existing land use acreages and to project future land use acreages. This task was initiated in Phase I and revised in Phase II primarily due to additional development and changes to several of the original planning areas. These changes were made after meeting with the Planning and Development Services Director in an effort to more closely match the planning areas utilized in the Wastewater Master Plan. In general, future land use is based upon those shown in the 1993 General Plan.

PART 2 - METHODOLOGY

The tasks completed under this portion of the plan were completed as follows:

2.1 Validation of Water Use Coefficients Developed for Long Term Planning in Phase I

The task was critical to insure not only valid planning criteria, but also to develop a hydraulic model with realistic water demands. The water use factors developed in Phase I were scrutinized to insure that they could be applied with confidence in developing local demands for the model. To validate the demand factors additional data was collected and analyzed. Of particular importance were the meter route records of 1996. Because the meter routes are organized in consistent geographic locations (generally a quarter mile section) and are segregated by land use, they were valuable for checking the actual use on a more extensive scale than the original sampling. Thus, in order to validate the factors, each meter route was broken down on an acreage basis and compared with the anticipated use generated by applying the original water use coefficients. The initial results of this analysis indicated that areas with large amounts of medium density SFR were underrepresented by the initial water use factor of 1.5 acft/yr. A graphical representation of the results of this exercise is presented in **Figure 1-1**. At the completion of this exercise for the Clovis area, it was found that the original water demand values when applied to currently developed land predicted the actual use (per meter data) to within 6%. Increasing the individual water use value identified resulted in a near match of predicted and historic use.

In addition to the meter route records, individual account records were examined to check additional samples within each land use category. These checks validated the numbers developed, as well as providing insight into variances that occur in different areas.

2.2 Develop Peaking Factors for Individual Land Uses

The development of peaking factors is a standard task for any approach to water planning. For general planning, typical factors are applied which have been taken from proximate systems. Whenever possible, it is most desirable to generate peaking factors based upon historic data for the actual system. For this plan, several different factors were developed using actual historic data. These included average day demand (ADD), maximum month (or peak month), maximum day (peak day or maximum day demand - MDD), and peak hour. Each of these factors is critical to different aspects of the planning process and will be discussed briefly below. Most of the factors were generated for the entire system as well as on an individual land use basis. The factors recommended for use are shown in **Table 1-1**. The multipliers (referred to as peaking factors) shown in the table relate the respective category to the average day demand.

Average Day Demand (ADD) - This value is generated for both the system and each land use and is derived from the total annual demand expressed in terms of either a daily production value (gallons or ac-ft) or in terms of a rate that would be sustained for a 24 hour period. For the existing combined Clovis/Tarpey systems it is estimated to be 15.3 million gallons or 10,600 GPM.

Maximum Month - This value consists of the highest month's production divided by the average monthly production. The values were checked by developing the same numbers from the meter records, which while not precise, did give some insight into the difference in seasonal peaking for individual land uses. The maximum month for 1996 was July, during which the combined Clovis/Tarpey systems produced 805 million gallons of water.

Maximum Day Demand (MDD) - Similar to the ADD, this value is expressed in terms of flow or total production and represents the highest rate or quantity of production over a 24 hour period. For this study, the maximum day of 29.14 million gallons was taken from daily production records provided by the City. This value correlates to the ADD very well with the resulting max day factor of 1.9 (a value of 2 is commonly assumed). This value is critical for planning because it is generally used along with fire flow requirements to establish the capacity of the water delivery system.

Peak Hour - The peak hour is best developed from historic data recorded during peak use events. Unfortunately, the 1996 hourly data was not available for use and mandated the development of this value for planning purposes. For a system as large as Clovis, the peak hour demand dictates the ultimate system capability with respect to water delivery capability. The peak hour was generated based upon knowledge of peak operating conditions obtained from discussion with the system operator and other data previously discussed. It is estimated to be approximately 30,000 GPM based upon the diurnal curve shown as **Figure 1-2**. When more data is available this summer, this value should be confirmed.

2.3 Identify Current Operating Conditions and Characteristics

This task included discussion with the system operator and evaluation of production data. The results of this task were critical in providing the knowledge of system operations sufficient to generate the diurnal curve discussed in the preceding section. In addition, key issues that were reviewed included the system losses and operating objectives. Based upon a review of records for 1996, it appears that 97% of produced water was accounted for in terms of billings. This is an indication of a well maintained system with minimal loss. Loss rates near 10% are not unusual on systems of this size. Another key operating condition identified was the need to operate GAC units sufficiently during low use periods to prevent the development of adverse constituents within the media. During low demand periods this effectively dictates the frequent cycling of all wells with GAC and prevents any extended recovery time for these units.

As a part of this task, annual, monthly, and daily variations in demand were reviewed as well as the major water users. Historic production was addressed in Phase I, and production for 1996 is shown in **Table 1-2**. **Figures 1-3** and **1-4** show the variation in production monthly and on a daily basis during low flow periods. Both are valuable tools to help understand the operating characteristics of the system as a whole.

The last consideration under this task was to identify the manner in which the two systems operate (Clovis & Tarpey). At present, the two systems are connected at three locations, only one of which is open. The open connection allows water to move in either direction while still being metered. A review of the records indicates that for the most part, both systems are able to operate independently, but with the interconnection redundancy and hence reliability is increased for both.

2.4 Identify Significant Industrial and Commercial Water Users

This task was a brief one completed by talking with staff and reviewing the billing records for 1996. **Figure 1-5** was prepared to help demonstrate the relative portion of water which is consumed by industrial and commercial users. While commercial use is considered significant, industrial use is minimal with a large portion of that being consumed by a single user, Wawona Foods. Significant commercial use is concentrated along the Shaw commercial areas and on Western Herndon Ave. **Table 1-3** shows the top five meter routes for combined commercial and industrial accounts. They are combined because the meter accounts are intermixed. In addition to reviewing the gross use by meter area, the use per meter was also reviewed to check for individual users which may present a concentrated demand. Several concentrated demands were identified along Shaw and also on Herndon.

Schools which use 2% of total production were also examined. Of greatest importance in this classification is the fact that several school sites have their own water wells, and thus do not contribute to system demand. Most of the Elementary schools utilize City water and use the majority of their water for irrigation as demonstrated by the high maximum month factor.

2.5 Determine Fire Flow Requirements

In order to determine fire flow requirements, P&P met with representatives of the Clovis Fire Department and found that Clovis has adopted the Uniform Fire Code (UFC) which dictates fire flow requirements for different land uses and facilities. Fire flow requirements are summarized in Table 3A of the UFC and will be utilized during the modeling phase as needed. In general, the fire flow requirements are a lesser demand condition than the peak flow condition and as such are usually met with no additional system requirements.

2.6 Quantify Existing and Ultimate Demands for Model Input

This task consisted primarily of extending the factors developed previously to the entire plan area. The results of the task are shown in **Table 1-4**. In order to check the validity of the peaking factors, the existing Clovis area was included in the analysis. In this way the same peaking factors were applied to the individual plan areas as well as the existing urban area, for which real historic values were known. This check provided increased confidence in the applicability of the various factors for the various land uses.

In order to match the planning areas identified in the Wastewater Master Plan (WWMP), some changes were made from the original planning areas shown in Phase I. The changes in planning areas were completed as directed by staff and are shown in **Figure 1-6**.

A final issue addressed under this task was the assumption regarding service to the rural residential areas outside the villages. After communicating with development department staff, it was agreed that allowance would be provided for the rural residential areas, with the understanding that planning would be limited to providing potable water only. Thus as this plan is expanded, allowance will be made to serve these areas in case of extended drought. Analysis in this plan will be limited to review of the potential costs for providing such service and identification of facilities required to serve these areas.

2.7 Other Miscellaneous

One of the important issues which will bear on later work is the assumption regarding growth and how it occurs. After discussion with the City, it was directed that we utilize the growth projections contained in the WWMP. These projections are completed on five year increments for the duration of the planning period and separated by village and planning area. They are given in terms of population which we will extrapolate to land based growth as needed. It should be noted that in later stages of development of this Phase II, these projections were modified to reflect a better understanding of future Clovis growth patterns.

PART 3 - CONCLUSIONS AND RECOMMENDATIONS

With the completion of this portion of work, sufficient data existed to develop both further facilities plans and the related hydraulic model. Review and analysis of the data has provided some insight into the operational needs of the system. It should be clearly understood that the peaking factors developed have been established to closely match the actual peaks within the system. Under such a scenario, there is little to no back up provided during peak events. The amount of redundance or factor of safety which the City wishes to provide is a policy level decision which should be made during this planning process. Again, the values identified in this Tech Memo No. 1 are established to meet only historic peaking conditions of the existing system. In addition, these factors are developed on a gross land basis. This means that all land uses, streets and roadways must be included in the total area calculation. Removal of parking lots or streets when calculating water demand will result in grossly underestimating the actual demand.

TABLES & FIGURES

TABLE 1-1 Peaking Factors by Land Use

Land Use	DU/ AC	Average Day Demand (ADD) (ACFT/AC)	Average Day Demand (ADD) GPM/AC	Max Month Multiplier	Max Day (MDD) Multiplier	Max Day (MDD) GPM/AC	Peak Hour Multiplier	Peak Hour GPM/AC
Rural Residential		0.5	0.31	1.10	1.25	0.39	2.00	0.62
V Low Density	0 - 2	3.1	1.92	1.90	2.10	4.04	3.20	6.15
Low Density	2-4	2.1	1.30	1.80	2.10	2.73	3.20	4.17
Medium Density	4-7	2.1	1.30	1.80	2.10	2.73	3.20	4.17
Med-High Density	7-15	3.4	2.11	1.60	1.70	3.58	2.40	5.06
High Density	15-23	5.1	3.16	1.60	1.70	5.37	2.40	7.59
Commercial		1.8	1.12	1.50	1.70	1.90	2.40	2.68
Office		1.8	1.12	1.50	1.50	1.67	2.00	2.23
Mixed Use		2.2	1.36	1.70	1.70	2.32	2.00	2.73
Industrial		1	0.62	1.50	1.60	0.99	2.00	1.24
Public		1.4	0.87	1.80	1.80	1.56	2.00	1.74
School		2.8	1.74	3.00	3.00	5.21	3.00	5.21
Parks		2.8	1.74	3.00	3.00	5.21	3.00	5.21
Typical Planning Values					2.00		3.50	
Calculated Values for 1996								
Clovis Only				1.65	1.82			
Tarpey Only				1.93	2.74			
Combined System				1.67	1.91			

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Table 1-2 Historic Water Production

TOTAL WATER PRODUCTION 1990 - 1996 (In millions of Gallons)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1990	195	201	207	330	407	464	574	583	515	401	330	220
1991	201	215	175	264	393	535	523	580	482	497	277	235
1992	187	215	210	296	527	573	565	610	559	435	253	196
1993	222	208	235	299	487	566	706	578	618	496	319	254
1994	236	218	260	375	441	621	**728**	**728**	660	401	313	237
1995	202	228	196	374	419	604	665	**836**	610	516	355	258
1996	231	206	264	405	582	699	**806**	779	624	499	260	221
1996 - Avg Month	465	465	465	465	465	465	465	465	465	465	465	465
1996 - Month/Avg	0.50	0.44	0.57	0.87	1.25	1.50	1.73	1.67	1.34	1.07	0.56	0.48

CLOVIS WELLS

1990	177	182	185	288	353	405	493	500	458	359	300	199
1991	182	196	160	237	348	466	451	503	421	446	253	216
1992	172	199	194	267	463	503	495	541	497	392	230	183
1993	207	194	219	271	434	514	625	507	553	459	295	235
1994	219	203	239	336	406	553	655	650	586	370	290	221
1995	188	213	183	349	388	541	589	751	544	472	328	244
1996	219	193	243	378	527	626	698	697	551	452	245	201

TARPEY WELLS

1990	18	20	21	42	54	59	81	83	56	42	30	21
1991	19	20	14	27	45	69	72	77	61	51	23	19
1992	16	16	16	29	64	71	71	69	61	42	23	13
1993	16	14	17	28	53	53	80	71	65	37	24	19
1994	17	14	21	38	35	68	73	77	74	31	23	16
1995	14	14	13	25	32	63	75	85	66	43	27	14
1996	13	14	21	28	55	73	108	82	73	47	15	20

Total Production	Total Annual Mil-Gal	Average Month Mil-Gal	Peak Month Mil-Gal	Peak Month Factor	Average Daily Production
1990	4,424	369	583	1.58	12.12
1991	4,377	365	580	1.59	11.99
1992	4,626	385	610	1.58	12.67
1993	4,990	416	706	1.70	13.67
1994	5,217	435	728	1.67	14.29
1995	5,263	439	836	1.91	14.42
1996	5,576	465	806	1.73	15.28

CLOVIS WELLS ONLY

1990	3,898	325	500	1.54	10.68
1991	3,880	323	503	1.56	10.63
1992	4,137	345	541	1.57	11.33
1993	4,514	376	625	1.66	12.37
1994	4,728	394	655	1.66	12.95
1995	4,792	399	751	1.88	13.13
1996	5,029	419	698	1.66	13.78

TARPEY WELLS ONLY

1990	526	44	83	1.89	1.44
1991	497	41	77	1.85	1.36
1992	489	41	71	1.74	1.34
1993	475	40	80	2.02	1.30
1994	489	41	77	1.90	1.34
1995	471	39	85	2.16	1.29
1996	547	46	108	2.37	1.50

1996 SUMMARY	Clovis		Tarpey		Combined	
	Mil-Gal	GPM	Mil-Gal	GPM	Mil-Gal	GPM
1996 Max Day	25.03	17,383	4.11	2,852	29.14	20,235
1996 Average Day	13.78	9,568	1.50	1,041	15.28	10,609
1996 Max Day Factor	1.8		2.7		1.9	

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Table 1-3
Significant Commercial/Industrial Water Use

Meter Route	Area Description, Boundary or Primary User	**Total Annual Use (gal)**	Calculated Use Factor (acft/yr)
14	Clovis Ave, Bullard, Sunnyside, Barstow	21,745,000	1.67
23	Clovis Ave, Barstow, Sunnyside, Shaw	52,356,000	1.61
26	Sierra Vista Mall	40,389,000	2.07
70	Costco Shopping Center & Ind. Park	33,848,000	1.48
47	Wawona Foods	75,783,000	5.29

Table 1-4 Future Water Demands

Acreage	Rural Resid.	Very Low Density	Low Density	Medium Density	Med-High Density	High Density	Comm	Office	Mixed Use	Industrial	Public	Schools	Parks	TOTAL
Existing Urban Area*	66	572	2,335	1,681	564	173	472	74	402	247	147	455	65	7,253
Clovis Area at Buildout	260	40	4,375	1,955	825	315	725	195	1,310	510	210	305	175	11,200
Northeast Triangle	0	1,345	430	0	0	10	20	50	20	0	0	0	0	1,875
Northwest Village	70	100	860	600	200	60	30	120	0	150	20	160	80	2,450
Northeast Village	0	1,600	610	280	50	70	40	0	140	280	0	230	170	3,470
Southeast Village	0	515	710	440	160	70	20	0	120	60	0	160	20	2,275
Northern Rural Area	2,940	0	0	0	0	0	0	0	0	0	0	112	0	3,052
Remaining Areas	8,710	0	0	0	0	0	0	0	0	0	0	40	0	8,750
Total	11,980	3,600	6,985	3,275	1,235	525	835	365	1,590	1,000	230	1,007	445	33,072
Percent of Total	36%	11%	21%	10%	4%	2%	3%	1%	5%	3%	1%	3%	1%	100%

* Includes Tarpey

Annual Required Water Demand (Acreage * Average Daily Demand/Acre) (ACRE-FEET)

Existing Urban Area	33	1,773	4,904	3,530	1,918	882	850	74	724	543	206	1,274	182	16,892
Clovis Area at Buildout	130	124	9,188	4,106	2,805	1,607	1,305	195	2,358	1,122	294	854	490	24,577
Northeast Triangle	0	4,170	903	0	0	51	36	50	36	0	0	0	0	5,246
Northwest Village	35	310	1,806	1,260	680	306	54	120	0	330	28	448	224	5,601
Northeast Village	0	4,960	1,281	588	170	357	72	0	252	616	0	644	476	9,416
Southeast Village	0	1,597	1,491	924	544	357	36	0	216	132	0	448	56	5,801
Northern Rural Area	1,470	0	0	0	0	0	0	0	0	0	0	314	0	1,784
Remaining Areas	4,355	0	0	0	0	0	0	0	0	0	0	112	0	4,467
Total Planning Area	5,990	11,160	14,669	6,878	4,199	2,678	1,503	365	2,862	2,200	322	2,820	1,246	56,890

Maximum Day Demand (Acreage * MDD Factor) (GPM)

Existing Urban Area	26	2,308	6,384	4,596	2,021	930	895	73	673	573	230	2,369	338	21,415
Clovis Area at Buildout	101	161	11,961	5,345	2,956	1,693	1,375	193	2,193	1,182	328	1,588	911	29,988
Northeast Triangle	0	5,428	1,176	0	0	54	38	50	33	0	0	0	0	6,778
Northwest Village	27	404	2,351	1,640	717	322	57	119	0	348	31	833	417	7,266
Northeast Village	0	6,457	1,668	765	179	376	76	0	234	649	0	1,198	885	12,488
Southeast Village	0	2,078	1,941	1,203	573	376	38	0	201	139	0	833	104	7,487
Northern Rural Area	1,139	0	0	0	0	0	0	0	0	0	0	583	0	1,722
Remaining Areas	3,375	0	0	0	0	0	0	0	0	0	0	208	0	3,583
Total Planning Area	4,642	14,528	19,096	8,953	4,425	2,822	1,584	362	2,661	2,318	359	5,244	2,317	69,312

Peak Hour Demand - (Acreage * Peak Hour Demand Factor) (GPM)

Existing Urban Area	41	3,518	9,727	7,003	2,853	1,313	1,264	92	897	674	255	2,369	338	30,344
Clovis Area at Buildout	161	246	18,226	8,144	4,173	2,390	1,942	242	2,924	1,391	365	1,588	911	42,702
Northeast Triangle	0	8,271	1,791	0	0	76	54	62	45	0	0	0	0	10,299
Northwest Village	43	615	3,583	2,500	1,012	455	80	149	0	409	35	833	417	10,130
Northeast Village	0	9,839	2,541	1,166	253	531	107	0	312	764	0	1,198	885	17,597
Southeast Village	0	3,167	2,958	1,833	809	531	54	0	268	164	0	833	104	10,721
Northern Rural Area	1,823	0	0	0	0	0	0	0	0	0	0	583	0	2,406
Remaining Areas	5,399	0	0	0	0	0	0	0	0	0	0	208	0	5,608
Total Planning Area	7,427	22,139	29,098	13,643	6,247	3,984	2,236	453	3,548	2,728	399	5,244	2,317	99,463

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Actual vs Predicted Water Use

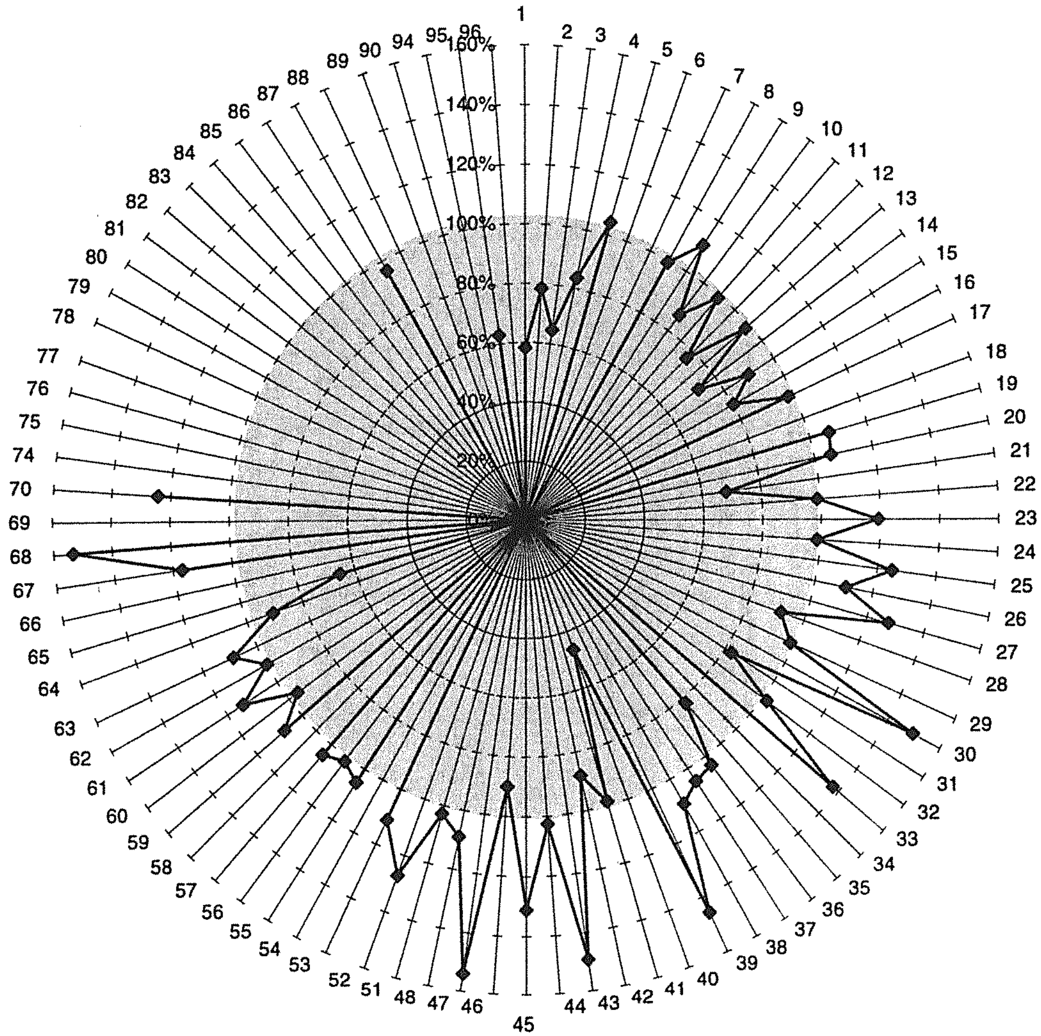

Figure 1-1

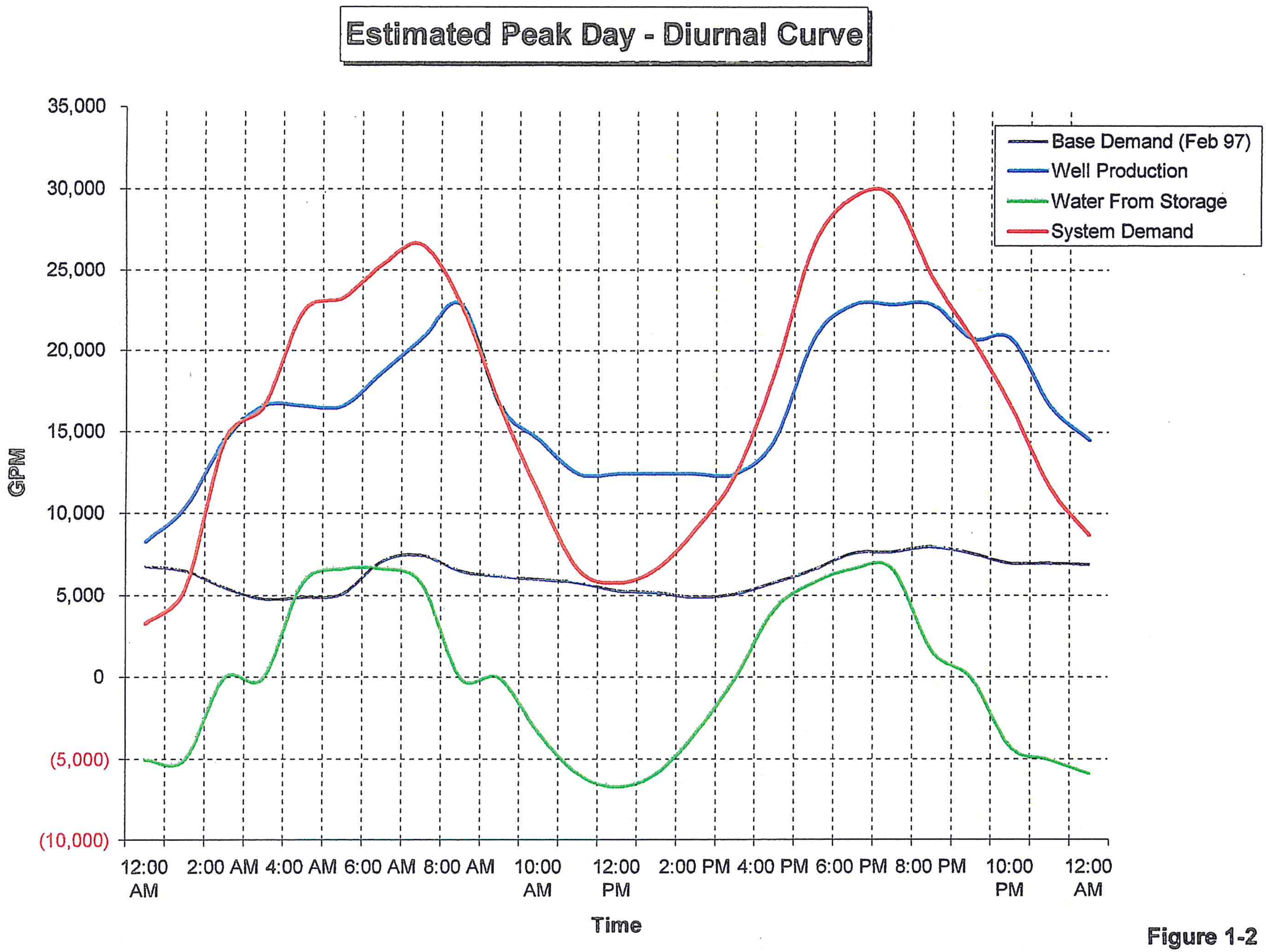

Figure 1-2

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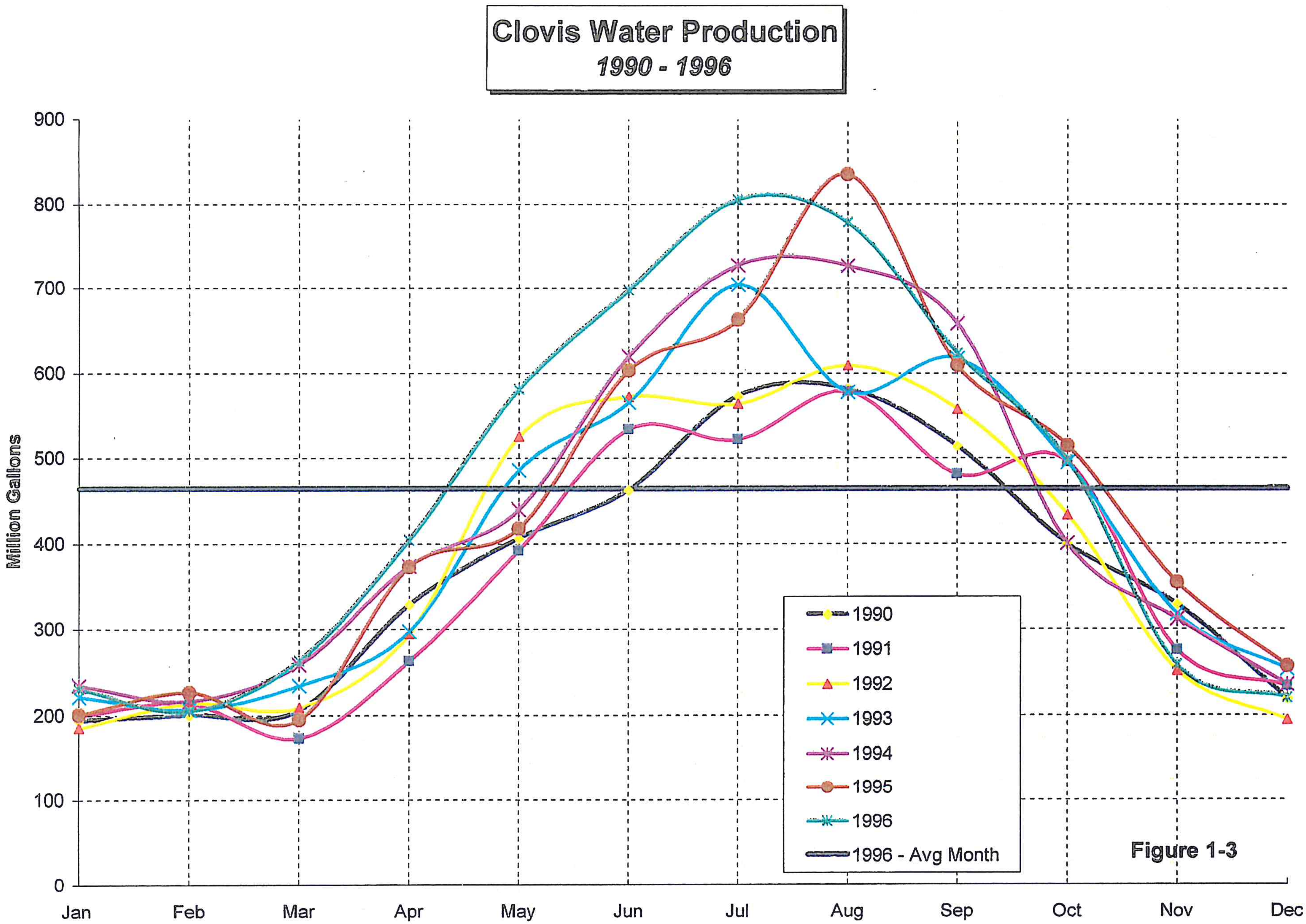
Clovis Water Production
1990 - 1996
Million Gallons
900
800
700
600
500
400
300
200
100
0
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
1990
1991
1992
1993
1994
1995
1996
1996 - Avg Month

Figure 1-3

City of Clovis Well Production Summary
February '97

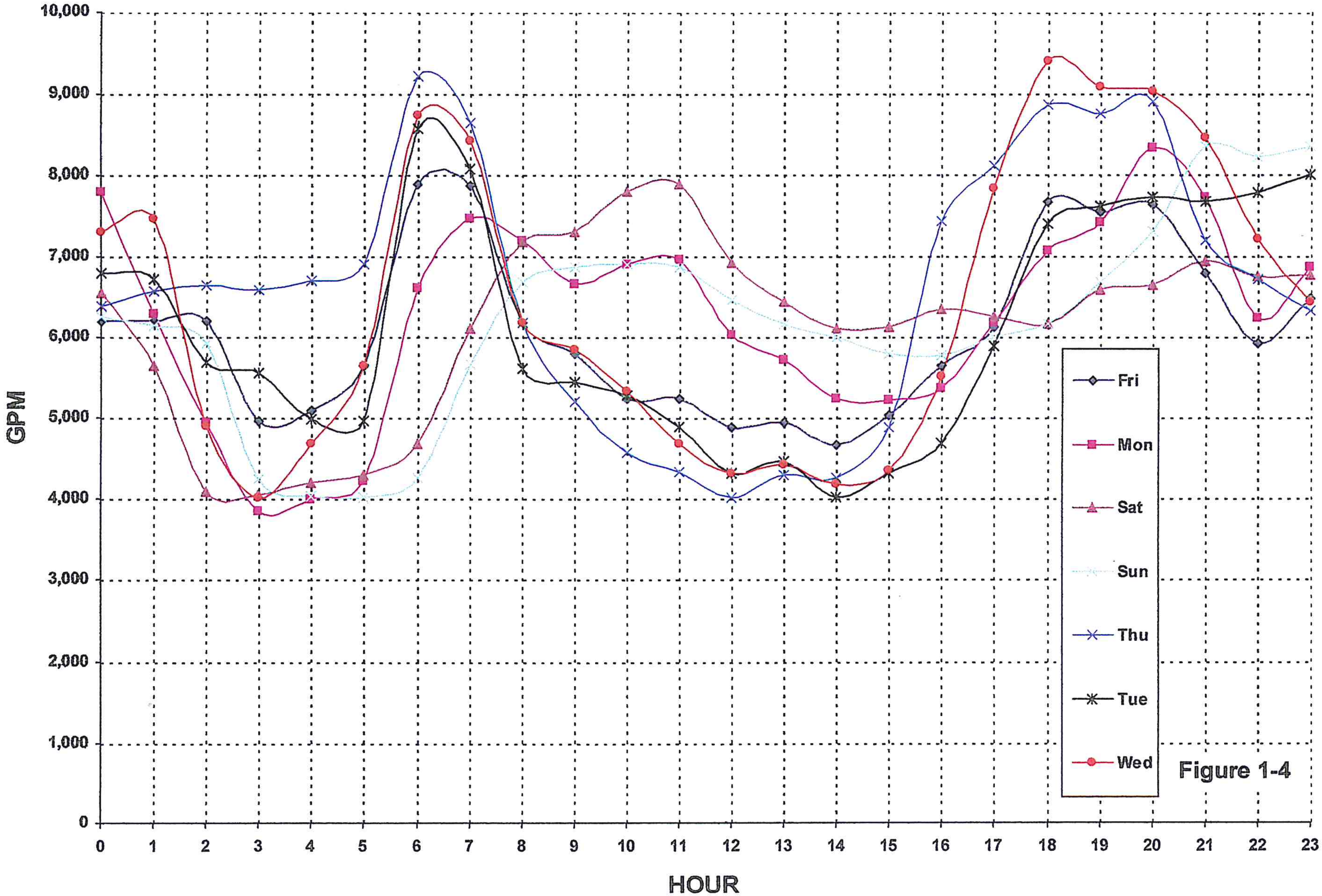

Figure 1-4

1996 Clovis Water Use

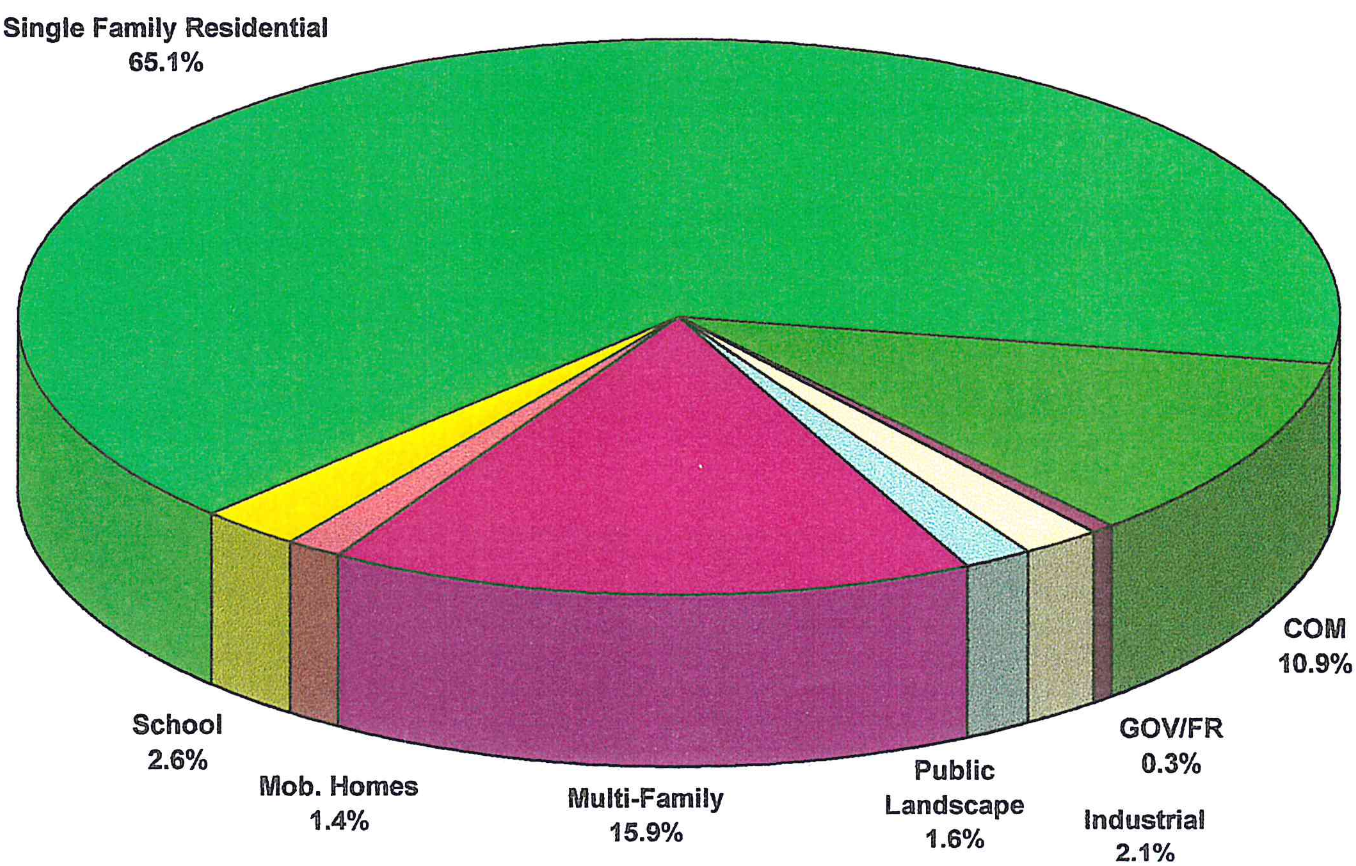

Figure 1-5

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Technical Memorandum 2

Current and Future Water Supply

PART 1 - INTRODUCTION

This memorandum focuses on identifying and quantifying the water supplies that will be required to serve the urban water needs within the general plan area. Phase I of the Water Master Plan Update provided an in depth review of many of the characteristics of both ground and surface water. This memorandum updates some information presented in the Phase I report, expand on other issues that have recently surfaced, and provide preliminary indications of delivery methods to meet supply demands noted in Tech Memo No. 1 (TM1).

As indicated in TM1, the planning area boundaries identified in Phase I were shifted slightly and demands changed for some urban land use designations. The revised boundaries are shown in TM1 as are the water demand values utilized for analysis in this report.

PART 2 - EXISTING WATER SUPPLY

The existing water system relies entirely upon groundwater wells for the urban supply. **Table 2-1** shows the existing well inventory for the system along with the current pumping capacity. Note that production capacity does not include water available from storage facilities, which is reserved to provide additional water during the peak hours.

The two factors with the greatest influence upon the production capability of the existing wells are groundwater levels and pumping plant characteristics (pump, motor components, age, and efficiency). The City has measured water levels at the wells on an intermittent basis for several years. The efficiency of the pumping plants are checked by regular pump tests. The City recently completed pump tests on most of the wells in the system with results showing an average overall plant efficiency of 65%.

Water level measurements taken in summer or fall show a noticeable decrease in water levels from spring measurements. This is a normal result of increased pumping during the summer months and must be accounted for in estimating water production capacity. The drop in water levels increases the cost to pump water, and also reduces the capability of the system to produce water. A drop in pumping levels of 20-50 feet could result in a significant reduction of system capacity. This change in system capacity could be critical during extended high demand periods.

Table 2-1
1998 - Well Inventory

Well ID#	Location	Estimated Well Capacity (GPM)	Standby Capacity* (GPM)	Planned Wells (GPM)	Status
1	Fifth & Hughes		350		On standby - exceeds DBCP MCL
2A	Fifth & Harvard	1,600			Replaced 1998
3	1190 5th Street	450			OK
4AA	South Corp. Yard	1,000			OK
5	410 Barstow Ave	450			OK
6	Tollhouse & Almond	300			OK
7	Letterman Park	2,000			OK
8A	294 N. Villa	1,900			OK
9	1st & Clovis	550			OK
10	2698 Peach Ave	1,000			OK
11	1722 Fowler Ave	1,150			OK
12	900 Gettysburg Ave	1,100			OK
13	San Gabriel / Temperence		300		On standby - poor quality & production
14	198 N Peach Ave	1,790			OK
15A	599 Timmy	1,550			OK
16	Armstrong & Ashlan	950			OK
17	1680 Willow	1,300			OK
18	3405 Clovis	1,000			Rehabbed with liner recently - OK
19	Clovis & Dakota	800			Hovers near MCL for DBCP
20	Armstrong & Barstow		500		Poor quality & production
21	640 W Alluvial	1,000			OK
22	842 Alluvial	925			OK
23	700 N Hughes	580			OK
24	Sunnyside & Herndon	1,000			OK
25	105 W Nees Ave	1,470			OK
26	850 N Peach	1,800			OK
27	611 N Peach	1,500			Fitted with GAC for DBCP
28	399 W. Shaw	2,500			Great - VFD
29	820 W. Pico	800			OK
T-5	5798 Tarpey Drive	1,000			
	Total Clovis Capacity	**31,465**	**1,150**	**-**	
	GPM/Capita	0.46	(Population = 68,807)		
T-1	4254 N. Minniwawa	200			
T-2	4205 N. Hamel		500		On standby - scheduled for GAC
T-3	Bernadine @ Phillip	1,000			
T-4	Gettysburg & Clovis			700	OFF line - 10* MCL for DBCP
T-6	4189 N. Hammel Way		1,000		On standby schedule for GAC
T-7	5598 E. Ashlan	500			
T-8	5435 E. Ashlan	500			
	Total Tarpey Capacity	**2,200**	**1,500**	**700**	
	GPM/Capita	0.55	(Population = 4,000)		
	Total System Capacity	**33,665**	**2,650**	**700**	
	GPM/Acre		5.0	(7,250 Acres in Existing Urban Area)	
	* Wells that can be operated in emergencies, but which produce water of lower quality.				

Figure 2-1 shows the springtime depths to water as measured at City wells from 1993 - 1997. An important trend that is clearly visible in **Figure 2-1** is the continued decline of groundwater levels despite three out of four years of above average precipitation and surface water availability. During the years from 1992 - 1996 runoff from the Kings River averaged 130% of normal.

2.1 Intentional Recharge Activity

In addition to natural recharge, the groundwater aquifer is recharged intentionally through both dedicated and dual use basins. Intentional Recharge is critical to maintaining the groundwater levels which have declined steadily for the last 50 years. In 1996, the quantity of water recharged surpassed 10,000 acre feet for the first time, due largely to the first full year of operation for the new Marion/Alluvial recharge facility. Further increasing this level of recharge activity will be critical to protecting and maintaining the groundwater supply. The City has completed Phase I of a Groundwater Recharge Investigation which identified areas which would be most favorable for groundwater recharge. In order to maintain a groundwater balance, the City will have to develop additional recharge in appropriate areas to insure that any additional pumpage is matched by added groundwater recharge facilities of equal or greater capacity.

Phase I studies estimated the present annual overdraft to be approximately 2500 AF per year. **Table 2-2** shows the most recent recharge activity. The planned sale of the Clovis Basin in southern Clovis and the resulting loss of recharge capacity is hoped to be offset by increasing recharge activities at the Marion/Alluvial site. Additional purchases of land adjacent to the Marion site and associated development for recharge activities may be capable of off-setting the estimated present overdraft. Several years operation will be needed to verify this assumption. Since the new basins are currently not in full operation, a review of groundwater trends would indicate that the overdraft is presently increasing.

Table 2-2
Historic Recharge Deliveries (ac-ft)

Year	Clovis Basin	FMFCD Basins	Marion Facility	Recharge thru Creeks	Total	Average of 10 Prior Years
1977	2,845				2,845	
1978	6,397				6,397	
1979	6,952				6,952	
1980	6,751				6,751	
1981	4,930				4,930	
1982	4,521	1,606		1,318	7,445	
1983	3,927	884		1,664	6,475	
1984	3,427	1,491		1,438	6,356	
1985	2,419	260		844	3,523	
1986	3,146	1,252		1,381	5,779	5,745
1987	1,601	782		119	2,502	5,711
1988	1,490	1,130		516	3,136	5,385
1989	3,961	1,261		344	5,566	5,246
1990	2,156	886		1,009	4,051	4,976
1991	3,278	1,694		3,158	8,130	5,296
1992	3,208	1,583		3,604	8,395	5,391
1993	1,879	2,275		4,640	8,794	5,623
1994	1,409	1,865		2,214	5,488	5,536
1995	1,967	2,105		4,908	8,980	6,082
1996	1,334	3,128	2,530	3,909	10,901	6,594
1997	733	1,626	1,979	4218	8,556	7,200
1998	738	1,713	2,745	5014	10,210	7,907
Avg	3140	1,502	2,638	2,370	6,462	5,985

FMFCD Basins: 1G, 2D, 3A, 3D, 3F, 4E, 5F, 5B/5C, 6D, 7C, BW, CL, S

It is estimated that sustainable long term groundwater yield in the urban area without intentional recharge is approximately 8,000 AF, assuming other inputs and outputs to the groundwater balance remain the same (calculated by well production less intentional recharge less overdraft). If current groundwater production approaches 17,000 AF/year, then intentional recharge activities should average 9,000 AF/year (17,000 - 8,000 AF) to limit overdraft. It can be concluded that any new development will require recharge and extraction facilities to be constructed to serve the new demand or other delivery mechanisms need be employed to match the increased demands.

To insure the continued success of the current water supply system, it is important to understand how the system works. Changes or alternations by recharge or extraction can cause water levels to change and contaminants to migrate. It is recommended that the City embark on expanding the current monitoring system where possible and providing a more structured protocol. The result will be better information. Included in Appendix A are recommendations for implementing a groundwater monitoring program. Implementation of an effective monitoring program is critical to collecting sufficient data to efficiently manage and protect groundwater resources.

PART 3 - FUTURE WATER SUPPLY

Prior to planning the incremental steps which will be taken to expand the water system, it is important to identify the ultimate sources of water. These sources include some proportion of groundwater, surface water, exchange water, reclaimed water and imported water. The overall supply was generally quantified in water balances completed in Phase I. It was estimated that in the plan area there is a 4,000 AF per year deficit. The urban demands were to be met by:

- Groundwater pumping - 8,000 AF
- Groundwater pumping with matching recharge activities - 15,000 AF
- Treated surface water - 20,000 AF
- Dual system 4,000 - AF

In that report the various supplies were identified and aggregated to provide a summary of supply for the entire plan area. Since that time, additional years of record (1992 - 1996) have been added to the historical record. Upon inclusion of these values, the long term water supply numbers have changed slightly.

To better understand the specific issues related to utilization of the full supply, the individual supply quantities and potential delivery mechanisms to the respective geographic areas will also be addressed later.

3.1 Surface Water

The availability of surface water was covered in some detail in Phase I. **Figure 2-2** (same as Figure 5-8 in Phase I CWMP) shows a summary of the estimated surface supply for the planned urban areas within the study area. The 20 year summary is based upon the historic entitlements for the area from 1974 to 1994. The most significant supply element is the Kings River Entitlement which is available to property within FID. The city currently uses this entitlement almost exclusively for groundwater recharge. The city has not historically been able to fully utilize all of the surface water available. It is assumed that in the future, recharge will be economically maximized, and the remainder of this supply will utilize other delivery mechanisms for use in the urban environment. Exchange water and water purchases will be required to maintain a long term positive water budget. As shown, the total estimated demand of 52,500 af can only be met through a combination of all the supplies shown. **Table 2-3** shows a breakdown of anticipated surface supplies for each planning area. Note that the Kings

TABLE 2-3
Estimated Surface Water Supply for Study Area

Area (Acres)	Existing Urban Area*	Clovis Area at Buildout	Northeast Triangle	Northwest Village	Northeast Village	Southeast Village	**Urban Areas Subtotal**	Northern Rural Area	Remaining Areas	**Total**
Total Area	8,000	11,645	2,035	2,840	6,250	3,740	26,510	3,540	16,470	46,520
Area within FID	7,320	10,700	0	2,300	0	3,620	16,620	700	4,820	22,140
* Includes Tarpey Area										
Surface Water Supply (ac-ft)										
Kings River Entitlement (2&3)										
Based upon Total District Area	13,513	19,752	0	4,246	0	6,683	30,681	1,292	8,898	**40,871**
Based upon Water Service Area	16,340	23,885	0	5,134	0	8,081	37,100	1,563	10,759	**49,422**
Class II - FID (4)	878	1,284	0	276	0	273	1,833	84	578	**2,500**
International & Garfield Wtr Dist. (5&6)				1,170	1,200		2,370	580		**2,950**
Water to FID Annex Lands (7)			22	0	79	0	101	0	107	**208**
Effluent Exchange (8) Alternative 1	1,000	1,000					1,000			**1,000**
Alternative 2		3,667	787	835	1,412	-	6,701	-	-	**6,701**
Reclaimed WW - (local facility)						800	800			**800**
Total Estimated Surface Supply										
Low	15,391	22,036	22	5,692	1,279	7,755	**35,985**	1,956	9,583	47,524
High	17,218	28,836	809	7,415	2,691	9,153	**48,905**	2,227	11,445	62,576

NOTES:

1) FID current estimated annual Kings River Entitlement = 452,700 ac-ft/year

2) FID area	Entire District	Receiving or capable of receiving water deliveries
	245,232 acres	202,801 acres
3) Estimated entitlement per acre	1.85 ac-ft/acre	2.23 ac-ft/acre

4) Class II water estimated at 0.12 ac-ft per acre for areas within FID
5) International WD entitlement of 1,200 af per year assigned to NE village
6) One half of Garfield WD is in planning area. One half of their 3,500 af annual entitlement (1,750 ac-ft) is split as shown between NW village and Northern RR.
7) FID water to annex lands based upon average historic deliveries of 200 AF per year.
8) Effluent Exchange Water - Low: Based upon 10% of historic reclamation quanitity of 10,000 af/yr.
High: Based upon 30% of urban demand (treated wastewater) being exchanged at 1:2 rate

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River entitlement is shown based upon two different factors. The first is a distribution based upon total acreage in the district which is the basis for the current water delivery contract between the City and FID. The second is the preferred basis of allocation, and divides the water supply between those areas which are either receiving or capable of receiving water deliveries. The difference in allocation methods could result in the addition of 6,000 ac-ft of water per year for the urban areas.

Two issues which the City must resolve in planning to utilize surface water efficiently are raw water conveyance and the variable nature of the supply. Raw water conveyance is significant in terms of protecting the supply from contaminants and insuring reliability. Annual and seasonal variations in the surface supply are also critical factors which must be addressed when planning for facilities.

3.2 Raw Water Conveyance

The two major sources of surface water for the planning area are the San Joaquin River and Kings River. Water from either source can be delivered to the area from one of three canals that cross or border the area. Because Fresno is also preparing to treat surface water, they have completed sanitary surveys for both the Enterprise and Friant Kern Canals. The surveys identify potential risks associated with the different systems.

San Joaquin River water delivered under contract with the Bureau of Reclamation is conveyed via the Friant-Kern Canal. The Friant Kern Canal flows through the northeast portion of the plan area; it has existing turnouts located at Big Dry Creek and Dog Creek. International Water District and Fresno Irrigation District also have turnouts from the Friant-Kern in the vicinity of Highway 168. In addition, Garfield Water District has a turnout located on the Copper alignment. Any of these turnouts could be utilized to deliver water to Clovis from the San Joaquin river system (CVP).

Kings river water can be readily delivered via the Enterprise or Gould Canals. The Enterprise is located centrally to the planning area. The Gould forms the southern boundary for parts of the planning area. Due to the importance of conveyance for any surface water, existing facilities, operations and maintenance activities were reviewed with FID. The major creeks that cross these canals include Dog, Redbank, Fancher, Mud, and Holland Creeks. All of these creeks have crossings under the canals that consist of some type of piped siphons. Most of the creeks pass through or under the Enterprise Canal, with much of the storm runoff captured and moved through the City of Fresno by the Gould Canal which is the designated drain for the area. Of the natural creeks, Mud Creek has been the most problematic; it sometimes flows into the Enterprise Canal in big storm events, and even into the Friant-Kern Canal at peak discharges. Fancher Creek has several flood control facilities, including a control structure on the Enterprise Canal which allows water to be spilled to Fancher Creek. **Table 2-4** summarizes important characteristics of the three canals.

Table 2-4
Canal Characteristics

Canal	Access	Storm runoff / potential contaminants	Capacity cfs, gpm & type	Operating months
Enterprise	Unlimited	Minimal runoff / dumped material	100 (44,800) unlined	Dec - Aug
Gould	Unlimited	Runoff from several natural drainages – primarily agricultural areas / pesticides & herbicides	150 (67,350) unlined	Jan - Sept
Friant Kern	Restricted	Local overland flow - primarily from rangeland / animal waste	3,500 (1,571,000) concrete lined	Year-round

FID entitlement water could also be delivered via the Friant Kern Canal if agreements can be established which allow an exchange of water further downstream. This could be accomplished by FID turning water into the Friant Kern at its intersection with the Kings river at the same rate that water is being delivered to Clovis from the Friant Kern Canal.

3.3 Variability of Surface Water Delivery

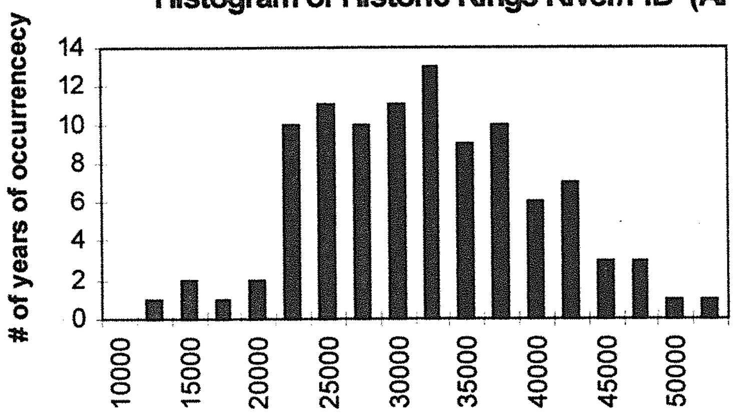
Unlike groundwater, surface water deliveries have a high degree of variability. As discussed in Phase I, the Kings River water entitlement is based upon "run of the river," which means that deliveries are based upon the actual flow rate in the river. **Figure 2-3** shows the average runoff on a monthly basis, along with the estimated demands. Efficient use of surface water requires much less variability. In order for the City to "level out" their supply, some storage of the runoff must be arranged. The current City contract with FID presently precludes storage of any water behind Pine Flat Dam. However, FID has approximately 140,000 AF of storage space; the City should negotiate terms in their new contract for storage of supplies.

In addition to the seasonal variations in surface supply, there are significant fluctuations in the annual deliveries which may vary from 40-160% of average. **Table 2-5** shows the historic entitlement variations for the total urban areas (within FID) taken as a percentage of both the total FID area, and the current delivery area. Historic records show that the most frequently occurring water years would deliver approximately 80% of the arithmetic "average." **Figure 2-4** shows a histogram of the historic entitlement to the urban area.

Table 2-5
Urban entitlement to
FID Kings River Water Supply (1895-1996) AF

Event	Entitlement based on % of FID Delivery Area	Entitlement based on % of FID
Minimum (Drought) Year	14,300	11,800
Maximum Runoff	60,500	50,000
Median	37,200	30,700
Arithmetic Average	36,700	30,300

Figure 2-4
Histogram of Historic Kings River/FID (AF)



Yearly Entitlement for Urban Area (Fresno-Clovis) based on Percent of FID
Note: based upon 101 years of record.

Figure 2-4 shows that based upon 101 years of historic deliveries, the entitlement for the urban area would be over 20,000 ac-ft in 95% of the years. Further, the entitlement would exceed 25,000 ac-ft in 73% of the years.

In order to utilize the majority of water to which the City will be entitled, the City must have capacity to efficiently utilize water in high flow years as well as normal years. One likely solution would be to provide primary facilities sized to handle the median entitlement, plus secondary facilities which can utilize excess water available in wet years. This can best be accomplished by sizing permanent direct water use and/or treatment facilities to utilize the median water supply. This class of facilities will include dedicated recharge facilities and water treatment plants. Secondary facilities such as dual use recharge or storm drainage facilities can then provide capacity to recharge the additional water supply available during wet years. Unfortunately, many storm basins are not constructed in soils conducive to recharge. Additional intentional recharge facilities may therefore be required that can only be utilized on an intermittent basis. Remaining alternatives which should be explored include dual use park/basins, and alternative water supply arrangements discussed previously. **Failure to provide sufficient capacity to utilize water available in wet years will result in an overall reduction in water supply.**

3.4 Exchange Water

Exchange water is defined as water which is obtained in exchange for wastewater treated and recovered from the Fresno/Clovis Regional plant. The City of Fresno currently gets credit on a "one for two" basis for water that is reclaimed from wells around the plant. At the present time, Clovis exports nearly 6,000 AF per year of wastewater to the plant. In the future it is hoped that the City will enter into an agreement to obtain credit for water that is reclaimed at or near the regional plant. The exchange water will then be made available by FID for City use through added deliveries of surface water to the City.

Existing reclamation activities at the Regional Wastewater Treatment Plant average 10,000 AF per year. It is assumed that Clovis would have rights to 10% of this supply (based upon their portion of inflow). For this report two values are provided in **Table 2-3**. The first is 1,000 ac-ft based upon 10% of current reclamation activity. The second value is based upon increased reclamation at the plant with the resulting reclaimed water exchanged as discussed above. In order to reach the second value, the City will need to arrange an exchange contract similar to Fresno's, as well as encourage the installation of additional reclamation facilities at the regional wastewater treatment plant.

3.5 Reclaimed Water

The City has continued to investigate options related to siting a local wastewater treatment plant in the plan area. For the purposes of this report, we assume that one small satellite plant will eventually be constructed in the southeast village area, and that the resulting reclaimed water will be available for either recharge, agricultural supply or use within a dual system.

3.6 Imported Water

Results of the Phase I investigation indicated that additional imported water will be required to cover long term deficiencies in the overall supply of the planning area. At the planning horizon, the estimated shortfall was previously estimated at approximately 4,000 AF. Sources of imported water were identified in Phase I and will be critical to meeting the stated development goals of the general plan. One further source of imported water is the City of Fresno's current contract supply from the Bureau of Reclamation. Fresno has stated that if they are unable to cover the costs of renewing their contract, they would be open to selling or transferring portions of their contract to others within the same groundwater basin. Further discussion of this topic is included in Technical Memorandum No. 3.

The potential opportunity for Clovis is to secure a Class I contract which would provide a highly reliable water source, even under extended drought conditions. The USBR has indicated that a reassignment would be fairly straightforward. In essence, the Fresno contract would be split into multiple smaller quantities based upon volume. Under such a scenario, Clovis could become a CVP contractor with all the inherent rights and ability to secure available flood waters (Section 215 water) as well as buy and sell their water like other CVP contractors.

3.7 Groundwater

A number of factors influence the potential to develop groundwater for public supply in the study area. These factors include subsurface geologic conditions, depth to water and water-level trends, aquifer characteristics, recharge, and groundwater quality. Subsurface geologic conditions below the water level are important in terms of well yields and conditions above the water level are important when considering potential recharge operations.

The findings of the Phase I report with respect to groundwater may be summarized as follows:

- The aquifer is thickest under the southwest portion of the City, generally south of Herndon and West of Clovis Ave.
- To the north and east of Clovis, the aquifer thins substantially and bedrock becomes shallow with a resulting reduction in water production capacity.
- Planned growth areas are less favorable for groundwater development than in the existing City of Clovis.
- Existing groundwater pumping levels exceed recharge rates, resulting in continued lowering of groundwater levels in most parts of the area.
- For the past 10 years pumping amounts within the city have continued to increase (to over 16,000 Acre-Feet in 1994), while intentional recharge has averaged 5,000 Acre-Feet/Year.
- Surface and subsurface geologic conditions favorable for intentional recharge are limited.
- The areas most favorable for intentional recharge activities are along Dry Creek and other stream channels.
- Continued sole reliance on the existing supply system of recharge and groundwater pumping for water supply for future urbanizing lands is hampered due to geologic constraints.

A number of constituents were identified in the Phase I study to be present at problem levels or to possibly be a concern in the future, including DBCP, EDB, Nitrate, Iron, Manganese, Arsenic, and Radon. MTBE (fuel additive), a fairly recent addition to potential pollutants, will need to be monitored, although it has not shown up in Clovis water supplies. The areas where each of these have been identified are outlined on the "Groundwater Constraints" map in the Phase I document. At the present time, the City utilizes wellhead treatment for DBCP removal for the water produced from seven wells. Arsenic and Radon do not currently exceed drinking water standards, but if proposed standards are adopted, these could create problems for utilizing groundwater.

The limited availability of adequate sustainable groundwater dictates the need for alternative water supplies. The ability to sustain the existing level of groundwater pumping will depend in large part upon maintaining adequate recharge activity. As more agricultural land is taken from production, recharge from irrigation and precipitation is reduced; equivalent replacement recharge must be supplied. Acquisition of additional recharge sites will become increasingly difficult in urbanized areas, due to cost. It can be estimated that the City will ultimately recharge an average of 13,500 AF annually through intentional recharge (this does not include storm runoff) with the addition of newly acquired sites. In addition to these lands, it can be speculated that areas identified as A1, A2, B1 and B4 listed in a report entitled "Groundwater Recharge Investigation" dated 1995 are the most favorable additional areas for intentional recharge. Together they may have the ability to add an additional 5,000 AF of recharge for a total projected recharge capacity of 13,000 AF per year. New recharge facilities will need to be distributed through the planning area to effectively recharge the whole area.

PART 4 - RELIABILITY OF WATER SUPPLY

There are several factors which must be addressed to insure the long term reliability of the public water supply. Reliability as discussed in this section refers to the raw water supply and the capability to provide raw water to the system, as opposed to reliability within the distribution system which is provided by storage, excess capacity and back up power units. Supply reliability is critical to insure that a portion of the total supply is always available. Potential threats to a water supply include source contamination, lack of adequate raw water conveyance system, and insufficient supply.

In order to insure the long term reliability of groundwater supply, the water level and quality must be protected. This requires continued recharge and monitoring of water levels, as well as insuring a balance of recharge with additional pumping. Continued decline of the water table if left unchecked will result in lower production capacity, higher pumping costs and could compromise the long term capacity that will be relied upon during drought periods. Maintenance of water quality can be accomplished through monitoring and management of potential contaminants along with other activities designed to reduce the risk of groundwater contamination.

Insuring the reliability of the surface sources requires securing an adequate source under all conditions including short term delivery variations and extended drought periods. To do so will require firming up the water supply as discussed previously. Protection against contamination of the source and conveyance system will be addressed initially in the sanitary surveys conducted on both the San Joaquin and Kings River systems.

PART 5 - DEVELOPMENT OF FUTURE SUPPLIES/RECONCILIATION

All of the additional supply alternatives identified require one of three forms of delivery mechanisms. The three alternatives are:

- Recharge and extraction
- Treatment for direct use
- Distribution as untreated water through a dual system

Table 2-6 shows the delivery mechanisms as developed in Phase I and a comparison to current findings. **Figure 2-5** shows a schematic representation of the various delivery mechanisms. Costs of the three alternatives were examined in Phase I and are highly dependant upon location and water quality. In most cases, groundwater has proven to be the most cost effective. However, as less desirable geologic conditions are encountered the cost advantage has decreased.

Table 2-7 shows the excess or deficiency of water supply for each planning area. The water supply values are based upon the low estimates of surface water as shown in **Table 2-3**. Note that the Northern RR surface supply identified is primarily FID water (1,376 ac-ft) along with some water from Garfield WD (580 ac-ft). The lands with the entitlements are located at either end of the planning area and are separated by a large area without any water rights. Therefore, extreme care should be taken in assuming any distribution of this water over the larger area. It is possible that if place of use restrictions are placed upon the FID water, that an additional source of imported water would be required to meet these demands.

Table 2-6
Water Supply Delivery Mechanisms for Urban Areas

Delivery Mechanism	Current Analysis	PHASE I
	2030	**2030**
Groundwater:		
Pumping without recharge	8,000	8,000
Pumping with recharge from existing basins	5,500	5,500
Pumping with recharge from newly acquired sites	2,500	
Pumping with recharge from additional (future) sites	5,000	10,000
Surface Water Treatment Plant	27,500	20,000
Dual System	4,000	4,000
TOTAL	52,500	47,500

Notes: 1) All values in acre-feet 2) Does not include service to rural residential property

Table 2-7
Reconciliation of Supply and Demand
(See Table 2-3)

Description Area	**Clovis**	**Northeast Triangle**	**NW Village**	**NE Village**	**SE Village**	**Northern RR**	**Total**
Water Supply							
Groundwater	8,000						8,000
Surface Supply	22,036	22	5,692	1,279	7,755	1,956	38,740
Total	30,036	22	5,692	1,279	7,755	1,956	46,740
Water Demands							
Urban Demand	24,447	5,246	5,566	9,416	5,801	314	50,790
Rural Residential	130	0	35	0	0	1,470	1,635
Total	24,577	5,246	5,601	9,416	5,801	1,784	52,425
Supply - Demand	5,459	(5,224)	91	(8,137)	1,954	172	(5,685)

The groundwater supply value is based upon the estimated sustainable groundwater yield discussed earlier. This value does not equal total groundwater pumpage. It only accounts for the water that is pumped in excess of intentional recharge. Total pumpage which includes this value as well as the additional pumpage with matching recharge is shown in **Table 2-6**.

Water demands are based upon the water use values discussed in TM#1. Water demand for rural residential customers is limited to potable supply only (0.5 ft/ac).

The last line of **Table 2-7** shows the reconciliation of anticipated water supplies with demand. Note that Clovis, the Northwest and Southeast Villages all have excess supplies of water compared to their anticipated ultimate demand. The remaining areas are expected to have some deficiency which will need to be supplied by other means. It is reasonable to anticipate that if Clovis can fully utilize its available supply, that there should be sufficient supply to make up the deficits in the Northeast Triangle. This scenario assumes that Clovis is able to supply either surface or groundwater to areas outside FID. If the City is unable to do so, then this area will need to be served by imported water. Serving water to the Northeast Village will encounter similar difficulties, with the addition that there is insufficient supply to meet the total demand. To overcome this deficit will require the acquisition of an additional water supply. Under a worst case scenario, the City may need to import up to 14,000 ac-ft of unrestricted water.

PART 6 - DELIVERY MECHANISMS BY PLANNING AREA

Following is a proposed balance of delivery mechanisms which will match the anticipated growth in demand with water supply. In order to develop an efficient overall system, the ultimate conditions are identified first, with successive iterations for each individual village area. Assumptions behind the planning are as follows:

- The changes in demand are based upon the growth assumptions provided in the Clovis Wastewater Masterplan. Actual growth will dictate the ultimate need and timing of facilities.
- Rural residential demands are limited to potable requirements only, and there is no plan made to serve the outside irrigation needs in rural residential areas.
- Values shown are average annual demand and supply. Additional water system components such as wells and storage facilities will be required to meet the daily and hourly peaks.
- The underlying objective in the timing of facilities is to prevent the continued or increased overdraft of groundwater in each planning area.
- Utilization of groundwater is maximized in proportion to feasible recharge potential. The amount of recharge shown is an annual value which means that the actual facilities should be sized proportionally higher to allow for the seasonal fluctuations in water supply.

6.1 Combined Urban Areas (refer to Figure 1-6)

Table 2-8 and **Figure 2-6** show the overall projections for the urban demands and summarizes the total portion of each supply element. The estimated ultimate balance will result in pumping 21,500 AF (44%) of groundwater annually along with treating 27,000 AF (56%) of surface water. The recharge capacity is slated to rise steadily with an ultimate capacity of 13,500 AF (average). If recharge is found to be uneconomical or surface sites are unavailable, then the treated water capacity must increase in proportion. Water is anticipated to be exported to rural residential areas and the NE Village which may also need an additional imported water supply.

6.2 Clovis and Northeast Area

Table 2-7 shows that the Northeast Triangle area is deficient in water and will depend upon water supplies generated in the Clovis area. Surface water will be utilized as a source for expanding recharge activity and a staged treatment facility. The treatment facility is anticipated to begin with an initial 5.0 MGD unit going online as soon as possible. This first stage facility (5 mgd) will be sufficient to provide for several years of growth with the concurrent development of additional storage, recharge and extraction facilities. The treatment capacity will be expanded as demand increases. In addition to the treatment capacity, groundwater recharge capabilities are planned to increase incrementally to 13,500 AF.

6.3 Northwest Village

This village appears to have a sufficient water supply and should have the capability to recharge sufficient quantities to rely upon groundwater. However, there may be some groundwater quality problems. Recharge capability should be confirmed by further field testing and is subject to the preservation of sites with favorable recharge conditions. Expansion of recharge facilities is scheduled to match the incremental increase in supply as closely as economically feasible.

6.4 Southeast Village

The southeast village is the only planning area where allowance is made for reclaimed water from a local or satellite wastewater treatment plant. In addition to this water supply, the area is entirely within FID and therefore has a substantial water entitlement. Recharge in this village is expected to be possible but only on a limited basis. Therefore, the majority of growth within the Village is anticipated to rely primarily upon treated water for potable needs. The amount of groundwater available will be dependant upon groundwater quality and recharge capability. Higher quality water may be available from deeper wells, but will still require a balance in terms of recharge. Dual systems will also be economical for any large landscape water users.

TABLE 2-8

Ultimate Urban Areas - Water Supply Elements[2]

(All quantities in acre-feet)

Year	Total Urban Demand	Source: Surface Entitl.[1]	Source: Pumped Grndwtr	Source: Imported Water	Treated Water	Dual System	Average Intentional Recharge	Excess Grndwtr Pumped*	Unused Surface
1995	16,200	15,400	16,200		-		6,000	2,200	9,400
1996	17,200	15,400	17,200		-		6,500	2,700	8,900
1997	18,730	15,400	18,730		-		7,200	3,530	8,200
1998	18,700	16,000	18,700		-		7,900	2,800	8,100
1999	19,600	16,700	19,600		-		8,500	3,100	8,200
2000	**20,500**	**18,600**	**20,500**		-		**9,000**	**3,500**	**9,600**
2001	21,300	19,500	21,300		-		10,000	3,300	9,500
2002	22,100	20,100	22,100		-		11,000	3,100	9,100
2003	23,000	20,800	18,000		5,000		11,000	0	4,800
2004	23,900	21,500	18,900		5,000		11,000	0	5,500
2005	25,200	22,500	20,200		5,000		12,000	200	5,500
2006	26,000	23,400	21,000		5,000		12,000	1,000	6,400
2007	26,800	24,100	21,800		5,000		12,000	1,800	7,100
2008	27,800	24,900	22,800		5,000		12,000	2,800	7,900
2009	28,800	25,200	23,800		5,000		12,000	3,800	8,200
2010	**29,900**	**26,800**	**18,900**		**10,000**	**1,000**	**12,000**	**0**	**3,800**
2011	30,800	27,300	19,800		10,000	1,000	12,000	0	4,300
2012	31,700	27,800	20,700		10,000	1,000	12,000	700	4,800
2013	32,700	28,300	21,700		10,000	1,000	12,000	1,700	5,300
2014	33,700	28,800	22,700		10,000	1,000	12,000	2,700	5,800
2015	34,800	29,300	18,300		15,000	1,500	12,000	0	800
2016	35,500	29,800	19,000		15,000	1,500	12,000	0	1,300
2017	36,500	30,300	20,000		15,000	1,500	12,000	0	1,800
2018	37,700	30,800	21,200		15,000	1,500	12,000	1,200	2,300
2019	39,200	31,300	22,700		15,000	1,500	12,000	2,700	2,800
2020	**40,300**	31,800	**18,300**	**3,000**	**20,000**	**2,000**	**13,000**	**0**	**(200)**
2021	41,400	32,300	19,400	3,000	20,000	2,000	13,000	0	300
2022	42,700	32,800	20,700	3,000	20,000	2,000	13,000	0	800
2023	44,200	33,300	22,200	3,000	20,000	2,000	13,000	1,200	1,300
2024	45,800	33,800	23,800	4,000	20,000	2,000	15,000	800	800
2025	46,800	34,300	19,800	5,000	25,000	2,000	12,000	0	300
2026	47,800	34,800	20,800	6,000	25,000	2,000	13,000	0	800
2027	48,900	35,300	21,900	6,000	25,000	2,000	13,000	900	1,300
2028	50,100	35,800	21,100	7,000	27,000	2,000	13,000	100	800
2029	51,300	36,300	20,300	8,000	27,000	4,000	13,000	0	300
2030[3]	**52,500**	**36,000**	**21,500**	**8,700**	**27,000**	**4,000**	**13,500**	**0**	**200**

* Excess groundwater pumped equal to total pumpage less recharge and perennial yield of 8,000 ac-ft per year

(1) See Table 2-3 for additional information on estimated surface water supplies

(2) Urban demand and supplies from Phase One Report

(3) Buildout conditions represent modified phasing included in this plan

6.5 Northeast Village

The Northeast Village is not anticipated to have significant water demands until nearly 2020. Existing water supply for this village is the least of the plan area with an estimated overall shortage of 8,000 AF per year at buildout. Currently the largest supply to the area is International ID. In addition portions of this village area are currently annexed to FID and thus capable of receiving FID water when available. A large portion of the village area is currently planted in permanent crops which are irrigated from a combination of surface and groundwater supplies. The aquifer in the area is limited in thickness and the surface soils are predominantly clay. These two factors combine to decrease the potential for extended groundwater use without well injection. Therefore, 100% of water needs are anticipated to be provided by surface water. This village also includes a large amount of very low density residential and schools which will likely utilize a dual distribution system to minimize treatment requirements.

FIGURE 2-1
Spring Water Levels

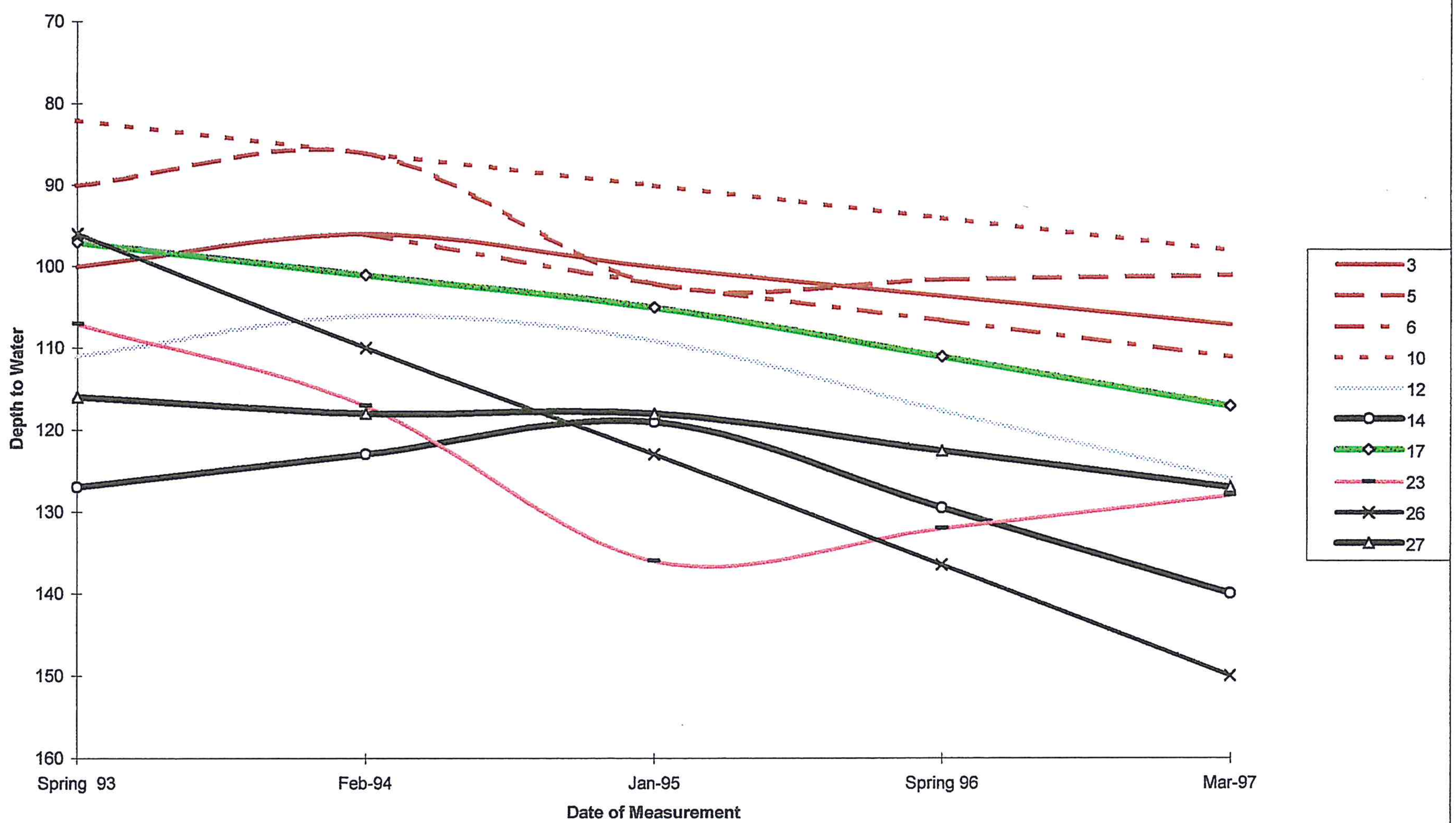


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Figure 2-2
Planned Raw Water Supply at Buildout

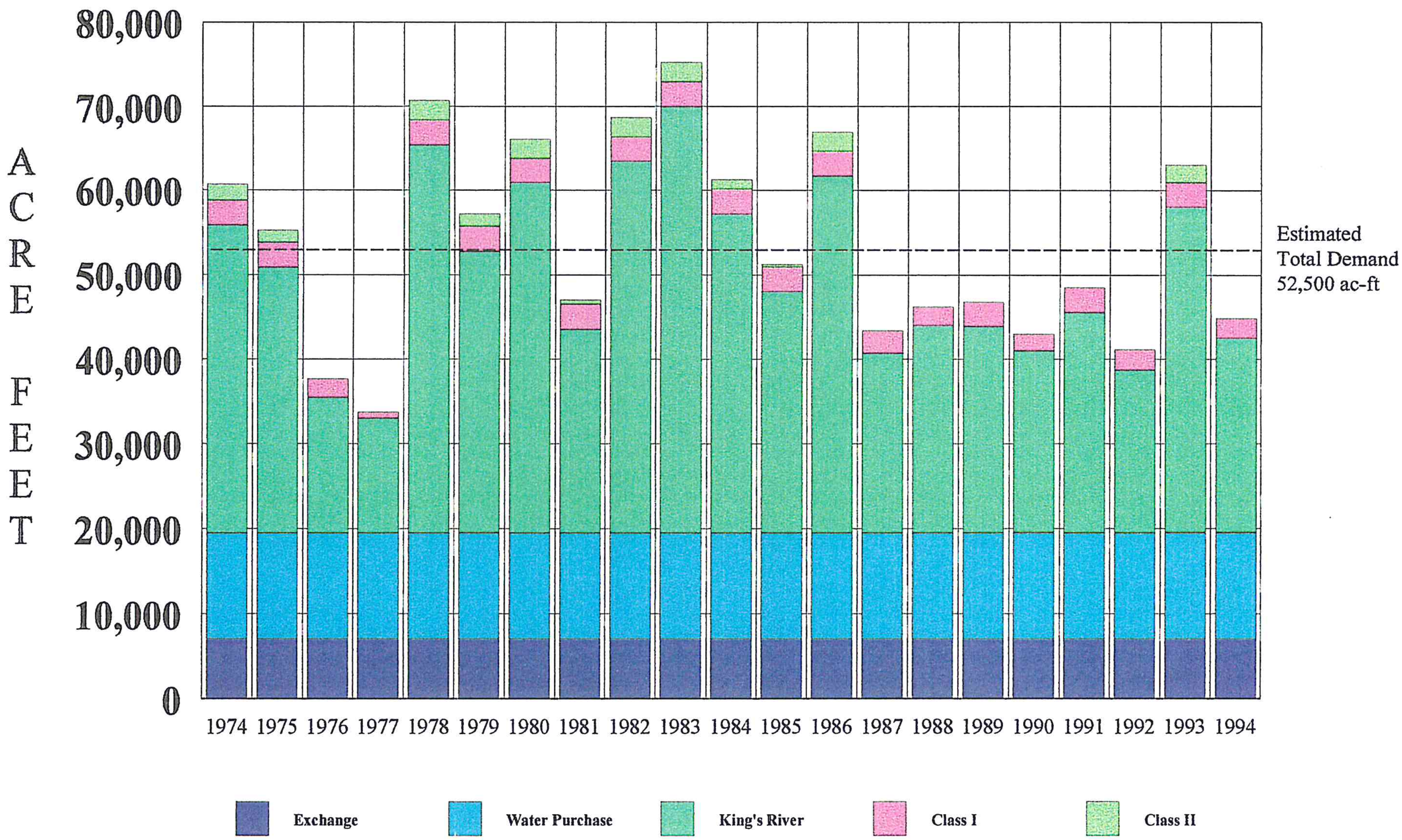

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FIGURE 2-3
Urban Area - Kings River Entitlement

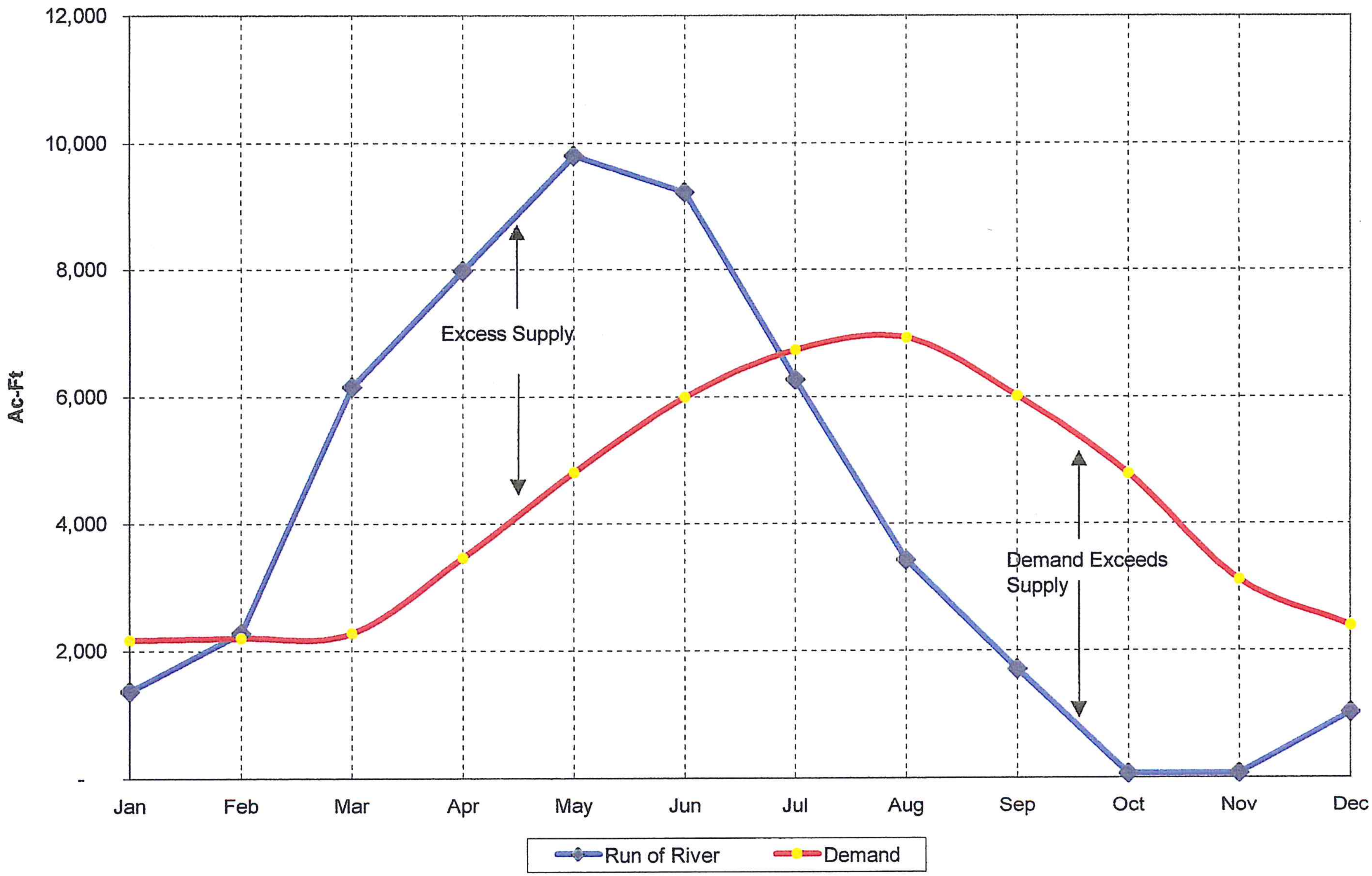

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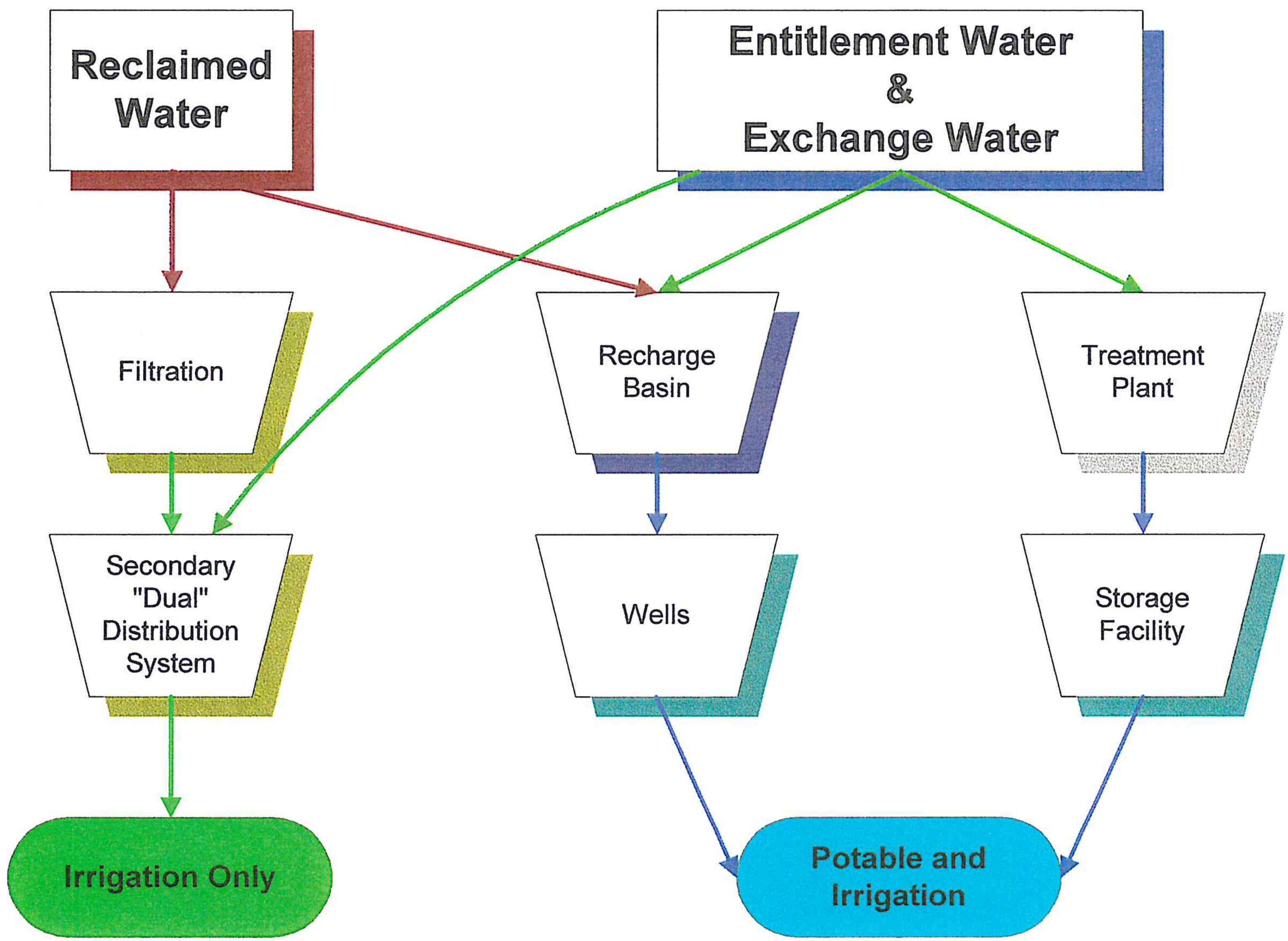
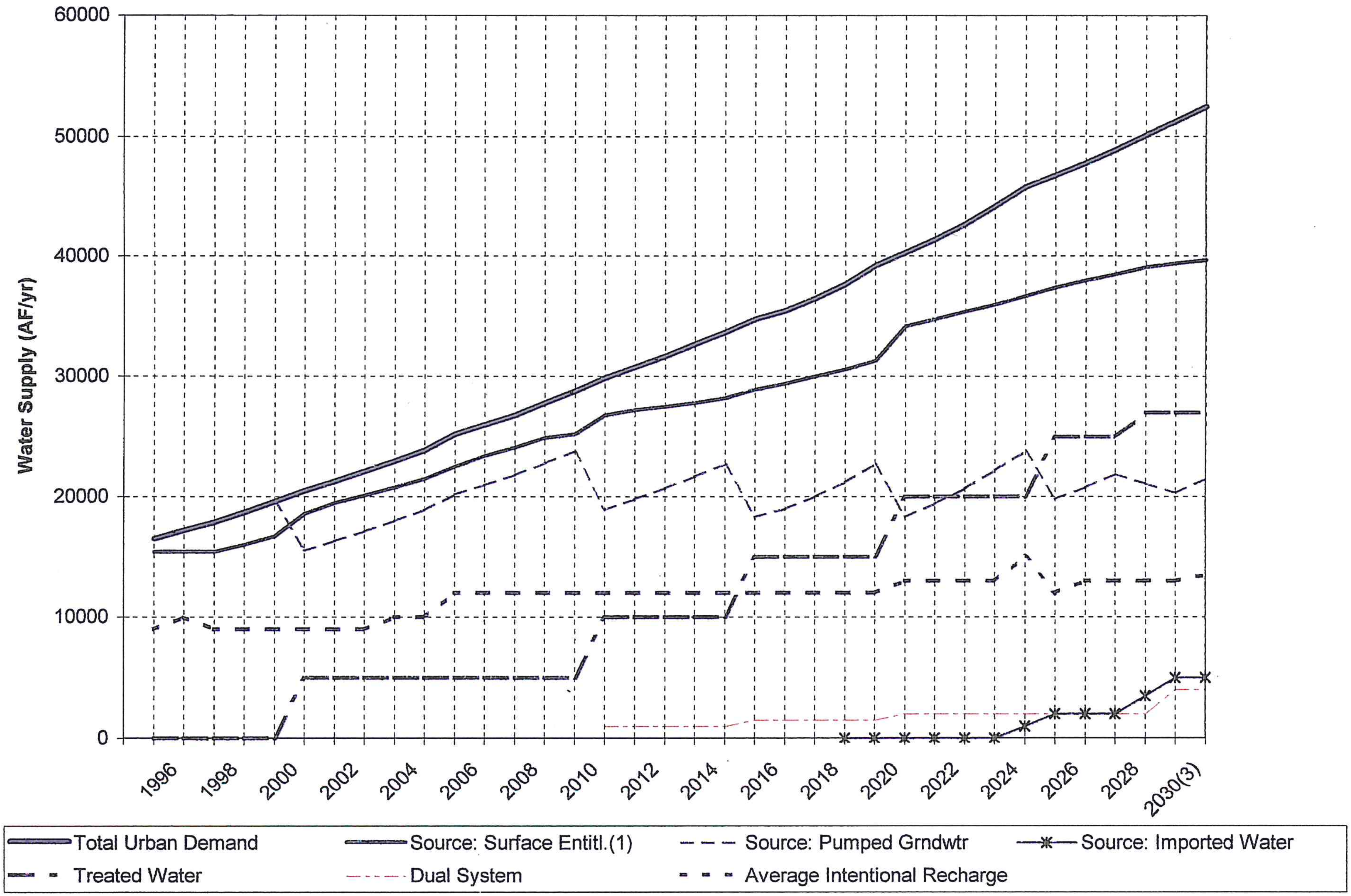
Figure 2-5
Water Delivery Mechanisms



process.vsd

Figure 2-6
Water Supply Elements in Combined Urban Areas



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Technical Memorandum 3
Water Treatment Plant Alternatives
and
Fresno Joint Water Treatment Plant Investigation

PART 1 - INTRODUCTION AND BACKGROUND

The Phase I Water Master Plan prepared by Provost & Pritchard identified the need for a surface water treatment plant (approximately 20 mgd) to serve projected needs of the City of Clovis. The first phase of this treatment plant (approximately 5 mgd) is needed to reduce or eliminate the present overdraft on the City's groundwater supply, and to satisfy the need for projected growth. Further expansion of the surface water treatment plant will be needed as growth occurs and demand increases throughout the City.

There are two apparent ways to satisfy the need for treated surface water supplies through the City of Clovis; a treatment plant built and operated by and for the City of Clovis, or joint use of a City of Fresno treatment plant, now being designed. This technical memo explores the relative merits of the second alternative. The information was developed from a "reconnaissance level" study, which was not a detailed engineering analysis but rather a broad look at the concept sufficient to provide reasonable indicators. The results of this study indicated that with all things considered, including cost, governmental control, unresolved water issues, and others, it was in Clovis' best interest to pursue a Clovis owned and operated plant. This study was presented to the City Council on November 10, 1997 for consideration. The Council concurred with the recommendation and directed City staff to complete the Master Plan on that basis.

PART 2 - FRESNO PROJECT STATUS AND TIMING

2.1 Discussions with Fresno Staff

During development of this memorandum, Provost & Pritchard and City of Clovis staff met with City of Fresno staff to discuss the possibility of participation and joint use of the new Fresno Water Treatment Plant. Lengthy discussions covered timing, cost sharing arrangements, sources and supplies of water, distribution of treated water, possible billing and the cost repayment options, and similar items. City of Fresno staff expressed a keen interest in a joint facility, and discussed apparent opportunities for a joint project to develop into a "win win" situation for both communities. Favorable response from Fresno staff allowed a more in depth analysis of the joint treatment plant to proceed.

2.2 Fresno Project: Content and Anticipated Costs

Site purchased The site for the Fresno plant was purchased several years ago and lies north of the Behymer Road alignment between Willow and Maple Streets. This site is favorable to both Fresno and Clovis, in that it is near the boundary of the two city spheres of influence, and generally upgradient from most of the potential service areas in either city. In addition, the site is adjacent to the Enterprise Canal and could be easily served with Kings River Water delivered by the Fresno Irrigation District (FID) through the Enterprise Canal.

Funding In 1995, the City of Fresno authorized and sold approximately $46 million in bonds, earmarked for construction of water system improvements. The funding allocates approximately $20 million for a new surface water treatment plant, and about $4-5 million each for a new raw water supply pipeline and for distribution system improvements. The authorized bonds have been sold; the "clock is running" for expenditure of the funds. The City of Fresno is aware the funds must be expended to avoid legal concerns with arbitrage.

EIR in place The City of Fresno has completed an environmental impact report for the site. This site was originally described in the EIR as a recharge site and provided for the possibility of use as a water treatment plant.

Consultant selected The City of Fresno selected a consultant to perform preliminary design studies and provide design services for the treatment plant. A contract for engineering services with the joint venture of Montgomery Watson/HDR was approved by Fresno City Council on June 3, 1997. The first phase of this contract required:

- Re-evaluation of the need for surface water in the City of Fresno; the study was expected to confirm earlier recommendations for a surface plant for the City.
- Pilot tests of candidate treatment processes on surface water from both the Friant/Kern Canal and FID/Enterprise Canal. The pilot test was to be highly restricted in duration and used only as an indication of treatability and likely process selection for each of the water supplies. The City of Fresno was unable to wait for a lengthy pilot test program and desired to move forward with process selection and plant design in the third quarter of calendar 1997.
- Completion of a watershed sanitary survey for the Kings River Water Supply, in accordance with California Department of Health Services (DHS) requirements. This study is required prior to approval as a public water supply source. Although a survey has been completed for the Friant/Kern Canal/Millerton Lake watershed, a similar study had never been conducted for the Kings River watershed/Enterprise Canal system. It was understood that this work was included in the City's current consultant contract. A watershed sanitary survey was completed and identified those conditions and activities which could potentially affect water supply. In the Kings River watershed, issues include grazing activities, mining activities and leaching,

crop land flood overflows, and agricultural tail water return. The duration of the watershed sanitary survey was expected to be 4-6 months with subsequent approvals taking another 2-3 months. The draft survey was completed in January, 1998.

2.3 Source of Water

The anticipated source of water for the Fresno Treatment Plant is a pipeline to the Friant/Kern Canal. A summary of water consumption and supply within the FID boundaries indicates that continued supply of 40-60,000 acre feet a year from U.S. Bureau of Reclamation (BuRec) is necessary to prevent overall depletion of groundwater reserves within the area. As a consequence, both the City of Fresno and FID have a strong desire to maintain this source of external water. At present, Fresno staff preference is to use BuRec contract water, but several issues must first be resolved with the BuRec and are discussed further below.

An alternate source of water for the Fresno plant is Kings River Water delivered by FID through the Enterprise Canal. The City of Fresno has several concerns with deliveries through the Enterprise Canal, related primarily to consistency of raw water quality. The Enterprise Canal runs through agricultural land for approximately twenty miles before reaching the treatment plant site. Several thousand acres of agricultural land lie uphill from the canal. Agricultural runoff, surface water runoff, and possible spills could conceivably find their way into the canal. Fresno staff feels that a pipeline directly from the Friant/Kern (F/K) Canal will minimize these concerns. The shortest pipeline between the canal and the site would be approximately 5 miles in length, generally along Copper Avenue. A more expensive alternative which would eliminate delivery downtime due to F/K canal maintenance would include a pipeline connected directly to the Friant Dam, about 10 miles in length. We have assumed the shorter connection will be eventually selected; a new canal turnout would be required to accommodate the supply pipeline.

Theoretically, the more consistent quality of water from the F/K canal will allow a less expensive "direct filtration" type of treatment process to be built. The choice of treatment process is discussed further below.

2.4 BuRec Contract Renewal

The City of Fresno does not have unblemished access to BuRec water. Although a long dissertation of the contract negotiation history between these two parties is outside the scope of this paper, it is correct to say that substantial issues remain between the City and the BuRec regarding long term deliveries of Bureau water. The City of Fresno is liable for outstanding costs and charges; these and other issues are discussed below. A cost summary is included in the appendix.

<u>Contract O&M charges</u> Operation and maintenance (O&M) charges were imposed by BuRec after their original delivery contract was signed with Fresno. Fresno has

chosen to take delivery of the water and pay only the contract delivery cost ($10.00 per acre-foot), without paying the actual capital and O&M costs (an additional $16.24 per acre-foot). These deficits have accrued over a long period and continue to increase at an 8% annual interest rate. The City of Fresno is now accumulating unpaid O&M charges at the rate of approximately $3,020,000 per year. Their unpaid balance at present is estimated to be approximately $45,000,000. Some form of settlement of this account with BuRec will be necessary prior to contract renewal.

<u>Hammer clause</u> The City of Fresno has agreed to sign a binding commitment requested by BuRec to commit to a contract renewal. This agreement will eliminate the threat of additional interest penalties which could be assessed by BuRec on subsequent deliveries.

<u>Metering of residential deliveries</u> A major factor in the contract relations regards BuRec conservation requirements, including metering of water deliveries and increasing tiered rate pricing. Recent voter initiatives prohibit the City from using meters as a device to determine residential monthly water bills. Unless changed by the voters, this circumstance will prohibit the City from imposing a tiered rate structure on residential users. Resolution of this issue may be difficult for the City of Fresno and will require creative mechanisms to avoid or resolve. Results of further discussions and activities with the BuRec are unknown at this time.

<u>Reassignment</u> One method for Fresno to avoid the metering issue is to reassign all or a portion of their water rights. Under one scenario, a portion of the Bureau allocation would be retained by the City of Fresno; the metering requirement for this allocation would be met by the City's present metering of multifamily, commercial, and industrial users. The remaining allocation (perhaps 20,000 AF) would be reassigned to the City of Clovis, Fresno Irrigation District, or perhaps other users. It is presumed that either of the reassigned users could demonstrate sufficient metering to meet the remaining BuRec requirements.

A further comment regarding reassignment is that Fresno has not confirmed the need for their entire 60,000 acre foot allotment and believe that their projected needs would be satisfied with approximately 40,000 acre feet of surface water deliveries. This surplus represents a possible source of additional water to serve the City of Clovis and also to resolve a sticky issue with the BuRec regarding metering. This combination represents a possible win win scenario which both cities can explore.

Fresno City staff is confident that the BuRec problems can be resolved and that the water treatment plant will be put on line using BuRec surface water supplies. The availability of surface water from the Kings River, delivered through FID's Enterprise Canal, is planned by City staff to be retained as a fall back position.

Until resolution of outstanding issues between Fresno and the BuRec, there is no assurance that the "Fresno-only" plant will operate using Friant/Kern Canal water and a direct filtration treatment process. It appears that Fresno is planning for raw water supply to be furnished with a raw water supply pipeline to the F/K canal and also to cover the contingency of an Enterprise Canal supply by providing a conventional treatment process.

PART 3 - COMPARISON OF TECHNICAL ISSUES

3.1 Location of Supply and Need

Growth is certain to occur on the outer perimeter of the presently populated Clovis service area. This does not require, however, that sources of additional water be located exclusively in the growth areas. Because treated water is easily distributed through piping networks, it is feasible for the sources of supply to be physically separated from demands. Satisfying the new demands requires adequate piping, booster pumping, and storage to transfer the water economically.

Delivery of water from the joint Fresno/Clovis treatment plant site to the growth areas in Clovis is presumed to require transmission facilities larger than Clovis' present distribution grid system. **Figure 3-1** illustrates one potential arrangement of pipelines which would deliver treated water from the Fresno site to the eastern parts of Clovis. Take-offs from this transmission main would be used to serve growing parts of northern Clovis; growth to the east of Clovis would be served by water delivered from the terminus of this pipeline into a distribution grid system similar to that now in place. The cost of this pipeline would be attributable solely to Clovis.

3.2 Annual Delivery Schedule of WTP

Should the Fresno Water Treatment Plant obtain delivery from the BuRec at Friant/Kern Canal, water deliveries will be available for approximately 11 months each year. This will allow the City of Clovis to offset peak summer demands and allow off-season deliveries. The availability of surface water during off-season months represents an opportunity for the city to allow many of its wells to rest, allowing recharge of the aquifer ("in-lieu" recharge of the groundwater). Note that this benefit is directly due to the use of surface water regardless of source or treatment plant location.

Year-round deliveries of surface water from the FID Enterprise Canal may be somewhat less reliable. Present understanding with FID is that the canal will be out of service for maintenance one or two months each year; this requirement is, we believe, negotiable with the FID. Annual deliveries will also be dependent on yearly precipitation in the Sierra. It is possible that FID deliveries would have to be curtailed for a few months in some years. Scheduled negotiations with FID have not progressed to the point where a firm delivery schedule can be defined. Note that delivery schedule concerns are directly due to the use of FID water and would apply equally to either a Clovis facility or a joint facility using FID supplies.

3.3 Ability to Accommodate Peaks

Planning for future water delivery systems in Clovis must accommodate the need to satisfy not only annual average usage but also peak demands during hot summer months. A joint Fresno/Clovis treatment plant and a Clovis-only treatment plant would be roughly equal in the ability to satisfy peaks. Our understanding at present is that the Fresno Treatment Plant will be designed as a "base-loaded" plant; this method of operation implies that the plant will be operated at a constant rate throughout the day and the flow volume leaving the plant will not fluctuate greatly between midnight and peak hour. The result is a less expensive treatment facility. Both Fresno and Clovis have the ability to accommodate this mode of operation because a surplus of groundwater wells allows existing wells to be cycled on and off to accommodate peaks.

The need for additional treated water storage is a second factor related to the ability to meet peaks. Assuming a base loaded treatment facility, the issue of additional water storage is a function of system operation, especially well production and scheduling. It has little relation to treated surface water or the location of the surface water treatment plant.

3.4 Treatment Process

Selection of treatment process is an inexact science which balances factors such as capital cost, operating cost, ability to accommodate interruptions, anticipated regulations, and similar factors. Fresno will soon begin a series of tests to assist in selection of the appropriate process for their facility. In advance of results of this testing program, we must make assumptions regarding the process to be used. If treating F/K canal water solely, it is likely that a "direct filtration" process would be selected. This process eliminates two process units used to flocculate and clarify the water prior to filtration, making the process less expensive to construct. Although lower in cost, the direct filtration facility has less ability to cope with high turbidity episodes in the raw water.

At the time this technical memo was prepared, a treatment plant designed exclusively for Clovis was also expected to use a conventional treatment process. Comparisons herein were made on that basis. As preparation of this Phase II Plan has progressed, technology and cost on alternative treatment processes have improved. Further research after submittal of this Technical memorandum No. 3 indicate that Microfiltration (MF) Technology will be essentially equal in cost to a conventional process. The MF plant has other advantages, including modularity, compact size, and ease of expansion.

3.5 Initial and Long Term Cost to Clovis

Treatment Process capital costs For purposes of this study either treatment plant will use a conventional treatment process. However, larger treatment plants are usually able to deliver finished water at a lower cost per gallon, due to economy of scale. Some fixed capital costs of treatment plant construction, (e. g. administration buildings and maintenance facilities), are spread over a larger production, and the proportional share of fixed cost per delivered gallon is comparatively less. This difference in cost is on the order of a few percentage points, and is not identifiable at this level of study.

Operating costs In addition to spreading capital costs, operating costs such as testing, operator training, sampling and similar, can be distributed. In general, larger plants have a slightly lower labor cost for a delivered unit of water than smaller treatment plants. This reduction of cost is partially due to economies of scale and partially due to overall higher levels of automation at larger facilities. Other operating costs, such as power and chemical costs, are essentially unrelated to plant size and are a function only of the amount of water treated.

Raw water A third cost factor is the delivered cost of the raw water. At this time it is assumed that Clovis would supply water for treatment at either location, so this issue presents no difference between the two options.

Cross-town transport of water As discussed earlier, the proposed Fresno Water Treatment Plant is located in north Fresno, near the intersection of Willow and Behymer Roads. This site, although attractive to both City of Fresno and City of Clovis, is assumed to require installation of a five mile pipeline for raw water delivery from the Friant/Kern Canal. The cost of this pipeline represents a possible increase in capital costs to the City of Clovis.

3.6 Cost Comparison

Table 3-1 represents a comparison of projected costs for a joint Fresno/Clovis plant, compared against a Clovis-only treatment plant drawing water exclusively from the Enterprise Canal. In preparing this table we have made several assumptions:

- The Clovis-only alternative requires no cross town transmission facilities. Reaches of pipes larger than the normal distribution grid will be required near the point of delivery for either alternative; these connections to the local system are assumed to be equal under the two alternatives.
- The Fresno/Clovis joint alternative includes a BuRec supply line and provisions for conventional treatment necessary for FID deliveries.
- The Clovis only alternative uses a conventional treatment process.

Note from **Table 3-1** that the capital cost of treatment and delivery facilities is slightly less for the Clovis-only alternative. The difference in projected costs is approximately 15%; this difference could be easily widened, or eliminated altogether, by differing assumptions. The slightly lower cost of a Clovis-only alternative is attributed to the fact that the lower capital costs of the Fresno plant due to economies of scale are offset by the increased cost of raw water and finished water pipelines. Although costs represented in the table are reconnaissance level estimates, it is our opinion that there is no significant cost advantage to joining with Fresno in operating of a joint treatment plant.

TABLE 3-1
City of Clovis
Cost Comparison of
Independant WTP
vs
Fresno Clovis WTP

Category	Item	Extension	Notes
Fresno/Clovis WTP			20 MGD, expandable to 40
	Raw water pipeline		
	subtotal	$ 6,062,500	
	Water treatment facility		
	subtotal	$ 18,689,125	
	Cross town transmission line		
	subtotal	$ 4,687,500	
Total cost		$ 29,439,125	
Clovis portion of costs (5MGD)		$ 10,875,406	25%of raw + treatment, all of cross town transmission
Independant Clovis WTP			5 MGD, expandable to 20
	Raw water supply		
	subtotal	$ 100,000	
	Treatment facility		
	subtotal	$ 8,794,250	
	Finished transmission		
	subtotal	$ 500,000	
Total plant cost		$ 9,394,250	

Other studies of treatment plant capital costs in the metropolitan area have been completed in the past; a summary of selected costs is presented in **Table 3-2**. As can be seen from the table, the anticipated capital cost of a treatment facility varies at approximately $2 per gallon of capacity. This generalization implies that the cost of a 5 million gallon per day facility is on the order of 10 million dollars. These rough estimates of cost are affected by the content of the assumptions which went into each theoretical facility. Some of the variables which account for differences in costs include the following:

- Initial oversizing of part of the facility to accommodate later expansion.
- Type of treatment process anticipated (this also affects operating cost projections).
- Size of site and price of land.
- Proximity of the site to raw water supply and to delivery points into the distribution system.
- The level of automation and instrumentation of the treatment facility.
- The extent of ancillary facilities included in the project, such as maintenance facilities, storage yards, laboratories, and administrative office space.

The numbers presented in **Table 3-2** are intended only to provide a reference point for comparison.

PART 4 - OTHER ISSUES AFFECTING JOINT OWNERSHIP

There are a number of other important issues which might influence the decision as to whether Clovis would be better served by a joint Fresno/Clovis plant or a separately owned treatment facility. Most of the remaining issues involve elements of risk or control. Issues related to uncertainty of the current BuRec contract have been discussed previously and remain important. Provost & Pritchard has discussed the concept of joint ownership with City of Clovis Utility Department staff, Planning Department staff, and others.

Table 3-3 presents a summary of those issues which we understand to be important to the City of Clovis. Because of the widely differing nature of viewpoints and issues, it is not possible to score an issue nor to sum scores for different categories. Summing scores would be the logical equivalent of "adding apples and oranges," and is meaningless. Nevertheless, the table presents our evaluation of the comparative attractiveness of either alternative when compared on the basis of an individual issue, and is useful to encourage discussion. Several issues are discussed in greater detail below:

TABLE 3-2
Previous Estimates of Cost
(Unadjusted for Inflation)

	Current estimate	Previous estimates		
		1989 Report Montogmery Watson	1994 Report Provost & Pritchard	1992 Report CH2M- Hill
Fresno/Clovis WTP				
Initial/ultimate capacity (initial/ ultimate)	20/40	20	N/A	25/50
Total plant cost	$29,500,000	$57,000,000	N/A	$60,000,000
Approximate capital per initial mgd	**$1.48**	**$2.85**		**$2.40**
Independant Clovis WTP				
Initial/ultimate capacity	5/20	5/5	10.7	N/A
Total plant cost	$9,394,250	$9,400,000	$28,300,000	N/A
Approximate capital per initial mgd	**$1.88**	**$1.88**	**$2.64**	

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TABLE 3-3

City of Clovis
Comparison of
Clovis WTP
vs
Fresno Clovis Joint WTP

Category and issue	Discussion
Costs	No significant difference between alternatives
Capital costs	
O&M costs	
Technical issues	
Location of WTP	Clovis only plant will be much more favorably located
Annual delivery schedule capability	No significant difference between alternatives
Ability to allow "in lieu" recharge	No significant difference between alternatives
Ability to accommodate peaks	No significant difference between alternatives
Ability to meet present / future treatment standards	No significant difference between alternatives
Raw water Delivery	Clovis-only alternative is closer to supplies
Treatment standards	No difference between locations
Issues related to service contract	
Participation in planning and decisionmaking for	A Clovis only plant has no significant contracting issues
expansions	
treatment process	
disinfection methods	
redundancy and reliability	
Ability to buy only what is needed	Possible advantage to joint WTP
Degree of obligation for deliveries- take or pay??	Negotiable- will depend on above e factor
Ownership of facility	Negotiable
Operation of facility	Likely to be by others
Staffing of facility	Clovis would probably not staff the joint facility
Cost sharing mechanisms	negotiable
Other issues	
Certainty of desired water supply	Clovis WTP option has much more certain water supply
Ownership of water supply	Clovis WTP allows ownership of water supply
Ownership/ control of WTP	Clovis WTP will be City owned, exclusively
Ability to serve areas outside FID limits	Primarily dependent on water source; Bureau water more able to accommodate perimeter areas.
Reliability of deliveries to Clovis	
Physical facilities	No significant difference between alternatives
Political commitment	Fresno commitment possible subject to change
Communications	Joint plant would require joint scheduling, staff meetings, etc.
Operations	Clovis only may provide quicker response
Maintenance	Joint plant may allow greater flexibility
Implementation related issues	
Ability to meet Clovis' scheduled need on time	No significant difference between alternatives
Need for siting and EIR studies	Fresno WTP is well ahead of Clovis

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4.1 Ownership of the Water

If the City of Fresno were to reassign a portion of its BuRec Water Rights to the City of Clovis, we believe that the City of Clovis will expect the equivalent of "clear title" to that water. Recent discussions with Bureau staff indicate that a reassignment of water rights will be handled by dividing the existing contract into parts, and replacing Fresno's name with Clovis' name in one of the parts. Clovis could then be a recognized holder of a Bureau contract; as such, Clovis could treat BuRec water directly, without involvement by Fresno. This would avoid the difficulties with Fresno's contract renewal process. Any option which relies on successful conclusion of Fresno's position with BuRec retains an element of risk.

4.2 WTP Ownership

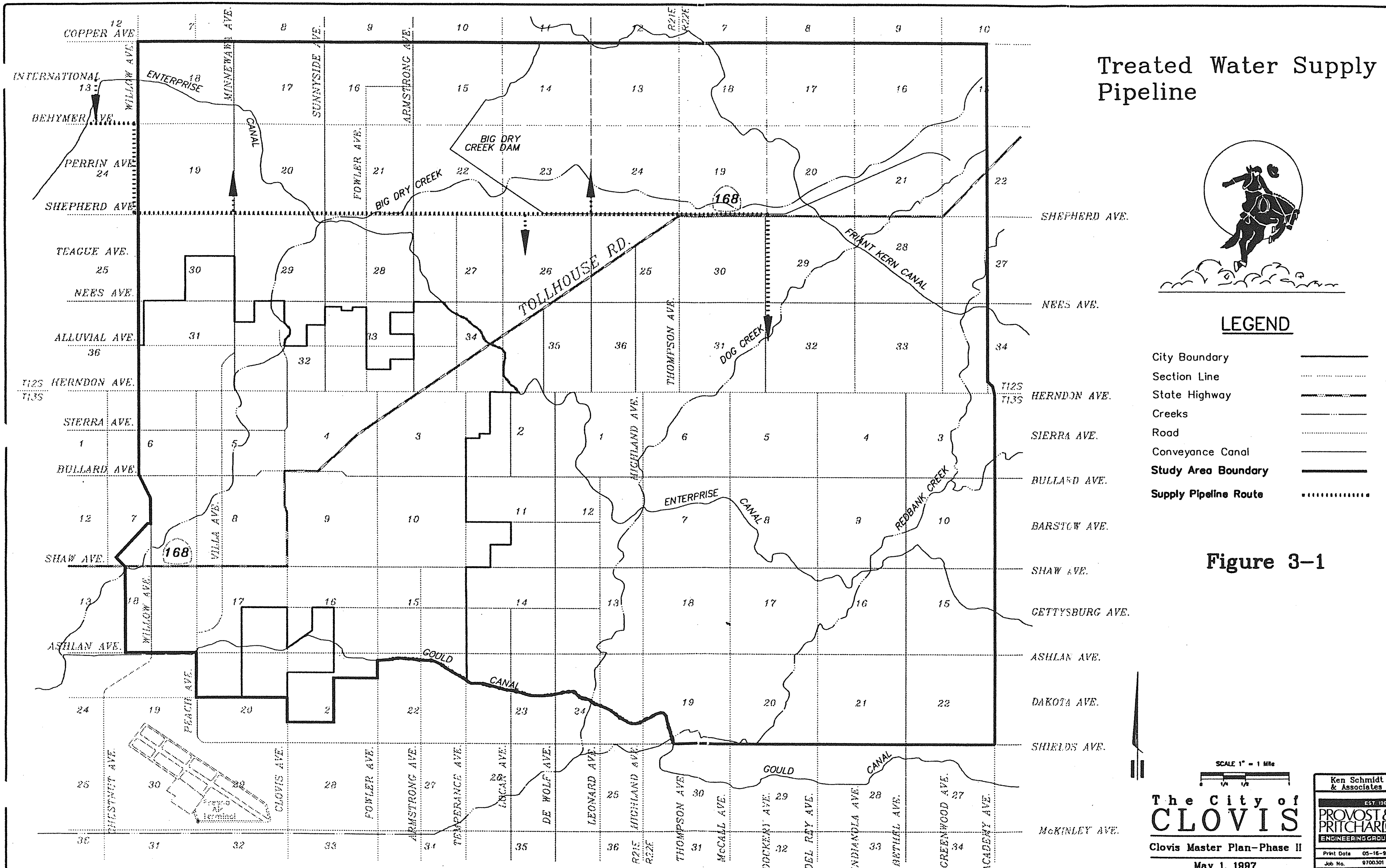
Possible ownership arrangements for a joint treatment facility have been discussed with City of Fresno staff. Several arrangements are possible:

- Joint ownership under a Joint Powers Authority, similar to the wastewater treatment plant.
- Sole ownership by the City of Fresno, with treated water delivered to Clovis under a wholesale supply contract.
- Ownership solely by the City of Fresno, with Clovis paying the operational cost for treatment of its own water supply at the Fresno facility, with subsequent deliveries back to Clovis.

Of these three alternatives, Fresno staff greatly prefer alternative 2, under which the City of Clovis would assume the role of a large wholesale customer. This alternative presents Clovis with very little control of the facility, its operation, or its cost, and is similar in concept to the situation under which Clovis now participates in the waste water facility. Although workable, it is possible that the City of Clovis may desire greater influence in the planning, design, operation, and pricing of the treatment facility.

4.3 Evaluation of Joint Option

After discussions with City of Clovis staff, and evaluation of the factors as presented in **Table 3-2**, it was our recommendation that Clovis proceed with a separately owned water treatment facility. We believe that overall delivered costs to the Clovis customer will not be greater and cost presents little reason to choose either alternative. Nevertheless, issues which have to do with control of water source, schedule, cost, and political feasibility seem to us to represent a clear preference for an independent treatment facility. As pointed out in staff discussions, the selection of a "Clovis-only" plant at the present time does not preclude possible joint ownership in the future, if that becomes preferable due to changes in conditions. At present, a "Clovis-only" WTP presents the clearest path to a reliable source of treated surface water.



Treated Water Supply Pipeline
LEGEND
City Boundary
Section Line
State Highway
Creeks
Road
Conveyance Canal
Study Area Boundary
Supply Pipeline Route
COPPER AVE.
INTERNATIONAL
BEHYMER AVE.
PERRIN AVE.
SHEPHERD AVE.
TEAGUE AVE.
NEES AVE.
ALLUVIAL AVE.
HERNDON AVE.
SIERRA AVE.
BULLARD AVE.
BARSTOW AVE.
SHAW AVE.
GETTYSBURG AVE.
ASHLAN AVE.
DAKOTA AVE.
SHIELDS AVE.
McKINLEY AVE.
WILLOW AVE.
MINNEWAWA AVE.
SUNNYSIDE AVE.
ARMSTRONG AVE.
FOWLER AVE.
THOMPSON AVE.
HIGHLAND AVE.
VILLA AVE.
PEACH AVE.
CHESTNUT AVE.
CLOVIS AVE.
TEMPERANCE AVE.
LOCAN AVE.
DE WOLF AVE.
LEONARD AVE.
McCALL AVE.
DOCKERY AVE.
DEL REY AVE.
INDIANOLA AVE.
BETHEL AVE.
GREENWOOD AVE.
ACADEMY AVE.
ENTERPRISE CANAL
BIG DRY CREEK DAM
BIG DRY CREEK
TOLLHOUSE RD.
FRIANT KERN CANAL
DOG CREEK
REDBANK CREEK
GOULD CANAL
168
T12S
T13S
R21E
R22E
Fresno Air Terminal
SCALE 1" = 1 Mile
The City of CLOVIS
Clovis Master Plan-Phase II
May 1, 1997
Ken Schmidt & Associates
PROVOST & PRITCHARD ENGINEERING GROUP
EST 1968
Print Date 05-16-97
Job No. 9700301
Dwg. No. 95-0036b

Figure 3-1

Technical Memorandum 4
Existing Water Distribution System: Hydraulic Modeling and Analyses

PART 1 - INTRODUCTION

This Technical Memo discusses the elements associated with the assembly and operation of a hydraulic model of the City water system and the associated elements including production (wells), distribution, storage, and pumping facilities. This memo will be presented in two parts (A & B) because of timing issues. Part A will address the issues and elements associated with the creation of the model. Part B will address calibration and modeling of future conditions including water treatment facilities, separation into pressure zones, and additional storage facilities. A discussion of our methodology, along with our findings, is presented below.

TECH MEMO NO. 4
PART A

PART 2 - BACKGROUND

The City of Clovis has maintained a computer model of their water system for many years; initially, this model was maintained on a proprietary software owned by a consulting firm. About three years ago, the City had the model converted into software more readily available. The new software, EPANET, used the previously modeled data set; as a consequence, existing inaccuracies in the data were carried into the City's revised model. Examples of inaccuracies include incorrect pipe sizes, seldom used pipe friction formulas, and difficult to understand methods for applying peak factors.

For modeling required in this study, the City has chosen to continue using EPANET software because of:

- Relative ease of operation and maintenance.
- User friendly for occasional users.
- Free of charge from the public domain.
- Well supported in the marketplace.

To avoid repetition of inaccuracies in the existing model, Provost & Pritchard has constructed an entirely new model using EPANET and has used the new model for all analyses and conclusions in this study.

PART 3 - EPANET

Version 1.1e is the current version of EPANET and was used in preparation of the model. EPANET has been in use in the marketplace for approximately ten years. Version 1.1e is a recent update and provides the capability to create contours from results and data, reviews the results to identify areas that are hydraulically isolated, and simulates growth of a constituent up to a limiting factor.

EPANET is a text based modeling program and lacks any capability to directly link to a graphical input/output program such as AutoCAD. This limits the transfer of data for processes such as model construction because construction is not graphical or "on – screen." As a result, the model construction process is not intuitive. Other programs are available which utilize the same algorithms and are linked with a graphical interface but these programs are more costly and often entail proprietary data files that are not as transferable. EPANET does allow the import of text files prepared for older versions of KY Pipe. During this project, P&P staff used other software tools to construct the model data sets which were then converted directly into EPANET. This exercise provides a convenient "reality check" of model construction and also allows visual checks of the model construction as it is constructed.

3.1 Database File Format

EPANET uses two independent sets of data or files for each simulation. The first file is referred to as the input file which contains physical information about the model elements and is used to describe all the actual elements in the model. The file can have any eight character name with an ".inp" extension. The input file is divided into several sections, some of which are required. Each section begins with the title of the section in brackets. For example, the pumps section which contains all the data describing how pumps will operate begins with the heading [PUMPS]. Within the input or map files any text following a semi colon is ignored by the program. Thus, in order to make the input file more user friendly, notes or descriptions are frequently included on the same line with data, separated by a semicolon. At times, whole lines will begin with a semi colon indicating that the line is for note purposes only, and will be ignored by the program.

The second file or map file contains coordinates for each node that is listed in the input file. The file name is referenced in the input file and is usually identified by a ".map" extension. Both files are required for a simulation. A more complete description of each file can be found in the EPANET users manual included with software.

EPANET is set up to allow a great deal of flexibility to the user. Nodes, for example, are described with elevation, demand, and demand code. Pipelines are described by ID number, length, diameter, roughness coefficient, and the nodes on either end. Note that the length given in the input file is independent of the length shown on the screen. The model uses the input length for the calculations but the appearance on the screen is dictated by the coordinates provided in the map file which is wholly unrelated to the calculations. Other model elements such as pumps, tanks, wells, and demand patterns can be described or modeled in a variety of ways.

For each simulation, changes must be made to the input or map files using a text editor or software able to import text files. Typical changes include updates and corrections to information, but more often consist of various trials of proposed solutions or future conditions. Each trial requires editing the input file, therefore it is critical to protect the base file and to document changes that are made for individual runs or simulations. The process is somewhat cumbersome, but workable. To facilitate the process, Provost & Pritchard has prepared an Excel spreadsheet that contains all the basic data, along with a macro that automates the creation of the necessary map and data input files. More details on this process will be included later in this memo.

PART 4 - MODELING - GENERAL ISSUES

4.1 Level of Detail

The model constructed for this project does not include every pipe in the City system. Because the model will be used only for overall planning purposes, it is not necessary to include every pipe. Rather it simplifies the process by skelatonizing the system to include only those pipes which will impact the actual operation of the system. Pipes and components included in the system should construct a complete model, but only of sufficient detail so that the model fairly represents actual conditions in the system. Future uses of the model may necessitate the adding of pipes to the model, especially when used for evaluating adequacy of a small part of the system. If further detail is needed in the future, the data set is easily manipulated. For the current model, all pipes 10" in diameter and larger were included, while six and eight inch pipes were included where they were determined to be of importance to transmitting water within the grid system.

4.2 Node and Link (Pipe) Numbering System

Nodes and pipes within the system are numbered generally from west to east and from south to north. For convenience, the City is broken into four corridors as shown below:

	Node and Link Numbers
Dakota - Shaw	200 - 1990
Shaw - Barstow	2000 - 2990
Barstow - Herndon	3000 - 4990
Herndon - Nees	5000 -

This numbering pattern allows quick reference to individual locations, and allows for future expansion. The model is constructed with "gaps" in the numbering system, in order to allow addition of nodes or elements to the model while still maintaining the general numbering system. In addition to being somewhat intuitive, this methodology allows unlimited addition of nodes and elements in growth areas.

4.3 City Map

For presentation purposes, and to facilitate input and development of coordinate basis, the basis of the map database was taken from a City map provided by the City on AutoCAD. The AutoCAD map is included with this memo, but is not connected by any direct link to the data files. Coordinates of the nodes were taken from the AutoCAD map to create the map file used by EPANET. If future nodes are added to the model, coordinates can be obtained from the AutoCAD map. Node and pipe numbers are also shown on the AutoCAD map. The node numbers are actual AutoCAD points which can be input and downloaded in the same fashion as survey points. The line numbers exist only as text in the AutoCAD file.

4.4 Demands

A critical part of any hydraulic model is identifying the quantity and location of system demands. For this model, the demands were developed as part of the research completed for Tech Memo #1. Demands were identified based upon meter records for 1996 for the entire network. From the annual demands, generic land use coefficients were developed. A spreadsheet model was prepared that included a take off of the land use within each section of the City. This process was facilitated by the City meter routes which are generally one quarter section each. Demand factors for each land use were applied and checked against the annual use as established by the billing records. The spreadsheet can easily be updated to reflect changes in land use and will automatically generate demands that can be further verified by future meter billing records, as well as input into a revised model.

PART 5 - MODEL CONSTRUCTION

5.1 Node & Pipe Data

Mapping coordinates for elements of the model were taken electronically from the City's existing AutoCad base map. The resulting node data was transferred from AutoCad into the Excel spreadsheet to facilitate map file development. A copy of the AutoCad map is included with this report along with a reduced printout. Elevations were interpolated from the USGS quad sheet.

Node and Pipe data were manually entered using a third party program which operates within AutoCad. The pipe data was taken initially from the City water plats and verified by review of plans and discussion with City personnel. Information on wells and storage facilities was also extracted from public utility records where available. Service lines connecting wells to the distribution system were generally estimated in terms of size and length.

In order to properly calibrate the model, it is necessary to determine the likely materials and approximate age of pipes in the existing system. The water plats which were the primary source of information for existing pipes contained some references to pipe material, but were inconsistent in places. Age of pipes was estimated based upon personal knowledge and review of historic aerial photos. In addition, we interviewed utilities staff to confirm findings and identify any missing pipes.

5.2 C Values and Fire Flow Tests

In order to initially estimate pipe roughness, we reviewed fire hydrant flow tests performed by the City Fire Department. Using their records for selected pipes in each of several areas, we were able to calculate the theoretical roughness which would be necessary to allow the pipe to produce the measured flow at the measured pressure drop. This exercise was then extrapolated to cover all pipes within the system. Pipes of similar age and material were assigned similar roughness coefficients. A table of pipe age/material/and assigned roughness factors (C values) is included in the appendix.

5.3 Storage Tanks and Booster Pumps

Limited information about existing water storage facilities and booster pumping systems was provided by staff. Each tank/booster combination is modeled using a fixed output for the pump. The desired level of precision for the model allows us to use a fixed output for each pump, without the need to reconstruct and calibrate a pump curve for each individual pump. In the future, if the City should desire to conduct "extended period simulations" with the model, more precise pump curves may be desirable. For this model, the fixed tank output was based upon SCADA records for peak periods.

5.4 Well and Pump Modeling

There are three methods which can be utilized to model a well. The method used should be determined based upon the desired output and quality of data to input. Following is a brief discussion of each method.

The most complicated method involves inputting a fixed reservoir (groundwater) elevation in conjunction with a pump and the respective pump curve. This method is preferred if the pumping water level is known and the pump curve is accurately measured. In the previous model this method was used on several wells in the system, but a check of available pump curves, measured pumping levels, and known well output showed that the input data was not representative of system operations. This method requires the highest maintenance for the model and would require multiple checks and alterations in the future as pumping characteristics and pumping water levels change. In addition, the use of this method adds complexity to the model without improving the output or results.

The second alternative for modeling a well is to set a fixed reservoir node with an HGL equal to the operating pressure at the well. This method will allow unlimited water to enter or leave the node while maintaining the system pressure. This method is simple, but requires careful monitoring to insure that the input of the fixed node is within actual operating range of the well.

The third and preferred alternative is to input the well as a node and apply a negative demand. This method simply forces a set amount of water into the system at the node and allows other factors to balance out the pressure. This method is easily input and allows easy updating for any changes based upon changes in actual well production. The process becomes one of simply matching actual operating capacity with current production. To control the well requires a simple control or valve which can be input into the pipe connecting the well to the system. This method was used for most wells because of the relative simplicity and ease of maintenance. In the future, as well as for production changes, the values are easily updated. In addition, a fixed pump output allows the model to reach equilibrium without "hunting" for a suitable operating point for the pumps. In a system which has numerous supply points (wells) the software has historically been unable to reach equilibrium satisfactorily.

5.5 Data Input Spreadsheets

Because of the repetitious nature of creating multiple simulations, we have constructed a spreadsheet model which includes the physical characteristics and data (length, age, capacity, and similar) to be input into the model for all wells, tanks, pumps, and selected pipes. The Excel spreadsheet incorporates several individual worksheets which contain different portions of data. In order to preserve the integrity of the model, it is recommended that a back up copy of the original spreadsheet be kept with updates documented carefully. Use of spreadsheet for data input is easier for the occasional user and provides convenient editing tools (such as cell copying and global formatting) for manipulating the data. Excerpts from the facilities input spreadsheet are contained in the appendix; an electronic copy of the file will be provided for the City's future use.

5.6 Confirmation of Data

After all identified facilities were input and graphically displayed, we reviewed the record drawings in the office of the City Engineer, to confirm the data and to assure ourselves the information was correct and complete. Information on age of facilities was also gained during this exercise. A thorough review of the information with knowledgeable City staff also added reliability to the base information.

5.7 Demands

Demands input into the model were developed as discussed previously. Demand figures were assigned to each identified node in accordance with land uses served by that node. Note that many nodes are not assigned a demand; these nodes are generally inserted as a matter of convenience for modeling and represent a junction between pipes, a change in size, or similar. Note also that the sum of demands assigned to the nodes is equal to those presented as the existing peak hour demand in TM #1.

5.8 Supplies - Wells

Water supply from wells are modeled as fixed nodes as outlined earlier. However, under normal conditions not all wells are active and hence some control system must be developed for the program to balance production and demand. In order to provide an initial balance of production and demand, wells were prioritized to identify which wells would be "baseloaded" and which would operate as needed or on standby. Wells were divided into geographic areas and ranked based upon capacity and efficiency. The wells ranked in the top two classifications are assumed to always be operating under peaking conditions, with the remainder being controlled by system demand and pressure requirements.

The general priority of wells used for modeling purposes is as follows:

1. Those wells which now have, or are soon projected the use, activated carbon units for DBCP removal. Staff normally operate these wells nearly continuously to maximize the use of the wells, to treat as much groundwater as possible, and to reduce migration of the contaminant plume.
2. Those wells with relatively shallow pumping levels; these wells are likely to produce water at lower pumping cost (i.e. the most efficient wells).
3. Back up wells which are only utilized when all other wells are insufficient to meet demand. These wells are known to either produce water of questionable quality, or have higher production costs.

A list of wells and their corresponding priority is found in the appendix. Prioritization of wells will be reviewed in greater depth at a later time once the model is fully operational and simulations of alternative priorities can be reviewed.

5.9 Remaining Tasks

There are several major elements related to the hydraulic model which remain to be completed. The first is the calibration of the model with peak hour data that will be collected this summer. It is anticipated that the peak day and hour will occur sometime in July or August, 1997 with the resulting analysis occurring sometime in early September, 1997 as the data becomes available. With complete operating parameters provided by the SCADA system, we will analyze the capability of the system under severe stress. The system will then be calibrated to the point at which it is able to match the actual conditions recorded to within an acceptable range. Only after this calibration is complete will the model be of value as a decision support tool. Data was also reviewed for Summer 1998 but due to cooler than normal weather, higher peak flows were not experienced.

Following calibration, the system will be examined to identify any bottlenecks, deficiencies, or problems which merit immediate attention. In addition, the capacity of the system to transport water across the system will be reviewed (such as would be required with the location of a concentrated source such as a treatment plant). Areas with potential low pressure problems will also be identified and examined in terms of potential solutions.

The second element which will be completed after calibration is the expansion of the model to account for growth and ultimate buildout conditions. In order to minimize the number of scenarios, the initial step will be from existing to ultimate. This will help identify the facilities required at ultimate buildout, which will vary according to water supply location that is selected during other phases of this project. As a part of this task, system pressure and operating constraints will be reviewed to examine the need for or viability of dividing the system into multiple pressure zones.

TECH MEMO NO. 4
PART B

PART 6 - DISTRIBUTION SYSTEM MODEL: INPUT INFORMATION

6.1 Surface Water Treatment Plant

Federal and State Regulations regarding surface water treatment continue to develop and become more complex. As a consequence, it is necessary to study not only existing regulations but also anticipated regulations when considering the nature and cost of a new facility. As a subconsultant to Provost & Pritchard, the firm of Black & Veatch has prepared a summary of existing and likely regulations affecting surface water treatment and also a summary of the impacts those regulations will have on the City of Clovis. The two summaries are found in Appendices A and B.

As mentioned previously, the buildout capacity of the surface water treatment plant is recommended to be 30 mgd. Black & Veatch performed an analysis of existing water quality records and available treatment process. Based on a 20 year present worth analysis of capital and operating costs, all alternatives considered were close in annual costs. The microfiltration process offers advantages in terms of site requirements, sludge handling, microfiltration expandability, phasing and operator attention requirements and with these advantages it is recommended this treatment process be utilized.

The recommended initial capacity of the treatment plant is 5 mgd. This initial increment was selected to allow construction multiple filtration units, so as to provide sufficient redundancy for continued operation during filtering. Further expansion of the treatment plant can occur in as little as 1-mgd increments, but it is suggested that 5 MGD units be used for purchasing power to a total installed capacity of 30 mgd.

Based on the recommendation of the B&V report, it appears that approximately 30 to 40 acres of space should be acquired for the treatment plant and solids handling operations. If additional space is available on the plant site, provision of distribution system storage is recommended. On-site storage will allow centralized operations and will require fewer site purchases.

In order to process the volume of sludge and filter backwash residuals that are generated from this treatment plant, the space of approximately 15 acres will eventually be reserved for solids handling. The site for the treatment plant must contain the treatment process and support facilities. The need for treated water storage and sludge handling can be satisfied on the treatment plant site or at a remote location, depending on the available site acreage, proximity to other storage sites, and other constraints.

6.2 Treatment Plant Siting

Using the above estimates of space constraints, we investigated the availability of parcels of ground within approximately 1/2 mile of the Enterprise Canal. A number of candidate sites were discussed with City staff and two general locations were identified as possible locations for the treatment plant. The generalized location is illustrated on **Figure 6-2** and was used for modeling purposes.

The location is on the fringe of existing development, and many individual parcels near each location are likely to undergo subdivision or other development within the next 5 to 10 years. (Several parcels near each site have already passed through the mapping process and are now being subdivided.) As a consequence, it is important that the City move quickly to secure a suitable site for the treatment facility.

Land uses in the general vicinity of the candidate site are primarily agricultural at present. For purposes of the remaining planning, and for delivery of water to the customers, any suitably sized parcel within the area is judged to be equally acceptable. After selection of a candidate site, a more detailed description of the site development will be prepared so that the site can pass through an environmental review process under CEQA.

6.3 Additional Wells and Recharge Facilities

The existing wells, even when supplemented by a surface water treatment plant, will not provide sufficient water to satisfy maximum day demand at the buildout condition. Additional wells are necessary to fully utilize groundwater supplies and to extract recharged water. **Figure 6-2** shows potential locations for additional wells, as recommended by Ken Schmidt, Consulting Hydrogeologist. For purposes of this analysis, each new production well is assumed to be approximately 1,000 gpm. Approximately 21 additional wells will be required to satisfy maximum day production capacity, when combined with a 30-mgd treatment plant. To supplement the above on-line wells, approximately 6 additional wells are needed in reserve to allow for routine well failure, occasional down time and similar difficulties. If wells installed have lower or higher capacities, the number of wells will need to be adjusted accordingly.

In addition to adding wells, the City must add recharge facilities. **Figure 6-4** also shows tentative or possible locations for recharge facilities. As discussed earlier, an additional total of approximately 160 acres will be necessary for the buildout condition. The acreage can be adjusted upward or downward depending on actual recharge performance of the selected sites.

6.4 Operating Storage

The water supplies and redundant facilities discussed above are selected to deliver sufficient water for the maximum day of any year. As detailed in Technical Memorandum No. 2, this report recommends that the surface water treatment plant be designed to continuously produce an eventual 30 mgd (million gallons per day). Sufficient on-line and standby wells must be available to satisfy the remainder of maximum daily demands, eventually requiring as many as 27 additional wells, depending on capacity.

Even after the addition of the treatment facility and additional wells, <u>the combination will not be sufficient to meet the needs of the customers during the peak hour</u>. Operational storage of treated water, combined with booster pumps as necessary, is required to meet the peak hour demand. Location and volume of storage provided is dependent on the physical layout of the distribution system, and will be discussed further below.

6.5 Pressure Zones

During the 1970's the distribution system in Clovis was designed to operate at a pressure gradient elevation of approximately 470-490. This elevation was sufficient to provide reliable pressures to the service area of the time. Growth since that time has caused the system to change. The operating pressure has gradually been raised over the years to provide adequate pressure to development in the northern and eastern parts of the City. Growth in these areas impacts the existing water system due to two physical conditions. The first is increased horizontal distance from the source to the demand. The majority of the higher producing wells are in the westerly and southwesterly parts of the City. The second is the greater elevation of the newer areas. Both of these factors stress the existing system in different ways and require planned responses to minimize their negative consequences.

An understanding of the concept of hydraulic gradient is necessary in order to understand the impacts and necessary modifications to the water system. Hydraulic gradient can be thought of as the maximum elevation at which the system can fill a column of water. The hydraulic grade can be defined in two ways; the first is the water surface elevation in feet, and the second is the pressure. In a static system, the gradient is relatively constant throughout the system, but the pressure varies with the ground surface. When water movement occurs, the gradient is depressed or raised at a location, and water flows in the "downhill" direction of the gradient. For any assigned condition, a computer model of the system can be used to view hydraulic gradients, pipe friction losses, and the resulting pressures.

As shown in **Figure 4-1**, City growth in higher elevation areas has required the system gradient to be increased in order to provide sufficient pressure in outlying areas. In addition, the concentration of water source in the western portion of the service area requires the movement of water north and east through the system. This movement creates pipe friction losses, which cause the grade line to slope downward to the northeast. In order to overcome the friction/transmission losses, the grade line in the western area must be further elevated, especially with higher flows. Both the higher elevation and the need for cross town transport of water require a gradient increase in the existing service area in order to serve the growth areas. This increased gradient has several negative impacts:

- Increased pressure in the southern and southwestern parts (older sections) of the City. The higher pressures are thought to aggravate the frequency and severity of leaks.
- The 500,000 gallon elevated water storage tank in Letterman Park no longer "floats" on the system. The pressure gradient is now above the overflow of the tank and this storage is effectively lost from the system. Without manipulation by staff, the tank would always remain filled, and be unavailable for satisfying short-term peaks.
- Pumping costs are increased due to the increased system pressure.

Some review of operating pressure gradients is therefore in order. As shown in **Figure 4-1**, the topography of the entire Clovis Study Area rises generally from the Southwest corner at elevation 340, to the Northeast corner, at approximately elevation 490 (near the intersection of Shepherd and Thompson). Existing ground elevations within the present service area are generally no higher than Elevation 390. If the future system were operated as a single distribution system, the difference in ground surface would produce a water pressure approximately 65 psi greater in the southwest than the northeast. If 40 psi system pressures were to be maintained in the northeast corner, static distribution system pressures in the southwest corner would be on the order of 105 psi. When the system was stressed and incurred losses, this pressure would likely by increased 20-30 psi.

Clearly, no single pressure can be chosen to satisfy the needed minimums in the higher parts of the city without excessive pressures in the southwest. It is therefore apparent that the distribution system should be divided into two or more pressure zones. The result of dividing the system into zones is the ability to deliver water at acceptable pressures without unnecessarily high pressures at any location. Additional benefits could also include re-gaining the use of the present Letterman storage tank, less leakage, and reduced pumping costs.

After discussion with staff and extensive model simulations, Provost & Pritchard recommends a partition be established that divides the present system into three pressure zones. The boundary between zone one and two should follow approximately the ground surface contour at elevation 380. The boundary between zone two and three would be the proximate alignment of Thompson Ave. **Figure 6-2** shows the recommended location of these pressure zone boundaries. The existing pressure zone (Zone 1) should be maintained at a pressure gradient elevation of approximately 490-510 feet; system pressures in the upper reaches of this zone will be approximately 40 psi. Reducing the existing system gradient to elevation 480 will allow the Letterman Park tank to fill and empty with daily fluctuations in demand; some adjustment of well pump set points would be necessary to accommodate this change.

The recommended hydraulic gradient in the middle zone (Zone 2) is approximately elevation 560, providing distribution system pressures of approximately 40 psi in the higher parts of Zone 2, and higher pressures in lower parts of the zone. Future growth in the distant northeast part of the study area will dictate the need for a third pressure zone with approximate elevation of 620. This third zone will need to be supplied from storage and pumping facilities, similar to those described herein for Zone 2. Because all potential Zone 3 demands are combined with Zone 2 for this analysis, it is not necessary to establish the boundary between the second and third zones at present.
Dividing the distribution system into separate pressure zones presents several operational concerns that do not exist in the present single zone system. The most important to recognize is that connections between the two pressure zones must include pressure regulating devices; pressure reducing valves are commonly used when water is released from an upper zone to the lower zone. Booster pumping systems are required to elevate water from the lower zone to the upper zone. Due to the mechanical nature of zone inter-ties, utilities commonly limit the number of such connections. For a system the size of Clovis, three to five connections are likely between the two pressure zones. Location, capacities, and equipment requirements at these inter-connections will be defined at the buildout condition in this memo, and defined for other planning intervals in Technical Memorandum No. 6.

6.6 Zone-Specific Demands and Supplies

Design of facilities for the two zones requires that demands and supplies be quantified for each zone. **Tables 4-1 & 4-2** represent a summary of water supply, storage and inter zonal transfer requirements for the buildout planning milestones. The information is presented for the overall water transfer requirements at winter (minimum), average day, max day, and peak hour conditions for the planning horizon conditions. Similar information for intermediate planning milestones and associated facilities will be discussed in Technical Memorandum No. 6.

1. **Table 4-1** Present conditions, after division into zones. Some limited growth is allowed for Zone 2.
2. **Table 4-2** Buildout conditions, with a 30 mgd water treatment facility.

Several comments can be deduced from inspection of the figures:

- Transfers of water between the zones is maximum on the maximum day at the buildout condition, when an average of 9300 gpm must be delivered from Zone 1 into Zone 2.
- Storage requirements for the two zones are also shown. For purposes of this representation, the storage requirements have been calculated as the difference between the max day and peak hour flow rates; this flow is assumed to be provided from storage for a 240-minute period. It should be recognized that Zone 1 storage requirements can be satisfied by above grade facilities (tanks), or aquifer storage and additional wells. The existing Letterman tank in Zone 1 appears to have adequate capacity to provide equalizing storage at the buildout conditions assumed. Use of this storage will require a lowering of hydraulic gradient or addition of pumps from the tank into the system.
- Storage required for Zone 2 will eventually amount to 5 million gallons; this is a combined total that includes volume which will eventually be dedicated to a Zone 3 feed tank and pumping station.
- The figures do not account for locations of any facilities; it may be possible to combine the storage for either or both zones at the treatment facility, for example.
- The water treatment plant will not be serviceable during canal down times, shown as "Winter" condition on the figures. As a consequence, all water consumed in Clovis in some winter months will be generated from groundwater sources. Since the predominance of these sources are in the Zone 1 area, a 3800 gpm transfer from Zone 1 to Zone 2 will be necessary during winter months.

6.7 Interzonal Transfers

Because of the dual nature of water supply for the Clovis area, several operational constraints will dictate the design of zone interconnections. The interzonal flows shown on **Tables 4-1 & 4-4** will be supplied by five interzonal connections. Detailed design of the facilities may provide reasons to adjust the planning flows to meet specific conditions, sites, and time schedules.

Booster pumps transferring water from Zone 1 to Zone 2 could be furnished with water in two ways:

- Directly from the Zone 1 distribution system, using inline booster pumps. This method of delivery would require larger pipes in Zone 1 to supply the pumping facilities required for peak periods, but may allow slightly lower energy costs.

- Indirectly, using a storage tank to supply the booster pumps. The storage tank would be filled at off-peak hours from Zone 1 (similar to the present operation of Tollhouse Reservoir). The storage provided would be available to satisfy Zone 2 peaking storage requirements. Since Zone 1 piping would not have to satisfy peak flows for delivery to Zone 2, existing pipe sizes in Zone 1 could be sufficient. The release of Zone 1 water pressure into ground storage represents an energy cost.

Resolution of the above two methods followed a basic logical analysis:

- Zone 2 will require peaking storage facilities; these will be located at grade (not elevated). Water delivery from ground storage into Zone 2 requires the use of pumps.
- Direct delivery of water from Zone 1 to Zone 2 (without storage) would require pumps with lower head requirements. The same pumps could not be used for direct delivery and for pumping from storage.
- It is preferable to have fewer pumping stations.

The above considerations lead to a general conclusion that each point of delivery between the zones should include a combined storage tank/pumping station. This combination will allow installation of a ground storage tank/pumping station at three eventual locations with the primary purpose of delivering Zone 1 water into Zone 2. The addition of the storage tank allows the pumping station to satisfy both average and peak hour demands. System modeling, as discussed below, will determine final recommended facilities and configurations for all 3 pressure zones.

PART 7 - DISTRIBUTION SYSTEM MODEL: MODELING CONDITIONS

7.1 Demand Scenarios

The above concepts were combined into a series of computer model runs for the "buildout" condition of the distribution system. Two critical operating conditions were examined; each posed a severe design condition on a different part of the system.

- Peak hour conditions, when customer deliveries throughout the system were greatest. This condition dictated the size of pumps needed from storage and the piping sizes needed to carry water from the booster stations. Supply conditions entered into the model are similar to those depicted in **Table 4-4**.
- Max day filling condition (outside the peak hour) when the storage tanks were filling. This condition generally presented the limiting design requirement for pipes approaching a storage tank site (Zone 1 piping), because the pipes must satisfy both the maximum day customer demands plus the added demand of tank filing.

7.2 Facility Configuration Assumptions

For modeling purposes, it was assumed that any Zone 1 node connected to at least three 12-inch pipelines will be adequate to supply an interconnection between zones. The carrying capacity of the three supply lines is on the order of 5000 gallons per minute. Up to five interzonal connections were assumed.

It was assumed that a Water Treatment Plant would be built near the central part of the city. At least two sites will be modeled. With this assumption, a large diameter pipeline will be required to carry water north from the treatment plant site into the remainder of Zone 2 and to Zone 3. In addition, a large diameter pipeline from the treatment plant will also be needed to carry water west into Zone 1.

For purposes of this analysis, each new production well in the northwest and city center areas is assumed to be approximately 1,000 gpm; new wells in the southeast area are assumed to have 500 gpm capacity.

It was also assumed that five storage tanks would be available; the existing Armstrong/Tollhouse tank; the existing Villa Tank and one in the vicinity of Nees and Fowler one at Shepherd and Thompson; and one at the surface water treatment plant. Another possible storage and booster site in the vicinity of Ashlan and Fowler was also examined. The tank, was determined to be unnecessary but the pump station is still required. In order to use the Barstow/Villa tank which still appears to be in good condition a booster pump with controls to an altitude valve are suggested.

PART 8 - DISTRIBUTION SYSTEM MODEL: RESULTS

Conceptual layouts of distribution facilities for the three pressure zones and alternative water treatment plant sites were analyzed by cursory modeling of the system. The conclusion of the model runs was that a central water treatment plant site near the Enterprise Canal would most satisfactorily serve the planning area combined with additional interzonal pumping. A discussion of the resulting and recommended facilities and their location follows: A graphical summary is shown on **Figure 6-2**.

8.1 Pipe Sizes

In general, the present City policy of using 12-inch pipelines on a half-mile grid should be continued. The capacity of this main grid to transport flows across the city under high demand conditions is justifiable and the 12 inch size appears to perform well. Nevertheless, the addition of interzonal transfer requirements, and the addition of a 30 mgd water treatment plant, pose conditions on the existing system that will require larger pipes. The locations of larger pipelines suggested by the model are shown on **Figure 6-2**. A 36-inch pipeline will be needed to carry flow from the Water Treatment Plant north into the remainder of Zone 2. A 24-inch pipeline will be required heading west from the Water Treatment plant, to serve Zone 1. Refer to **Figure 6-2** for further details.

8.2 Interzonal Connections, Storage and Pumping

Interzonal Connections

As mentioned previously, five interzonal connections were investigated and are recommended between Zones 1 and 2. Installation of pressure regulating valves (PRV's) at these connections are required so that when the residual system pressures in Zone 1 become less than about 25 psi during peak use periods. Pressure can be maintained by use of flow from the Upper Zone. Low pressures distant from supply sources is already becoming the case in the northeast. Geography will put off installation of PRV's in the southeast to later years. **Figure 6-2** indicates the locations for the Zone 1-2 connections. The first is the existing Armstrong/Tollhouse storage tank. This tank is conveniently located near the center of the boundary between Zones 1 & 2 and will easily service those portions of areas of existing development which are recommended to be separated into the second pressure zone. Four other Zone 1-2 links are shown; two in the central part of the city, near the intersection of Locan and Bullard & Barstow, and one in the southern part of the city, near the intersection of Ashlan and Locan and the last at the proposed new tank location of Nees and Fowler.

Tanks Analysis of the buildout condition shows that three additional storage tanks will be needed to deliver water into Zone 2, two located near the Zone 1-2 boundary and a third at the Zone boundary from Zone 2 to 3. A new tank should be built at a north site near the intersection of Nees and Fowler; the existing Armstrong/Tollhouse tank should serve adequately as the second tank, with modification of pumps as necessary. These two tanks will serve as reservoirs from which the interzonal pumps can draw and will provide operating storage for Zone 2 during peak hour conditions. The storage tank at the SWTP is needed to provide operational flexibility to the plant and provide a supply source for the SE village should probably arise with the pump station on Ashlan Avenue. In addition, a storage tank and pumping station will ultimately be needed to boost Zone 2 water into Zone 3; this facility shall be built generally in the area of Shepherd and Thompson, but will not be needed until substantial development occurs in the Northeast Village area. Buildout storage capacity requirements are estimated in the following table.

Table 4-2
Estimated Storage Needs

TANK LOCATION	STORAGE AT BUILDOUT, gals
Nees/Fowler	1.0 million
Armstrong/Tollhouse (existing)	2 million
Water Treatment Plant	1 million
Northeast (Zone 2/3) Shepherd//Thompson	2 million
Barstow/Villa (Letterman)	0.5 million

Pumping stations It is interesting to note that the model results for peak hour conditions reveal that Zone 1 peak hour demands are satisfied entirely by Zone 1 wells and the Letterman Tank Pumping station (No flow from the Water Treatment Plant enters Zone 1 under this condition). All Water Treatment Plant flow is delivered into Zone 2, along with pumping from storage tanks at the Nees/Fowler and Armstrong/Tollhouse sites.

As shown on **Figure 6-2**, pumping stations that serve Zone 2 should be located at the Nees/Fowler and Armstrong/Tollhouse tank sites. A third station should be located near the intersection of Ashlan and Locan, operating purely as a booster station; no additional storage is required at this site. To make the existing storage at Letterman Park effective for Zone 1, a small station should be added, pumping from the tank into the system during peak hours.

In the future, a fourth and fifth pumping station each located on Thompson Avenue at Shepherd and at Nees Avenues will be needed to boost Zone 2 Water into the Zone 3; since this zone will have essentially no internal water supply, the combined capacities of these stations are projected to be the largest facilities with capacity to serve all of Zone 3 peak demands.

The buildout capacities of the pumping stations, in gallons per minute, should be approximately as shown in the following table.

Table 4-3
Estimated Pump Station Needs

Pumping station location	Approx. capacity (gpm)	Approximate capacity (gpm)
	Into Zone 1	Into Zone 2
Letterman Park	500	-
Tollhouse	4,000	4000
North Site (Nees / Fowler)	-	4000
South Site (Ashlan / Locan)	-	2500 (booster; no storage)
Northeast (Thompson) (2 stations)	-	Total capacity of 13,500 (Serving Zone 3)

PART 9 - OTHER WATER DELIVERY FACILITIES

In addition to the potable water facilities discussed above, it is important to recognize that systems must also be installed to provide and distribute non-potable water within the City. Because the annual water budget at the buildout condition requires the use of approximately 4000 acre-feet of reclaimed or raw water, it will be necessary to identify approximately 600 acres of land to be irrigated from these sources, in lieu of potable water. Suitable locations for such irrigation may include school grounds, highway medians and rights- of- way, parks, cemeteries, and other public spaces. It may also be appropriate to provide dual distribution systems in subdivided areas that develop to low densities, so that raw water can be provided for landscape irrigation. Close identification of such areas is outside the scope of this memorandum. We have, however, identified some such areas conceptually, in order to estimate the required piping, pumping, and facilities which may be needed. A summary of this concept is presented on **Figure 6-4**.

Figure 4-1. Existing & Proposed Hydraulic Gradient Elevations

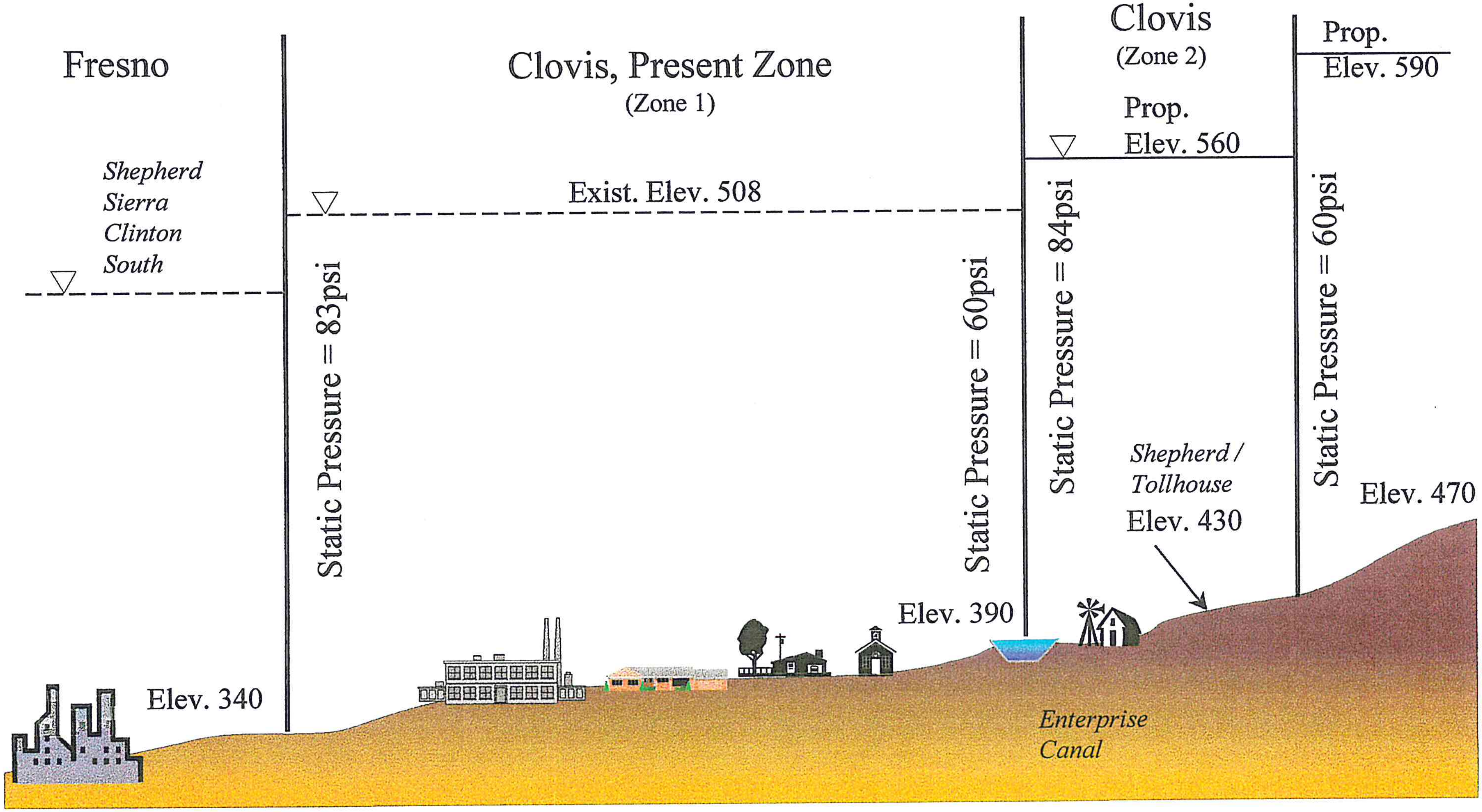


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TABLE 4-1
PRESENT CONDITIONS
Water Use and Transfers, by Zone
(gpm, unless indicated otherwise)

ZONE 1	*Demand*	*Production*
Peak Hour	32,000	
Wells		24,000
Storage		8,000
Max Day	22,000	23,000
Avg Day		
Winter		
Storage	2.5 MG	
Hours @ Peak Flow		5.2

TRANSFERS ===>>>
Pk Hr
Wells
Storage
Max
Avg
Winter

ZONE 2	*Demand*	*Production*
Peak Hour		
Wells		
Storage		
Max Day		
Avg Day		
Winter		
Storage	0 MG	
Hours @ Peak Flow		0

SWTP to Z1		0 MGD Water Treatment Plant	*Production*	*% Capacity*	SWTP to Z2
		Peak Hr			
		Max			
		Avg			
		Winter			

Note: Production is from wells only.

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TABLE 4-4
BUILDOUT CONDITIONS
Water Use and Transfers, by Zone
(gpm, unless indicated otherwise)

ZONE 1	*Demand*	*Production*
Peak Hour	50,400	
Wells		50,000
Storage		400
Max Day	35,600	44,900
Avg Day	17,900	14,200
Winter	10,400	14,200
Storage	0.5 MG	
Hours @ Peak Flow		20.8

TRANSFERS	===>>>
Pk Hr	- 0 -
Wells	
Storage	
Max	9,300
Avg	
Winter	3,800

ZONE 2	*Demand*	*Production*
Peak Hour	43,400	
Wells		2,500
Storage		20,100
Max Day	30,100	
Avg Day	14,600	
Winter	7,300	
Storage	5 MG	
Hours @ Peak Flow		4.1

SWTP to Z1		30 MGD Water Treatment Plant	*Production*	*% Capacity*	SWTP to Z2
		Peak Hr	20,800	*100%*	20,800
		Max	20,800	100%	20,800
3,700		Avg	18,300	88%	14,600
		Winter	3,500	17%	3,500

Note: Zone 2 includes Reagan Education Center, Assemi Development, New Apts and some Existing Housing.

Assume following operational levels at WTP

Pk Hour	100%	Avg Day	88%
Max Day	100%	Winter	17%

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Technical Memorandum 5
System Analysis and Recommendations or Planning Horizon Conditions

PART 1 - INTRODUCTION AND OBJECTIVES

This Technical Memorandum addresses and provides general recommendations for growth of the water utility in Clovis. In doing so, a discussion is included of water supply sources, treatment facilities, and storage and distribution facilities. This memorandum will form the basis of the Plan facilities.

In general, this memorandum discusses only the facilities needed for potable water supply; those facilities needed for nonpotable supply (reclaimed water or raw water delivered for irrigation) are highly dependent on development conditions, and are discussed only conceptually.

Information provided includes planning assumptions and resulting facilities needed for the projected conditions at the buildout planning horizon year 2030. Demands and facilities for intermediate years and costs for facilities by planning increment will be presented in Technical Memorandum No. 6. Before proceeding with a detailed discussion of water supply sources and facilities, it is important to list the objectives that provide an overall framework for the studies performed:

- Provide a safe and reliable water supply for present and future Clovis residents.
- Provide adequate water pressure throughout the City for normal uses and for fire protection.
- Avoid water use restrictions in all but the most severe drought conditions.
- Provide water quality sufficient to satisfy present and anticipated regulations.
- Protect and maintain groundwater aquifer quality and long term balance of water budget.
- Maximize utilization of surface water supply, for either recharge or as a source of treated water.
- Operate facilities in a cost effective manner.
- Provide facilities that are technically feasible given the complex nature of groundwater conditions underlying the planning area.
- Limit the impacts on the existing system.

PART 2 - PLANNING ISSUES AND POLICIES

The above objectives must be met by a combination of recommended facilities and practices while recognizing there are numerous concerns.

These include threats to the existing groundwater supply such as:

- Continuing overdraft of the aquifer indicated by declining water table levels.
- Contamination of existing groundwater sources, most notably DBCP.
- Occasional failure of wells, temporary power outages
- Limited capacity of the groundwater aquifer.
- Limited recharge capabilities, due to geologic constraints.

The addition of a surface water treatment plant, as recommended in Technical Memorandum No. 2, adds several additional planning issues to the water utility:

- Drought and extended water shortages, either for a single year or multiple years, which diminish the surface water supply.
- Contamination of the surface water supply, either in the watershed or the raw water delivery facilities. Sources of contamination include natural or biological contaminants such as those from mining or agricultural activities. Threats due to hazardous materials entering the raw water supply, such as agricultural pesticides, must also be considered.
- Failures of the raw water delivery mechanism, or outages in the canal supply, due to canal maintenance activities.

2.1 Planning Tools for the Above Concerns

2.1.1 Redundancy

Some degree of redundancy must be provided in order to meet the stated water supply goals. It is necessary to determine the appropriate levels of operating redundancy as a component of facilities planning. The overriding considerations are the assurance of a firm supply and minimizing outages. Redundancy is not a new concept to the City; at present, the City has more wells available than needed to meet present day maximum demands. These redundant wells provide operating flexibility during well equipment failures or other situations that would make a single well unavailable.

Reliable water service requires adequate standby power to deliver potable water during power outages. The City recently embarked on a program of adding standby engine power to existing wells; this program should continue. The addition of a surface treatment plant will also require alternate power feeds to the plant, or inclusion of standby power facilities to run the treatment process and pumping components. Similarly, any new wells or pumping facilities must be provided with standby power.

Redundant facilities represent a capital facilities expenditure that is unavailable for use. As a consequence it is important to make wise choices when selecting any degree of redundancy. For surface water treatment plant redundancy see "Surface Water Supply Assurance Below".

2.1.2 Groundwater Supply Assurance

Long term reliability and dependability of groundwater supplies is contingent on two major factors. The first is control of activities which could contaminate the groundwater sources. The City may not be able to exercise control over activities outside its jurisdiction but which could affect water quality in the aquifer which supplies the City The City's experience with DBCP contamination is an example of this sort, but it illustrates the present ability to cope with unforeseen quality limitations when they occur.

The second and more direct concern with groundwater supplies is continually declining pumping levels in the aquifer. Long-term water supply planning for the area includes even larger reliance on groundwater resources. As discussed in Technical Memorandum No. 2, the recommendation of this report is that the City provide sufficient groundwater recharge facilities to overcome the existing overdraft of groundwater and stabilize groundwater at present levels. Facilities needed to do so are discussed later in this memo.

2.1.3 Surface Water Supply Assurance

<u>Deliveries</u> Assurance of a reliable surface water supply to the residents of Clovis requires that the delivery, treatment, and distribution system must all be designed to accommodate unexpected restrictions or limitations. First and foremost, contract amendments must be made with the Fresno Irrigation District (the proposed surface water supplier) to assure that both quantity and quality of raw water supply are properly recognized and controlled. In particular, the following issues must be resolved:

- Overall annual delivery schedule, including both quantity and time of deliveries.
- Availability of FID surface storage attributable to Clovis. This factor most probably includes allocation of storage volume within Pine Flat Reservoir.
- Canal improvements to eliminate contamination from runoff during precipitation events.
- Control to acceptable levels of runoff contaminants entering the canal.
- Available supplies, during normal, dry, and wet years.
- Deliveries of exchange water, as a condition of discharge of effluent from the Regional Waste Water Treatment Plant.
- Capacity and contractual arrangements for groundwater banking and subsequent delivery to the City.
- Conveyance arrangements for water purchased elsewhere by the City and delivered through FID canals.

When the above mechanisms are sufficiently addressed, they will assure the City of Clovis an adequate supply in normal and wet years; an extended drought in the Central Valley could present a condition under which FID cannot meet contractual obligations to delivery water. Supplies during extended droughts must be purchased on the open market; delivery through the FID system can be arranged under nearly any purchase scenario. The purchase of short-term surface water supplies is a wide departure from present City operations. Although the City could pursue all necessary details of this supply independently, it may be easier and more reliable to arrange for FID to serve as the City's agent for this purpose. Such an arrangement would allow the District to continue to serve in the water delivery arena where its strength lies, and to allow the City to remain a retail water provider, also where the present strength lies.

Without a strong agreement between the City and FID, the installation of a surface water treatment plant will require the City to be much more active in acquiring, scheduling, and delivering raw water. That responsibility will become even more significant during extended drought situations.

Treatment Process Redundancy Normal treatment plant design provides redundant equipment and processes so that the plant is able to produce its rated capacity without the largest unit of any particular process. Redundant equipment allows treatment to continue at full capacity even when undergoing routine service maintenance or repairs. As a process example, four pumps rated at 1000 gpm would be given a firm capacity of 3000 gpm. Designs using this concept allow continued full capacity of the treatment plant during process upsets or equipment downtime.

2.2 Summary of Water Sources

At the planning horizon the annual water consumption of the Clovis area is expected to be approximately 52,500 af/year. Sources of this water, as shown on **Table 5-1**, are as follows:

Table 5-1

Water source	Average Annual Quantity (A-F)
Surface Water Supply (Treated)	27,000
Groundwater (Safe Yield)	8,000
Groundwater (Recharged)	13,500
Surface Water supply (direct uses)	4,000

Figure 5-1 describes a hypothetical annual average condition to satisfy the demand at the planning horizon in Clovis. The planning process must account, however, for the differences in supply which occur between wet, normal, and dry years. **Figure 2-2** shows how the ultimate requirements of water supply for the planning conditions would be satisfied by the recommended facilities, given surface water availability for the past 20 years. The figure shows a uniform annual demand of 52,500 acre-feet per year, and

illustrates the variable nature of surface water supplies. Several conclusions can be drawn from the information on the figure:

- Excess groundwater must be recharged (banked) when available, so that groundwater is available during drought years.
- Surface water supplies are adequate in approximately 60% of years; approximately 10% of years severe drought conditions occur and are inadequate to fully operate the SWTP.
- The use of approximately 4,000 af/year of reclaimed water and other exchanges is necessary to maintain an overall balance.
- A 30-mgd (33,600 AF/year) water treatment plant will be necessary to fully offset the shortage of water from available groundwater sources.

Key to the acceptability of this supply scenario is the acceptance of several operational and policy factors:

- The water treatment facility is "base loaded." That is to say, it is used to its maximum capacity whenever possible. This allows optimal use of surface water supplies and maximal recharge of the aquifer.
- Use of groundwater is absolutely minimized during winter months; only those wells that are pumped to maintain flow through carbon treatment units are used at minimum flows. This allows natural (in-lieu) recharge of the aquifer to occur.
- Recharge of the aquifer continues through recharge basins whenever possible. At the buildout condition, approximately 160 acres of additional recharge basins will be needed.

There exists a potential for a tradeoff between purchase of additional recharge sites for use when surplus water is available, and purchase of a larger treatment plant with corresponding surface water supplies in all years. Three scenarios regarding these tradeoffs were developed and discussed at length with staff.

- Use of only existing recharge facilities; supplemented by a 40 mgd water treatment plant.
- Use of a moderate amount of additional recharge basins, supplemented by a 30 mgd water treatment plan.
- Purchase and use of recharge facilities sufficient to handle all available surface water sources, supplemented by a 30-mgd water treatment plant.

After discussion with City staff it was agreed that the best scenario involved a "middle of the road" approach, where some additional recharge facilities are purchased, but not to the maximum extent necessary to fully utilize wet year water sources. The recommendations contained herein are structured around this concept.

2.3 Source Prioritization

The City of Clovis will have several water sources available to supply the future demands. Design of facilities is contingent, to some extent, on the priority that is placed on each of these separate sources of supply. In designing the recommendations herein, we have assumed that sources of water will be used in the following priority, highest to least priority:

1. Surface water treatment plant supply.
2. Wells with DBCP removal facilities (carbon filters).
3. Groundwater supplies and recharged groundwater, as necessary.

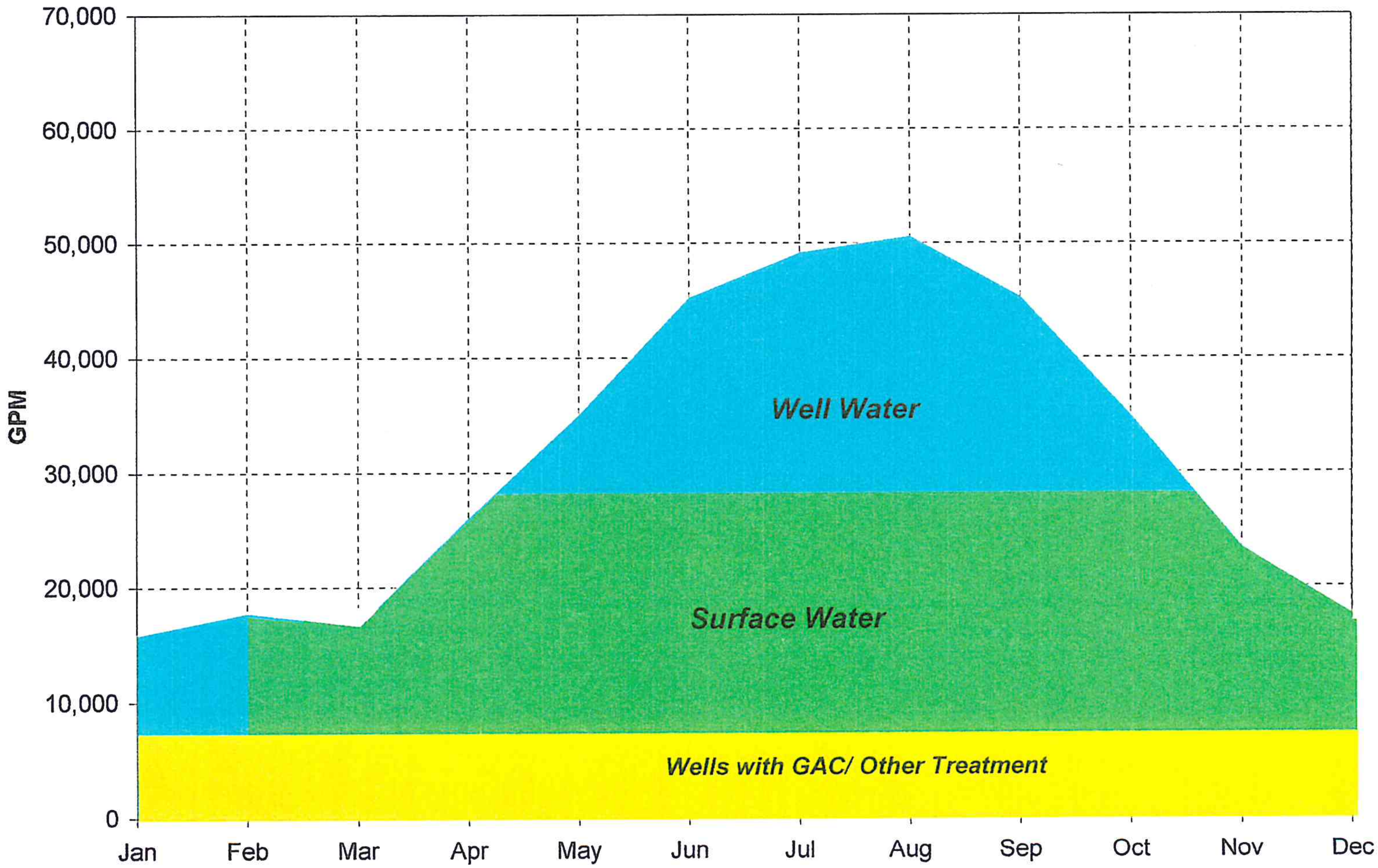
The above priority will allow the most beneficial use of surface and groundwater supplies. It is important to realize that extended pumping of the DBCP wells during winter months could satisfy a large portion of winter demand. This action would, however, reduce the deliveries of surface water during winter months and recharge of the aquifer would be insufficient to maintain a balanced water level. As a consequence, it is important to reduce pumping of DBCP wells in winter months to approximately 25% duty cycle and maintain surface water plant operations for all additional water demands. This practice will allow sufficient delivery of surface water during winter months to prevent groundwater depletion.

2.4 Summary of Water Planning Policies

After considering the above issues and needs, this plan presents a number of policy recommendations regarding water source, delivery, and distribution.

1. The City should move quickly to add specific information to their present agreement with FID, as discussed earlier.
2. The City should pursue the purchase of recharge sites identified in previous reports.
3. The City should select and purchase a site for a surface Water Treatment Plant.
4. The City should establish policies to encourage the use of untreated and or reclaimed water where feasible.
5. The City should require standby power for many new water facilities; the appropriate level of standby power may be considered on a case by case basis; but in no circumstance should the total standby capacity during a power blackout be less than the annual average day demand, which at present is approximately 12,000 gpm or 8 wells (assuming an average production of 1,500 gpm).

Figure 5-1. Annual Monthly Water Demand and Supply At Buildout (Year 2030)



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Technical Memorandum 6
Facilities Plan with Capital Improvement Plan

PART 1 - INTRODUCTION

This Technical Memorandum is the culmination of all the past efforts to initiate, acquire, study, evaluate, and propose alternative means to accomplish the goal of efficiently and effectively serving the growing water supply needs of the City of Clovis. In this memorandum, much of the previous work related to time and growth projections, facilities, and alteration and expansion of the distribution system will be condensed so that an easy reference can be attained to address how and when additional facilities need to be completed.

Also, a significant attempt was made to limit the text in this document and provide more graphical and tabular information for ease of reference. It should be noted that the ultimate facilities, as well as phasing of improvements included herein, is based on the latest available information. Projections of new development patterns is cyclic and very subject to economic conditions. Any projections beyond 3-5 years is speculative. Significant deviation of size, timing, and/or location of facilities may occur due to changing plans and development. This Plan is more dependent on the underlying assumptions than in the past due to the (1) reliance on wells that are mainly in the western part of the study area and;(2) the need for conveyance of water through the system to the east. At buildout conditions, the service area will have doubled in size. The separation of the system into zones further adds to the system complexity. As a result, it is recommended that the Plan be revisited and updated as new development occurs, particularly if it falls outside the parameters used in the development of this Plan.

Costs shown herein are based on 1998 dollars.

PART 2 - TIME AND GROWTH PROJECTIONS

Our initial work (Technical Memorandum No. 2, Table 2-8) identified the estimated total urban demand through buildout (year 2030). During preparation of this document, our basic assumption regarding location, direction, and schedule of growth has been the current General Plan for the City of Clovis and the assumptions used in the subsequent Wastewater Masterplan. Table 6-1 lists the supporting information on which Table 2-8 is based. Shown in Table 6-1 is the year and corresponding population related to each village. It should also be noted that the special study areas identified subsequent to the adoption of the General Plan have been included in the population numbers shown. Recent growth rates have not correlated well with projections contained in the General

Plan. To enable this plan to be used for both short and long term planning, a revised rate schedule based on Equivalent Dwelling Units (EDU) and associated water demand was developed. This schedule is shown in **Table 6-2**. EDU as defined herein is a residence with 3.1 people or 4.5 acres of office or commercial properties.

The estimated annual water demands calculated in **Table 6-2** were determined using the growth projections identified previously and applying the appropriate water demands. Land use designations shown in the Master Plan were the basis for the calculations. **Figure 6-1** is a graphical representation of the information contained in **Table 6-2**. From review of this data, it is assumed that much of the near term development is located around the newly completed Buchanan Educational Center and the Reagan Educational Center that is presently under construction. As the initiation of construction of State Highway 168 commences, it could be expected that growth pressures will be experienced between Tollhouse Road and Shepherd Avenue to the east of the Enterprise Canal.

Table 6-1

Urban Population Change within
City of Clovis
General Plan Area

General Plan Year	Existing Clovis Area		NW Urban Area		SE Urban Area		NE Urban Area		TOTAL
	Cumulative	Increment	Cumulative	Increment	Cumulative	Increment	Cumulative	Increment	
1995	63,000	13,800	-	1,500	-	-	-	-	63,000
2000	76,800	13,100	1,500	1,500	-	2,000	-	-	78,300
2005	89,900	10,600	3,000	3,900	2,000	3,100	-	600	94,900
2010	100,500	10,600	6,900	2,200	5,100	5,100	600	700	112,500
2015	111,100	-	9,100	4,600	10,200	5,100	1,300	10,400	131,700
2020	111,100	-	13,700	4,600	15,300	8,200	11,700	14,400	151,800
2025	111,100	-	18,300	4,500	23,500	3,100	26,100	14,300	179,000
2030	**111,100**		**22,800**		**26,600**		40,400		200,900

Growth Rates - From Wastewater Master Plan

Table 6-2
Population Growth represented by
Equivalent Dwelling Units (EDU) and Associated Water Demands

Planning Year (yr)	City Projected Growth (EDU/yr)	Cumulative Growth (EDU)	General Plan Projection (EDU)	Increm'tl Demand (AF)	Estimated Total Urban Demand (AF)
1998	550	550			17,900
1999	550	1,100			
2000	700	1,800	6,367	500	18,400
2001	700	2,500			
2002	700	3,200			
2003	700	3,900			
2004	700	4,600			
2005	1,000	5,600	13,275	1,600	20,000
2006	1,000	6,600			
2007	1,000	7,600			
2008	1,000	8,600			
2009	1,000	9,600			
2010	1,400	11,000	20,848	2,800	22,800
2011	1,400	12,400			
2012	1,400	13,800			
2013	1,400	15,200			
2014	1,400	16,600			
2015	2,000	18,600	28,588	4,800	27,600
2016	2,000	20,600			
2017	2,000	22,600			
2018	2,000	24,600			
2019	2,000	26,600			
2020	2,700	29,300	36,952	6,100	33,700
2021	2,700	32,000			
2022	2,700	34,700			
2023	2,700	37,400			
2024	2,700	40,100			
2025	3,500	43,600	48,271	10,600	44,300
2026	3,500	47,100			
2027	3,500	50,600			
2028	3,500	54,100			
2029	3,500	57,600			
2030	-	57,600	57,384	8,200	52,500

PART 3 - FACILITIES

3.1 Existing System Components

Traditionally, the City of Clovis has utilized wells, tanks (both elevated and ground level) and booster stations for the production, storage, and supply of water for the City. The distribution system consists of 12" mains spaced at ½ mile intervals with 8" sub-mains spaced at the ¼ mile. This system allows for significant looping and the ability to convey water supplies significant distance. Other supporting system features include recharge facilities (both single purpose and dual purpose flood control basins) and granular activated carbon (GAC) treatment facilities on wells containing DBCP. The addition of the GAC treatment facilities has allowed the system water quality to be maintained and meet regulatory requirements. Special treatment facilities have been installed on one well (No. 16) to remove objectionable iron and manganese.

3.2 Pipes

For single family residential areas, a minimum of one 8-inch cross should be provided inside the 12-inch main grid square. For high density residential, commercial, and industrial areas, additional 8-inch or larger crosses or loops may be needed to adequately serve the development. The interior grid pipelines need to be individually sized to ensure delivery of the required fireflow at adequate residual pressure. During peak-hour demand conditions, minimum residual pressures of 30 psi are required for all types of development. It is the intent of this Plan to provide a residual pressure of 40 psi under this design condition.

The existing grids were assumed to be upgraded with 12-inch pipe to bring them up to the 12-inch grid guideline. In some cases, increases in pipe size above the 12-inch grid are necessary. These pipes have been identified and are planned for upsizing either when the main is scheduled to be replaced, or when the need arises to increase the conveyance capacity of the system. Costs for all the mains and replacement pipelines are included herein.

3.3 Wells

From the Insurance Services Office (ISO) grading schedule, the recommended pumping capacity should be able to serve the maximum-day demand plus the basic fireflow. For reliability, ISO's standard is for maximum-day demand plus the basic fireflow to be delivered with the main pumping facility out of operation and with no flow from storage. Pumping capacity, in conjunction with outflow from storage, should also be able to supply peak-hour demand while maintaining adequate residual pressure.

Supply capacity must also be available for power outages. This can take the form of wells fitted with an auxiliary power supply, elevated tanks, or ground level tanks with booster pumps powered by an auxiliary power supply. Emergency storage in tanks should be considered separate from operational storage, so that storage is available anytime during the maximum-day demands of summer.

3.4 Distribution System Improvements

The General Plan anticipates that as of the year 2030, the planning area will be built out. Recent trends have resulted in slower growth and rates used are represented in **Table 6-2.** While Table 6-2 still shows 2030 as the buildout year, in actuality, buildout could occur a number of years later (see Section 4-1). Existing roads are generally located at ½ mile intervals in the presently undeveloped areas. Extension of the existing grid should provide adequate water delivery to most of the developing areas in the northwest and southeast villages. Several reaches of the new distribution piping emanating from the well field will be oversized to limit friction losses and resulting reduced pressures in the northeast. Correspondingly, the same will occur in the vicinity of all the storage tanks and booster stations where flows will be concentrated. The most significant change is evidenced in the northeast portion of the study area. Here, the main pipe grid will consist of 16, 18 and 20-inch diameter mains to convey larger quantities over greater distances. Dominating the distribution system will be a 36-inch conveyance pipeline that is planned to extend from the planned surface water treatment plant to the limits of the northeast village. For comparison purposes, an existing 12-inch main in the existing grid carries about 1,000 gpm at a velocity of about 3.5 fps. The planned 36-inch pipeline will convey about 17,000 gpm with a corresponding velocity of 5.25 fps.

A model of the projected system was developed by adding pipes to the configuration previously constructed and calibrated as discussed in Section 4.0. Both the peak hour and maximum day (fill cycle) were run. Results revealed that the improvements shown in **Figure 6-2** will meet the demands in the farthest portion of the service area while still maintaining 40 psi pressure. Most notable of the findings of the model runs was the inability to maintain minimum pressures in the area bounded by Fowler, Temperance, Herndon, and Shaw Avenues. This problem was alleviated by providing for a zonal connection from the SWTP to Zone 1 at Bullard and Barstow Avenues at Locan Avenue.

3.5 Supply and Storage Requirements

The best mix of wells, storage, and surface treatment was determined for ultimate conditions based upon costs. As defined previously, this scenario is planned to provide additional supply capability to meet the maximum day condition and provide storage to meet the peak hour condition. From previous work, this relates to a surface water plant of 30 MGD plus 27 additional wells with an estimated combined capacity of 44 mgd.

Discussions with DHS staff, visits to several operating and test facilities, and conversations with City Staff all indicate a preference for a Microfiltration (MF) treatment process for the surface water treatment plant. When compared to a conventional treatment process, MF offers several advantages:

- Compact size
- Modular units; capacity need not be added until needed
- Highly automated; less operator attention
- Essentially equal cost, although costs for MF continue to decrease
- A “fail safe” barrier to pathogenic agents such as Cryptosporidia Giardia.

In view of the above benefits, we recommend the new treatment facility incorporate this emerging technology.

For the study, the addition of pumping capacity assumes that production from existing wells remains constant over time. Water production from wells north of Herndon are assumed to be capable of 1,000 gpm. Wells east of Locan are assumed to have a capacity of 500 gpm. Additionally, it is assumed that half of the wells installed north of Herndon will require the installation of GAC treatment facilities. To achieve a groundwater balance, it is assumed that any new well will be accompanied with an increase in intentional groundwater recharge to balance overdraft.

3.6 Ultimate Condition (year 2030)

With the expansion of the water system, to buildout conditions, new system components are planned to be incorporated. These additional facilities are identified below:

Description	Size or Number
Surface water treatment plant	30 MGD
Additional wells	27 (24 MGD)
Additional storage tanks	
Nees/Fowler	1 MG
Shepherd/Thompson	1 MG
SWTP	1 MG
Additional intentional groundwater recharge	160 acres
Booster stations	
Letterman Park	500 gpm
Nees @ Fowler	4,000 gpm
Tollhouse/Armstrong	4,000 gpm
Ashlan/Locan	2,500 gpm
Shepherd/Thompson	8,000 gpm
Nees @ Thompson	5,500 gpm
Conveyance pipeline	36-inch diameter
Pressure reducing stations	5

For purposes of this discussion only, the main system grid is accounted for herein. The submains and local piping is assumed to be borne by developers of the individual subject properties.

PART 4 - PHASING

4.1 Factors Affecting Construction Sequence

Phasing of facilities and the resultant capital improvement plan are tied to two issues; first, actual facilities needed to supply water, and second, the timing and location at which the improvements occur. With this in mind, planning periods were chosen to be shorter in the near future and become longer during the end of the planning period. The years 2000, 2005, 2010, 2020, and 2030 have been identified as the planning horizons used in this study. Although sufficient for planning purposes, it is recognized that actual growth patterns will vary, possibly greatly, from those assumed herein. The recommendations contained herein must be adjusted to accommodate for the total growth as it actually occurs. This will have the result of changing the schedule of specific improvements, but not the sequencing or the end goal. Capital programs for specific years may vary from the planned values shown.

4.2 Criteria for Constructing New Facilities

Throughout the system expansion, the City must decide whether a particular project must be completed immediately or whether it can be delayed. This question is straightforward when it comes to the pipe network, but becomes more difficult when evaluating supply issues such as additional recharge or a surface water treatment plant. This question, as it relates to expansion of the surface water treatment facility, should be answered using the following method.

The total system capacity, including the expansion increment under consideration, should be plotted on a demand versus time graph. The expansion should be commissioned in time to provide surplus capacity for the first few years of the expansion, allowing depleted groundwater levels to recover. This methodology will also allow moderate overdraft of the groundwater during the last years of the increment cycle. **Figure 6-3** helps to illustrate this methodology. The steps shown as water supply capability relate to the addition of additional SWTP capacity or the addition of recharge facilities.

It is necessary to divide the recommended improvements into increments to construct a capital planning program for improvements. There are differing and competing issues that need to be addressed when planning for system expansion. The most significant of these is to reduce the long term overdraft. This can be accomplished by intentional recharge or through the addition of a surface water treatment plant. In this vain, a primary objective is to more fully utilize the raw water supplies currently available to the City and convert this supply to a potable use. Understanding these issues, the following goals and guidelines were developed:

- Eliminate long term overdraft through conversion and use of available raw surface water supplies.
- Finish construction of existing intentional recharge areas.
- When additional supplies are needed to maintain water budget, construct first phase of surface water treatment plant. Given dropping pumping levels at City wells, the timing of construction should be accelerated.
- Construct those improvements needed to make the first phase of surface water treatment function effectively.
- Add wells as the system expands to meet peak day requirements.
- Remove and replace bottlenecks as time and funds allow. In general, those improvements needed to transport water into the area of Fowler and Sierra are most important, so that the full value of the existing Tollhouse tank can be realized.
- Add water storage tanks as needed to meet peak hour demands.
- Install the zone boundary and related booster pump stations when the demand in the second zone approximates a peak hour flow rate in excess of 1,000 gpm.
- Distribution system grid improvements will be made in conjunction with growth.
- Conveyance improvements within existing city area to be made in conjunction with street improvements.
- Future phase of the water treatment plant construction will be added as the need arises.

4.3 Surface Water Treatment Plant Increments

As discussed in Technical Memorandum # 5, the overall recommended capacity of the surface water treatment facility is 30 mgd at the planning horizon. The first recommended increment was 10 mgd; this increment could be reduced to 5 mgd, depending on the process selected. (A membrane process, for example, is much more amenable to operation at smaller increments at than a conventional filtration plant.) Future expansions are detailed in this document as increments of 5 mgd each.

Choice of a membrane process will allow the City flexibility to expand the treatment plant in smaller increments, more closely matching the growth patterns experienced and budgets available.

4.4 Distribution and Pumping for New Village

The zone boundary has been established at the proximate western boundary of the southeast village and extending north. This being the case, pumping from the lower zone (Zone 1) to the upper zone (Zone 2) will need to be accomplished until the surface water treatment plant is constructed. Problems associated with maintaining adequate flows and pressures have recently come to light with several planned developments in the northeast. It can be assumed that the first booster station to provide service to the second zone will probably be at the existing Tollhouse tank site or at a new tank site to the northwest.

Additionally, if siting of the surface water plant is ultimately in the southeast, and to delay the need to construct the 36 inch transmission line linking the SWTP to the second zone, in the northeast it can be expected that operations in the near term will include providing water service from the surface water treatment plant to the lower zone (Zone 1) and boosting it through the booster station previously described.

4.5 Adjustments to Planned Schedule

Several adjustments will be necessary during the implementation of the recommendations herein. Adjustments could be due to a variety of reasons, as follows:

- Changes in water availability or costs.
- Changes in treatment technology, or the costs of treatment.
- Large changes in the cost of pumping energy, or in the cost of labor for operations and maintenance.
- Changes in regulations; this is especially true as it relates to groundwater disinfection monitoring requirements. The need for disinfectant contact time (CT) at each well site could dramatically change the economic operation of the existing well system.
- Regulatory changes could also greatly affect the cost of the surface water treatment facility- both capital and operational costs. The most likely change foreseen would be the implementation of strict controls for the presence of Giardia or Chriptosporidium. Either could dictate the need for differing treatment technology, ozonation, or both.
- Changes in the growth rate, or location of growth experienced in the City planning area. Affects on the City system due to outside influences, such as agricultural pumping or operations of nearby cities.

If the City should experience any of the above conditions, it will be necessary to re-evaluate impacts on the recommended improvements, and to adjust the plan as appropriate.

4.6 Phased Facilities

As discussed in Technical Memorandum #5, there are numerous specific issues related to the construction, validation, and output generated from the hydraulic model of the potable water system. In the previous memorandum, present and buildout conditions were discussed. In this current memorandum, the intermediate years will be presented and discussed. Included in **Table 6-3** is information that lists the population and associated peak hour demand for every year from present to the end of the planning horizon. The highlighted rows indicate the planning increments that are being used for this study. Additionally, certain system facilities are identified in specific years that will enable the peak capacity to be met. Since the system configuration consists of numerous wells it is expected that the controlling criteria will be the peak hour flow condition. The maximum day demand condition is less reflective of peak flow conditions and correlates more directly with supply capability. For this reason, **Table 6-4** lists the max-day flows as well as the raw water requirements to maintain a water balance. There are other permutations that can be made with the facilities shown in the table. The approach used in defining facilities assumed:

- Existing intentional recharge projects will overcome existing overdraft.
- The first phase of the Surface Water Treatment Plant is planned to be on line in 2 years due to time required to design and permit the facility. These actions should commence immediately.
- Wells will be installed to keep up with peak hour demands in the short term.
- A booster pump station to Zone 2 is expected within the next 10 years. If the industrial park at Temperance and Tollhouse Road is realized, a new booster station will be needed immediately.
- Recharge for the newly installed wells will occur after the wells have been constructed (operate in an overdraft condition).
- New storage for Zone 2 may be needed as early as the year 2008.

Tables 6-5 through **6-8** provides information that details the estimated water demands and potential supply sources for the years identified as the planning increments.

4.7 Summary

From the information discussed, a phasing plan that is graphical in nature was developed. It is included as **Figure 6-4**. It should be noted that the facilities so noted would be planned to be constructed and in operation by the year shown.

Table 6-3
Summary of Planning Horizons
& Associated Facilities

Actual Year (Yr)	Cumulative Growth (EDU)	Annual Demand (AF)	Daily Demand Pk Hr (GPM)	FACILITIES Wells (Qty)	Rchg. (Ac)	SWTP (MGD)	Booster (GPM)	Storage (M.G.)	Cumulative Supply* (GPM)	Storage (GPM)	Total Capacity (GPM)
1998	550	17,900	32,000	26				2.5	26,000	7,000	33,000
1999	1,100	18,100	32,400	1					27,000	7,000	34,000
2000	**1,800**	**18,400**	**32,900**						**27,000**	**7,000**	**34,000**
2001	2,500	18,700	33,400						27,000	7,000	34,000
2002	3,200	19,000	33,900			5			30,500	7,000	37,500
2003	3,900	19,300	34,400						30,500	7,000	37,500
2004	4,600	19,600	34,900		40				30,500	7,000	37,500
2005	**5,600**	**20,000**	**35,600**				**500**		**30,500**	**7,500**	**38,000**
2006	6,600	20,400	36,300						30,500	7,500	38,000
2007	7,600	20,500	36,600	1					31,500	7,500	39,000
2008	8,600	21,200	37,700				4,000	1	31,500	11,500	43,000
2009	9,600	21,900	38,800	1					32,500	11,500	44,000
2010	**11,000**	**22,800**	**40,300**						**32,500**	**11,500**	**44,000**
2011	12,400	23,700	41,800			5			36,000	11,500	47,500
2012	13,800	24,600	43,300						36,000	11,500	47,500
2013	15,200	25,200	44,300	1					37,000	11,500	48,500
2014	16,600	26,200	46,100	2					39,000	11,500	50,500
2015	18,600	27,600	48,600	1					40,000	11,500	51,500
2016	20,600	29,000	51,100	2			2,500		42,000	11,500	53,500
2017	22,600	29,900	52,800	1					43,000	11,500	54,500
2018	24,600	31,000	55,000	2					45,000	11,500	56,500
2019	26,600	32,100	57,200			10			52,000	11,500	63,500
2020	**29,300**	**33,700**	**60,100**		**60**				**52,000**	**11,500**	**63,500**
2021	32,000	34,900	63,000	1					53,000	11,500	64,500
2022	34,700	36,900	65,900	2			8,000	2	55,000	19,500	74,500
2023	37,400	38,900	68,800	3					58,000	19,500	77,500
2024	40,100	41,000	70,900	3	60				61,000	19,500	80,500
2025	43,600	44,300	74,900	3					64,000	19,500	83,500
2026	47,100	47,600	78,900			10	5,500		71,000	19,500	90,500
2027	50,600	47,500	82,900	1					72,000	19,500	91,500
2028	54,100	50,000	86,900	1					73,000	19,500	92,500
2029	57,600	52,500	90,900	1					74,000	19,500	93,500
2030	**57,600**	**52,500**	**93,900**						**74,000**	**19,500**	**93,500**

Note: 1 well requires about 7.5 AC recharge and which can recharge 25 AF/AC/YR
* Includes water supply from wells and SWTP

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Table 6-4
Summary of Planning Horizons
& Raw Water Supply Projections

Actual Year (Yr)	Cumulative Growth (EDU)	Annual Demand (AF)	Daily Demand Ave (GPM)	Daily Demand Max (GPM)	FACILITIES Wells (Qty)	FACILITIES Rchg. (Ac)	FACILITIES SWTP (MGD)	FACILITIES Alt. Sys (AF/Y)	Raw Wtr Req'mt (AF)
1998	550	17,900	11,600	23,400	26				10,000
1999	1,100	18,100	11,700	23,600	1				10,000
2000	**1,800**	**18,400**	**11,800**	**23,800**					**10,000**
2001	2,500	18,700	11,900	24,000					10,000
2002	3,200	19,000	12,000	24,200			5		14,500
2003	3,900	19,300	12,100	24,400					14,500
2004	4,600	19,600	12,200	24,600		40			15,500
2005	**5,600**	**20,000**	**12,400**	**24,900**				**1,000**	**16,500**
2006	6,600	20,400	12,600	25,200					16,500
2007	7,600	20,500	12,700	25,600	1				16,500
2008	8,600	21,200	13,100	26,400					16,500
2009	9,600	21,900	13,500	26,400	1				16,500
2010	**11,000**	**22,800**	**14,000**	**26,400**					**16,500**
2011	12,400	23,700	14,500	26,400			5		21,000
2012	13,800	24,600	15,000	26,400					21,000
2013	15,200	25,200	15,400	31,000	1				21,000
2014	16,600	26,200	16,000	32,200	2				21,000
2015	18,600	27,600	16,900	34,000	1				21,000
2016	20,600	29,000	17,800	35,800	2				21,000
2017	22,600	29,900	18,300	37,000	1				21,000
2018	24,600	31,000	19,100	38,500	2				21,000
2019	26,600	32,100	19,900	40,000			10		30,000
2020	**29,300**	**33,700**	**20,900**	**42,000**		**60**		**1,000**	**32,500**
2021	32,000	34,900	21,900	44,000	1				32,500
2022	34,700	36,900	22,900	46,000	2				32,500
2023	37,400	38,900	23,900	48,000	3				32,500
2024	40,100	41,000	24,600	49,700	3	60			34,000
2025	43,600	44,300	26,200	52,900	3				34,000
2026	47,100	47,600	27,800	56,100			10		43,000
2027	50,600	47,500	29,400	59,300	1				43,000
2028	54,100	50,000	31,000	62,500	1				43,000
2029	57,600	52,500	32,600	65,700	1			1,500	44,500
2030	**57,600**	**52,500**	**32,600**	**65,700**					**44,500**

Note: 1 well requires about 7.5 AC recharge and which can recharge 25 AF/AC/YR

* Current raw water availability is 15,000 AF/Yr under average conditions

TABLE 6-5
YEAR 2000 CONDITIONS
Water Use and Transfers, by Zone
(gpm, unless indicated otherwise)

ZONE 1	*Demand*	*Required Production*
Peak Hour	32,900	
Wells		26,000
Storage		6,900
Max Day	23,000	23,000
Avg Day	11,400	11,400
Winter	6,780	6,780
Storage	2.5 MG	
Hours @ Peak Flow		6.0

TRANSFERS	===>>>
Pk Hr	
Wells	
Storage	- 0 -
Max	- 0 -
Avg	- 0 -
Winter	- 0 -

ZONE 2	*Demand*	*Production*
Peak Hour	- 0 -	
Wells		- 0 -
Storage		- 0 -
Max Day	- 0 -	- 0 -
Avg Day	- 0 -	- 0 -
Winter	- 0 -	- 0 -
Storage	0 MG	
Hours @ Peak Flow		0

SWTP to Z1		**0 MGD Water Treatment Plant**	*Production*	*% Capacity*	**SWTP to Z2**
- 0 -		Peak Hr	- 0 -		- 0 -
- 0 -		Max	- 0 -		- 0 -
- 0 -		Avg	- 0 -		- 0 -
- 0 -		Winter	- 0 -		- 0 -

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TABLE 6-6
YEAR 2005 CONDITIONS
Water Use and Transfers, by Zone
(gpm, unless indicated otherwise)

ZONE 1	Demand	Required Production
Peak Hour	35,600	
Wells		25,100
Storage		8,000
Max Day	24,900	22,100
Avg Day	12,360	9,700
Winter	7,350	5,770
Storage	2.5 MG	
Hours @ Peak Flow		5.2

TRANSFERS ===>>>	
Pk Hr	
Wells	
Storage	1,000
Max	700
Avg	340
Winter	170

ZONE 2	Demand	Production
Peak Hour	1,000	
Wells		- 0 -
Storage		- 0 -
Max Day	700	- 0 -
Avg Day	340	- 0 -
Winter	170	- 0 -
Storage	0 MG	
Hours @ Peak Flow		0

SWTP to Z1		5 MGD Water Treatment Plant	Production	% Capacity		SWTP to Z2
3,500		Peak Hr	3,500	100%		- 0 -
3,500		Max	3,500	100%		- 0 -
3,000		Avg	3,000	90%		- 0 -
1,750		Winter	1,750	50%		- 0 -

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TABLE 6-7
YEAR 2010 CONDITIONS
Water Use and Transfers, by Zone
(gpm, unless indicated otherwise)

ZONE 1	Demand	Required Production
Peak Hour	35,300	
Wells		24,800
Storage		8,000
Max Day	28,000	27,500
Avg Day	13,000	12,400
Winter	7,000	6,066
Storage	2.5 MG	
Hours @ Peak Flow		6.4

TRANSFERS	===>>>
Pk Hr	
Wells	- 0 -
Storage	- 0 -
Max	- 0 -
Avg	- 0 -
Winter	- 0 -

ZONE 2	Demand	Production
Peak Hour	5,000	
Wells		- 0 -
Storage		4,000
Max Day	3,000	- 0 -
Avg Day	2,400	- 0 -
Winter	816	- 0 -
Storage	1 MG	
Hours @ Peak Flow		4.1

SWTP to Z1		5 MGD Water Treatment Plant	Production	% Capacity		SWTP to Z2
2,500		Peak Hr	3,500	100%		1,000
3,940		Max	3,500	100%		3,000
3,846		Avg	3,000	90%		2,400
2,654		Winter	1,750	50%		816

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TABLE 6-8
YEAR 2020 CONDITIONS
Water Use and Transfers, by Zone
(gpm, unless indicated otherwise)

ZONE 1	*Demand*	*Required Production*
Peak Hour	47,000	
Wells		34,220
Storage		8,000
Max Day	33,000	28,120
Avg Day	16,683	10,573
Winter	9,693	2,571
Storage	0.5 MG	
Hours @ Peak Flow		20

TRANSFERS	===>>>
Pk Hr	
Wells	- 0 -
Storage	- 0 -
Max	- 0 -
Avg	- 0 -
Winter	- 0 -

ZONE 2	*Demand*	*Production*
Peak Hour	13,100	
Wells		
Storage		4,000
Max Day	9,000	- 0 -
Avg Day	4,300	- 0 -
Winter	1,900	- 0 -
Storage	3.0 MG	
Hours @ Peak Flow		6.6

SWTP to Z1		20 MGD Water Treatment Plant	*Production*	*% Capacity*		SWTP to Z2
4,780		Peak Hr	13,880	100%		9,100
4,880		Max	13,880	100%		9,000
6,110		Avg	10,410	75%		4,300
7,122		Winter	9,022	65%		1,900

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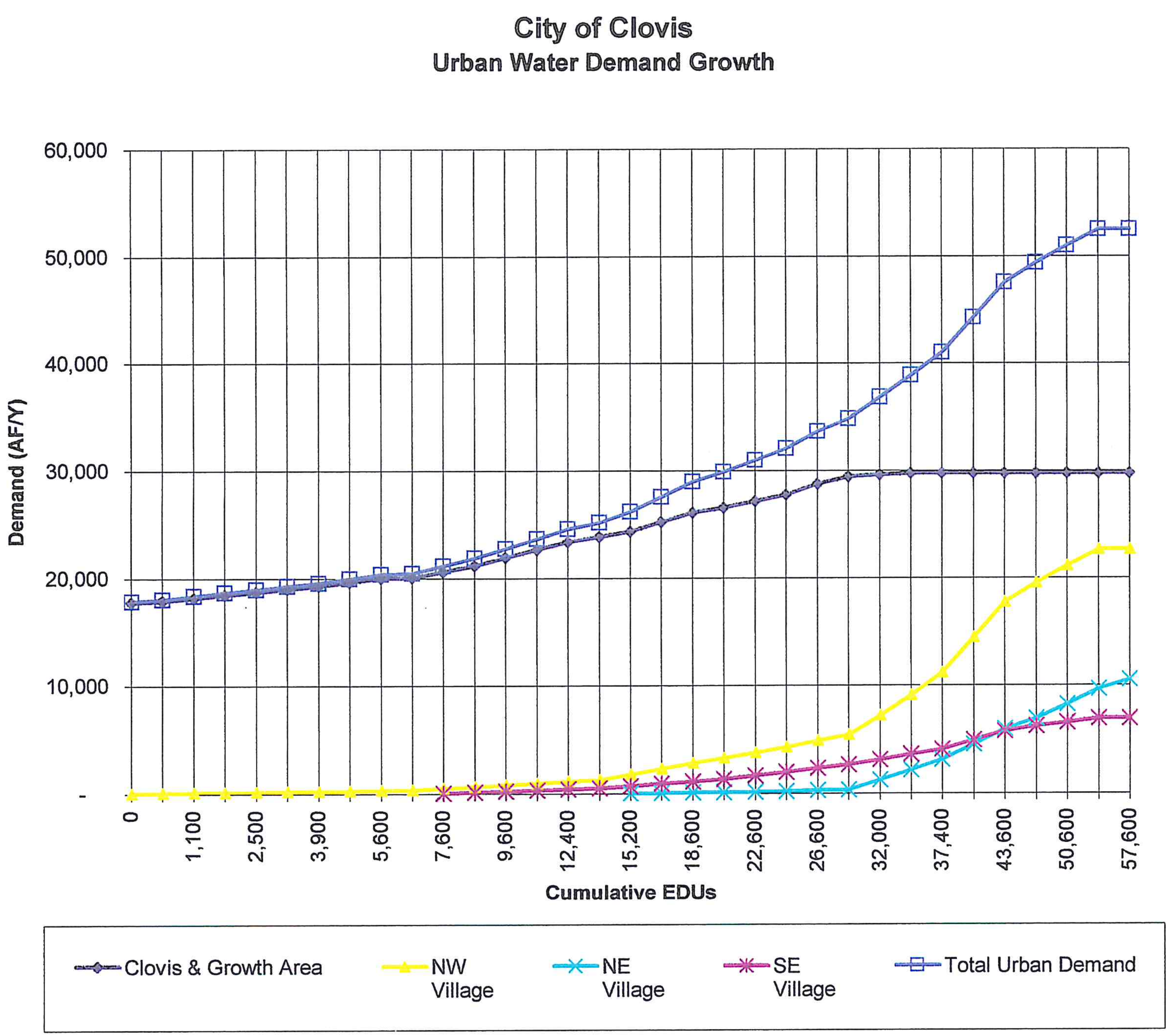
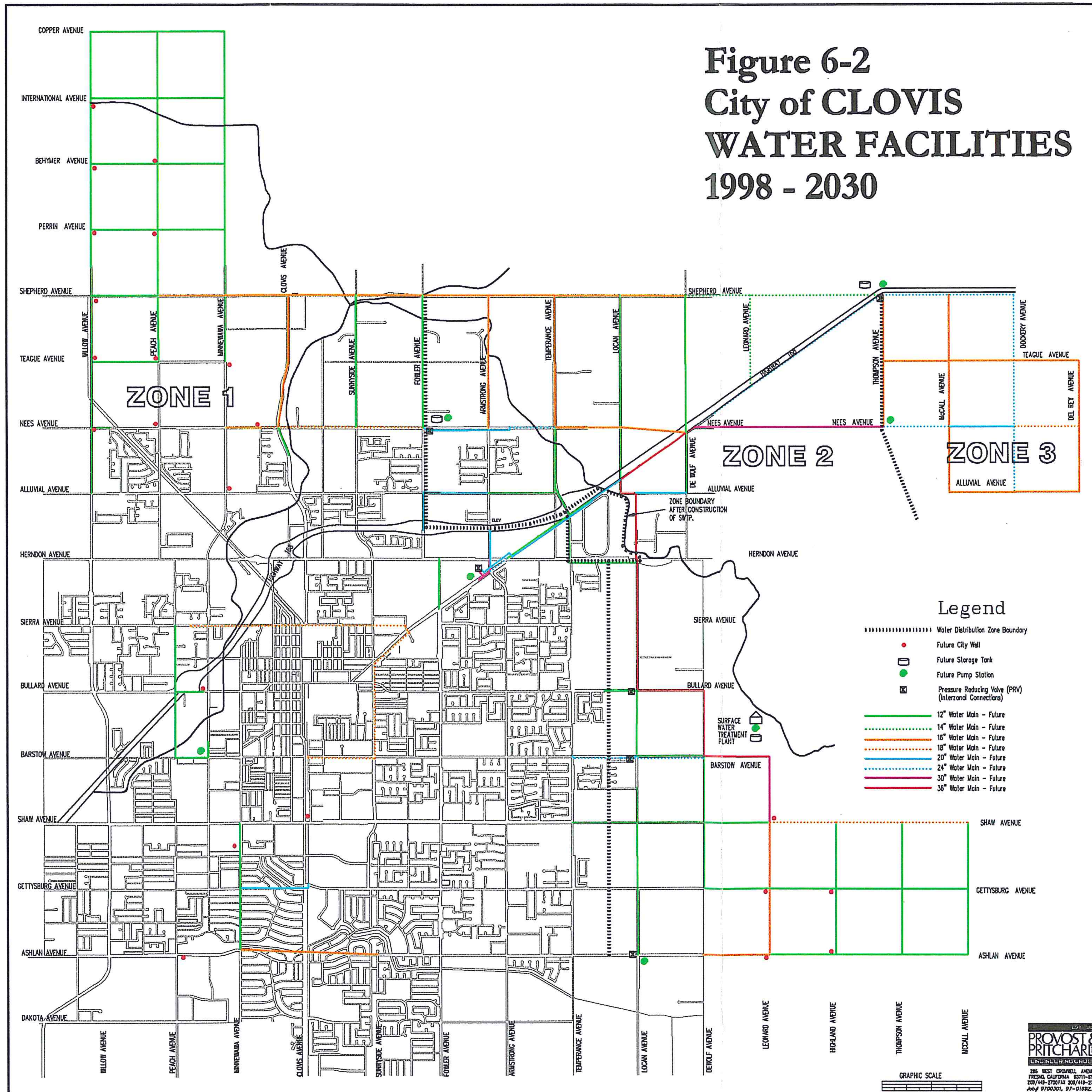


Figure 6-1. Annual Water Demand Projections

Figure 6-2
City of CLOVIS
WATER FACILITIES
1998 - 2030



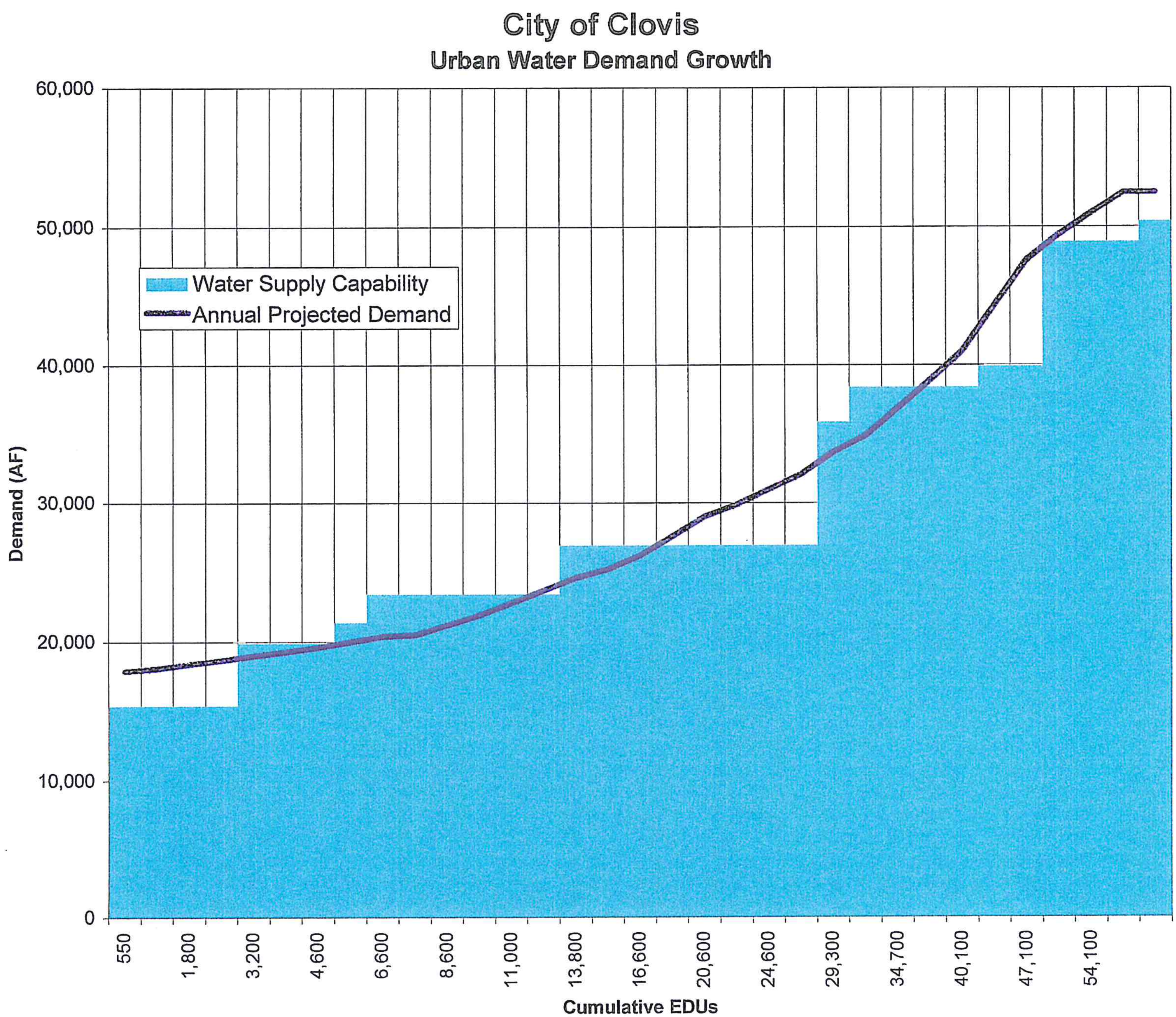
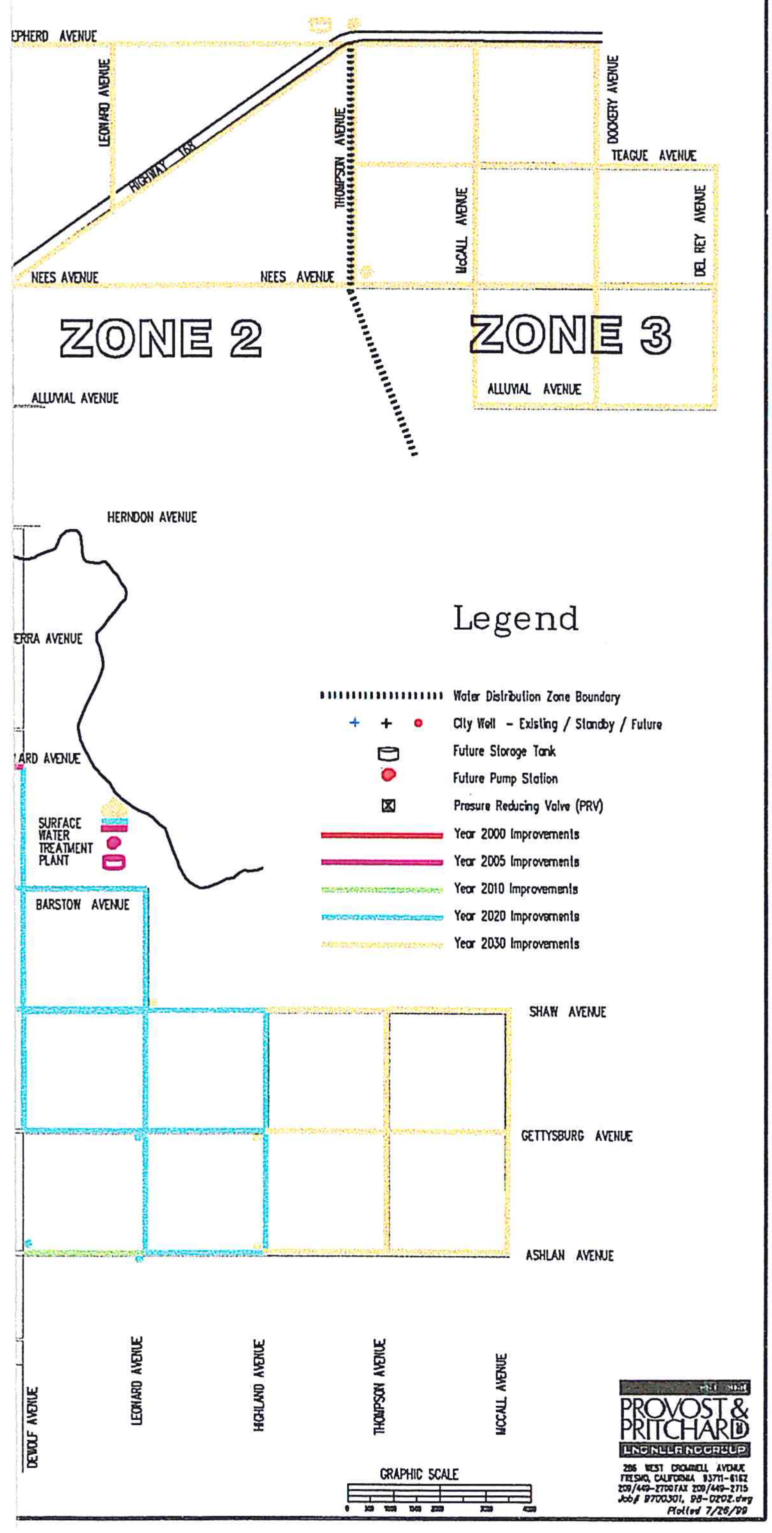


Figure 6-3. Water Supply Capabilities

6-4
CLOVIS
TIES PHASING PLAN
030



Technical Memorandum 7
Capital Improvement Plan

PART 1 - COSTS

1.1 Capital Cost Estimate

Order-of-magnitude unit cost estimates were developed for pipelines, storage reservoirs, wells, booster stations, and the surface water treatment plant for June 1998 conditions. A 25 percent contingency and 15 percent engineering and administration factor were applied to unit costs.

The cost estimates presented in this study and included in **Table 7-1** are developed from cost curves, vendors, information obtained from previous studies, and recent experience on other projects. The costs should be considered order-of-magnitude and have an expected accuracy range of +20 percent to -10 percent as defined by the American Association of Cost Engineers.

The cost estimates have been prepared for guidance in project evaluation and implementation from the information available at the time of the estimate. As constructed, final costs of the project will depend on actual labor and material costs, competitive market conditions, specific details of recommended modifications, final project scope, implementation schedule, and other variable factors. As a result, the final capital and operating project costs will vary from the estimates presented. Therefore, project feasibility and funding needs must be reviewed carefully prior to specific financial decisions to help ensure proper project evaluation and adequate funding.

1.2 Water Pipe Costs

Costs shown include valves, fittings, moderate utility interference, street resurfacing, and Class 150 pressure pipe installed complete. There are two columns that represent both growth and infill projects. The higher costs shown for replacement of pipelines within the existing system result in greater amounts of work related to developed conditions. This could take the form of utilities, construction site conditions, traffic control and construction staging activities.

1.3 Water Well Cost

Costs for water wells are based on cost data provided by the City of Clovis Engineering Department for recent well construction projects and on cost curves developed by Provost and Pritchard. Well costs assume a 600-foot deep well capable of delivering either 500 or 1,000 gpm. The basic well unit cost is approximately $354,000 and includes

Table 7-1
Construction Cost Summary Sheet

Conveyance & Transmission System

Water Pipes	Size inches	Growth $/ft	Infill $/ft
	12	52	63
	14	63	75
	16	74	86
	18	83	96
	20	93	107
	24	105	n/a
	30	120	n/a
	36	140	n/a

Supply Facilities

	Capacity		Cost	Unit Cost	Annual Water Supply AF
Water Well	500	gpm	$ 354,000		350
Recharge	40	AC	$ 3,000,000	(75,000 $/AC)	1,000
SWTP					
Initial	5	MGD	$ 10,000,000		4,500
Incremental	5	MGD	$ 4,000,000		
Dual System	20	AC	$ 40,000	(2000 $/AC)	60

Other Costs

	Capacity		Cost	Unit Cost
Reservoir	2	MG	$ 2,000,000	
Booster Station	4,000	gpm	$ 360,000	90 $/gpm
Pressure Red. Station			$ 100,000	
For Wells				
Standby Power			$ 50,000	
Wellhead Treatment			$ 750,000	
Telemetry			--	

test wells, drilling, casing, gravel packing, well development, pump, motor, gearhead, electrical equipment, chlorination equipment, site acquisition, system connection, and power connection. Auxiliary features such as standby power and wellhead treatment are shown under support features. For budgeting purposes, it is assumed that half of the future wells in the City of Clovis and the northwest village will require wellhead treatment and all will be fitted with standby power.

1.4 Reservoir Cost

Reservoir costs are estimated to be $1 per gallon based on similar projects and previous estimates for the City of Clovis, costs for a 2 MG facility are estimated at $2,000,000 and includes sitework, pump manifold, reservoir, and land acquisition.

1.5 Recharge Basin

Based upon recent purchases and a study completed for the City of Clovis by Provost & Pritchard dated March 1997, initial purchase and development costs are estimated to approximately $75,000/ac.

1.6 Surface Water Treatment Plant

Reference is made to Technical Memorandum No. 5; in the Appendix, costs were developed for various alternative treatment methodologies. Costs included here are for the membrane process and amount to the $10 million (estimate $7 million for 5 MGD) for the initial phase. Remaining phases are projected to approximate $5 million for each 5 MGD expansion.

1.7 Booster Pump Station

Costs developed based on past cost histories and in conjunction with costs developed for the SWTP pump station. A value of $90 gpm is utilized for the 4,000 gpm booster plants at Nees & Tollhouse. Relative factors have been applied for the other 2,500, 5,500, and 8,000 gpm pump stations.

1.8 Cost of Facilities by Increment

Table 7-2 presents a consolidated list of the recommended water supply improvements, sorted by approximate increments that the improvement will be needed. A summary indicates the magnitude of expenditures needed for each such increment. These totals, especially for the first five years, should form the basis of the capital planning and budgeting process for the City. **Table 7-3** lists the consolidated conveyance and transmission facilities by year. It should be noted that other improvements to the existing system, such as telemetry, internal system piping, and replacement items, are not included. The costs of these on-going programs should be considered additive to the costs indicated.

Table 7-2
Capital Improvement Plan
Supply Facilities Estimated Construction Cost

Type	Area	Phase Period	EW Street	NS Street	Reason	Unit	$/Unit	Total
Well	City	2000	Bullard	Villa	Growth	1		350,000
		2000 Total						**350,000**
PumpStn	NE	2005	Tollhouse	Armstrong	Freeway	4000	90	360,000
Recharge	NW	2005	Clovis	Alluvial	Groundwater Overdraft	40	75,000	3,000,000
WTF	NE	2005	Initial		Groundwater Overdraft	5		10,000,000
		2005 Total						**13,360,000**
PumpStn	NE	2010	Nees	Fowler	Growth	4500	90	405,000
PumpStn	City	2010	Barstow	Villa	Growth	500	90	45,000
AltSys	Other	2010	Reagan Complex		Growth	500	1,500	750,000
Storage	NE	2010	Nees	Fowler	Growth	1	1,000,000	1,000,000
PRV	NE	2010	Nees	Fowler	Growth	1	25,000	25,000
Well	NW	2010	Nees	Marion	Growth	1		350,000
Well	City	2010	Shaw	Clovis	Growth	1		350,000
		2010 Total						**2,925,000**
AltSys	Other	2020	Buchanan Complex		Growth	500	1,500	750,000
AltSys	Other	2020	Rural Residential		Growth	1000	1,500	1,500,000
PumpStn	SE	2020	Ashlan	Locan	Growth	2500	90	225,000
Recharge	City	2020	Nees	Clovis	Growth	60	75,000	4,500,000
PRV	SE	2020	Bullard	Locan	Growth	1	25,000	25,000
PRV	SE	2020	Barstow	Locan	Growth	1	25,000	25,000
PRV	SE	2020	Ashlan	Locan	Growth	1	25,000	25,000
Well	SE	2020	Ashlan	Dewolf	Growth	1		350,000
Well	SE	2020	Ashlan	Leonard	Growth	1		350,000
Well	City	2020	Ashlan	Peach	Growth	1		350,000
Well	SE	2020	Gettysburg	Leonard	Growth	1		350,000
Well	Other	2020	Herndon	Clovis	Growth	1		350,000
Well	NW	2020	Nees	Willow	Growth	1		350,000
Well	City	2020	Shaw	Minnewawa	Growth	1		350,000
Well	Other	2020	Sierra	Willow	Growth	1		350,000
Well	NW	2020	Minnewawa	Alluvial	Growth	1		1,100,000
WTF	NE	2020	Expansion		Growth	15		12,000,000
		2020 Total						**22,950,000**
AltSys	Other	2030	Rural Residential		Growth	1500	1,500	2,250,000
PumpStn	NE	2030	Nees	Thompson	Growth	5500	90	495,000
PumpStn	NE	2030	Shepherd	Thompson	Growth	8000	90	720,000
Recharge	City	2030	Sierra	Clovis	Growth	60	75,000	4,500,000
Storage	NE	2030	Shepherd	Thompson	Growth	2		2,000,000
Well	SE	2030	Ashlan	Highland	Growth	1		350,000
Well	NW	2030	Behymer	Peach	Growth	1		350,000
Well	NW	2030	Behymer	Willow	Growth	1		350,000
Well	SE	2030	Gettysburg	Highland	Growth	1		350,000
Well	NW	2030	International	Willow	Growth	1		350,000
Well	SE	2030	Shaw	Leonard	Growth	1		350,000
Well	NW	2030	Shepherd	Peach	Growth	1		350,000
Well	NW	2030	Willow	Perrin	Growth	1		350,000
Well	NW	2030	Willow	Teague	Growth	1		350,000
Well	NW	2030	Nees	Peach	Growth	1		1,100,000
Well	NW	2030	Perrin	Peach	Growth	1		1,100,000
Well	NW	2030	Shepherd	Willow	Growth	1		1,100,000
Well	NW	2030	Teague	Clovis	Growth	1		1,100,000
Well	NW	2030	Teague	Minnewawa	Growth	1		1,100,000
Well	NW	2030	Teague	Peach	Growth	1		1,100,000
WTF	NE	2030	Expansion		Growth	10		8,000,000
		2030 Total						**27,715,000**
		Grand Total						**67,300,000**

Table 7-3
Distribution System Capital Improvement Plan
Conveyance & Distribution Facilities Construction Cost

Type	Area	Year	Along	From	To	Reason	Exist Dia.	New Dia.	Block	Length	Unit Cost	Total
Pipeline	Infills	2000	Fowler	Herndon	Sierra	Growth		12	0.8	2,112	63	133,056
Pipeline	Infills	2000	Gettysburg	Minnewawa	Clovis	Growth		12	1	2,640	63	166,320
Pipeline	NE	2000	Armstrong	Shepherd	Nees	Growth		16	2	5,280	74	390,720
Pipeline	NE	2000	Shepherd	Armstrong	Temperance	Growth		16	1	2,640	74	195,360
Pipeline	NE	2000	Temperance	Shepherd	Nees	Growth		16	2	5,280	74	390,720
Pipeline	Bottlenecks	2000	Barstow	Clovis	Sunnyside	Age	10	18	1	2,640	83	219,120
		2000 Total										**1,495,296**
Pipeline	Bottlenecks	2005	Barstow	Peach	Minnewawa	Growth	10	12	1.25	3,300	52	171,600
Pipeline	Bottlenecks	2005	Bullard	Peach	Villa	Growth	10	12	0.5	1,320	52	68,640
Pipeline	Infills	2005	Bullard	Temprance	Locan	TP Siting		12	0.5	1,320	63	83,160
Pipeline	SE	2005	Locan	Shaw	Ashlan	Growth		12	2	5,280	52	274,560
Pipeline	NE	2005	Minnewawa	Barstow	Ninth	Tank	12	12	0.5	1,320	52	68,640
Pipeline	SE	2005	Shaw	Temperance	Locan	Growth		12	1	2,640	52	137,280
Pipeline	NW	2005	Willow	Alluvial	Teague	Growth		12	2	5,280	52	274,560
Pipeline	SE	2005	Locan	Bullard	Barstow	TP Siting		16	1	2,640	74	195,360
Pipeline	NE	2005	Shepherd	Temperance	Locan	Growth		16	1	2,640	74	195,360
Pipeline	NW	2005	Nees	Clovis	Fowler	Growth		18	2	5,280	83	438,240
Pipeline	SE	2005	Barstow	Temperance	Locan	Growth		24	1	2,640	124	327,360
Pipeline	Trans	2005	Bullard	Locan	Dewolf	Growth		36	1	2,640	136	359,040
		2005 Total										**2,593,800**
Pipeline	NE	2010	Alluvial	Armstrong	Temperance	Growth		12	1	2,640	52	137,280
Pipeline	NW	2010	Minnewawa	Shepherd	Perrin	Growth		12	1	2,640	52	137,280
Pipeline	NW	2010	Minnewawa	Teague	Shepherd	Growth		12	1	2,640	52	137,280
Pipeline	NW	2010	Peach	Teague	Perrin	Growth		12	2	5,280	52	274,560
Pipeline	NW	2010	Perrin	Willow	Minnewawa	Growth		12	2	5,280	52	274,560
Pipeline	NW	2010	Shepherd	Willow	Peach	Growth		12	1	2,640	52	137,280
Pipeline	NW	2010	Teague	Willow	Peach	Growth		12	1	2,640	52	137,280
Pipeline	NE	2010	Temperance	Herndon	Hwy 168	Growth		12	0.8	2,112	52	109,824
Pipeline	NE	2010	Temperance	Hwy 168	Alluvial	Growth		12	0.5	1,320	52	68,640
Pipeline	NW	2010	Willow	Shepherd	Perrin	Growth		12	1	2,640	52	137,280
Pipeline	SE	2010	Ashlan	Dewolf	Leonard	Growth		16	1	2,640	74	195,360
Pipeline	NW	2010	Clovis	Nees	Shepherd	Growth		16	2	5,280	74	390,720
Pipeline	NW	2010	Nees	Minnewawa	Clovis	Pump Station		16	0.5	1,320	74	97,680
Pipeline	NE	2010	Nees	Temperance	Dewolf	Growth		16	2	5,280	74	390,720
Pipeline	NW	2010	Shepherd	Peach	Clovis	Growth		16	2	5,280	74	390,720
Pipeline	Bottlenecks	2010	Sunnyside	Barstow	Bullard	Growth		18	1	2,640	83	219,120
Pipeline	Bottlenecks	2010	Sunnyside	Bullard	Hwy 168 & Sierra	Growth		18	1	2,640	83	219,120
Pipeline	NE	2010	Nees	Fowler	Temperance	Growth		24	2	5,280	124	654,720
		2010 Total										**4,109,424**
Pipeline	NE	2020	Alluvial	Locan	Dewolf	Growth		12	0.2	528	52	27,456
Pipeline	SE	2020	Ashlan	Leonard	Highland	Growth		12	1	2,640	52	137,280
Pipeline	NW	2020	Behymer	Willow	Minnewawa	Growth		12	2	5,280	52	274,560
Pipeline	SE	2020	Dewolf	Barstow	Gettysburg	Growth		12	2	5,280	52	274,560
Pipeline	NW	2020	Fowler	Nees	Shepherd	Growth		12	2	5,280	63	332,640
Pipeline	SE	2020	Gettysburg	Dewolf	Highland	Growth		12	2	5,280	52	274,560
Pipeline	Infills	2020	Herndon	Temprance	Locan	Growth		12	1	2,640	63	166,320
Pipeline	SE	2020	Highland	Ashlan	Shaw	Growth		12	2	5,280	52	274,560
Pipeline	NE	2020	Locan	Shepherd	Alluvial	Growth		12	3	7,920	52	411,840
Pipeline	NW	2020	Minnewawa	Perrin	Behymer	Growth		12	1	2,640	52	137,280
Pipeline	Bottlenecks	2020	Minnewawa	Shaw	Gettysburg	Growth	8	12	1	2,640	52	137,280
Pipeline	NW	2020	Peach	Perrin	International	Growth		12	2	5,280	52	274,560
Pipeline	SE	2020	Shaw	Locan	Highland	Growth		12	3	7,920	52	411,840
Pipeline	Bottlenecks	2020	Sierra	Dewitt	Clovis	Freeway	8	12	0.5	1,320	52	68,640
Pipeline	NW	2020	Sunnyside	Nees	Shepherd	Growth		12	2	5,280	63	332,640
Pipeline	Infills	2020	Temprance	Nees	Alluvial	G&F&WTP		12	1	2,640	63	166,320
Pipeline	NW	2020	Willow	Perrin	International	Growth		12	2	5,280	52	274,560
Pipeline	Infills	2020	Ashlan	Minnewawa	Sunnyside	Growth		14	2	5,280	75	396,000
Pipeline	SE	2020	Leonard	Shaw	Ashlan	Growth		16	2	5,280	74	390,720
Pipeline	NW	2020	Shepherd	Clovis	Armstrong	Growth		16	3	7,920	86	681,120
Pipeline	Bottlenecks	2020	Sierra	Clovis	Sunnyside	Growth	12	18	1	2,640	83	219,120
Pipeline	Bottlenecks	2020	Sierra	Minnewawa	Dewitt	Growth	8	18	1.25	3,300	83	273,900
Pipeline	SE	2020	Barstow	Locan	Dewolf	Growth		24	1	2,640	124	327,360
Pipeline	SE	2020	Leonard	Barstow	Shaw	TP Siting		24	1	2,640	124	327,360
Pipeline	Infills	2020	Tollhouse	Herndon	Locan	Growth		24	2.5	6,600	124	818,400
Pipeline	Trans	2020	Barstow	Dewolf	Leonard	TP Siting		36	1	2,640	136	359,040
Pipeline	Trans	2020	Dewolf	Bullard	Barstow	Growth		36	1	2,640	136	359,040
Pipeline	Trans	2020	Locan	Alluvial	Bullard	Growth		36	3	7,920	136	1,077,120
		2020 Total										**9,206,076**
Pipeline	NE	2030	Alluvial	Locan	Dewolf	Growth		12	0.8	2,112	52	109,824
Pipeline	SE	2030	Ashlan	Highland	McCall	Growth		12	2	5,280	52	274,560
Pipeline	NW	2030	Copper	Willow	Minnewawa	Growth		12	2	5,280	52	274,560
Pipeline	NE	2030	Dewolf	Alluvial	Nees	Growth		12	0.9	2,376	52	123,552
Pipeline	NE	2030	Dewolf	Shepherd	Alluvial	Growth		12	3	7,920	52	411,840

Table 7-3
Distribution System Capital Improvement Plan
Conveyance & Distribution Facilities Construction Cost

Type	*Area*	*Year*	*Along*	*From*	*To*	*Reason*	*Exist Dia.*	*New Dia.*	*Block*	*Length*	**Unit Cost**	**Total**
Pipeline	NE	2030	Dewolf	Shepherd	Hwy 168	Growth		12	1.3	3,432	52	178,464
Pipeline	SE	2030	Gettysburg	Highland	McCall	Growth		12	2	5,280	52	274,560
Pipeline	NW	2030	International	Willow	Minnewawa	Growth		12	2	5,280	52	274,560
Pipeline	SE	2030	McCall	Ashlan	Shaw	Growth		12	2	5,280	52	274,560
Pipeline	NW	2030	Minnewawa	Behymer	Copper	Growth		12	2	5,280	52	274,560
Pipeline	Bottlenecks	2030	Minnewawa	Gettysburg	Ashlan	Growth	10	12	1	2,640	52	137,280
Pipeline	NW	2030	Peach	International	Copper	Growth		12	1	2,640	52	137,280
Pipeline	Bottlenecks	2030	Peach	Sierra	Barstow	Growth		12	2	5,280	52	274,560
Pipeline	SE	2030	Shaw	McCall	Highland	Growth		12	2	5,280	52	274,560
Pipeline	SE	2030	Thompson	Ashlan	Shaw	Growth		12	2	5,280	52	274,560
Pipeline	NW	2030	Willow	International	Copper	Growth		12	1	2,640	52	137,280
Pipeline	NE	2030	Shepherd	Leonard	Thompson	Growth		14	2	5,280	63	332,640
Pipeline	NE	2030	Alluvial	McCall	Del Rey	Growth		16	2	5,280	74	390,720
Pipeline	NE	2030	Del Rey	Teague	Alluvial	Growth		16	2	5,280	74	390,720
Pipeline	NE	2030	Leonard	Shepherd	Hwy 168	Growth		16	1.3	3,432	74	253,968
Pipeline	NE	2030	McCall	Shepherd	Alluvial	Growth		16	3	7,920	74	586,080
Pipeline	NE	2030	Shepherd	Dewolf	Leonard	Growth		16	1	2,640	74	195,360
Pipeline	NE	2030	Shepherd	Locan	Dewolf	Growth		16	1	2,640	74	195,360
Pipeline	NE	2030	Teague	Thompson	Del Rey	Growth		16	3	7,920	74	586,080
Pipeline	NE	2030	Thompson	Nees	Shepherd	Growth		16	2	5,280	74	390,720
Pipeline	NE	2030	Hwy 168	Dewolf	Leonard	Growth		18	1.25	3,300	83	273,900
Pipeline	NE	2030	Hwy 168	Leonard	Thompson	Growth		18	2	5,280	83	438,240
Pipeline	NE	2030	Nees	Del Rey	Dockery	Growth		18	1	2,640	83	219,120
Pipeline	NE	2030	Nees	McCall	Dockery	Growth		20	1	2,640	93	245,520
Pipeline	NE	2030	Dockery	Shepherd	Alluvial	Growth		24	3	7,920	124	982,080
Pipeline	NE	2030	Nees	Thompson	McCall	Growth		24	1	2,640	124	327,360
Pipeline	NE	2030	Shepherd	Thompson	Dockery	Growth		24	2	5,280	124	654,720
Pipeline	NE	2030	Nees	Dewolf	Thompson	Growth		30	2.5	6,600	124	818,400
Pipeline	Trans	2030	Hwy 168	Locan	Nees	Growth		36	1.75	4,620	136	628,320
		2030 Total										**11,615,868**
		Grand Total										**29,020,464**

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PART 2 - RECOMMENDED CAPITAL IMPROVEMENT PROGRAM

Based on information presented in preceding chapters of this report, the capital improvements described below are recommended. This plan will enable the City of Clovis to provide adequate water service for the projected population growth in accordance with the General Plan and to correct existing deficiencies. The Capital Improvement Program for major facilities is shown in **Table 7-4**.

The costs in this report are based on June 1998 price levels. Costs of improvements installed after 1998 should be escalated to account for anticipated inflation.

2.1 Water Supply

As detailed in Table 6-3 it is important that surface water supplies be added to the options available to the City. Even though this process has commenced, such as Letterman Park, and Marion recharge project, on-going projects will still take several years to accomplish. In the intervening time, wells will need to be added to keep pace with demand. Immediate activities consisting of pilot scale testing for a surface water treatment plant should commence to determine if the membrance process is suitable for Enterprise Canal water. Once the surface plant is operational, more options will be available to manage the supply. Since wells will be added, it is planned that the City will continue to operate at a groundwater deficit related to the overall water budget. It is therefore recommended that the City drill additional wells to provide sufficient supply to meet projected needs until 2002.

This improvement plan does not include costs for replacing existing wells. A capital replacement or reserve fund for the replacement of wells that reach the end of their useful lives should be provided in the annual budget.

A booster pump station located at the existing water storage facility at Tollhouse/Armstrong is recommended immediately to accommodate growth in the northeast portions of the City. The estimated construction cost for this facility is $360,000. With contingencies and administrative costs included it is suggested that $500,000 be budgeted.

2.2 Telemetry System

The City's telemetry system central processing unit (CPU) and software package should be upgraded to handle the increased number of pump stations in the distribution system. The upgraded system would also have the capacity to monitor hourly conditions so that more reliable estimates of design requirements could be made as expansion progresses. It is recommended that this upgrade be coincident with the construction of the surface plant. By coordinating these activities, it will ensure that the operating controls of the most significant supply source be thoroughly integrated with the remainder of the system.

Table 7-4
Capital Improvement Program
Estimated Major Facilities Cost

Growth of 1,800 EDU (est. Year 2000)

One (1) well with auxiliary power to serve growth	350,000	
Contingency (25%)	87,500	
Subtotal		437,500
Administration and Engineering (15%)	52,500	
Total		490,000

Growth of 5,600 EDU (est. Year 2005)

Pumpstation at Toolhouse & Armstrong	360,000	
First (1st) phase of SWTP	10,000,000	
36" distribution system	359,040	
40 acres recharge capability	3,000,000	
Subtotal		13,719,040
Contingency (25%)	3,429,760	
Subtotal		17,148,800
Administration and Engineering (15%)	2,051,200	
Total		19,200,000

Growth of 11,000 EDU (est. Year 2010)

Two (2) wells with auxiliary power to serve growth	700,000	
PRV at Nees & Fowler	25,000	
Pumpstation at Barstow & Minewawa	45,000	
500 AF alternative supply system	750,000	
Pumpstation & Storage at Nees & Fowler	1,405,000	
Subtotal		2,925,000
Contingency (25%)	731,250	
Subtotal		3,656,250
Administration and Engineering (15%)	433,750	
Total		4,090,000

Table 7-4
Capital Improvement Program
Estimated Major Facilities Cost

Growth of 29,300 EDU (est. Year 2020)

1500 AF alternative supply system		2,250,000	
Second thru fourth phase of SWTP (15 MGD)		12,000,000	
Nine (9) wells with auxiliary power to serve growth		3,900,000	
Three (3) PRV on Bullard/Barstow/Ashlan & Locan		75,000	
Pumpstation at Ashlan & Locan		225,000	
36" distribution system		1,795,200	
60 acres recharge capability		4,500,000	
	Subtotal		24,745,200
Contingency (25%)		6,186,300	
	Subtotal		30,931,500
Administration and Engineering (15%)		3,708,500	
Total			34,640,000

Growth of 57,600 EDU (est. Year 2030)

1500 AF alternative supply system		2,250,000	
Final phases of SWTP (10 MGD)		8,000,000	
Fifteen (15) wells with auxiliary power to serve growth		9,750,000	
Pumpstation & Storage at Shepherd & Thompson		2,720,000	
Pumpstation at Nees & Thompson		495,000	
36" distribution system		628,320	
60 acres recharge capability		4,500,000	
	Subtotal		28,343,320
Contingency (25%)		7,085,830	
	Subtotal		35,429,150
Administration and Engineering (15%)		4,250,850	
Total			39,680,000

Appendix

City of Clovis

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Appendix A
Supplemental Information
Technical Memorandum No. 2
Current and Future Water Supply

GROUNDWATER MONITORING PROGRAM

Existing Monitoring

The City presently periodically measures water levels in existing wells and also periodically runs PG&E pump tests on those wells. In addition, the City conducts comprehensive water quality analysis on water from the actively used wells pursuant to CDHS requirements. The City also measures depth to water in a number of observation wells in the vicinity of the new Marion Recharge facility. Pumpage for each well and intentional recharge in both dedicated facilities and storm runoff basins are also measured. In the non-urbanized areas, water levels are monitored in some wells semi-annually by DWR. In general, up-to-date information on groundwater quality is lacking.

Data Gaps

In terms of future groundwater management needs, there are several data gaps in the present monitoring program. In the urbanized area, these involve primarily water levels and quality of the shallow groundwater. In the rural areas, these primarily involve groundwater quality. Knowledge of groundwater quality in the areas not yet urbanized can provide extremely important information as the City expands into these areas.

Other Limitations

There are several other limitations inherent with the City's current monitoring efforts. First, unless the wells are shut down for some period of time, the pumping water levels measured are influenced by the recovery rate of the well; hence the readings are of limited value. Second, the water produced by many of the production wells is drawn from deeper intervals and provides limited information where the contaminants are concentrated in the shallow groundwater. This is of particular importance with the two most serious contaminants, (DBCP and Nitrate) which both originate at the surface and move into the groundwater.

Recommended Monitoring

In order to better monitor the condition of the groundwater, it is recommended that the City expand the present network of monitoring wells, and implement an improved program of monitoring static levels, pumping water levels, and groundwater quality. A data management system is also recommended. Although some sampling is already required by the state water system operating permit, additional sampling would be useful for tracking the location and level of contaminants, for measuring the success of recharge and for optimization of pump use.

Monitoring Network

A more comprehensive network of monitoring wells should be established. Besides City wells, which generally are from several hundred to about 600 feet deep, valuable information can also be obtained from private domestic wells, after urbanization occurs. Many of the shallow wells have a driller's log available and are equipped with submersible pumps. The tops of selected wells could be vaulted if necessary and converted to allow pumping with a portable generator if the electricity has to be disconnected. Presently, these wells are destroyed according to the County of Fresno Water Well ordinance. Preservation of some of these wells could save considerable expense in drilling new shallow monitoring wells. Monitoring of shallow wells is highly useful because these wells often tap relatively shallow groundwater (i.e. within about 50 feet of the water table). Measuring water levels in them can provide information on the shallow groundwater most affected by surface sources or recharge. Monitoring shallow water quality can provide a fore warning of problems that could affect deeper City wells in the future, if unmitigated. Prior to new subdivision approval, the developers should be required to provide a detailed map identifying these wells. The City would then review this information and work with the developer to convert selected wells to monitoring wells.

In older areas, monitoring wells can be installed within any public right-of-way. Consideration should also be made for preservation of the wells within the FID and DWR water level monitoring networks. These wells have been monitored for long periods of time and provide significant historical data.

Once the well network has been established, regular readings should be taken at least twice a year, with water quality sampling performed in areas where DBCP or nitrate has been identified. In this manner, a cost-effective shallow groundwater monitoring program could be developed.

Water Levels

It is recommended that static water levels be measured semi-annually in all existing (including inactive) wells. The spring measurements would generally be in January or February and would be done at a time to reflect the seasonal shallowest water levels.

The fall measurements would generally be done in early September, and would be done to reflect the seasonal deepest water levels. Water levels in existing observation wells and new ones would be measured at the same time. All wells should be measured within a period of several days during each measuring round. For actively used wells, the pump should be turned off for at least 12 hours before measurement. Records should be kept of how long the well had been off prior to measurement. Elevations of the measuring point should be determined to the nearest tenth of a foot, so that water level elevation maps can be prepared. In the future, it may be desirable to make one map for the shallow groundwater and another for the deeper groundwater tapped by many new City wells.

It would be useful to measure pumping levels monthly during June through September. In this case, the pumping level should be measured after the well is pumping for at least four hours. Records should be kept of how long the well had been pumping prior to measurement.

Groundwater Quality

Both shallow monitoring wells and wells in rural areas should periodically be sampled for water quality analysis. Monitoring of the shallow wells in the urbanized areas would focus on: 1) the impact of recharge of good quality water in dedicated facilities and storm runoff basins 2) Water quality problem areas, such as nitrate, DBCP, EDB, and manganese. The shallow monitoring wells should be sampled semi-annually in the spring and fall. The constituents for each well would be selected based on the location.

In the urbanizing areas, sampling would focus on areas within about two miles of urbanized areas, and upgradient (normally to the northeast), or in areas projected for development (i.e. some of the village areas). This sampling would probably be done on a triennial basis - the purpose being to periodically provide data for updated maps of constituent problem areas and concentration (i.e.. for nitrate and DBCP). This type of sampling was previously done once in the Herndon-Shepard plan area for DBCP and EDB and in the east Clovis area for DBCP, EDB, Iron, and manganese. Wells selected for sampling would have information available on depth and perforation interval, and ideally would have previous sampling results available.

Data Management

The monitoring data should be verified for accuracy and then placed in a suitable data management system. Such a system should allow easy preparation of water level and chemical constituent hydrographs. Also water level elevation and depth to water maps should be periodically prepared.

Appendix A

Supplemental Information
Technical Memorandum No. 3
Water Treatment Plant Alternatives

and

Fresno Joint Water Treatment Plant Investigation

FRESNO/ BUREC CONTRACT COST SUMMARY

The Central Valley Improvement Act passed and was signed into law on October 30, 1992. This act radically changed the method in which the Federal Government contracts with water users in the Central Valley. Certain provisions were established that have bearing on whether Fresno should consider renewing this supply. Following is a brief summary of key elements of the current contract which the City of Fresno has with the USBR.

EXISTING USBR - CITY OF FRESNO CONTRACT

Amount of Water (AF)	60,000
Original Contract Rate ($/AF)	$ 10.00
Type / Allowable Uses	Municipal & Industrial
Contract Expiration	2006
O & M Deficit (Accrued as of 9/30/95)	$ 21,682,000
Accrual rate in 1995	$ 3,017,000 / Yr
Current Interest Rate	8%
Capital Repayment responsibility	$ 24,465,000
O & M Rate (1994)	$ 16.01 /AF
Friant Surcharge	$ 4.00/AF (increasing incrementally to 9.00 in 1999).
Restoration Fund	$ 6.23 /AF
Overall Water Rate	$ 26.24 /AF

Appendix A
Supplemental Information
Technical Memorandum No. 4
Existing Water Distribution System: Hydraulic Modeling and Analyses

Table 1
Well Production Capacity and Priority List
As of January 1997

Classification	Clovis Wells*				Tarpey Wells*
	NE	SE	NW	SW	
Prime Pumpers - Base Line	24	11	8A,14,	28, 15, 17	T-5
Secondary	22	12,18	25, 26	4AA, 10, 29	T-3, T-6
Marginal	3, 23	19	21(GAC)	5, 6, 9	T-1, T-7, T-8
Stand By	1	13	27		T-2
Off Line or Future	20	16		2, 7	T-4

* Wells are listed in order of priority for each classification.

Table 2
Pipe Age / Material / Roughness Coefficients

Material	Installed	Age	Estimated "C" Factor
Ductile Iron	1980	15	120
	1990	5	130
Transite / AC	1950	45	80
	1960	35	90
	1970	25	100
	1980	15	110
PVC	1980	15	120
	1990	5	130

Appendix A

Supplemental Information
Technical Memorandum No. 5
System Analysis and Recommendations for Planning Horizon Conditions

Black & Veatch

Technical Memorandum
Water Quality & Regulatory Requirements

Clovis, California
Water System Master Plan

B&V Project 34404.102
February 10, 1998

Prepared By: Doug Elder

Introduction

A. Background

This technical memorandum is one of several special studies being conducted as part of the development of a Water System Master Plan for the City of Clovis. The Master Plan addresses the development of a surface water supply system to augment the City's existing groundwater supply. The raw water supply will be the Kings River, delivered to the treatment plant site through the Enterprise Canal, an unlined canal which serves Clovis and the northern portions of the City of Fresno.

B. Purpose

The purposes of this memorandum are: (1) to present the results of an evaluation of existing raw water quality data for the Enterprise Canal, and (2) to provide an overview of current and impending water quality and treatment regulations which will impact the design of the new surface water treatment facility.

Water Quality

The majority of the water present in the Enterprise Canal originates as rainfall and snowmelt on the Sierra Nevada Mountain range in the San Joaquin and Kings River basins, and is generally of relatively high quality. However, significant localized rainfall events can result in runoff into the canal, which can lead to short-term increases in turbidity (the intensity and duration of these turbidity peaks have not yet been defined).

A. Water Quality Monitoring Results

While existing water quality data for the Enterprise Canal are limited, the City of Fresno is currently evaluating design requirements for a new treatment facility utilizing the canal as the raw water supply, and has implemented a water quality monitoring program to assist in defining treatment requirements for the new facility. A summary of water quality monitoring data provided by Fresno for January through September 1997 is presented in Table 1.

Table 1
Raw Water Quality for Enterprise Canal (01/97 - 09/97)[1]

Constituent	Average Concentration	Range[2]
Turbidity, NTU	3.5	0.3 - 15
pH, units	7.4	6.8 - 8.2
Total Alkalinity, mg/L as $CaCO_3$	19	9.9 - 67
Total Hardness, mg/L as $CaCO_3$	19	7 - 84
Calcium, mg/L as Ca	4.4	2.1 - 14
Iron, mg/L as Fe	0.20	ND - 0.89
Color (apparent), units	16.2	5 - 30
Total Organic Carbon, mg/L	1.3	1.2 - 1.3[3]
Threshold Odor Number	1.5	1 - 2
Aluminum, mg/L as Al	0.24	ND - 1.10

[1]Source: City of Fresno monitoring data.
[2]Total of 11 samples unless otherwise noted.
[3]3 TOC samples collected during September 1997.

Samples collected during the 1997 monitoring period were also analyzed for a broad spectrum of organic chemicals (primarily pesticides/herbicides, or their degradation by-products); these contaminants were not detected in any of the samples. However, Department of Health Services staff have reportedly expressed concerns regarding the potential for degradation of water quality in the canal attributable to runoff from agricultural and cattle grazing areas. (Runoff from cattle grazing areas is a potential source of *Cryptosporidium*, a microbial contaminant which will be regulated under the impending Interim and Long-Term Enhanced Surface Water Treatment regulations.) While nitrate was detected in several of the samples, the observed concentrations (i.e., 1 to 2 mg/L) were significantly less than the current Maximum Contaminant Level (MCL) for drinking water of 45 mg/L.

Information on the water's disinfection by-product formation potential is also limited at this time. The low total organic carbon (TOC) concentrations for the three samples collected during September 1997, however, suggest that disinfection by-product formation should be relatively low when using chlorine as the primary disinfectant and for maintenance of a disinfectant residual within the distribution system. (Waters with TOC concentrations of approximately 2 mg/L or less generally have been shown to yield concentrations of chlorine-based disinfection by-products which are lower than current and anticipated future allowable levels.) Limited evaluation of the water's potential to form total trihalomethane compounds (halogenated compounds formed during disinfection using free chlorine) conducted during September 1996 showed 7-day TTHM formation potentials ranging from less than 0.010 mg/L to approximately 0.069 mg/L (the highest TTHM concentration was observed for a sample with a free chlorine residual of approximately 2 mg/L after 7 days). While these data suggest that significant problems in complying with current and anticipated future requirements for TTHM levels in the treated water would not be anticipated, it is emphasized that these conclusions are based on limited monitoring data, and that additional testing to assess formation potentials for TTHMs and other disinfection by-products would be recommended prior to initiating design of the new surface water treatment facility.

A significant issue to be considered in the design of the Clovis surface water treatment facility is the potential incompatibility of the existing groundwater supply and the treated surface water. The groundwater supply exhibits significant levels of alkalinity and calcium hardness (typical alkalinity is 110-120 mg/L as $CaCO_3$, and calcium hardness is typically 50-70 mg/L as $CaCO_3$), which encourages the deposition of protective calcium carbonate coatings within the distribution system. However, the proposed surface water supply exhibits relatively low alkalinity and hardness, and treatment to remove turbidity will likely result in additional reductions in alkalinity. Intermixing of these two water sources in the distribution system could result in the dissolution/removal of existing protective coatings within the system, thereby encouraging corrosion and difficulties in complying with current regulations governing lead and copper concentrations at consumer taps. Appropriate adjustments in the composition of the treated surface water prior to distribution will therefore need to be made to ensure that corrosion-related problems do not occur. This can be accomplished through the addition of alkalinity and adjustment of treated surface water pH.

B. Recommendations

Clovis should consider joint participation with the City of Fresno in their ongoing Enterprise Canal water quality monitoring program, and in the impending pilot-scale treatment process evaluations planned by Fresno. The information generated by these studies will provide valuable insite regarding site-specific treatment requirements for the new surface water source. A joint approach to these studies would also avoid the need for separate monitoring/analysis of the raw water supply, thereby resulting in significant cost savings for both entities.

Water Treatment Regulatory Requirements

A detailed discussion of both current and impending regulations under the 1986 and 1996 Safe Drinking Water Act (SDWA), and EPA's current regulatory promulgation schedule are presented in the attachment to this Technical Memorandum. Several aspects of these regulations which will affect the design and operation of the City's new surface water treatment facility are summarized below.

In California, the State Department of Health Services (DHS) is responsible for enforcement of the federal water quality and treatment regulations. In order to maintain primacy, the State must adopt drinking water regulations which are at least as stringent as the federal regulations. (DHS may also promulgate regulations which are more stringent that the federal regulations, and has exercised this option in the development of Maximum Contaminant Levels (MCLs) for several organic contaminants.

A. Current Regulations

1. Surface Water Treatment Rule

The primary purpose of the Surface Water Treatment Rule (SWTR) is to protect the public from waterborne diseases. Under the SWTR, the new treatment plant will be required to comply with mandatory performance requirements for filtration and disinfection. Filtered water turbidity must be equal to or less than 0.5 NTU for a minimum of 95 percent of the monthly turbidity samples. (However, it is expected that under the impending Interim Enhanced Surface Water Treatment Rule, the turbidity limit will be reduced to 0.3 NTU by the time that the plant is placed in service, as discussed below.) Disinfection conditions which will ensure effective inactivation of *Giardia* cysts and enteric viruses must also be maintained continuously. Required levels of disinfection required are specified by DHS (current minimum requirements for plants practicing "conventional treatment" are to maintain disinfection conditions which will result in a 68 percent (0.5-log) inactivation of *Giardia* cysts and a 99 percent (2-log) inactivation of enteric viruses; however, DHS can specify that higher levels of disinfection be provided, based on the type of treatment process utilized and/or the degree of cyst contamination in the source water).

The SWTR also requires that a minimum disinfectant residual of 0.2 mg/L be maintained in the treated water entering the distribution system, and that a "detectable" residual be maintained within the distribution system for a minimum of 95% of the monthly coliform samples analyzed. The requirement that a detectable distribution system disinfectant residual be maintained may dictate that provisions for continuous chlorination of water from all of the City's existing wells be added. (The disinfectant residual in the treated surface water would dissipate rapidly following blending with untreated groundwater within the distribution system.) DHS will therefore need to evaluate disinfection requirements for the existing wells with respect to overall system requirements.

2. Total Trihalomethanes Rule

The Clovis system will be required to comply with the current MCL for total trihalomethanes of 0.10 mg/L. (However, it is expected that under the impending Disinfectants/Disinfection By-Products Rule, the current MCL will be reduced to

0.080 mg/L by the time that the plant is placed in service, as discussed below.) Compliance with this regulation is not expected to present any significant difficulties, based on the following:

- Compliance with this regulation is based on monitoring results for the entire distribution system, and blending of the surface water supply with the existing groundwater supply will result in low average disinfection by-product concentrations
- Preliminary testing indicates that the proposed surface water supply has a relatively low tendency to form disinfection by-products (as discussed in the "Water Quality" section above).

3. 1996 Safe Drinking Water Act Amendments

The 1996 SDWA Amendments establish specific schedules for promulgation of new regulations governing disinfection by-products (DBPs), microbial contaminants, arsenic, radon, and disinfection of groundwater supplies, and requires EPA and the Centers for Disease Control to conduct a joint study of the potential health impacts of sulfate in drinking water supplies. Utilities will be allowed up to three years to achieve compliance with new regulations following final promulgation, with provisions for extensions of up to two years (with EPA and/or local regulatory agency approval) if "significant" capital improvements are required. (Under the 1986 Amendments, utilities were typically allowed only 18 months to achieve compliance.) The 1996 Amendments also include requirements for monitoring of "unregulated contaminants", for preparation of annual reports advising consumers of the quality of the distributed water (i.e., "Consumer Confidence Reports") for utilities serving more than 500 consumers, and for regulation of in-plant recycling of filter backwash and/or sludge treatment residuals.

4. California Design Requirements

Current California design standards for new water treatment facilities include the following criteria:

- For conventional and direct filtration processes, provide facilities to achieve an average daily treated water turbidity goal of 0.2 NTU.
- Provide filter-to-waste capability for individual filters, or provisions for addition of a coagulant to the filter backwash water to minimize the duration and intensity of the turbidity spike which occurs when a filter is returned to service following backwashing.

- Provide facilities to adequately clean the filter media during backwashing (typically this involves provisions for air scouring of the media prior to and during the initial stages of backwashing).
- For systems which recycle filter backwash through the treatment process, provisions for separation of solids prior to recycle must be included.
- When a coagulation process is used, process selection is to be based on results of pilot-scale or bench-scale testing which demonstrates the effectiveness of the proposed coagulation process over the full range of expected water quality conditions.

5. *Cryptosporidium* Action Plan

The California Legislature recently passed legislation which directs DHS to implement their "*Cryptosporidium* Action Plan" dated April 1995. This plan was developed to address the need for optimization of treatment plant performance to ensure effective removal of *Cryptosporidium*, an intestinal parasite implicated in several recent waterborne disease outbreaks. The Plan does not contain any specific requirements which supersede the current SWTR; instead, it emphasizes plant optimization in the context of reducing the risk of waterborne disease. The following treatment goals are included in the Plan:

- Sedimentation/clarification basin effluent turbidity: 1 to 2 NTU
- Combined filter effluent turbidity: 0.1 NTU
- Reclaimed backwash water turbidity: Less than 2.0 NTU
- Turbidity after filter backwash, filter to waste: Less than 0.3 NTU

While these criteria are goals, and not actual treatment requirements, design of new surface water treatment facilities should incorporate these goals in the planning and design process.

B. Pending Regulations

Regulations which will be of primary concern over the next several years to utilities treating surface water supplies are Stage 1 of the impending Disinfectants/Disinfection By-Products Rule (D/DBPR) and the Interim Enhanced Surface Water Treatment rules. Under the 1996 SDWA Amendments, EPA must promulgate these rules by November 1998, and compliance must be achieved by November 2001 (November 2003 if "substantial improvements" are required for compliance, with DHS approval).

1. Disinfectants / Disinfection By-Products

Stage 1 of the D/DBPR will reduce the current Maximum Contaminant Level (MCL) for total trihalomethanes (TTHMs) from 0.10 mg/L to 0.080 mg/L, and will introduce new MCLs of 0.060 mg/L for total haloacetic acids (referred to as HAA5, as five of the nine known haloacetic acid compounds are to be regulated) and 0.010 mg/L for bromate, a by-product of disinfection using ozone. (Under Stage 2 of the D/DBPR, the MCLs for total trihalomethanes and HAA5 may be further reduced, as discussed below.) This regulation will also establish a treatment technique requiring that most utilities which treat surface water supplies using conventional methods, i.e., coagulation/filtration, operate in an "enhanced coagulation" mode to achieve specified total organic carbon (TOC) percent removals. Required TOC removal percentages will be dependent on raw water ("source water") TOC and alkalinity concentrations, as shown in Table 2. (Utilities with annual running average source water TOC concentrations of 2 mg/L or less will not be required to comply with the enhanced coagulation criteria in

Table 2
TOC Removal Requirements for Enhanced Coagulation

Source Water TOC, mg/L*	Percent TOC Removal Required at Indicated Source Water Alkalinity		
	0 - 60 mg/L	>60 - 120 mg/L	>120 mg/L
>2 - 4	35	25	15
>4 - 8	45	35	25
>8	50	40	30

*Based on annual running average of monthly source water TOC.

Table 2.) Specific UV absorbance (SUVA, defined as the ratio of the water's ultraviolet absorbance at 254 nm (UV_{254}) to its dissolved organic carbon (DOC) concentration) was also recently added as a criteria for determining if enhanced coagulation treatment will be required. For plants treating surface water supplies, enhanced coagulation treatment would not be required if the raw water has an average SUVA of less than 2.0 liter/(mg)(m). While additional testing will be

required to determine the need for enhanced coagulation treatment for the Enterprise Canal supply, preliminary testing indicates that raw water TOC is less than 2.0 mg/L, which would eliminate the need to practice enhanced coagulation.

The Stage 2 D/DBPR (currently scheduled for promulgation during May 2002, and effective 3 years later) will likely reduce the allowable limits for certain DBPs beyond those promulgated in the Stage 1 rule. (The extent to which the MCLs may be reduced
cannot be predicted with any certainty at this time, as the final MCLs will be determined through negotiations between EPA and the affected parties.)

2. Enhanced Surface Water Treatment Rule

The Enhanced Surface Water Treatment Rule is being developed in response to increasing concerns regarding the microbial quality of treated water supplies, and the desire to avoid compromising the microbial quality of the treated water while attempting to comply with more stringent disinfection by-product regulations. Under the pending Interim Enhanced Surface Water Treatment Rule (IESWTR), the allowable finished water turbidity will be reduced from the present 0.5 NTU to 0.3 NTU for utilities serving more than 10,000 consumers. (This standard applies to the combined filtered water, and a minimum of 95 percent of the monthly turbidity measurements must meet the revised turbidity criteria.) The turbidity of the combined filter effluent cannot exceed 1 NTU at any time (the current Surface Water Treatment Rule allows for a maximum combined filter effluent turbidity of 5 NTU). The rule will also include specific performance criteria for individual filters. Utilities will be required to maintain filtration conditions which will ensure that a minimum 2-log (99%) removal of *Cryptosporidium* is achieved, and must monitor haloacetic acid concentrations within the distribution system over four consecutive quarters beginning within 90 days of promulgation of the IESWTR. Credit will continue to be allowed for disinfection contact time provided at any point in the treatment process. (The originally-proposed D/DBPR included provisions that would not allow utilities to claim disinfection credit for contact time provided prior to enhanced coagulation treatment.) It is also expected that the IESWTR will include a requirement that all utilities treating surface water supplies conduct sanitary surveys of their water supply and treatment facilities every 3 to 5 years.

As discussed in the attached regulatory summary, EPA plans to initiate a regulatory negotiation process in the near future to develop a long-term ESWTR which may include a treatment technique designed to minimize the potential for public exposure to *Cryptosporidium* in drinking water. EPA intends to promulgate this regulation (currently being referred to as the Stage 2 Long-Term Enhanced Surface Water Treatment Rule, or LT2ESWTR) concurrently with the Stage 2

D/DBPR, i.e., during May 2002. As *Cryptosporidium* is extremely resistant to disinfection with free chlorine, promulgation of specific inactivation requirements for *Cryptosporidium* could result in the need for advanced disinfection processes, such as ozonation, for utilities treating surface water supplies.

3. Filter Backwash Water Rule

In-plant recycling of waste flows such as filter backwash has recently come under increased scrutiny due to concerns regarding the potential for return of microbial contaminants such as Giardia and Cryptosporidium to the head of the treatment process. To address these concerns, EPA must develop a regulation governing the recycling of filter backwash water within the treatment process of public water systems by August 2000. It is expected that this regulation may, as a minimum, require that provisions for separation of solids from filter backwash flows be included if recycling of these flows to minimize water losses is practiced. (Note that this is already required under current California regulations.) However, the extent to which this impending regulation could impact the design of the City's new surface water treatment facility cannot be predicted with any certainty at this time, as EPA has made no official comments regarding the potential requirements of the rule. The City should therefore monitor the development of this rule closely over the next two years.

4. Groundwater Disinfection Rule

In order to fulfill the amended SDWA mandate that disinfection requirements be imposed on all public water systems, a rule to regulate the disinfection of groundwater supplies is being developed by EPA. Under the 1996 SDWA Amendments, EPA must finalize this rule by January 2001. For the City's existing groundwater supply, it is considered likely that addition of chlorine feed capability will be required at each well to comply with this regulation. While this regulation will not directly impact the design of the new surface water treatment facility, the disinfectant residual in the treated surface water supply must be compatible with the disinfectant residual resulting from chlorination of the groundwater supply.

Treatment Implications

Based on review of the limited raw water quality information and consideration of both current and impending regulatory requirements, the City's new surface water treatment facility should be designed to incorporate the following:

- Sufficient residence time and process control flexibility to correct for changes in treatment requirements brought about by variations in raw water quality.
- Provisions for reduction/removal of raw water turbidity prior to filtration.
- Efficient filtration to ensure reliable removal of microbial contaminants.
- Provisions for removal of tastes/odors and organic contaminants through addition of powdered activated carbon.
- Ability to adjust treated water pH and alkalinity to ensure compatibility of the treated water with the existing groundwater supply.
- Compatibility of the treated water disinfectant residual with that of the current water supply.
- Flexibility to incorporate provisions for increased treatment (such as ozone disinfection and/or post-filtration activated carbon contactors), if required to comply with future regulations.

Appendix A

Supplemental Information

Technical Memorandum No. 6
Computer Model Output

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*****************************************************************
*                         E P A N E T                           *
*                                                               *
*                 Hydraulic and Water Quality                   *
*                                                               *
*                 Analysis for Pipe Networks                    *
*                                                               *
*                        Version 1.1e                           *
*                                                               *
*****************************************************************
```

CLOVIS WATER MASTER PLAN - HYDRAULIC MODEL
Last Updated: 3/2/98

```
Input Data File .................... 3-2A.INP
Output Report File ................. 3-2a.rpt
Verification File ..................
Hydraulics File ....................
Map File ........................... Clovis.map
Number of Pipes .................... 696
Number of Nodes .................... 476
Number of Tanks .................... 2
Number of Pumps .................... 0
Number of Valves ................... 0
Headloss Formula ................... Hazen-Williams
Hydraulic Timestep ................. 1.00 hrs
Hydraulic Accuracy ................. 0.005000
Maximum Trials ..................... 20
Quality Analysis ................... None
Specific Gravity ................... 1.00
Kinematic Viscosity ................ 1.10e-05 sq ft/sec
Chemical Diffusivity ............... 1.30e-08 sq ft/sec
Total Duration ..................... 0.00 hrs
Reporting Criteria:
   Selected Nodes
   Selected Links
```

Node Results:

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi
7010	360.00	0.00	475.01	49.83
7020	358.00	0.00	475.11	50.74
7030	360.00	-945.00	476.62	50.53
7040	340.00	-1050.00	481.27	61.21
7050	354.00	-500.00	471.70	51.00

7060	348.00	-325.00	484.75	59.25
7070	350.00	0.00	471.08	52.46
7080	363.00	-1495.00	487.47	53.93
7090	362.00	0.00	477.50	50.04
7100	344.00	0.00	470.97	55.02
7110	365.00	-1330.00	471.42	46.11
7120	355.00	-1030.00	471.89	50.65
7130	375.00	0.00	466.73	39.75
7140	360.00	-1160.00	489.34	56.04

Node Results: (continued)

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi	
7150	349.00	-1250.00	484.25	58.60	
7160	360.00	0.00	467.08	46.40	
7170	344.00	-1380.00	480.46	59.13	
7180	345.00	-855.00	490.21	62.92	
7190	345.00	-410.00	486.00	61.10	
7200	372.00	0.00	469.18	42.11	
7210	356.00	-930.00	487.64	57.04	
7220	370.00	-645.00	485.75	50.15	
7230	367.00	-325.00	484.41	50.87	
7240	370.00	-880.00	484.91	49.79	
7250	364.00	-1200.00	488.86	54.10	
7260	363.00	-1130.00	488.28	54.28	
7270	360.00	0.00	486.73	54.91	
7280	348.00	-2180.00	476.90	55.85	
7290	340.00	-780.00	474.16	58.13	
7510	345.00	-120.00	491.55	63.50	
7520	346.00	0.00	490.61	62.66	
7530	348.00	-1030.00	491.53	62.19	
7540	348.00	0.00	476.64	55.74	
7550	350.00	-1025.00	472.76	53.19	
7560	346.00	-700.00	495.23	64.66	
7570	345.00	-500.00	490.11	62.87	
7580	345.00	-660.00	497.41	66.04	
7610	468.00	0.00	468.00	0.00	Reservoir
7620	496.00	-8313.00	506.00	4.33	Tank

Link Results:

Link	Start Node	End Node	Diameter in	Flow gpm	Velocity fps	Headloss /1000ft

-							
	1070	730	740	10.00	388.54	1.59	1.7
5							
	1330	750	910	12.00	535.30	1.52	0.9
3							
	1380	780	950	12.00	1025.00	2.91	3.1
0							
	2010	2400	3000	12.00	-132.57	0.38	0.0
8							
	3693	3600	3605	12.00	-310.44	0.88	0.2
9							
	4890	4410	7620	20.00	-8313.00	8.49	10.6
6							

****
*   E P A N E T
*
*   Hydraulic and Water Quality
*
*   Analysis for Pipe Networks
*
*   Version 1.1e
*
****

Fill Cycle S.E. wells of

CLOVIS WATER MASTER PLAN - HYDRAULIC MODEL

Nees 2,500 Fill
4,000 Boost around tank

Input Data File 7-6XMDD.INP
Output Report File 7-6xmdd.rpt
Verification File ....
Hydraulics File ....
Map File Clovis2.map
Number of Pipes 865
Number of Nodes 600
Number of Tanks 5
Number of Pumps 8
Number of Valves 13
Headloss Formula .... Hazen-Williams
Hydraulic Timestep .... 1.00 hrs
Hydraulic Accuracy 0.005000
Maximum Trials 20
Quality Analysis .... None
Specific Gravity 1.00
Kinematic Viscosity .... 1.10e-05 sq ft/sec
Chemical Diffusivity .... 1.30e-08 sq ft/sec
Total Duration .... 0.00 hrs
Reporting Criteria:
Selected Nodes
Selected Links

Tollhouse
5,000 Fill
2,500 Boost out to 2

Cross town pipes
Sierra & Sunnyside
⇒ 18"
Tollhouse Main
⇒ 24"

Node Results:

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi
50	342.00	0.00	473.46	56.96
60	340.00	46.00	473.23	57.73
70	340.00	0.00	473.22	57.72
80	340.00	0.00	473.20	57.71
85	340.00	0.00	473.24	57.73
90	340.00	0.00	473.21	57.72

100	340.00	37.50	473.12	57.68
110	343.00	31.50	472.62	56.17
120	345.00	0.00	470.72	54.48
130	342.00	0.00	472.23	56.43
140	345.00	0.00	470.57	54.41
150	345.00	0.00	470.47	54.37
160	345.00	324.00	470.36	54.32
170	345.00	0.00	473.39	55.63
180	342.00	28.50	473.42	56.95

Node Results: (continued)

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi
190	338.00	366.00	472.90	58.45
200	337.00	0.00	473.32	59.07
210	336.00	126.00	472.95	59.34
220	336.00	30.00	474.93	60.20
225	335.00	166.00	472.77	59.69
230	337.00	22.50	475.18	59.87
235	335.00	0.00	473.27	59.91
240	341.00	280.00	473.96	57.61
245	337.00	0.00	473.30	59.06
250	345.00	115.50	473.79	55.80
255	344.00	0.00	475.37	56.92
260	346.00	0.00	470.35	53.88
270	347.00	0.00	466.18	51.64
280	350.00	0.00	469.82	51.92
290	347.00	234.00	470.77	53.63
300	345.00	0.00	475.39	56.50
305	345.00	0.00	476.12	56.81
310	340.00	193.50	475.44	58.69
320	341.00	0.00	476.15	58.56
325	340.00	0.00	476.15	58.99
330	338.00	0.00	475.86	59.74
340	340.00	49.50	476.09	58.97
350	343.00	49.50	476.44	57.82
355	344.00	0.00	476.44	57.39
360	344.00	0.00	476.18	57.27
370	346.00	0.00	476.40	56.50
375	348.00	0.00	473.54	54.40
380	350.00	33.00	470.68	52.29
390	352.00	0.00	465.32	49.10
400	354.00	18.00	461.02	46.37
410	352.00	208.00	462.22	47.76
420	352.00	169.50	461.02	47.24
430	358.00	0.00	457.95	43.31
440	355.00	0.00	458.14	44.69

450	354.00	0.00	462.40	46.97
460	354.00	0.00	462.09	46.84
470	354.00	108.00	461.72	46.68
500	350.00	0.00	463.90	49.35
505	348.00	0.00	463.90	50.22
510	350.00	0.00	468.80	51.47
520	354.00	0.00	474.22	52.09
530	350.00	0.00	475.20	54.25
540	350.00	537.25	462.28	48.65
550	350.00	0.00	475.38	54.33
560	350.00	0.00	475.26	54.28
565	345.00	0.00	476.92	57.16

Node Results: (continued)

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi
570	345.00	0.00	470.56	54.40
580	345.00	0.00	470.53	54.39
600	345.00	0.00	470.74	54.48
610	348.00	0.00	476.78	55.80
620	350.00	0.00	476.49	54.81
630	347.00	0.00	470.20	53.38
640	342.00	0.00	469.65	55.31
650	345.00	537.25	469.17	53.80
660	345.00	0.00	471.09	54.64
670	342.00	0.00	469.55	55.27
680	345.00	0.00	469.41	53.91
690	346.00	0.00	469.43	53.48
700	345.00	0.00	469.47	53.93
710	350.00	57.00	466.92	50.66
715	348.00	0.00	466.25	51.24
720	350.00	0.00	465.40	50.00
730	350.00	0.00	464.58	49.65
740	350.00	0.00	461.96	48.51
750	350.00	0.00	459.21	47.32
755	350.00	0.00	459.23	47.33
760	350.00	0.00	466.36	50.42
770	350.00	262.50	461.84	48.46
780	353.00	0.00	457.04	45.08
790	350.00	470.75	449.18	42.97
800	349.00	0.00	459.67	47.95
805	350.00	0.00	455.78	45.84
810	348.00	0.00	459.79	48.44
820	345.00	0.00	462.13	50.75
830	345.00	0.00	463.49	51.34
840	345.00	0.00	463.82	51.48
850	345.00	0.00	464.42	51.74

860	345.00	182.00	464.43	51.75
870	345.00	85.00	465.62	52.27
880	346.00	0.00	471.47	54.37
890	345.00	127.00	462.15	50.76
900	350.00	268.00	456.43	46.11
910	350.00	0.00	459.01	47.24
920	355.00	0.00	451.12	41.65
930	357.00	154.00	442.81	37.18
940	355.00	0.00	443.51	38.35
950	354.00	0.00	453.58	43.15
960	355.00	364.00	451.12	41.65
970	360.00	0.00	441.92	35.50
980	360.00	0.00	441.94	35.50
990	364.00	6.00	440.65	33.21
1000	356.00	0.00	451.12	41.21

Node Results: (continued)

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi
1010	357.00	58.50	444.95	38.11
1020	360.00	0.00	449.35	38.72
1030	365.00	320.00	443.78	34.14
1040	355.00	7.50	458.14	44.69
1050	360.00	150.00	450.99	39.43
1060	360.00	105.00	448.73	38.45
1070	360.00	75.00	447.43	37.88
1080	365.00	0.00	440.02	32.51
1090	365.00	39.00	439.34	32.21
1100	366.00	0.00	438.61	31.46
1110	370.00	37.50	428.55	25.37
1120	370.00	430.00	429.01	25.57
1130	370.00	0.00	433.98	27.72
1140	375.00	134.00	429.77	23.73
1150	363.00	176.00	434.09	30.80
1160	365.00	33.00	431.26	28.71
1170	359.00	154.00	429.76	30.66
1180	360.00	154.00	420.87	26.38
1190	360.00	0.00	419.98	25.99
1200	358.00	0.00	425.28	29.15
1210	365.00	0.00	419.04	23.42
1220	365.00	0.00	412.64	20.64
1221	365.00	0.00	508.16	62.03
1222	365.00	0.00	508.16	62.03
1230	378.00	308.00	422.52	19.29
1240	369.00	178.00	422.16	23.03
1250	369.00	712.00	420.25	22.21
1260	362.00	692.00	416.17	23.47

2400	343.00	162.00	476.93	58.03
2410	344.00	0.00	476.58	57.45
2420	344.00	0.00	476.58	57.45
2430	344.00	0.00	476.48	57.40
2450	350.00	248.00	477.04	55.05
2500	350.00	370.50	470.93	52.40
2510	350.00	0.00	469.41	51.74
2520	351.00	0.00	464.91	49.36
2530	351.00	0.00	463.74	48.85
2540	353.00	188.00	461.40	46.97
2545	354.00	0.00	461.44	46.55
2550	353.00	0.00	460.99	46.79
2560	354.00	54.00	456.62	44.46
2570	355.00	0.00	457.87	44.57
2580	355.00	0.00	456.62	44.03
2585	358.00	0.00	457.86	43.27
2590	355.00	0.00	452.57	42.28
2595	357.00	0.00	451.68	41.02

Node Results: (continued)

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi
2600	355.00	106.50	452.48	42.24
2610	355.00	123.00	449.50	40.95
2620	358.00	0.00	449.45	39.63
2625	359.00	0.00	452.03	40.31
2630	358.00	0.00	449.30	39.56
2640	360.00	0.00	441.91	35.49
2650	360.00	0.00	442.65	35.81
2660	360.00	0.00	444.13	36.45
2670	362.00	300.00	438.37	33.09
2680	362.00	0.00	439.23	33.46
2690	363.00	0.00	441.79	34.14
2700	365.00	0.00	437.89	31.58
2701	366.00	0.00	437.87	31.14
2710	365.00	0.00	438.55	31.87
2720	365.00	0.00	434.18	29.98
2730	366.00	0.00	429.93	27.70
2740	367.00	0.00	430.56	27.54
2750	370.00	306.00	429.90	25.95
2760	368.00	0.00	429.90	26.82
2770	370.00	0.00	427.94	25.10
2780	370.00	0.00	428.11	25.18
2790	370.00	0.00	428.12	25.18
2800	370.00	0.00	426.59	24.52
2810	370.00	418.00	425.61	24.10
2820	372.00	0.00	425.23	23.06

2830	372.00	100.50	425.23	23.06
2840	372.00	0.00	424.57	22.78
2850	376.00	0.00	422.88	20.31
2860	377.00	692.00	422.14	19.56
2870	377.00	0.00	422.57	19.74
2880	380.00	308.00	422.45	18.39
2882	380.00	0.00	596.28	93.71
2890	380.00	52.50	422.12	18.25
2900	380.00	0.00	421.76	18.10
2902	380.00	0.00	496.89	50.65
3000	346.00	56.00	473.82	55.38
3010	350.00	157.50	471.40	52.60
3020	350.00	0.00	469.88	51.94
3030	350.00	0.00	469.63	51.83
3040	350.00	0.00	469.68	51.86
3050	350.00	0.00	468.25	51.24
3055	350.00	0.00	466.91	50.66
3060	350.00	120.00	466.91	50.66
3070	351.00	15.00	466.61	50.10
3080	350.00	367.50	473.74	53.62
3090	350.00	0.00	478.33	55.61

Node Results: (continued)

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi
3100	350.00	0.00	481.56	57.01
3101	350.00	0.00	481.78	57.10
3102	350.00	0.00	481.56	57.01
3104	350.00	0.00	481.78	57.10
3110	350.00	0.00	473.99	53.73
3115	346.00	0.00	475.82	56.25
3120	346.00	0.00	475.75	56.22
3130	346.00	171.00	473.97	55.45
3135	347.00	0.00	473.85	54.96
3136	347.00	0.00	473.85	54.96
3137	347.00	0.00	473.85	54.96
3140	346.00	0.00	475.79	56.24
3150	355.00	7.50	458.74	44.95
3160	355.00	0.00	454.50	43.12
3170	355.00	0.00	454.77	43.23
3180	355.00	153.00	455.49	43.54
3190	354.00	0.00	456.55	44.44
3200	354.00	0.00	457.67	44.92
3260	354.00	0.00	460.12	45.98
3270	353.00	0.00	460.39	46.53
3280	350.00	0.00	464.09	49.43
3290	350.00	0.00	458.67	47.09

3300	350.00	188.00	459.43	47.42
3310	350.00	50.00	464.42	49.58
3320	355.00	0.00	455.65	43.61
3330	354.00	0.00	457.01	44.64
3340	355.00	0.00	452.61	42.30
3350	360.00	0.00	445.73	37.15
3360	360.00	105.00	444.72	36.71
3365	360.00	0.00	444.67	36.69
3370	360.00	0.00	437.45	33.56
3375	360.00	0.00	437.46	33.57
3380	360.00	0.00	437.32	33.50
3390	360.00	0.00	437.38	33.53
3395	360.00	0.00	437.85	33.73
3400	360.00	54.00	438.29	33.92
3410	361.00	0.00	437.90	33.32
3415	360.00	0.00	437.92	33.76
3420	361.00	0.00	438.31	33.50
3430	356.00	180.00	452.07	41.63
3435	356.00	0.00	453.08	42.07
3440	355.00	0.00	454.09	42.94
3450	355.00	0.00	454.53	43.13
3460	355.00	0.00	455.46	43.53
3470	356.00	0.00	455.40	43.07
3475	355.00	0.00	444.15	38.63

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Node Results: (continued)

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi
3480	356.00	0.00	453.03	42.04
3485	355.00	0.00	446.08	39.47
3490	355.00	0.00	455.75	43.65
3500	363.00	244.00	433.41	30.51
3510	362.00	0.00	433.26	30.88
3520	364.00	0.00	431.48	29.24
3530	365.00	0.00	429.61	28.00
3540	365.00	0.00	428.78	27.64
3550	368.00	88.00	426.56	25.38
3560	370.00	0.00	427.07	24.73
3570	372.00	0.00	427.14	23.89
3575	372.00	0.00	427.26	23.95
3580	370.00	0.00	427.62	24.97
3590	367.00	0.00	430.59	27.56
3600	365.00	0.00	434.28	30.02
3605	365.00	0.00	433.87	29.84
3610	364.00	0.00	434.86	30.70
3620	364.00	0.00	434.93	30.73
3630	364.00	0.00	435.31	30.90

3640	360.00	102.00	436.78	33.27
3650	362.00	0.00	433.91	31.16
3660	361.00	0.00	437.34	33.08
3670	360.00	0.00	433.80	31.98
3680	360.00	360.00	433.87	32.01
3690	365.00	0.00	431.99	29.03
3700	365.00	0.00	432.85	29.40
3720	367.00	0.00	429.76	27.20
3730	368.00	244.00	429.19	26.51
3740	374.00	0.00	424.70	21.97
3750	378.00	156.00	424.15	20.00
3760	376.00	0.00	423.88	20.75
3770	375.00	0.00	423.62	21.07
3773	375.00	0.00	423.62	21.07
3780	374.00	0.00	424.56	21.91
3790	373.00	0.00	424.94	22.51
3800	372.00	0.00	426.43	23.58
3810	374.00	418.00	423.89	21.62
3820	374.00	0.00	424.52	21.89
3830	350.00	0.00	467.42	50.88
3840	351.00	0.00	465.73	49.71
3850	352.00	0.00	464.64	48.81
3860	352.00	158.00	464.41	48.71
3870	357.00	39.00	461.73	45.38
3880	358.00	0.00	460.41	44.37
3890	355.00	0.00	464.76	47.56
3900	352.00	0.00	466.61	49.66

Node Results: (continued)

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi
3910	352.00	52.00	466.24	49.50
3920	352.00	0.00	465.15	49.03
3930	355.00	304.00	462.82	46.72
3940	352.00	0.00	466.31	49.53
3950	355.00	0.00	464.06	47.25
3960	360.00	0.00	456.84	41.96
3970	360.00	0.00	456.38	41.76
3980	360.00	188.00	459.66	43.18
3990	358.00	150.00	450.26	39.98
4000	358.00	0.00	448.96	39.41
4010	358.00	0.00	448.82	39.35
4020	357.00	0.00	449.17	39.94
4030	357.00	144.00	449.97	40.28
4035	358.00	0.00	458.87	43.71
4040	355.00	0.00	457.73	44.51
4050	355.00	0.00	457.48	44.40

4060	356.00	94.50	451.29	41.29
4070	357.00	0.00	450.48	40.50
4080	356.00	218.00	450.19	40.81
4090	360.00	0.00	445.82	37.18
4100	360.00	0.00	444.78	36.73
4110	365.00	63.00	440.21	32.59
4120	365.00	0.00	439.64	32.34
4130	367.00	0.00	436.65	30.18
4140	365.00	0.00	436.56	31.01
4150	364.00	7.50	436.56	31.44
4160	364.00	0.00	436.54	31.43
4170	360.00	0.00	436.70	33.23
4180	360.00	0.00	446.90	37.65
4190	361.00	0.00	444.67	36.25
4200	363.00	230.00	438.47	32.70
4210	362.00	0.00	436.95	32.48
4250	367.00	0.00	439.59	31.45
4260	360.00	0.00	446.97	37.69
4270	360.00	0.00	446.83	37.62
4280	367.00	43.50	432.81	28.51
4290	368.00	217.50	431.83	27.66
4300	373.00	0.00	429.31	24.40
4310	371.00	0.00	429.04	25.15
4320	370.00	0.00	429.43	25.75
4330	370.00	0.00	429.80	25.91
4340	365.00	0.00	430.03	28.18
4350	370.00	0.00	430.03	26.01
4360	370.00	0.00	429.85	25.93
4380	368.00	2.00	436.18	29.54
4390	373.00	0.00	429.62	24.53

Node Results: (continued)

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi
4400	374.00	0.00	427.09	23.01
4410	376.00	0.00	425.81	21.58
4420	375.00	0.00	391.34	7.08
4425	0.00	0.00	425.81	184.50
4430	376.00	0.00	425.83	21.59
4440	375.00	250.00	425.83	22.02
4445	376.00	0.00	425.61	21.50
4450	375.00	0.00	425.97	22.08
4460	375.00	0.00	425.97	22.09
4470	375.00	0.00	426.06	22.12
4480	373.00	0.00	426.06	22.99
4490	371.00	396.00	426.96	24.25
4500	371.00	0.00	427.01	24.27

4510	371.00	18.00	427.94	24.67
4520	372.00	0.00	427.34	23.98
4530	372.00	132.00	426.91	23.79
4540	378.00	0.00	425.39	20.54
4550	377.00	0.00	425.27	20.92
4560	377.00	376.00	424.95	20.78
4570	376.00	0.00	424.95	21.21
4580	376.00	0.00	425.00	21.23
4590	375.00	0.00	425.87	22.04
4600	382.00	0.00	424.84	18.56
4610	382.00	0.00	424.76	18.53
4620	382.00	0.00	424.61	18.46
4630	353.00	0.00	465.76	48.86
4640	359.00	0.00	460.59	44.02
4650	360.00	0.00	461.56	44.01
4660	360.00	0.00	461.38	43.93
4670	360.00	126.00	461.09	43.80
4680	360.00	0.00	457.77	42.36
4685	363.00	6.00	453.66	39.28
4690	360.00	0.00	458.72	42.77
4700	360.00	130.00	458.47	42.67
4710	360.00	162.00	457.22	42.13
4720	362.00	112.50	450.24	38.23
4725	362.00	4.50	454.29	39.99
4730	365.00	0.00	443.70	34.10
4740	365.00	0.00	443.61	34.06
4750	366.00	0.00	443.61	33.63
4755	368.00	102.00	441.02	31.64
4760	367.00	338.00	440.82	31.99
4770	370.00	112.50	436.18	28.68
4780	369.00	9.00	436.08	29.06
4800	385.00	82.50	424.55	17.14
4810	390.00	0.00	579.52	82.12

Node Results: (continued)

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi
4820	385.00	0.00	584.30	86.36
4830	382.00	156.00	589.09	89.73
4831	382.00	0.00	422.79	17.67
4840	390.00	0.00	593.97	88.38
4850	388.00	826.50	596.28	90.25
4860	393.00	0.00	598.82	89.18
5000	352.00	174.00	463.27	48.21
5010	359.00	0.00	462.87	45.01
5020	362.00	0.00	462.54	43.57
5025	363.00	0.00	462.51	43.12

5030	358.00	133.50	462.21	45.16
5040	355.00	52.00	462.97	46.78
5050	356.00	178.00	462.91	46.32
5080	360.00	67.50	463.76	44.96
5090	360.00	0.00	463.25	44.74
5100	361.00	0.00	462.97	44.18
5110	361.00	0.00	462.83	44.12
5120	361.00	286.00	460.45	43.09
5130	361.00	0.00	460.46	43.10
5140	363.00	192.00	460.14	42.09
5150	365.00	0.00	460.10	41.21
5160	365.00	0.00	460.36	41.32
5170	365.00	152.00	460.50	41.38
5180	365.00	0.00	460.12	41.22
5190	365.00	0.00	459.94	41.14
5200	365.00	0.00	459.51	40.95
5210	365.00	76.00	452.05	37.72
5215	370.00	616.00	450.90	35.06
5220	375.00	471.20	450.81	32.85
5230	370.00	2.00	457.18	37.78
5240	368.00	70.00	459.04	39.45
5250	367.00	0.00	459.78	40.20
5300	368.00	0.00	456.67	38.42
5310	375.00	0.00	452.35	33.52
5400	375.00	0.00	445.64	30.61
5410	375.00	156.00	445.00	30.33
5420	375.00	0.00	443.20	29.55
5430	375.00	12.00	446.98	31.19
5440	375.00	0.00	443.45	29.66
5450	375.00	0.00	443.28	29.59
5500	362.00	404.00	453.85	39.80
5600	368.00	0.00	443.45	32.69
5610	368.00	0.00	443.53	32.73
5620	368.00	194.00	439.31	30.90
5630	368.00	0.00	439.31	30.90
5640	370.00	0.00	434.13	27.79

Node Results: (continued)

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi
5650	370.00	0.00	434.86	28.10
5660	370.00	0.00	434.88	28.11
5670	369.00	0.00	436.74	29.35
5680	372.00	206.00	432.19	26.08
5690	372.00	140.00	431.63	25.84
5700	370.00	0.00	435.93	28.57
5710	373.00	350.00	429.45	24.46

5720	373.00	37.50	429.63	24.54
5730	376.00	288.00	427.43	22.28
5740	378.00	0.00	421.82	18.99
5750	378.00	154.00	418.89	17.72
5751	378.00	0.00	392.05	6.09
5752	378.00	0.00	580.45	87.72
5754	378.00	0.00	418.84	17.70
5756	378.00	0.00	580.45	87.72
5758	378.00	0.00	616.69	103.42
5760	376.00	0.00	431.65	24.11
5770	373.00	0.00	431.86	25.50
5780	375.00	408.00	422.82	20.72
5800	380.00	135.00	575.55	84.73
5810	382.00	200.00	575.53	83.86
5820	383.00	118.00	575.59	83.45
5830	386.00	0.00	575.72	82.20
5840	385.00	0.00	574.94	82.30
5850	380.00	16.50	575.97	84.92
5860	374.00	280.00	428.74	23.72
5865	376.00	0.00	425.85	21.60
5866	376.00	0.00	576.45	86.85
5870	380.00	0.00	425.44	19.69
5880	381.00	246.00	424.72	18.94
5890	384.00	72.00	424.27	17.45
5900	385.00	230.00	424.05	16.92
5990	394.00	0.00	569.91	76.22
6000	391.00	0.00	573.85	79.23
6010	385.00	218.00	574.89	82.28
6020	385.00	0.00	575.24	82.43
6030	385.00	0.00	576.49	82.97
6040	385.00	0.00	706.42	139.27
6050	385.00	0.00	389.94	2.14
6100	398.00	790.40	563.76	71.83
6110	405.00	1054.50	557.48	66.07
6120	400.00	1563.70	568.44	72.99
6130	390.00	0.00	574.73	80.04
6140	400.00	659.30	568.77	73.13
6150	404.00	0.00	568.14	71.12
6160	410.00	1054.50	550.99	61.09

Node Results: (continued)

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi
6170	420.00	1054.50	547.37	55.19
6180	436.00	864.50	539.65	44.91
6182	436.00	0.00	471.93	15.57
6184	436.00	0.00	471.11	15.21

6186	436.00	0.00	539.65	44.91
6188	435.00	0.00	539.65	45.34
6190	435.00	0.00	539.65	45.34
6200	425.00	1024.10	547.89	53.25
6202	425.00	0.00	559.60	58.32
6204	425.00	0.00	559.59	58.32
6210	430.00	0.00	547.65	50.98
6220	430.00	2050.10	540.03	47.67
6230	425.00	0.00	539.92	49.79
6240	435.00	2050.10	537.42	44.38
6250	438.00	0.00	540.88	44.58
6260	445.00	0.00	538.52	40.52
6270	450.00	2050.10	537.29	37.82
6280	445.00	0.00	539.62	41.00
6290	440.00	2050.10	539.77	43.23
6300	455.00	539.00	539.60	36.66
7010	360.00	0.00	444.67	36.69
7020	358.00	-1200.00	459.38	43.93
7030	360.00	-945.00	437.79	33.71
7040	340.00	-1050.00	474.54	58.30
7050	354.00	-500.00	460.22	46.02
7060	348.00	0.00	473.85	54.53
7070	350.00	-2000.00	482.10	57.24
7080	363.00	-1495.00	459.34	41.74
7090	362.00	0.00	446.83	36.76
7100	344.00	-1000.00	476.20	57.28
7110	365.00	-1330.00	438.78	31.97
7120	355.00	-1030.00	461.88	46.31
7130	375.00	0.00	422.16	20.43
7140	360.00	-1160.00	461.95	44.17
7150	349.00	-1250.00	470.13	52.48
7160	360.00	-1000.00	431.55	31.00
7170	344.00	-1380.00	477.20	57.71
7180	345.00	-855.00	471.95	55.01
7190	345.00	0.00	463.82	51.48
7200	372.00	0.00	427.26	23.95
7210	356.00	-930.00	463.36	46.52
7220	370.00	-645.00	443.34	31.78
7230	367.00	-325.00	443.55	33.17
7240	370.00	-880.00	436.13	28.65
7250	364.00	-1200.00	457.36	40.45
7260	363.00	-1130.00	462.67	43.19

Node Results: (continued)

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi
7270	360.00	-1500.00	464.30	45.19

7280	348.00	-2180.00	478.12	56.38
7290	340.00	-780.00	476.31	59.06
7320	340.00	-1000.00	473.72	57.94
7330	350.00	-1000.00	470.08	52.03
7340	355.00	-1000.00	458.11	44.68
7350	350.00	-1000.00	466.87	50.64
7360	367.00	-1000.00	453.92	37.66
7370	350.00	-1000.00	466.01	50.27
7380	360.00	-1000.00	460.92	43.73
7390	365.00	-1000.00	460.35	41.32
7400	374.00	-1000.00	445.89	31.15
7410	370.00	-1000.00	456.92	37.66
7420	372.00	-1000.00	452.61	34.93
7430	368.00	-1000.00	452.92	36.80
7440	375.00	-1000.00	451.50	33.15
7450	376.00	-1000.00	450.07	32.09
7460	380.00	-1000.00	449.92	30.30
7470	385.00	0.00	446.78	26.77
7475	385.00	0.00	498.39	49.13
7480	375.00	0.00	496.88	52.81
7485	380.00	0.00	494.29	49.52
7490	370.00	0.00	496.90	54.99
7495	378.00	0.00	493.97	50.25
7510	345.00	-120.00	470.66	54.45
7520	346.00	-500.00	475.80	56.24
7530	348.00	-1030.00	467.77	51.89
7540	348.00	0.00	463.90	50.22
7550	350.00	-1025.00	457.79	46.71
7560	346.00	-700.00	478.69	57.49
7570	345.00	0.00	466.25	52.54
7580	345.00	-660.00	478.85	58.00
7701	390.00	0.00	599.41	90.74
7702	390.00	0.00	598.82	90.48
8000	360.00	471.20	460.66	43.62
8010	365.00	0.00	456.67	39.72
8020	368.00	860.70	452.66	36.69
8030	372.00	0.00	451.24	34.34
8040	378.00	860.70	449.81	31.12
8050	390.00	0.00	446.78	24.60
8060	390.00	860.70	445.25	23.94
8070	380.00	0.00	449.67	30.19
8080	375.00	0.00	451.24	33.04
8090	372.00	0.00	451.05	34.25
8100	376.00	860.70	444.18	29.54
8110	379.00	537.70	444.42	28.34

Node Results: (continued)

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi

8120	382.00	860.70	444.76	27.19	
8130	392.00	0.00	445.08	23.00	
8200	372.00	0.00	441.65	30.18	
8210	370.00	0.00	442.75	31.52	
8220	377.00	807.50	441.02	27.74	
8225	378.00	0.00	436.69	25.43	
8230	379.00	0.00	434.74	24.15	
8235	378.00	0.00	434.17	24.34	
8240	377.00	0.00	426.81	21.58	
8245	378.00	0.00	572.81	84.41	
9000	385.00	480.70	496.89	48.48	
9009	383.00	0.00	598.02	93.17	
9010	383.00	361.00	498.39	50.00	
9020	385.00	241.30	495.70	47.96	
9030	385.00	361.00	494.58	47.48	
9040	390.00	0.00	494.45	45.26	
9050	390.00	0.00	493.55	44.87	
9060	385.00	722.00	493.32	46.93	
9070	380.00	480.70	494.29	49.52	
9080	378.00	480.70	496.88	51.51	
9090	380.00	480.70	496.91	50.65	
9100	375.00	0.00	419.13	19.12	
9102	375.00	0.00	496.91	52.82	
9200	368.00	722.00	499.23	56.86	
9210	370.00	480.70	496.90	54.99	
9220	378.00	480.70	493.97	50.25	
9230	380.00	722.00	493.00	48.96	
9240	380.00	0.00	493.27	49.08	
7620	454.00	-1336.58	482.00	12.13	Tank
7630	370.00	2500.00	390.00	8.67	Tank
7640	380.00	2500.00	392.00	5.20	Tank
7650	450.00	4000.00	471.00	9.10	Tank
7700	600.00	-19503.68	600.00	0.00	Reservoir

Link Results:

Link	Start Node	End Node	Diameter in	Flow gpm	Velocity fps	Headloss /1000ft
1	7010	3365	10.00	-0.00	0.00	0.00
2	7020	4035	10.00	1200.00	4.90	10.07
3	7030	3375	10.00	945.00	3.86	6.48
4	7040	85	12.00	1050.00	2.98	3.24
5	7050	3260	10.00	500.00	2.04	1.9

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6	7060	3137	10.00	0.00	0.00	0.00

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Link Results: (continued)

Link	Start Node	End Node	Diameter in	Flow gpm	Velocity fps	Headloss /1000ft
7	7070	3100	12.00	2000.00	5.67	10.67
8	7080	4690	12.00	1495.00	4.24	6.23
9	7090	4270	10.00	-0.00	0.00	0.00
10	7100	255	12.00	1000.00	2.84	2.96
11	7110	2710	14.00	1330.00	2.77	2.37
12	7120	470	12.00	1030.00	2.92	3.12
14	7140	4650	12.00	1160.00	3.29	3.89
15	7150	3040	12.00	1250.00	3.55	4.47
16	7160	1160	12.00	1000.00	2.84	2.96
17	7170	2400	12.00	1380.00	3.91	5.37
20	7200	3575	10.00	-0.00	0.00	0.00
21	7210	5090	12.00	930.00	2.64	2.23
22	7220	5450	12.00	645.00	1.83	1.13
23	7230	5610	12.00	325.00	0.92	0.32
24	7240	5700	12.00	880.00	2.50	2.01
25	7250	5230	12.00	1200.00	3.40	3.57
26	7260	5025	12.00	1130.00	3.21	3.20
27	7270	5080	12.00	1500.00	4.26	5.40
28	7280	2450	12.00	2180.00	6.18	10.79

29	7290	325	12.00	780.00	2.21	1.6	1
32	7320	50	12.00	1000.00	2.84	2.5	5
33	7330	280	12.00	1000.00	2.84	2.5	5
34	7340	2585	12.00	1000.00	2.84	2.5	5
35	7350	3070	12.00	1000.00	2.84	2.5	5
36	7360	4685	12.00	1000.00	2.84	2.5	5
37	7370	4630	12.00	1000.00	2.84	2.5	5
38	7380	8000	12.00	1000.00	2.84	2.5	5
39	7390	5150	12.00	1000.00	2.84	2.5	5
40	7400	5400	12.00	1000.00	2.84	2.5	5
41	7410	5300	12.00	1000.00	2.84	2.5	5
42	7420	5310	12.00	1000.00	2.84	2.5	5
43	7430	8020	12.00	1000.00	2.84	2.5	5
44	7440	8080	12.00	1000.00	2.84	2.5	5
45	7450	8040	12.00	1000.00	2.84	2.5	5
46	7460	8070	12.00	1000.00	2.84	2.5	5
47	7470	8050	12.00	-0.00	0.00	0.0	0
48	7480	9080	12.00	-0.00	0.00	0.0	0
49	7490	9210	12.00	-0.00	0.00	0.0	0
50	7475	9010	12.00	0.00	0.00	0.0	0
51	7485	9070	12.00	0.00	0.00	0.0	0
52	7495	9220	12.00	0.00	0.00	0.0	0
54	7650	6184	24.00	-4000.00	2.84	1.1	3
55	50	60	12.00	212.87	0.60	0.1	5
60	60	70	10.00	21.95	0.09	0.0	1
70	70	90	10.00	21.95	0.09	0.0	1
80	80	90	8.00	-21.95	0.14	0.0	2

Link Results: (continued)

Link	Start Node	End Node	Diameter in	Flow gpm	Velocity fps	Headloss /1000ft
90	85	80	12.00	170.63	0.48	0.10
100	80	100	12.00	192.59	0.55	0.12
110	100	60	12.00	-144.92	0.41	0.07
120	100	110	12.00	300.01	0.85	0.28
130	50	110	12.00	473.07	1.34	0.64
140	110	120	12.00	741.58	2.10	1.47
150	120	130	12.00	-603.07	1.71	1.00
160	130	85	12.00	-879.37	2.49	2.01
161	880	130	12.00	-276.30	0.78	0.24
170	120	140	12.00	323.53	0.92	0.43
180	140	150	8.00	62.52	0.40	0.15
190	150	160	8.00	62.52	0.40	0.15
200	160	170	8.00	-271.67	1.73	2.25
210	170	50	12.00	-150.88	0.43	0.11
215	245	50	12.00	-163.18	0.46	0.12
220	170	180	12.00	-120.80	0.34	0.07
230	180	190	8.00	106.67	0.68	0.40
240	190	245	8.00	-92.90	0.59	0.31
250	190	200	8.00	-92.91	0.59	0.31
255	235	245	12.00	-70.28	0.20	0.03
260	200	235	8.00	29.14	0.19	0.04

270	200	210	8.00	89.73	0.57	0.2	9
275	225	235	8.00	-99.42	0.63	0.3	5
280	225	210	8.00	-66.58	0.42	0.1	7
290	210	220	6.00	-102.86	1.17	1.5	1
300	220	230	8.00	-70.88	0.45	0.1	9
310	230	200	8.00	211.77	1.35	1.4	2
320	230	240	12.00	444.82	1.26	0.9	3
330	240	190	6.00	73.52	0.83	0.8	1
340	240	250	12.00	170.88	0.48	0.1	3
350	250	180	12.00	255.97	0.73	0.2	8
360	250	260	12.00	863.11	2.45	2.6	5
370	260	160	8.00	-10.19	0.07	0.0	1
380	260	270	12.00	958.83	2.72	3.2	2
390	270	140	8.00	-261.01	1.67	2.0	9
400	270	280	8.00	-275.81	1.76	2.7	6
410	290	280	8.00	134.01	0.86	0.7	2
420	260	290	8.00	-85.53	0.55	0.3	2
430	290	300	10.00	-571.08	2.33	3.5	7
440	255	300	12.00	-63.70	0.18	0.0	3
445	255	250	12.00	1063.69	3.02	4.6	5
450	300	310	10.00	-47.43	0.19	0.0	4
460	240	310	6.00	-79.59	0.90	1.1	2
470	310	320	10.00	-204.57	0.84	0.5	3
480	320	325	12.00	-30.03	0.09	0.0	1
485	325	230	12.00	749.97	2.13	2.4	3

Link Results: (continued)

Link	Start Node	End Node	Diameter in	Flow gpm	Velocity fps	Headloss /1000ft
490	320	330	6.00	32.79	0.37	0.22
500	220	330	6.00	-61.98	0.70	0.71
510	330	340	6.00	-29.19	0.33	0.18
520	340	350	8.00	-78.69	0.50	0.27
530	350	320	12.00	207.32	0.59	0.23
540	350	355	8.00	8.86	0.06	0.00
545	355	360	8.00	143.69	0.92	0.82
546	355	2430	8.00	-134.82	0.86	0.73
550	360	310	8.00	115.95	0.74	0.55
560	2430	370	8.00	32.06	0.20	0.05
570	370	300	12.00	440.19	1.25	0.76
572	370	305	8.00	119.42	0.76	0.59
574	305	300	8.00	147.16	0.94	0.86
576	360	305	8.00	27.74	0.18	0.03
580	370	375	10.00	702.73	2.87	4.40
583	375	380	10.00	702.73	2.87	4.40
585	2450	370	12.00	537.00	1.52	0.81
590	380	290	12.00	-117.55	0.33	0.07
600	380	390	12.00	995.11	2.82	4.11
610	390	280	8.00	-375.61	2.40	3.48
620	2545	390	12.00	-960.91	2.73	3.23
625	400	2545	12.00	-866.96	2.46	2.67
630	280	410	8.00	482.59	3.08	7.76

640	410	420	8.00	293.66	1.87	3.1	0
650	420	400	8.00	4.55	0.03	0.0	0
660	400	430	12.00	833.32	2.36	2.4	8
665	2570	430	12.00	-180.06	0.51	0.1	5
670	430	440	12.00	-180.71	0.51	0.1	5
675	430	2585	12.00	310.08	0.88	0.4	0
680	420	440	6.00	119.61	1.36	2.3	8
690	410	450	6.00	-19.07	0.22	0.0	8
700	450	460	12.00	712.99	2.02	1.5	8
710	460	470	12.00	406.70	1.15	0.5	6
720	505	500	6.00	-0.00	0.00	0.0	0
725	7540	505	8.00	-0.00	0.00	0.0	0
730	500	510	4.00	-82.33	2.10	8.6	1
740	510	520	4.00	-53.97	1.38	3.9	4
745	270	460	12.00	763.60	2.17	1.5	5
750	520	530	6.00	-53.97	0.61	0.5	5
760	530	540	6.00	234.80	2.66	8.3	1
770	530	550	12.00	-288.77	0.82	0.4	2
775	7520	550	8.00	500.00	3.19	8.2	8
780	550	560	12.00	211.23	0.60	0.2	3
785	7560	565	10.00	700.00	2.86	5.2	1
790	500	540	6.00	82.33	0.93	1.2	0
810	7510	570	8.00	120.00	0.77	0.5	9

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Link Results: (continued)

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	Start	End	Diameter	Flow	Velocity	Headlos

st-2767615703625548889335678

Link	Node	Node	in	gpm	fps	/1000f
820	570	580	12.00	248.48	0.70	0.3
830	580	540	6.00	220.12	2.50	7.3
840	120	600	12.00	-369.89	1.05	0.6
850	570	600	8.00	-128.48	0.82	0.6
860	600	610	8.00	-561.27	3.58	10.2
870	610	620	8.00	98.73	0.63	0.4
880	7580	610	8.00	660.00	4.21	13.8
890	580	630	6.00	28.36	0.32	0.1
900	630	510	4.00	28.36	0.72	1.2
910	600	640	6.00	62.91	0.71	0.7
920	640	650	6.00	43.24	0.49	0.3
930	650	620	6.00	-214.33	2.43	7.0
940	620	565	8.00	-115.61	0.74	0.5
945	565	560	8.00	584.39	3.73	11.0
950	560	660	8.00	406.23	2.59	5.6
960	650	660	8.00	-249.20	1.59	2.2
970	640	670	6.00	19.67	0.22	0.0
980	670	680	6.00	19.67	0.22	0.0
990	650	680	6.00	-30.48	0.35	0.1
1000	680	690	6.00	-10.81	0.12	0.0
1010	690	700	6.00	-10.81	0.12	0.0
1020	700	660	6.00	-157.03	1.78	3.9
1030	700	710	6.00	146.22	1.66	3.4
1040	560	710	6.00	389.39	4.42	21.1
1050	710	715	10.00	478.61	1.96	2.5

1052	715	7570	10.00	-0.00	0.00	0.0	0
1055	715	720	10.00	478.61	1.96	2.5	8
1060	720	730	10.00	770.91	3.15	6.2	3
1070	730	740	10.00	459.14	1.88	2.3	9
1080	740	755	8.00	294.35	1.88	3.1	1
1090	720	760	10.00	-292.30	1.19	1.0	4
1100	760	770	10.00	737.70	3.01	5.7	4
1110	770	740	12.00	-164.79	0.47	0.1	5
1120	770	780	10.00	639.98	2.61	4.4	1
1130	7530	760	12.00	1030.00	2.92	4.3	8
1140	7550	780	12.00	1025.00	2.91	4.3	4
1150	730	790	6.00	311.78	3.54	14.0	4
1160	790	805	6.00	-158.97	1.80	4.0	4
1170	750	800	12.00	-330.03	0.94	0.3	8
1175	755	805	6.00	158.97	1.80	2.8	8
1180	800	810	12.00	-330.03	0.94	0.3	8
1190	810	820	12.00	-737.30	2.09	1.6	8
1200	820	830	12.00	-699.55	1.98	1.5	3
1210	830	840	12.00	-864.30	2.45	2.2	6
1220	840	850	12.00	-864.30	2.45	2.2	6
1230	850	860	8.00	-28.53	0.18	0.0	3

Link Results: (continued)

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	Start	End	Diameter	Flow	Velocity	Headlos
						s
Link	Node	Node	in	gpm	fps	/1000f
						t

-							
	1240	860	870	8.00	-210.53	1.34	1.1
9							
	1250	850	870	12.00	-835.77	2.37	2.1
2							
	1260	870	880	12.00	-1131.30	3.21	3.7
2							
	1270	7180	880	12.00	855.00	2.43	2.2
1							
	1280	7190	840	12.00	-0.00	0.00	0.0
0							
	1290	820	890	12.00	-37.75	0.11	0.0
1							
	1300	830	890	8.00	164.75	1.05	0.7
6							
	1310	810	900	8.00	407.27	2.60	4.0
5							
	1320	900	910	8.00	-327.75	2.09	2.7
1							
	1330	750	910	12.00	465.40	1.32	0.7
2							
	1335	755	750	8.00	135.38	0.86	0.5
3							
	1340	910	920	12.00	1528.66	4.34	6.4
9							
	1350	920	930	12.00	1542.14	4.37	6.5
9							
	1360	930	940	12.00	-467.02	1.32	0.7
2							
	1370	900	940	8.00	467.02	2.98	5.2
1							
	1380	780	950	12.00	1664.98	4.72	7.6
0							
	1390	950	960	12.00	859.62	2.44	2.2
4							
	1400	960	920	12.00	13.48	0.04	0.0
0							
	1410	960	970	8.00	482.13	3.08	5.5
3							
	1420	930	980	12.00	537.83	1.53	0.9
4							
	1430	980	970	12.00	79.46	0.23	0.0
3							
	1440	970	990	12.00	561.59	1.59	1.0
2							
	1450	470	1000	12.00	1328.70	3.77	5.0
1							
	1460	1000	1010	12.00	1348.87	3.83	5.1
5							
	1470	950	1000	12.00	805.37	2.28	1.9
8							
	1480	1010	990	12.00	1074.29	3.05	3.3
8							
	1490	1000	1020	12.00	785.20	2.23	1.8
9							

1500	1040	440	12.00	61.11	0.17	0.02
1510	1040	460	12.00	-1069.88	3.04	3.35
1520	1040	1050	12.00	1001.28	2.84	2.97
1530	1050	1060	12.00	851.28	2.41	2.20
1540	2585	2625	12.00	1307.72	3.71	4.86
1542	430	2585	12.00	338.27	0.96	0.40
1545	2625	1070	12.00	1057.69	3.00	3.28
1550	1070	1060	12.00	-1084.71	3.08	3.44
1560	1060	1020	12.00	-338.43	0.96	0.40
1570	1020	1030	8.00	446.77	2.85	4.80
1580	1030	1010	8.00	-216.07	1.38	1.25
1590	1030	1080	8.00	342.85	2.19	2.94
1600	1080	990	12.00	-448.90	1.27	0.67
1610	1080	1090	12.00	791.75	2.25	1.92
1620	1090	1100	12.00	393.06	1.12	0.53
1630	1070	2690	12.00	1230.29	3.49	4.34
1635	2690	1100	12.00	901.48	2.56	2.44
1640	1100	1110	12.00	1148.41	3.26	3.82
1650	1110	1120	12.00	-324.12	0.92	0.37

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Link Results: (continued)

Link	Start Node	End Node	Diameter in	Flow gpm	Velocity fps	Headloss /1000ft
1660	1090	1120	8.00	359.69	2.30	3.21
1670	990	1130	12.00	1180.98	3.35	4.0

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1680	1130	1140	12.00	1223.95	3.47	4.3
1690	1140	1120	12.00	394.43	1.12	0.5
1700	980	1150	8.00	458.38	2.93	5.0
1710	1150	1130	8.00	42.98	0.27	0.0
1720	1150	1160	8.00	239.40	1.53	1.5
1730	1140	1160	12.00	-550.53	1.56	0.9
1740	930	1170	12.00	1317.33	3.74	4.9
1750	1170	1180	12.00	1427.71	4.05	5.7
1760	1160	1170	12.00	655.87	1.86	1.3
1770	1170	1200	8.00	391.49	2.50	3.2
1780	1200	1180	8.00	391.49	2.50	3.2
1790	1180	1190	12.00	1665.20	4.72	6.5
1800	1190	1210	12.00	387.70	1.10	0.4
1810	1190	1260	12.00	1277.51	3.62	4.0
1820	1110	2830	12.00	992.75	2.82	2.5
1822	2830	1230	12.00	889.85	2.52	2.0
1830	1140	1240	12.00	1246.05	3.53	3.8
1840	1240	1250	12.00	1068.05	3.03	2.8
1850	1230	1250	12.00	553.90	1.57	0.8
1860	1250	1210	12.00	530.79	1.51	0.7
1870	7130	1240	12.00	-0.00	0.00	0.0
1880	1210	1260	12.00	918.49	2.61	2.1
1900	1250	9100	12.00	379.16	1.08	0.4
1905	9100	9102	12.00	0.00		Close
1910	9102	9090	12.00	-0.00	0.00	0.0
1920	9090	9200	12.00	-562.20	1.59	0.8
1930	1222	9200	16.00	2484.73	3.96	3.3

1940	1220	1260	16.00	-1503.99	2.40	1.34
2000	350	2400	12.00	-344.38	0.98	0.48
2010	2400	3000	12.00	706.74	2.00	1.83
2020	370	2450	8.00	-156.27	1.00	0.81
2025	370	2450	12.00	-537.00	1.52	0.81
2030	2450	3140	8.00	161.87	1.03	0.86
2035	2450	3115	12.00	539.85	1.53	0.82
2040	3140	3120	8.00	40.94	0.26	0.07
2050	2500	380	12.00	207.83	0.59	0.19
2060	2500	3110	12.00	-803.19	2.28	2.32
2070	2500	2510	8.00	224.86	1.44	1.58
2080	2510	2520	6.00	224.86	2.55	6.43
2090	390	2520	12.00	234.43	0.67	0.24
2100	2520	3280	12.00	459.30	1.30	0.82
2110	390	2530	8.00	175.37	1.12	1.19
2120	2530	3280	8.00	-77.39	0.49	0.26
2130	2530	2540	8.00	252.76	1.61	2.35

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Link Results: (continued)

Link	Start Node	End Node	Diameter in	Flow gpm	Velocity fps	Headloss /1000ft
2140	2540	3270	6.00	64.76	0.73	0.77
2150	2545	2550	8.00	93.95	0.60	0.38
2160	400	2550	8.00	20.19	0.13	0.02
2170	2550	3260	8.00	114.14	0.73	0.5

4	2180	2570	2560	8.00	180.06	1.15	1.2
5	2190	2560	3460	8.00	130.97	0.84	0.6
9	2200	2560	2580	6.00	-4.91	0.06	0.0
1	2210	430	2580	8.00	185.62	1.18	1.3
2	2220	2580	3450	8.00	180.72	1.15	1.2
6	2230	2585	2600	8.00	340.63	2.17	4.0
7	2240	2600	2590	8.00	-52.49	0.34	0.1
3	2250	2590	3440	8.00	-250.41	1.60	2.3
0	2260	2590	2595	8.00	197.91	1.26	1.4
9	2270	2595	3430	8.00	-121.03	0.77	0.6
0	2280	2595	2630	8.00	318.94	2.04	3.6
1	2290	2630	2640	8.00	404.40	2.58	5.5
9	2300	2610	2630	8.00	85.46	0.55	0.3
2	2305	2600	2610	8.00	286.62	1.83	2.4
8	2310	2610	2620	8.00	78.16	0.50	0.2
7	2320	2625	2620	8.00	250.03	1.60	2.3
0	2340	2620	2660	8.00	328.19	2.09	3.8
0	2350	1070	2660	12.00	837.11	2.37	2.5
0	2360	2660	2650	12.00	1165.30	3.31	4.6
1	2370	2650	2640	12.00	831.41	2.36	2.4
7	2380	2640	3420	12.00	1235.81	3.51	5.1
4	2390	2650	2670	8.00	333.90	2.13	3.2
9	2400	2690	2680	8.00	328.80	2.10	3.2
0	2403	2400	2420	10.00	166.88	0.68	0.3
1	2405	2410	2420	10.00	0.00	0.00	0.0
0	2407	2420	2430	12.00	166.88	0.47	0.1
3	2410	2680	2670	8.00	181.59	1.16	1.0

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2420	2670	3610	8.00	215.49	1.38	1.46
2430	2680	2700	8.00	147.21	0.94	0.72
2431	2700	2701	8.00	107.14	0.68	0.40
2440	2700	3600	12.00	958.79	2.72	2.74
2450	2700	2710	12.00	-918.72	2.61	2.53
2460	2710	1100	12.00	-146.14	0.41	0.08
2470	2701	2720	6.00	210.68	2.39	4.85
2471	2710	2701	6.00	103.54	1.17	1.30
2480	2720	3600	8.00	-72.83	0.46	0.17
2490	2720	2740	8.00	283.51	1.81	2.07
2500	2710	2750	8.00	453.88	2.90	4.94
2510	2750	2730	8.00	-62.32	0.40	0.13
2520	2730	2740	8.00	-314.40	2.01	2.51
2530	2740	3590	8.00	-30.89	0.20	0.03
2540	2730	2770	8.00	252.09	1.61	1.67

Link Results: (continued)

Link	Start Node	End Node	Diameter in	Flow gpm	Velocity fps	Headloss /1000ft
2550	2750	2780	8.00	210.19	1.34	1.19
2560	2750	2760	8.00	-0.00	0.00	0.00
2570	1110	2790	12.00	442.28	1.25	0.65
2580	2790	2780	12.00	87.45	0.25	0.03
2590	2780	2770	12.00	297.65	0.84	0.31
2600	2770	3580	12.00	268.72	0.76	0.2

6	2700	2770	2800	8.00	281.02	1.79	2.0
4	2710	2800	3800	8.00	67.57	0.43	0.1
5	2720	2800	2810	8.00	213.44	1.36	1.2
2	2730	2790	2810	8.00	354.83	2.26	3.1
3	2740	2810	2820	8.00	150.27	0.96	0.6
4	2750	2820	2830	8.00	-2.40	0.02	0.0
0	2760	2820	2840	8.00	152.67	0.97	0.6
6	2770	2840	3790	8.00	-101.46	0.65	0.3
1	2780	2840	2850	8.00	254.13	1.62	1.6
9	2800	1230	2850	12.00	-277.51	0.79	0.2
8	2810	2850	3770	12.00	-443.75	1.26	0.5
7	2820	2850	2860	12.00	420.38	1.19	0.5
1	2830	2860	2870	12.00	-332.07	0.94	0.3
3	2840	3770	2870	12.00	506.43	1.44	0.7
2	2850	2870	2880	12.00	174.36	0.49	0.1
0	2860	2860	2890	12.00	60.45	0.17	0.0
1	2870	2890	2880	12.00	-288.17	0.82	0.2
6	3000	3000	3010	12.00	801.81	2.27	1.7
0	3010	3010	3020	12.00	644.31	1.83	1.1
3	3020	3020	3030	12.00	644.31	1.83	1.3
1	3030	3030	3040	12.00	-138.57	0.39	0.0
8	3040	3040	3050	12.00	1111.43	3.15	3.6
0	3050	3050	3060	12.00	805.66	2.29	1.9
8	3055	3055	3060	12.00	-0.01	0.00	0.0
0	3056	3055	3060	8.00	0.01	0.00	0.0
0	3060	3060	3070	12.00	242.98	0.69	0.2
2	3065	3060	3070	8.00	76.60	0.49	0.2
2							

	3070	3070	3080	12.00	-1163.22	3.30	4.6
0							
	3080	3080	3090	12.00	-1989.37	5.64	12.4
1							
	3090	3090	3100	12.00	-3336.57	9.47	32.2
9							
	3092	3101	7620	12.00	-1336.58	3.79	4.3
6							
	3093	3102	3100	12.00	-0.00	0.00	0.0
0 CV							
	3097	3104	3100	12.00	1336.58	3.79	4.3
6							
	3100	3110	3090	12.00	-1347.20	3.82	6.0
3							
	3110	3120	3115	10.00	-160.19	0.65	0.2
9							
	3112	3115	3110	10.00	379.65	1.55	1.4
1							
	3120	3120	3130	10.00	322.07	1.32	1.0
4							
	3130	3130	3135	10.00	151.07	0.62	0.2
6							
	3132	3135	3000	12.00	151.07	0.43	0.0
9							
	3134	3135	3136	10.00	-0.00	0.00	0.0
0							

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Link Results: (continued)

	Link	Start Node	End Node	Diameter in	Flow gpm	Velocity fps	Headlos /1000f
-st-							
	3136	3136	3137	10.00	-0.00	0.00	0.0
0							
	3140	3120	3140	12.00	-120.93	0.34	0.0
7							
	3150	3070	3150	12.00	1342.84	3.81	6.0
0							
	3160	3150	3160	12.00	950.97	2.70	3.1
7							
	3170	3160	3170	12.00	-466.55	1.32	0.8
5							
	3180	3170	3180	12.00	-466.55	1.32	0.8
5							
	3190	3180	3190	12.00	-541.78	1.54	1.1
2							
	3200	3190	3200	12.00	-587.92	1.67	1.3

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3240	3200	3260	12.00	-1874.52	5.32	9.4
3250	3260	3270	12.00	-396.60	1.13	0.6
3255	3270	3260	8.00	105.54	0.67	0.3
3260	3260	3280	12.00	-907.89	2.58	2.9
3265	3280	3270	8.00	437.39	2.79	5.4
3270	3280	3110	10.00	-923.66	3.77	7.2
3280	3150	3290	6.00	20.66	0.23	0.0
3290	3290	3300	6.00	-94.06	1.07	1.5
3300	3300	3310	6.00	-368.95	4.19	19.1
3310	3310	3280	6.00	39.71	0.45	0.3
3320	3310	3080	8.00	-458.65	2.93	7.0
3330	3290	3320	6.00	114.72	1.30	2.2
3340	3320	3180	8.00	114.72	0.73	0.5
3350	3300	3330	6.00	86.89	0.99	1.3
3360	3330	3260	6.00	-149.66	1.70	3.6
3370	3330	3190	8.00	236.54	1.51	1.4
3380	3160	3340	12.00	1417.52	4.02	5.6
3390	3340	3350	12.00	1345.81	3.82	5.1
3400	3350	3360	12.00	1345.81	3.82	5.1
3402	3360	3365	6.00	43.93	0.50	0.3
3405	3365	3475	6.00	43.93	0.50	0.3
3406	3475	3485	6.00	-102.28	1.16	1.7
3407	3475	3395	6.00	146.21	1.66	3.4
3410	3360	3370	12.00	1181.32	3.35	4.7
3420	3370	3375	12.00	-138.80	0.39	0.0
3425	3375	3380	12.00	599.00	1.70	1.3
3430	3380	3390	12.00	-218.35	0.62	0.2

3440	3390	3395	12.00	-291.06	0.83	0.3	5
3445	3395	3415	12.00	-144.86	0.41	0.1	0
3450	3415	3410	12.00	73.64	0.21	0.0	3
3460	3410	3420	18.00	-1364.82	1.72	0.6	3
3470	3420	3430	10.00	-1045.71	4.27	9.1	7
3475	3400	3420	10.00	-54.00	0.22	0.0	4
3480	3430	3440	8.00	-369.10	2.36	3.3	7
3485	3440	3435	12.00	977.64	2.77	3.3	3
3486	3435	3430	12.00	977.64	2.77	3.3	3
3490	3440	3450	12.00	-955.66	2.71	2.7	2
3500	3450	3460	12.00	-774.94	2.20	1.8	5

Link Results: (continued)

Link	Start Node	End Node	Diameter in	Flow gpm	Velocity fps	Headlos /1000f	s t
3505	3460	3440	12.00	748.68	2.12	2.0	3
3510	3460	3200	12.00	-1286.60	3.65	5.5	4
3520	3180	3470	8.00	36.95	0.24	0.0	5
3530	3470	3480	6.00	213.59	2.42	5.8	4
3535	3480	3485	6.00	320.77	3.64	12.4	0
3540	3485	3415	6.00	218.50	2.48	6.1	0
3550	3480	3440	6.00	-107.18	1.22	1.6	3
3560	3190	3490	8.00	282.69	1.80	2.0	6
3570	3490	3470	8.00	176.64	1.13	0.8	6
3580	3490	3460	8.00	106.05	0.68	0.3	

4							
4	3590	3380	3500	12.00	817.35	2.32	2.0
3	3600	3500	3510	12.00	354.64	1.01	0.4
2	3610	3510	3520	12.00	645.33	1.83	1.3
4	3620	3520	3530	12.00	839.66	2.38	2.1
2	3630	3530	3540	12.00	916.25	2.60	2.5
4	3640	3540	3550	12.00	703.31	2.00	1.5
5	3650	3550	3560	12.00	-359.60	1.02	0.4
0	3660	3560	3570	12.00	-159.27	0.45	0.1
0	3665	3570	3575	12.00	-423.19	1.20	0.6
0	3670	3575	3580	12.00	-423.19	1.20	0.6
9	3680	3580	3590	12.00	-910.88	2.58	2.4
5	3690	3590	3600	12.00	-941.77	2.67	2.6
0	3693	3600	3605	12.00	412.52	1.17	0.5
5	3695	3605	3700	12.00	820.10	2.33	2.0
3	3700	3600	3610	12.00	-468.33	1.33	0.7
3	3710	3610	3620	12.00	-252.84	0.72	0.2
7	3720	3620	3630	12.00	-660.42	1.87	1.3
5	3730	3630	3420	12.00	-862.70	2.45	2.2
5	3740	3390	3640	6.00	72.71	0.83	0.9
4	3745	3640	4170	18.00	809.19	1.02	0.2
0	3750	3640	3650	8.00	340.07	2.17	2.9
7	3760	3650	3510	8.00	290.68	1.86	2.1
8	3770	3640	3660	18.00	-1178.56	1.49	0.4
9	3780	3660	3410	18.00	-1438.46	1.81	0.6
8	3790	3650	3670	8.00	49.39	0.32	0.0
1	3800	3670	3680	8.00	-102.19	0.65	0.3
1	3810	3680	3630	8.00	-202.28	1.29	1.1

3820	3660	3680	8.00	259.90	1.66	1.76
3830	3520	3690	12.00	-450.76	1.28	0.68
3840	3690	3700	12.00	-722.15	2.05	1.62
3860	3620	3605	12.00	407.58	1.16	0.56
3870	3670	3700	8.00	151.57	0.97	0.65
3880	3690	3720	8.00	271.39	1.73	1.91
3890	3720	3730	8.00	194.80	1.24	1.03
3900	3700	3730	8.00	249.53	1.59	1.63
3910	3730	3560	8.00	200.32	1.28	1.09

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Link Results: (continued)

Link	Start Node	End Node	Diameter in	Flow gpm	Velocity fps	Headloss /1000ft
3920	3550	3740	12.00	612.90	1.74	1.03
3930	3740	3750	12.00	476.57	1.35	0.65
3940	3750	3760	12.00	245.42	0.70	0.19
3950	3760	3770	12.00	265.13	0.75	0.22
3960	3770	3780	12.00	-685.06	1.94	1.27
3970	3780	3790	12.00	-722.52	2.05	1.40
3980	3790	3800	12.00	-823.97	2.34	1.78
3990	3800	3580	12.00	-756.40	2.15	1.52
4000	3740	3810	8.00	136.33	0.87	0.46
4010	3810	3820	8.00	-301.38	1.92	2.00
4020	3820	3780	8.00	-37.46	0.24	0.04
4030	3570	3820	8.00	263.92	1.68	1.5

6							
	4040	3810	3760	8.00	19.71	0.13	0.0
1							
	4050	3030	3830	12.00	782.87	2.22	1.8
8							
	4060	3830	3840	12.00	559.31	1.59	1.0
1							
	4070	3840	3850	12.00	632.31	1.79	1.2
7							
	4080	3850	3860	12.00	737.34	2.09	1.6
8							
	4090	3860	3870	12.00	714.12	2.03	1.5
9							
	4100	3870	3880	12.00	818.09	2.32	2.0
4							
	4110	3880	3890	8.00	-291.41	1.86	2.1
8							
	4120	3890	3060	8.00	-366.07	2.34	3.3
2							
	4130	3050	3900	8.00	305.78	1.95	2.3
8							
	4140	3900	3910	8.00	140.49	0.90	0.5
6							
	4150	3910	3830	8.00	-223.56	1.43	1.3
3							
	4160	3910	3920	8.00	312.05	1.99	2.4
7							
	4170	3920	3850	8.00	105.03	0.67	0.3
3							
	4180	3920	3930	8.00	207.02	1.32	1.1
6							
	4190	3930	3870	8.00	142.97	0.91	0.5
8							
	4200	3900	3940	8.00	165.29	1.06	0.7
6							
	4210	3940	3950	6.00	165.29	1.88	3.1
0							
	4220	3950	3890	6.00	-74.66	0.85	0.7
1							
	4230	3950	3930	8.00	239.95	1.53	1.5
2							
	4240	3880	3960	12.00	1417.80	4.02	5.6
4							
	4250	3960	3970	18.00	1417.80	1.79	0.6
8							
	4260	3970	3980	12.00	-936.97	2.66	3.0
8							
	4270	3980	3070	12.00	-1124.97	3.19	4.3
2							
	4280	3970	3990	18.00	3068.20	3.87	2.8
2							
	4290	3990	4000	18.00	3012.62	3.80	2.7
2							
	4300	4000	4010	12.00	298.18	0.85	0.3
2							

4310	4010	4020	12.00	-470.19	1.33	0.73
4320	4020	4030	12.00	-749.11	2.13	1.73
4330	4030	3340	12.00	-946.76	2.69	2.67
4340	3340	4035	8.00	-875.05	5.59	19.56
4345	4035	4040	8.00	324.95	2.07	3.13
4350	4040	4050	8.00	103.56	0.66	0.38
4360	3150	4050	8.00	363.70	2.32	3.85

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Link Results: (continued)

Link	Start Node	End Node	Diameter in	Flow gpm	Velocity fps	Headloss /1000ft
4370	4050	4060	8.00	467.27	2.98	6.13
4380	4060	4070	8.00	141.29	0.90	0.67
4390	4060	4080	8.00	231.48	1.48	1.67
4400	4080	4030	12.00	281.73	0.80	0.33
4410	4040	4080	6.00	221.39	2.51	7.45
4420	4080	4070	6.00	-46.86	0.53	0.42
4430	4070	3990	8.00	94.42	0.60	0.32
4440	4000	4090	18.00	3076.04	3.88	2.83
4450	4090	4100	18.00	4009.16	5.05	4.62
4460	4100	4110	18.00	3508.44	4.42	3.61
4470	4110	4120	18.00	3339.26	4.21	3.30
4480	4120	4130	18.00	2862.95	3.61	2.48
4481	4120	4130	8.00	261.30	1.67	2.49
4490	4130	4140	10.00	138.92	0.57	0.2

2	4491	4130	4140	12.00	265.31	0.75	0.2
2	4500	4140	4150	10.00	13.13	0.05	0.0
0	4501	4140	4150	12.00	25.08	0.07	0.0
0	4510	4150	4160	8.00	62.33	0.40	0.1
5	4511	4150	4160	12.00	214.18	0.61	0.1
5	4520	4160	4170	18.00	-756.30	0.95	0.2
1	4521	4160	4170	12.00	-260.10	0.74	0.2
1	4530	4170	3375	8.00	-207.20	1.32	1.1
6	4540	4030	4180	8.00	335.38	2.14	2.8
2	4550	4180	4190	8.00	449.54	2.87	4.8
6	4560	4190	4200	8.00	465.10	2.97	5.1
7	4570	4200	4150	8.00	245.79	1.57	1.5
9	4580	3370	4210	18.00	1320.12	1.66	0.6
9	4590	4210	4160	18.00	1524.45	1.92	0.9
0	4620	4210	4200	8.00	-204.33	1.30	1.1
3	4630	4200	4120	8.90	-215.01	1.11	0.8
7	4640	4110	4250	8.00	106.18	0.68	0.3
4	4650	4190	3360	6.00	-15.56	0.18	0.0
5	4660	4020	4260	8.00	278.92	1.78	2.0
1	4670	4010	4270	12.00	768.38	2.18	1.8
2	4680	4180	4260	10.00	-114.17	0.47	0.1
5	4690	4260	4270	10.00	164.75	0.67	0.3
0	4700	4270	4090	12.00	933.13	2.65	2.6
0	4710	4160	4280	18.00	2817.35	3.55	2.4
1	4720	4280	4290	18.00	3139.87	3.96	2.9
4	4730	4290	4300	12.00	789.83	2.24	1.6
5	4740	4300	4310	12.00	372.06	1.06	0.4
8							

```
     4750     4310     4320      12.00     -424.41        1.20         0.6
1
     4760     4320     4330      12.00     -652.08        1.85         1.3
4
     4770     4330     4340      12.00     -256.43        0.73         0.2
4
     4780     4340     3520       8.00     -256.43        1.64         1.7
2
     4790     4340     4350      12.00        0.00        0.00         0.0
0
```



```
  Link Results: (continued)
  ---------------------------------------------------------------
-
              Start      End  Diameter       Flow  Velocity  Headlos
s
      Link     Node     Node        in        gpm       fps   /1000f
t
  ---------------------------------------------------------------
-
     4800     3500     4360       8.00      218.71        1.40         1.2
8
     4810     4360     4330      12.00      395.65        1.12         0.5
3
     4820     4360     4290       8.00     -176.94        1.13         0.8
7
     4830     4140     4280       8.00      366.02        2.34         3.3
2
     4840     4130     4380      18.00     2023.95        2.55         1.3
1
     4841     4130     4380      12.00      696.07        1.97         1.3
1
     4850     4380     4290      18.00     2739.75        3.45         2.2
9
     4860     4290     4390      24.00     4695.35        3.33         1.5
3
     4865     4390     4300      24.00     4695.35        3.33         1.5
3
     4870     4300     4400      24.00     4813.34        3.41         1.6
0
     4880     4400     4410      24.00     4694.67        3.33         1.5
2
     4890     7630     4420      20.00    -5000.00        5.11         4.1
6
     4891     4425     4410      20.00       -0.00        0.00         0.0
0  CV
     4900     4410     4430      18.00     -305.33        0.38         0.0
4
     4910     4430     4440      12.00      -15.06        0.04         0.0
0
     4920     4440     4450      12.00     -188.26        0.53         0.1
```

2							
	4930	4450	4460	12.00	-63.22	0.18	0.0
2							
	4940	4460	4470	12.00	-198.24	0.56	0.1
3							
	4950	4470	4480	12.00	-58.62	0.17	0.0
1							
	4960	4480	3550	12.00	-362.01	1.03	0.3
9							
	4970	4320	4490	8.00	227.67	1.45	1.3
8							
	4980	4490	4500	8.00	-73.32	0.47	0.1
7							
	4990	4500	4470	8.00	139.62	0.89	0.5
6							
	5000	3720	3530	8.00	76.59	0.49	0.1
8							
	5010	3540	4500	8.00	212.94	1.36	1.2
2							
	5020	4310	4510	12.00	796.48	2.26	1.9
4							
	5030	4510	4520	12.00	607.55	1.72	1.1
8							
	5040	4520	4450	12.00	512.53	1.45	0.8
6							
	5050	4510	4530	8.00	170.93	1.09	0.7
0							
	5060	4530	4400	8.00	-118.66	0.76	0.3
6							
	5070	4490	4520	8.00	-95.02	0.61	0.2
7							
	5080	4530	4440	8.00	157.60	1.01	0.6
0							
	5090	4440	4445	8.00	80.80	0.52	0.1
8							
	5095	4540	4445	8.00	-80.80	0.52	0.1
8							
	5100	4540	4550	8.00	89.35	0.57	0.2
1							
	5110	4550	4560	8.00	224.36	1.43	1.1
6							
	5120	4560	4570	8.00	6.03	0.04	0.0
0							
	5130	4570	4580	8.00	-53.63	0.34	0.0
8							
	5140	4580	4590	8.00	-145.72	0.93	0.5
2							
	5150	4480	4590	10.00	303.39	1.24	0.6
8							
	5160	4590	4560	8.00	157.67	1.01	0.6
0							
	5170	4460	4550	8.00	135.01	0.86	0.4
5							
	5180	4450	4540	12.00	387.49	1.10	0.4
4							

5190	4540	4600	12.00	378.94	1.07	0.42
5200	4600	4610	12.00	194.90	0.55	0.12
5210	4610	4620	12.00	254.56	0.72	0.20

Link Results: (continued)

Link	Start Node	End Node	Diameter in	Flow gpm	Velocity fps	Headloss /1000ft
5220	4620	3750	12.00	346.66	0.98	0.36
5230	4570	4610	8.00	59.66	0.38	0.10
5240	4580	4620	8.00	92.09	0.59	0.22
5250	3840	4630	12.00	-73.00	0.21	0.02
5260	4630	3860	8.00	134.78	0.86	0.52
5270	3880	4640	12.00	-308.30	0.87	0.34
5280	4640	4650	12.00	-596.95	1.69	1.14
5290	4650	4660	12.00	563.05	1.60	1.02
5300	4660	4670	12.00	271.21	0.77	0.26
5310	4680	4690	14.00	-1070.07	2.23	1.58
5320	4690	4700	12.00	424.93	1.21	0.61
5330	4700	4710	12.00	586.78	1.66	1.10
5340	4710	3970	12.00	713.43	2.02	1.58
5350	4660	4700	8.00	291.84	1.86	2.18
5360	4640	4710	8.00	288.65	1.84	2.51
5370	4000	4720	10.00	-361.60	1.48	1.29
5380	4100	4730	12.00	500.72	1.42	0.82
5390	4730	4740	12.00	196.65	0.56	0.1

5	5400	4740	4750	12.00	-40.23	0.11	0.0
1	5410	4730	4760	8.00	304.07	1.94	2.3
6	5420	4760	4250	8.00	280.00	1.79	2.0
2	5430	4250	4770	8.00	386.17	2.46	3.6
7	5435	4755	4760	8.00	77.04	0.49	0.1
9	5440	4740	4760	8.00	236.88	1.51	1.4
8	5450	4380	4770	10.00	-7.47	0.03	0.0
0	5451	4380	4770	12.00	-14.27	0.04	0.0
0	5460	4770	4780	10.00	86.58	0.35	0.0
9	5461	4770	4780	12.00	165.36	0.47	0.0
9	5470	4430	5865	18.00	-290.27	0.37	0.0
4	5480	4600	4800	12.00	184.04	0.52	0.1
1	5600	4630	5000	12.00	792.22	2.25	1.6
6	5610	5000	5040	12.00	250.73	0.71	0.2
0	5620	5040	5010	12.00	164.90	0.47	0.0
9	5630	5010	5090	12.00	-381.71	1.08	0.4
3	5640	5090	5100	12.00	548.29	1.56	0.8
4	5650	5100	5110	12.00	274.02	0.78	0.2
3	5660	5110	5020	12.00	319.76	0.91	0.3
1	5670	5020	5080	12.00	-629.76	1.79	1.0
8	5680	5080	5030	12.00	612.83	1.74	1.0
3	5710	5040	5050	8.00	33.82	0.22	0.0
3	5720	5110	5050	8.00	-45.74	0.29	0.0
6	5730	5080	5050	8.00	189.92	1.21	0.8
5	5750	5020	5025	12.00	144.74	0.41	0.0
7	5780	5025	5200	12.00	1274.74	3.62	4.0
0	5790	5200	5210	12.00	1489.72	4.23	5.3
3							

```
     5800     5210     5500      12.00     -690.61        1.96        1.2
9
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  Page 29            CLOVIS WATER MASTER PLAN - HYDRAULIC MODEL


  Link Results: (continued)
  --------------------------------------------------------------------
-
              Start      End  Diameter         Flow  Velocity  Headlos
s
     Link      Node     Node        in          gpm       fps   /1000f
t
  --------------------------------------------------------------------
-
     5810     5500     4680      12.00   -1094.61        3.11        3.0
2
     5820     4680     4670      12.00    -992.03        2.81        2.5
1
     5830     4670     5030      12.00    -846.82        2.40        1.8
8
     5840     4680     4725      12.00     967.48        2.74        2.7
8
     5850     4725     4685      12.00     488.88        1.39        0.7
9
     5860     4725     4720      10.00     474.10        1.94        1.5
6
     5900     5100     5120       8.00     274.27        1.75        1.6
8
     5910     5120     5130       8.00     -11.73        0.07        0.0
0
     5920     5130     5140      12.00     309.62        0.88        0.2
9
     5930     5140     5150      12.00     117.62        0.33        0.0
5
     5940     5150     5300      12.00     694.20        1.97        1.3
0
     5950     5150     5240      12.00     693.21        1.97        1.3
0
     5960     5240     5230      12.00     791.22        2.24        1.6
5
     5970     5230     5220      12.00    1989.22        5.64        9.1
1
     5980     5220     5310      14.00    -676.24        1.41        0.5
8
     5990     5300     5310      12.00     786.45        2.23        1.6
4
     6000     5220     5215      14.00    -220.30        0.46        0.0
7
     6010     5215     5210      14.00    -836.30        1.74        0.8
7
     6020     5240     5250       8.00    -168.01        1.07        0.6
8
     6030     5250     5160       8.00    -168.01        1.07        0.6
```

8	6040	5150	5160	12.00	-269.78	0.77	0.2
3	6045	5160	5170	12.00	-437.80	1.24	0.5
5	6050	5170	5020	12.00	-804.78	2.28	1.7
1	6060	5170	5190	8.00	113.06	0.72	0.3
3	6070	5190	5200	8.00	214.98	1.37	1.0
7	6080	5190	5180	8.00	-101.91	0.65	0.2
7	6090	5170	5180	8.00	101.91	0.65	0.2
7	6100	5220	5400	12.00	1211.84	3.44	3.6
4	6110	5400	5410	8.00	122.21	0.78	0.3
8	6120	5410	5430	8.00	-292.11	1.86	1.8
9	6130	5430	5210	12.00	-1268.03	3.60	3.9
6	6140	5430	5440	12.00	963.92	2.73	2.3
8	6150	5410	5420	8.00	258.32	1.65	1.5
0	6155	5420	8210	12.00	563.64	1.60	0.8
8	6160	5420	5440	12.00	-305.32	0.87	0.2
8	6170	5630	5450	12.00	-1269.48	3.60	3.9
7	6180	5450	5440	12.00	-624.48	1.77	1.0
7	6190	5440	5600	12.00	34.12	0.10	0.0
0	6200	5600	5610	12.00	-453.30	1.29	0.5
9	6210	5610	4750	12.00	-128.30	0.36	0.0
6	6220	5600	5620	8.00	487.41	3.11	4.8
6	6230	5620	5630	8.00	-6.44	0.04	0.0
0	6240	5620	5670	8.00	299.86	1.91	1.9
8	6250	5670	5650	8.00	299.86	1.91	1.9
8	6260	5630	5640	12.00	1263.04	3.58	3.9
3	6270	5640	5770	12.00	807.16	2.29	1.7
2							

Link Results: (continued)

Link	Start Node	End Node	Diameter in	Flow gpm	Velocity fps	Headloss /1000ft
6280	5650	5640	12.00	532.22	1.51	0.79
6290	5650	5660	12.00	-232.36	0.66	0.17
6300	5660	5700	12.00	-586.24	1.66	0.95
6310	4750	4755	14.00	1314.35	2.74	2.00
6320	4755	4780	12.00	1135.31	3.22	3.74
6330	5700	4780	12.00	-243.34	0.69	0.19
6340	5700	5710	10.00	537.09	2.19	1.96
6350	4780	5720	12.00	1134.91	3.22	3.22
6360	5710	5720	12.00	-380.45	1.08	0.43
6370	5660	5680	8.00	353.88	2.26	2.69
6380	5680	5690	8.00	147.88	0.94	0.54
6390	5640	5690	12.00	988.09	2.80	2.40
6500	5690	5730	12.00	995.97	2.83	2.53
6510	5730	5740	12.00	1275.52	3.62	4.00
6520	5740	5750	14.00	1453.88	3.03	2.41
6530	5740	5780	8.00	-178.36	1.14	0.76
6540	5780	5770	8.00	-586.36	3.74	6.85
6550	5770	5760	12.00	220.80	0.63	0.16
6600	5752	5830	12.00	826.94	2.35	1.80
6610	5830	5840	12.00	524.36	1.49	0.77
6625	5990	6100	12.00	951.85	2.70	2.33
6630	5830	5820	12.00	238.34	0.68	0.1

8	6640	5820	5810	12.00	120.34	0.34	0.0
5	6650	5810	5800	12.00	-79.66	0.23	0.0
2	6660	5800	5850	14.00	-495.38	1.03	0.3
3	6670	5850	5866	14.00	-511.88	1.07	0.3
5	6672	5866	5865	12.00	0.00		Close
d	6680	5860	5720	12.00	-716.96	2.03	1.3
8	6690	5865	5860	12.00	-736.73	2.09	1.4
5	6700	5865	5870	12.00	446.46	1.27	0.5
7	6710	5870	5880	12.00	446.46	1.27	0.5
7	6730	5880	4800	12.00	200.46	0.57	0.1
3	6740	4800	5890	12.00	302.00	0.86	0.2
8	6750	5890	5900	12.00	230.00	0.65	0.1
7	7000	5010	8000	12.00	546.62	1.55	0.8
3	7010	8000	5130	12.00	321.36	0.91	0.3
1	7020	8000	8010	12.00	754.06	2.14	1.5
1	7030	8010	5300	12.00	-0.85	0.00	0.0
0	7040	8010	8020	12.00	754.90	2.14	1.5
2	7050	8020	8090	12.00	462.56	1.31	0.6
1	7060	8020	8030	12.00	431.64	1.22	0.5
4	7070	8030	8080	12.00	-1.07	0.00	0.0
0	7080	8030	8040	12.00	432.72	1.23	0.5
4	7090	8040	8070	12.00	125.15	0.36	0.0
5	7100	8040	8050	12.00	446.87	1.27	0.5
7	7110	8050	8060	12.00	446.87	1.27	0.5

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Link Results: (continued)

Link	Start Node	End Node	Diameter in	Flow gpm	Velocity fps	Headloss /1000ft	
7120	8060	8130	12.00	133.72	0.38	0.06	
7130	8070	8060	12.00	547.55	1.55	0.84	
7140	8120	8130	12.00	-133.72	0.38	0.06	
7150	8070	8120	12.00	842.97	2.39	1.86	
7160	8070	8080	12.00	-265.38	0.75	0.22	
7170	8110	8120	12.00	-116.00	0.33	0.05	
7180	8080	8110	12.00	586.42	1.66	0.95	
7190	8080	8090	12.00	147.13	0.42	0.07	
7200	8100	8110	12.00	-164.72	0.47	0.09	
7210	8090	8100	14.00	1516.59	3.16	2.60	
7220	5300	8090	12.00	906.91	2.57	2.13	
7230	5310	8100	12.00	1110.20	3.15	3.10	
7300	5400	8200	16.00	3292.36	5.25	5.70	
7310	8200	5760	18.00	3299.99	4.16	3.22	
7320	8200	8210	12.00	-563.64	1.60	0.88	
7330	5750	5760	18.00	-4108.25	5.18	4.84	
7331	5750	5754	24.00	2500.00	1.77	0.48	CV
7333	7640	5751	24.00	-2500.00	1.77	0.48	
7350	5710	5730	12.00	567.55	1.61	0.89	
7360	5730	5800	12.00	0.00		Closed	
7400	5000	5030	12.00	367.49	1.04	0.40	
7410	4685	4750	12.00	1482.88	4.21	5.29	
7420	4300	5860	12.00	299.77	0.85	0.27	
7500	5840	6000	24.00	2809.80	1.99	0.5	

```
9
     7510      5830      5840       20.00    2285.44          2.33         0.9
8
     7520      5830      5752       20.00   -3173.05          3.24         1.8
0
     7530      5220      5400       12.00    1202.72          3.41         3.5
9
     8000      5800      6010       12.00     280.72          0.80         0.2
4
     8010      6000      6010       12.00    -367.28          1.04         0.4
0
     8020      6010      6020       12.00    -304.56          0.86         0.2
8
     8030      6000      6120       16.00    1893.76          3.02         2.0
5
     8040      6120      6150       16.00     398.67          0.64         0.1
1
     8050      6120      6130       12.00    -963.12          2.73         2.3
8
     8060      6130      6140       12.00     936.05          2.66         2.2
6
     8070      6140      6150       12.00     276.75          0.79         0.2
4
     8080      6020      6130       24.00    1683.56          1.19         0.2
3
     8090      6130      6150       36.00   14776.79          4.66         1.7
7
     8100      6000      6100       14.00    1283.32          2.67         1.9
1
     8110      6100      6110       14.00    1444.78          3.01         2.3
8
     8120      6110      6120       12.00    -894.51          2.54         2.0
8
     8130      6110      6170       14.00    1284.79          2.68         1.9
1
     8140      6170      6180       14.00    1110.82          2.32         1.4
6
     8150      6150      6160       18.00    4343.05          5.48         5.3
6
     8160      6160      6170       14.00     880.53          1.84         0.9
5
     8170      6160      6180       18.00    2408.02          3.04         1.8
0
     8180      6150      6200       30.00   11109.16          5.04         2.5
3

  Page 32            CLOVIS WATER MASTER PLAN - HYDRAULIC MODEL


  Link Results: (continued)
  ---------------------------------------------------------------
-
              Start      End  Diameter       Flow  Velocity  Headlos
s
```

		Link	Node	Node	in	gpm	fps	/1000f
t								
-		8190	6200	6204	30.00	0.00		Close
d	CV	8200	6190	6300	24.00	289.45	0.21	0.0
1		8201	6180	6188	24.00	289.46	0.21	0.0
1	CV	8202	6188	6190	24.00	289.45	0.21	0.0
1		8204	6182	6184	16.00	4000.00	6.38	8.1
7	CV	8210	6180	6200	16.00	-1635.11	2.61	1.5
6		8220	6300	6280	24.00	-249.55	0.18	0.0
1		8230	6280	6290	16.00	-272.06	0.43	0.0
6		8240	6270	6280	16.00	-1200.46	1.92	0.8
8		8250	6210	6290	16.00	2322.16	3.71	2.9
9		8260	6250	6280	18.00	1177.94	1.49	0.4
8		8270	6260	6270	16.00	849.64	1.36	0.4
6		8280	6204	6210	24.00	8449.95	5.99	4.5
2		8290	6210	6250	20.00	3847.32	3.93	2.5
6		8300	6250	6260	18.00	1651.39	2.08	0.9
0		8310	6210	6220	16.00	2280.46	3.64	2.8
9		8320	6230	6250	18.00	-1017.99	1.28	0.3
7		8330	6240	6260	16.00	-801.75	1.28	0.4
2		8340	6220	6230	16.00	230.36	0.37	0.0
4		8350	6230	6240	16.00	1248.35	1.99	0.9
5		8400	8100	8220	18.00	1930.82	2.43	1.2
0		8405	8200	8220	18.00	556.00	0.70	0.1
2		8410	8220	8225	16.00	1679.32	2.68	1.6
4		8415	8225	8230	16.00	1091.86	1.74	0.7
4		8420	8230	5990	16.00	0.00		Close
d		8425	8225	8235	12.00	587.46	1.67	0.9

```
5
     8430     8235     5760      12.00     587.46        1.67        0.9
5
     8435     8230     8240      12.00    1091.86        3.10        3.0
0
     8440     8240     5750      12.00    1091.86        3.10        3.0
0
     8445     5990     8245      14.00    -951.85        1.98        1.1
0
     8450     8245     5830      14.00    -951.85        1.98        1.1
0
     9000     7700     7701      36.00   19503.68        6.15        2.9
5
     9001     7701     7702      36.00   19503.68        6.15        2.9
5
     9050     6020     6030      24.00   -1988.12        1.41        0.3
1
     9051     6030     5866      20.00     511.88        0.52        0.0
6
     9054     7630     6050      20.00    2500.00        2.55        1.1
5
     9080     6130     4810      36.00  -14992.41        4.73        1.8
1
     9100     4810     4820      36.00  -14992.41        4.73        1.8
1
     9150     4820     4830      36.00  -14992.41        4.73        1.8
1
     9200     3750     4831      12.00     421.81        1.20        0.5
2
     9210     4830     4831      12.00       0.00                  Close
d
     9230     4830     4840      36.00  -15148.41        4.77        1.8
5
     9260     2880     4831      16.00    -421.81        0.67        0.1
3
     9280     4840     4850      42.00  -15148.41        3.51        0.8
7
     9300     2882     3773      24.00       0.00                  Close
d
     9310     4850     2882      24.00       0.00        0.00        0.0
0
```


Link Results: (continued)

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  ---------------------------------------------------------------
-
               Start      End   Diameter       Flow  Velocity  Headlos
s
      Link      Node     Node         in        gpm       fps   /1000f
t
  ---------------------------------------------------------------
-
```

9320	4850	4860	42.00	-15974.91	3.70	0.9
9340	2890	2900	12.00	296.12	0.84	0.2
9350	1230	2900	12.00	305.45	0.87	0.2
9380	4860	9009	30.00	3528.78	1.60	0.3
9390	2900	2902	12.00	0.00		Close
9400	2902	9000	12.00	-0.00	0.00	0.0
9420	9000	9010	12.00	-444.50	1.26	0.5
9440	9010	9020	18.00	1772.84	2.24	1.0
9460	9020	9030	18.00	1102.70	1.39	0.4
9480	9030	9040	18.00	337.88	0.43	0.0
9500	2900	9100	12.00	601.57	1.71	1.0
9540	9000	9090	12.00	-36.20	0.10	0.0
9570	9010	9080	16.00	950.44	1.52	0.5
9590	9020	9070	12.00	428.84	1.22	0.5
9600	9030	9060	12.00	403.81	1.15	0.4
9610	9040	9050	12.00	337.88	0.96	0.3
9620	9090	9080	12.00	45.30	0.13	0.0
9640	9070	9080	12.00	-596.89	1.69	0.9
9660	9060	9070	12.00	-351.26	1.00	0.3
9680	9050	9060	12.00	160.34	0.45	0.0
9700	1220	9100	12.00	-980.73	2.78	2.4
9720	9080	9210	16.00	-81.86	0.13	0.0
9740	9070	9220	12.00	193.78	0.55	0.1
9760	9060	9230	12.00	193.42	0.55	0.1
9780	9050	9240	12.00	177.54	0.50	0.1
9820	9200	9210	16.00	1200.52	1.92	0.8
9840	9210	9220	12.00	637.97	1.81	1.1
9860	9220	9230	12.00	351.04	1.00	0.3

```
7
      9880      9230      9240       12.00    -177.54         0.50        0.1
0
      9900       270       450       12.00     732.06         2.08        1.4
3
      9902       120       910       14.00    1391.00         2.90        2.2
2
      1932      1220      1221                2484.73        60 hp      -95.5
3  Pump
      3094      3101      3102                   0.00                   Close
d  Pump
      4892      4420      4425                   0.00                   Close
d  Pump
      7334      5751      5756                   0.00                   Close
d  Pump
      7336      5750      5758                4000.00       200 hp     -197.8
0  Pump
      8182      6200      6202                8449.95        25 hp      -11.7
0  Pump
      8206      6184      6186                   0.00                   Close
d  Pump
      9052      6050      6040                2500.00       200 hp     -316.4
8  Pump
      1934      1221      1222       12.00    2484.73         7.05        0.0
0  FCV
      3096      3104      3101       12.00   -1336.58         3.79        0.0
0  FCV
      4895      4410      4420       12.00    5000.00        14.18       34.4
8  FCV
      7332      5754      5751       12.00    2500.00         7.09       26.7
9  FCV
      7335      5756      5752       24.00      -0.00         0.00        0.0
0  FCV
      7337      5758      5752       24.00    4000.00         2.84       36.2
3  FCV
      8184      6202      6204       30.00    8449.95         3.84        0.0
0  PSV

   Page 34           CLOVIS WATER MASTER PLAN - HYDRAULIC MODEL


   Link Results: (continued)
   ---------------------------------------------------------------
-
               Start       End  Diameter       Flow  Velocity  Headlos
s
      Link      Node      Node        in        gpm       fps   /1000f
t
   ---------------------------------------------------------------
-
      8203      6180      6182       16.00    4000.00         6.38       67.7
2  FCV
      8207      6186      6188       24.00      -0.00         0.00        0.0
0  FCV
```

	9002	7702	4860	30.00	19503.68	8.85	0.0
0	FCV						
	9053	6040	6030	20.00	2500.00	2.55	129.9
4	FCV						
	9301	3773	3770	20.00	-0.00	0.00	0.0
0	PRV						
	9381	9009	9010	24.00	3528.78	2.50	99.6
2	PRV						

```
**********************************************************************
*                          E P A N E T                               *
*                 Hydraulic and Water Quality                        *
*                 Analysis for Pipe Networks                         *
*                        Version 1.1e                                *
**********************************************************************
```

CLOVIS WATER MASTER PLAN - HYDRAULIC MODEL

```
Input Data File ................... 7-6PKH.INP
Output Report File ................ 7-6pkh.rpt
Verification File .................
Hydraulics File ...................
Map File .......................... Clovis2.map
Number of Pipes ................... 869
Number of Nodes ................... 599
Number of Tanks ................... 5
Number of Pumps ................... 7
Number of Valves .................. 8
Headloss Formula .................. Hazen-Williams
Hydraulic Timestep ................ 1.00 hrs
Hydraulic Accuracy ................ 0.005000
Maximum Trials .................... 20
Quality Analysis .................. None
Specific Gravity .................. 1.00
Kinematic Viscosity ............... 1.10e-05 sq ft/sec
Chemical Diffusivity .............. 1.30e-08 sq ft/sec
Total Duration .................... 0.00 hrs
Reporting Criteria:
   Selected Nodes
   Selected Links
```

Node Results:

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi
50	342.00	0.00	522.73	78.31
60	340.00	46.00	522.89	79.24
70	340.00	0.00	522.94	79.27
80	340.00	0.00	523.18	79.37
85	340.00	0.00	523.43	79.48
90	340.00	0.00	523.05	79.31

100	340.00	50.00	522.98	79.28
110	343.00	44.10	522.83	77.92
120	345.00	0.00	522.84	77.06
130	342.00	0.00	522.88	78.37
140	345.00	0.00	522.24	76.80
150	345.00	0.00	520.02	75.84
160	345.00	550.80	517.63	74.80
170	345.00	0.00	521.17	76.33
180	342.00	39.90	520.70	77.43

Node Results: (continued)

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi
190	338.00	622.20	519.14	78.49
200	337.00	0.00	519.91	79.26
210	336.00	214.20	518.68	79.16
220	336.00	40.00	520.93	80.13
225	335.00	282.20	518.49	79.51
230	337.00	31.50	521.32	79.87
235	335.00	0.00	520.56	80.40
240	341.00	476.00	520.34	77.71
245	337.00	0.00	521.18	79.81
250	345.00	161.70	520.40	76.00
255	344.00	0.00	521.43	76.88
260	346.00	0.00	517.27	74.21
270	347.00	0.00	514.71	72.67
280	350.00	0.00	515.50	71.71
290	347.00	397.80	515.93	73.20
300	345.00	0.00	521.30	76.39
305	345.00	0.00	521.81	76.61
310	340.00	270.90	521.30	78.56
320	341.00	0.00	522.15	78.49
325	340.00	0.00	522.19	78.94
330	338.00	0.00	521.79	79.64
340	340.00	69.30	521.93	78.83
350	343.00	69.30	522.39	77.73
355	344.00	0.00	522.32	77.27
360	344.00	0.00	521.99	77.12
370	346.00	0.00	521.92	76.23
375	348.00	0.00	518.69	73.96
380	350.00	44.00	515.45	71.69
390	352.00	0.00	510.35	68.61
400	354.00	24.00	505.87	65.81
410	352.00	353.60	505.42	66.48
420	352.00	237.30	504.90	66.25
430	358.00	0.00	502.92	62.80
440	355.00	0.00	502.86	64.07

450	354.00	0.00	505.18	65.50
460	354.00	0.00	505.18	65.50
470	354.00	144.00	504.78	65.33
500	350.00	0.00	568.74	94.78
505	348.00	0.00	633.28	123.61
510	350.00	0.00	544.23	84.16
520	354.00	0.00	531.02	76.70
530	350.00	0.00	528.63	77.40
540	350.00	706.10	522.72	74.84
550	350.00	0.00	528.65	77.41
560	350.00	0.00	528.18	77.21
565	345.00	0.00	529.61	79.99

Node Results: (continued)

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi
570	345.00	0.00	523.23	77.23
580	345.00	0.00	523.23	77.23
600	345.00	0.00	522.89	77.08
610	348.00	0.00	529.06	78.45
620	350.00	0.00	528.80	77.47
630	347.00	0.00	527.18	78.07
640	342.00	0.00	520.05	77.15
650	345.00	706.10	518.77	75.29
660	345.00	0.00	521.89	76.64
670	342.00	0.00	519.81	77.04
680	345.00	0.00	519.46	75.59
690	346.00	0.00	519.56	75.20
700	345.00	0.00	519.72	75.71
710	350.00	76.00	516.66	72.21
715	348.00	0.00	515.80	72.71
720	350.00	0.00	512.16	70.27
730	350.00	0.00	510.51	69.55
740	350.00	0.00	505.00	67.16
750	350.00	0.00	494.87	62.77
755	350.00	0.00	494.96	62.81
760	350.00	0.00	512.24	70.30
770	350.00	345.00	504.95	67.14
780	353.00	0.00	499.22	63.36
790	350.00	618.70	481.92	57.16
800	349.00	0.00	497.07	64.16
805	350.00	0.00	490.49	60.88
810	348.00	0.00	497.66	64.85
820	345.00	0.00	504.34	69.04
830	345.00	0.00	507.79	70.54
840	345.00	0.00	508.62	70.90
850	345.00	0.00	509.44	71.25

860	345.00	309.40	509.42	71.24
870	345.00	85.00	511.30	72.06
880	346.00	0.00	520.11	75.44
890	345.00	127.00	504.48	69.10
900	350.00	455.60	492.22	61.62
910	350.00	0.00	493.75	62.28
920	355.00	0.00	490.55	58.74
930	357.00	261.80	485.49	55.68
940	355.00	0.00	485.84	56.69
950	354.00	0.00	495.51	61.32
960	355.00	618.80	490.78	58.84
970	360.00	0.00	485.36	54.32
980	360.00	0.00	485.31	54.30
990	364.00	8.40	485.15	52.49
1000	356.00	0.00	494.47	60.00

Node Results: (continued)

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi
1010	357.00	78.00	488.64	57.04
1020	360.00	0.00	493.70	57.93
1030	365.00	544.00	486.85	52.80
1040	355.00	10.00	502.83	64.06
1050	360.00	200.00	495.71	58.80
1060	360.00	147.00	493.70	57.93
1070	360.00	105.00	493.19	57.71
1080	365.00	0.00	485.09	52.03
1090	365.00	54.60	484.93	51.97
1100	366.00	0.00	484.94	51.54
1110	370.00	52.50	476.48	46.14
1120	370.00	731.00	476.48	46.14
1130	370.00	0.00	479.94	47.64
1140	375.00	227.80	477.19	44.28
1150	363.00	299.20	479.74	50.58
1160	365.00	44.00	478.64	49.24
1170	359.00	261.80	477.53	51.36
1180	360.00	261.80	472.74	48.85
1190	360.00	0.00	472.37	48.69
1200	358.00	0.00	475.12	50.75
1210	365.00	0.00	471.56	46.17
1220	365.00	0.00	471.24	46.04
1222	365.00	0.00	507.65	61.81
1230	378.00	523.60	472.17	40.80
1240	369.00	302.60	472.95	45.04
1250	369.00	1210.40	471.54	44.43
1260	362.00	1176.40	471.17	47.30
2400	343.00	226.80	522.99	77.99

2410	344.00	0.00	522.49	77.34
2420	344.00	0.00	522.49	77.34
2430	344.00	0.00	522.36	77.28
2450	350.00	421.60	522.43	74.71
2500	350.00	518.70	515.44	71.69
2510	350.00	0.00	514.07	71.09
2520	351.00	0.00	510.01	68.90
2530	351.00	0.00	508.72	68.34
2540	353.00	319.60	505.88	66.24
2545	354.00	0.00	506.35	66.01
2550	353.00	0.00	506.13	66.35
2560	354.00	72.00	502.49	64.34
2570	355.00	0.00	502.90	64.09
2580	355.00	0.00	502.48	63.90
2585	358.00	0.00	502.87	62.77
2590	355.00	0.00	499.41	62.57
2595	357.00	0.00	498.83	61.46
2600	355.00	142.00	498.98	62.39

Node Results: (continued)

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi
2610	355.00	172.20	496.37	61.26
2620	358.00	0.00	496.29	59.92
2625	359.00	0.00	497.69	60.09
2630	358.00	0.00	496.37	59.96
2640	360.00	0.00	491.63	57.04
2650	360.00	0.00	491.72	57.07
2660	360.00	0.00	492.29	57.32
2670	362.00	510.00	484.79	53.21
2680	362.00	0.00	485.67	53.59
2690	363.00	0.00	487.94	54.14
2700	365.00	0.00	484.71	51.87
2701	366.00	0.00	484.70	51.43
2710	365.00	0.00	485.04	52.02
2720	365.00	0.00	482.54	50.93
2730	366.00	0.00	477.73	48.41
2740	367.00	0.00	478.76	48.43
2750	370.00	520.20	477.31	46.50
2760	368.00	0.00	477.31	47.37
2770	370.00	0.00	476.59	46.18
2780	370.00	0.00	476.54	46.16
2790	370.00	0.00	476.46	46.13
2800	370.00	0.00	475.57	45.74
2810	370.00	710.60	473.72	44.94
2820	372.00	0.00	473.74	44.09
2830	372.00	134.00	473.84	44.13

2840	372.00	0.00	473.73	44.08
2850	376.00	0.00	472.69	41.89
2860	377.00	1176.40	471.57	40.98
2870	377.00	0.00	472.27	41.28
2880	380.00	523.60	471.99	39.86
2882	380.00	0.00	594.61	92.99
2890	380.00	70.00	471.75	39.76
2900	380.00	0.00	471.77	39.76
2902	380.00	0.00	510.74	56.65
3000	346.00	95.20	520.91	75.79
3010	350.00	220.50	519.05	73.25
3020	350.00	0.00	518.19	72.88
3030	350.00	0.00	518.04	72.81
3040	350.00	0.00	518.15	72.86
3050	350.00	0.00	516.87	72.31
3055	350.00	0.00	515.59	71.75
3060	350.00	168.00	515.59	71.75
3070	351.00	20.00	515.18	71.14
3080	350.00	514.50	517.42	72.54
3090	350.00	0.00	520.35	73.81
3100	350.00	0.00	522.34	74.67

Node Results: (continued)

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi
3101	350.00	0.00	481.95	57.18
3102	350.00	0.00	522.39	74.69
3104	350.00	0.00	522.34	74.67
3110	350.00	0.00	517.80	72.71
3115	346.00	0.00	521.28	75.95
3120	346.00	0.00	521.28	75.95
3130	346.00	239.40	520.91	75.79
3135	347.00	0.00	520.97	75.38
3136	347.00	0.00	521.24	75.50
3137	347.00	0.00	521.40	75.57
3140	346.00	0.00	521.32	75.97
3150	355.00	10.50	507.87	66.24
3160	355.00	0.00	503.97	64.55
3170	355.00	0.00	503.97	64.55
3180	355.00	204.00	503.96	64.54
3190	354.00	0.00	504.12	65.05
3200	354.00	0.00	504.47	65.20
3260	354.00	0.00	506.15	65.93
3270	353.00	0.00	506.28	66.42
3280	350.00	0.00	509.29	69.02
3290	350.00	0.00	505.63	67.43
3300	350.00	319.60	505.40	67.33

3310	350.00	85.00	509.34	69.04
3320	355.00	0.00	504.04	64.58
3330	354.00	0.00	504.36	65.15
3340	355.00	0.00	503.21	64.22
3350	360.00	0.00	498.95	60.21
3360	360.00	140.00	498.32	59.94
3365	360.00	0.00	499.43	60.42
3370	360.00	0.00	491.90	57.15
3375	360.00	0.00	491.68	57.06
3380	360.00	0.00	491.04	56.78
3390	360.00	0.00	490.99	56.76
3395	360.00	0.00	491.00	56.76
3400	360.00	75.60	490.70	56.63
3410	361.00	0.00	490.68	56.19
3415	360.00	0.00	490.95	56.74
3420	361.00	0.00	490.74	56.22
3430	356.00	240.00	499.56	62.21
3435	356.00	0.00	500.42	62.58
3440	355.00	0.00	501.27	63.38
3450	355.00	0.00	501.62	63.53
3460	355.00	0.00	502.47	63.90
3470	356.00	0.00	503.26	63.81
3475	355.00	0.00	496.98	61.52
3480	356.00	0.00	501.16	62.90

Node Results: (continued)

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi
3485	355.00	0.00	497.34	61.67
3490	355.00	0.00	503.34	64.28
3500	363.00	414.80	483.75	52.32
3510	362.00	0.00	483.42	52.61
3520	364.00	0.00	481.16	50.76
3530	365.00	0.00	479.06	49.42
3540	365.00	0.00	478.29	49.09
3550	368.00	149.60	476.34	46.94
3560	370.00	0.00	476.66	46.22
3570	372.00	0.00	476.71	45.37
3575	372.00	0.00	476.81	45.42
3580	370.00	0.00	476.78	46.27
3590	367.00	0.00	479.18	48.61
3600	365.00	0.00	482.82	51.05
3605	365.00	0.00	482.68	50.99
3610	364.00	0.00	483.81	51.91
3620	364.00	0.00	484.07	52.03
3630	364.00	0.00	484.86	52.37
3640	360.00	173.40	487.21	55.12

3650	362.00	0.00	483.76	52.76
3660	361.00	0.00	487.50	54.81
3670	360.00	0.00	482.62	53.13
3680	360.00	612.00	482.61	53.13
3690	365.00	0.00	481.38	50.43
3700	365.00	0.00	481.92	50.66
3720	367.00	0.00	479.05	48.55
3730	368.00	414.80	477.83	47.59
3740	374.00	0.00	474.58	43.58
3750	378.00	218.40	474.14	41.66
3760	376.00	0.00	474.14	42.52
3770	375.00	0.00	474.24	43.00
3773	375.00	0.00	594.30	95.02
3780	374.00	0.00	474.54	43.57
3790	373.00	0.00	474.73	44.08
3800	372.00	0.00	475.70	44.93
3810	374.00	710.60	473.38	43.06
3820	374.00	0.00	474.26	43.44
3830	350.00	0.00	516.36	72.08
3840	351.00	0.00	515.13	71.12
3850	352.00	0.00	514.47	70.40
3860	352.00	268.60	514.33	70.34
3870	357.00	52.00	513.51	67.82
3880	358.00	0.00	513.40	67.34
3890	355.00	0.00	514.67	69.18
3900	352.00	0.00	515.64	70.90
3910	352.00	88.40	515.34	70.77

Node Results: (continued)

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi
3920	352.00	0.00	514.65	70.47
3930	355.00	516.80	512.79	68.37
3940	352.00	0.00	515.42	70.81
3950	355.00	0.00	513.83	68.82
3960	360.00	0.00	512.99	66.29
3970	360.00	0.00	511.69	65.73
3980	360.00	319.60	512.30	65.99
3990	358.00	200.00	502.68	62.69
4000	358.00	0.00	501.77	62.30
4010	358.00	0.00	501.51	62.18
4020	357.00	0.00	501.56	62.64
4030	357.00	192.00	501.77	62.73
4035	358.00	0.00	509.13	65.48
4040	355.00	0.00	507.82	66.21
4050	355.00	0.00	507.29	65.99
4060	356.00	132.30	502.76	63.59

4070	357.00	0.00	502.65	63.11
4080	356.00	370.60	501.82	63.18
4090	360.00	0.00	498.79	60.14
4100	360.00	0.00	497.41	59.54
4110	365.00	88.20	493.02	55.47
4120	365.00	0.00	492.55	55.27
4130	367.00	0.00	491.18	53.81
4140	365.00	0.00	490.94	54.57
4150	364.00	10.50	490.83	54.96
4160	364.00	0.00	490.78	54.93
4170	360.00	0.00	490.77	56.66
4180	360.00	0.00	500.11	60.71
4190	361.00	0.00	498.05	59.39
4200	363.00	391.00	491.71	55.77
4210	362.00	0.00	491.31	56.03
4250	367.00	0.00	492.84	54.53
4260	360.00	0.00	500.30	60.79
4270	360.00	0.00	500.29	60.79
4280	367.00	58.00	486.25	51.67
4290	368.00	304.50	484.55	50.50
4300	373.00	0.00	480.99	46.79
4310	371.00	0.00	480.22	47.32
4320	370.00	0.00	480.39	47.83
4330	370.00	0.00	480.69	47.96
4340	365.00	0.00	480.75	50.16
4350	370.00	0.00	480.75	47.99
4360	370.00	0.00	480.75	47.99
4380	368.00	3.40	491.17	53.37
4390	373.00	0.00	481.42	46.98
4400	374.00	0.00	478.61	45.33

Node Results: (continued)

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi
4410	376.00	0.00	477.87	44.14
4420	375.00	0.00	390.00	6.50
4425	0.00	0.00	477.87	207.06
4430	376.00	0.00	477.83	44.12
4440	375.00	425.00	476.57	44.01
4445	376.00	0.00	476.02	43.34
4450	375.00	0.00	476.35	43.91
4460	375.00	0.00	476.26	43.88
4470	375.00	0.00	476.23	43.86
4480	373.00	0.00	476.18	44.71
4490	371.00	673.20	476.22	45.59
4500	371.00	0.00	476.42	45.68
4510	371.00	25.20	478.75	46.69

4520	372.00	0.00	477.86	45.87
4530	372.00	224.40	477.90	45.89
4540	378.00	0.00	475.47	42.24
4550	377.00	0.00	475.13	42.52
4560	377.00	639.20	474.45	42.22
4570	376.00	0.00	474.58	42.71
4580	376.00	0.00	474.72	42.77
4590	375.00	0.00	475.89	43.72
4600	382.00	0.00	474.68	40.16
4610	382.00	0.00	474.58	40.11
4620	382.00	0.00	474.47	40.07
4630	353.00	0.00	515.13	70.25
4640	359.00	0.00	513.42	66.91
4650	360.00	0.00	513.67	66.59
4660	360.00	0.00	513.27	66.41
4670	360.00	168.00	511.76	65.76
4680	360.00	0.00	509.72	64.87
4685	363.00	8.00	506.14	62.02
4690	360.00	0.00	511.62	65.70
4700	360.00	221.00	511.62	65.70
4710	360.00	275.40	511.65	65.71
4720	362.00	157.50	502.72	60.98
4725	362.00	6.30	506.64	62.67
4730	365.00	0.00	496.72	57.08
4740	365.00	0.00	496.71	57.07
4750	366.00	0.00	496.79	56.67
4755	368.00	142.80	494.53	54.82
4760	367.00	574.60	493.48	54.80
4770	370.00	150.00	491.32	52.57
4780	369.00	12.60	491.39	53.03
4800	385.00	110.00	474.21	38.66
4810	390.00	0.00	584.74	84.38
4820	385.00	0.00	587.22	87.62

Node Results: (continued)

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi
4830	382.00	265.20	589.71	90.00
4831	382.00	0.00	472.41	39.17
4840	390.00	0.00	592.31	87.66
4850	388.00	1313.70	594.91	89.66
4860	393.00	0.00	598.73	89.14
5000	352.00	232.00	512.07	69.36
5010	359.00	0.00	510.52	65.65
5020	362.00	0.00	510.37	64.29
5025	363.00	0.00	510.35	63.84
5030	358.00	178.00	511.90	66.68

5040	355.00	88.40	510.94	67.57
5050	356.00	302.60	510.64	67.00
5080	360.00	90.00	512.35	66.01
5090	360.00	0.00	510.88	65.38
5100	361.00	0.00	510.60	64.82
5110	361.00	0.00	510.53	64.79
5120	361.00	486.20	506.20	62.91
5130	361.00	0.00	506.70	63.13
5140	363.00	326.40	506.64	62.24
5150	365.00	0.00	506.75	61.42
5160	365.00	0.00	507.38	61.69
5170	365.00	258.40	507.63	61.80
5180	365.00	0.00	507.57	61.77
5190	365.00	0.00	507.54	61.76
5200	365.00	0.00	507.47	61.73
5210	365.00	129.20	501.43	59.12
5215	370.00	1047.20	499.88	56.28
5220	375.00	748.96	499.88	54.11
5230	370.00	3.40	505.05	58.52
5240	368.00	119.00	506.10	59.84
5250	367.00	0.00	506.82	60.58
5300	368.00	0.00	503.26	58.61
5310	375.00	0.00	500.80	54.51
5400	375.00	0.00	497.80	53.21
5410	375.00	265.20	497.01	52.87
5420	375.00	0.00	496.53	52.66
5430	375.00	20.40	498.53	53.53
5440	375.00	0.00	496.76	52.76
5450	375.00	0.00	496.71	52.74
5500	362.00	686.80	503.42	61.28
5600	368.00	0.00	496.74	55.78
5610	368.00	0.00	496.79	55.81
5620	368.00	329.80	493.84	54.53
5630	368.00	0.00	494.33	54.74
5640	370.00	0.00	491.90	52.82
5650	370.00	0.00	491.83	52.79

Node Results: (continued)

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi
5660	370.00	0.00	491.78	52.77
5670	369.00	0.00	492.68	53.59
5680	372.00	350.20	489.89	51.08
5690	372.00	238.00	489.98	51.12
5700	370.00	0.00	491.78	52.77
5710	373.00	595.00	485.86	48.90
5720	373.00	50.00	485.47	48.73

5730	376.00	489.60	488.53	48.76
5740	378.00	0.00	489.87	48.47
5750	378.00	261.80	492.32	49.53
5751	378.00	0.00	391.86	6.01
5752	378.00	0.00	590.95	92.27
5754	378.00	0.00	492.32	49.53
5756	378.00	0.00	611.64	101.23
5758	378.00	0.00	590.95	92.27
5760	376.00	0.00	493.15	50.76
5770	373.00	0.00	492.00	51.56
5780	375.00	693.60	487.44	48.72
5800	380.00	189.00	585.07	88.86
5810	382.00	340.00	584.89	87.91
5820	383.00	200.60	584.90	87.48
5830	386.00	0.00	585.05	86.25
5840	385.00	0.00	584.17	86.30
5850	380.00	23.10	586.31	89.39
5860	374.00	476.00	481.53	46.59
5865	376.00	0.00	477.83	44.12
5866	376.00	0.00	587.67	91.72
5870	380.00	0.00	476.72	41.91
5880	381.00	418.20	474.71	40.60
5890	384.00	96.00	473.54	38.80
5900	385.00	391.00	472.95	38.11
5990	394.00	0.00	581.43	81.21
6000	391.00	0.00	582.93	83.16
6010	385.00	370.60	583.75	86.12
6020	385.00	0.00	584.05	86.25
6030	385.00	0.00	587.78	87.86
6040	385.00	0.00	609.61	97.32
6050	385.00	0.00	389.83	2.09
6100	398.00	1256.32	571.57	75.21
6110	405.00	1676.10	566.10	69.80
6120	400.00	2485.46	577.13	76.75
6130	390.00	0.00	582.25	83.30
6140	400.00	1047.94	577.33	76.84
6150	404.00	0.00	577.69	75.26
6160	410.00	1676.10	566.77	67.93
6170	420.00	1676.10	563.90	62.35

Node Results: (continued)

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi
6180	436.00	1374.10	564.75	55.79
6182	436.00	0.00	580.59	62.65
6184	436.00	0.00	580.59	62.65
6186	436.00	0.00	630.04	84.08

6188	435.00	0.00	614.31	77.69
6190	435.00	0.00	613.90	77.52
6200	425.00	1627.78	566.75	61.42
6202	425.00	0.00	584.50	69.11
6204	425.00	0.00	584.50	69.11
6210	430.00	0.00	578.97	64.55
6220	430.00	3258.58	570.77	61.00
6230	425.00	0.00	572.10	63.74
6240	435.00	3258.58	568.35	57.78
6250	438.00	0.00	576.97	60.22
6260	445.00	0.00	572.80	55.38
6270	450.00	3258.58	572.27	52.98
6280	445.00	0.00	582.83	59.72
6290	440.00	3258.58	576.59	59.19
6300	455.00	539.00	592.32	59.50
7010	360.00	-350.00	499.49	60.44
7020	358.00	-1200.00	509.63	65.70
7030	360.00	-945.00	492.00	57.20
7040	340.00	-1050.00	524.73	80.04
7050	354.00	-500.00	506.25	65.97
7060	348.00	-325.00	521.89	75.35
7070	350.00	-2000.00	522.87	74.91
7080	363.00	-1495.00	512.24	64.67
7090	362.00	-550.00	500.41	59.97
7100	344.00	-1000.00	522.26	77.24
7110	365.00	-1330.00	485.28	52.12
7120	355.00	-1030.00	504.94	64.97
7130	375.00	-300.00	473.30	42.60
7140	360.00	-1160.00	514.06	66.75
7150	349.00	-1250.00	518.60	73.49
7160	360.00	-1000.00	478.93	51.53
7170	344.00	-1380.00	523.25	77.67
7180	345.00	-855.00	520.59	76.08
7190	345.00	-410.00	509.19	71.14
7200	372.00	-500.00	476.91	45.46
7210	356.00	-930.00	511.00	67.16
7220	370.00	-645.00	496.77	54.93
7230	367.00	-325.00	496.81	56.25
7240	370.00	-880.00	491.98	52.85
7250	364.00	-1200.00	505.23	61.20
7260	363.00	-1130.00	510.51	63.91
7270	360.00	-1500.00	512.89	66.25

Node Results: (continued)

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi
7280	348.00	-2180.00	523.51	76.05

7290	340.00	-780.00	522.35	79.01
7320	340.00	-1000.00	522.99	79.29
7330	350.00	-1000.00	515.76	71.82
7340	355.00	-1000.00	503.12	64.18
7350	350.00	-1000.00	515.43	71.68
7360	367.00	-1000.00	506.39	60.40
7370	350.00	-1000.00	515.38	71.66
7380	360.00	-1000.00	507.08	63.73
7390	365.00	-1000.00	507.00	61.53
7400	374.00	-1000.00	498.05	53.75
7410	370.00	-1000.00	503.51	57.85
7420	372.00	-1000.00	501.05	55.92
7430	368.00	-1000.00	497.74	56.21
7440	375.00	-1000.00	497.26	52.97
7450	376.00	-1000.00	495.24	51.67
7460	380.00	-1000.00	494.90	49.79
7470	385.00	-1000.00	495.39	47.83
7475	385.00	-500.00	521.54	59.16
7480	375.00	-500.00	511.48	59.14
7485	380.00	-500.00	509.81	56.25
7490	370.00	-500.00	508.77	60.13
7495	378.00	-500.00	508.20	56.42
7510	345.00	-120.00	523.33	77.27
7520	346.00	-500.00	529.06	79.32
7530	348.00	-1030.00	513.65	71.78
7540	348.00	-700.00	641.00	126.96
7550	350.00	-1025.00	499.98	64.99
7560	346.00	-700.00	531.38	80.32
7570	345.00	-500.00	516.35	74.25
7580	345.00	-660.00	531.12	80.65
7701	390.00	0.00	599.36	90.72
7702	390.00	0.00	598.73	90.44
8000	360.00	748.96	506.82	63.62
8010	365.00	0.00	503.02	59.80
8020	368.00	1368.06	497.48	56.10
8030	372.00	0.00	496.73	54.05
8040	378.00	1368.06	494.99	50.69
8050	390.00	0.00	495.13	45.55
8060	390.00	1368.06	489.38	43.06
8070	380.00	0.00	494.65	49.68
8080	375.00	0.00	497.00	52.86
8090	372.00	0.00	497.05	54.18
8100	376.00	1368.06	493.17	50.77
8110	379.00	854.66	490.68	48.39
8120	382.00	1368.06	488.79	46.27

Node Results: (continued)

Node	Elev. ft	Demand gpm	Grade ft	Pressure psi

8130	392.00	0.00	489.18	42.11	
8200	372.00	0.00	495.67	53.59	
8210	370.00	0.00	496.29	54.72	
8220	377.00	1283.50	493.11	50.31	
8225	378.00	0.00	493.07	49.86	
8230	379.00	0.00	492.99	49.39	
8235	378.00	0.00	493.11	49.88	
8240	377.00	0.00	492.65	50.11	
8245	378.00	0.00	583.24	88.93	
9000	385.00	764.06	510.74	54.48	
9009	383.00	0.00	595.84	92.22	
9010	383.00	573.80	521.47	60.00	
9020	385.00	383.54	514.12	55.95	
9030	385.00	573.80	511.13	54.65	
9040	390.00	0.00	510.78	52.33	
9050	390.00	0.00	508.21	51.22	
9060	385.00	1147.60	507.60	53.12	
9070	380.00	764.06	509.74	56.22	
9080	378.00	764.06	511.41	57.81	
9090	380.00	764.06	508.71	55.77	
9100	375.00	0.00	471.54	41.83	
9102	375.00	0.00	508.71	57.94	
9200	368.00	1147.60	507.65	60.51	
9210	370.00	764.06	508.70	60.10	
9220	378.00	764.06	508.13	56.39	
9230	380.00	1147.60	506.55	54.83	
9240	380.00	0.00	507.38	55.19	
7620	454.00	-571.11	482.00	12.13	Tank
7630	370.00	-4500.00	390.00	8.67	Tank
7640	380.00	-4500.00	392.00	5.20	Tank
7650	560.00	-8000.00	581.00	9.10	Tank
7700	600.00	-20315.37	600.00	0.00	Reservoir

Link Results:

Link	Start Node	End Node	Diameter in	Flow gpm	Velocity fps	Headloss /1000ft
1	7010	3365	10.00	350.00	1.43	1.03
2	7020	4035	10.00	1200.00	4.90	10.07
3	7030	3375	10.00	945.00	3.86	6.48
4	7040	85	12.00	1050.00	2.98	3.24
5	7050	3260	10.00	500.00	2.04	1.99

6	7060	3137	10.00	325.00	1.33	0.90
7	7070	3100	12.00	2000.00	5.67	10.67

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Link Results: (continued)

Link	Start Node	End Node	Diameter in	Flow gpm	Velocity fps	Headloss /1000ft
8	7080	4690	12.00	1495.00	4.24	6.23
9	7090	4270	10.00	550.00	2.25	2.38
10	7100	255	12.00	1000.00	2.84	2.96
11	7110	2710	14.00	1330.00	2.77	2.37
12	7120	470	12.00	1030.00	2.92	3.12
14	7140	4650	12.00	1160.00	3.29	3.89
15	7150	3040	12.00	1250.00	3.55	4.47
16	7160	1160	12.00	1000.00	2.84	2.96
17	7170	2400	12.00	1380.00	3.91	5.37
20	7200	3575	10.00	500.00	2.04	1.99
21	7210	5090	12.00	930.00	2.64	2.23
22	7220	5450	12.00	645.00	1.83	1.13
23	7230	5610	12.00	325.00	0.92	0.32
24	7240	5700	12.00	880.00	2.50	2.01
25	7250	5230	12.00	1200.00	3.40	3.58
26	7260	5025	12.00	1130.00	3.21	3.20
27	7270	5080	12.00	1500.00	4.26	5.40
28	7280	2450	12.00	2180.00	6.18	10.79
29	7290	325	12.00	780.00	2.21	1.6

1	32	7320	50	12.00	1000.00	2.84	2.5
5	33	7330	280	12.00	1000.00	2.84	2.5
5	34	7340	2585	12.00	1000.00	2.84	2.5
5	35	7350	3070	12.00	1000.00	2.84	2.5
5	36	7360	4685	12.00	1000.00	2.84	2.5
5	37	7370	4630	12.00	1000.00	2.84	2.5
5	38	7380	8000	12.00	1000.00	2.84	2.5
5	39	7390	5150	12.00	1000.00	2.84	2.5
5	40	7400	5400	12.00	1000.00	2.84	2.5
5	41	7410	5300	12.00	1000.00	2.84	2.5
5	42	7420	5310	12.00	1000.00	2.84	2.5
5	43	7430	8020	12.00	1000.00	2.84	2.5
5	44	7440	8080	12.00	1000.00	2.84	2.5
5	45	7450	8040	12.00	1000.00	2.84	2.5
5	46	7460	8070	12.00	1000.00	2.84	2.5
5	47	7470	8050	12.00	1000.00	2.84	2.5
5	48	7480	9080	12.00	500.00	1.42	0.7
1	49	7490	9210	12.00	500.00	1.42	0.7
1	50	7475	9010	12.00	500.00	1.42	0.7
1	51	7485	9070	12.00	500.00	1.42	0.7
1	52	7495	9220	12.00	500.00	1.42	0.7
1	54	7650	6184	24.00	8000.00	5.67	4.0
9	55	50	60	12.00	-166.70	0.47	0.0
9	60	60	70	10.00	-76.27	0.31	0.0
5	70	70	90	10.00	-76.27	0.31	0.0
5	80	80	90	8.00	76.27	0.49	0.1
6	90	85	80	12.00	415.54	1.18	0.5
0							

Link Results: (continued)

Link	Start Node	End Node	Diameter in	Flow gpm	Velocity fps	Headloss /1000ft
100	80	100	12.00	339.27	0.96	0.35
110	100	60	12.00	136.43	0.39	0.06
120	100	110	12.00	152.84	0.43	0.08
130	50	110	12.00	-149.04	0.42	0.08
140	110	120	12.00	-40.30	0.11	0.01
150	120	130	12.00	-77.71	0.22	0.02
160	130	85	12.00	-634.46	1.80	1.10
161	880	130	12.00	-556.75	1.58	0.86
170	120	140	12.00	680.29	1.93	1.70
180	140	150	8.00	330.92	2.11	3.24
190	150	160	8.00	330.92	2.11	3.24
200	160	170	8.00	-296.06	1.89	2.63
210	170	50	12.00	-760.14	2.16	2.09
215	245	50	12.00	-555.59	1.58	1.17
220	170	180	12.00	464.09	1.32	0.84
230	180	190	8.00	191.94	1.23	1.18
240	190	245	8.00	-222.30	1.42	1.55
250	190	200	8.00	-129.46	0.83	0.57
255	235	245	12.00	-333.29	0.95	0.46
260	200	235	8.00	-119.27	0.76	0.49
270	200	210	8.00	172.24	1.10	0.9

7	275	225	235	8.00	-214.02	1.37	1.4
5	280	225	210	8.00	-68.18	0.44	0.1
7	290	210	220	6.00	-110.13	1.25	1.7
2	300	220	230	8.00	-90.78	0.58	0.3
0	310	230	200	8.00	182.43	1.16	1.0
8	320	230	240	12.00	397.03	1.13	0.7
5	330	240	190	6.00	78.50	0.89	0.9
2	340	240	250	12.00	-94.33	0.27	0.0
4	350	250	180	12.00	-232.25	0.66	0.2
3	360	250	260	12.00	820.81	2.33	2.4
1	370	260	160	8.00	-76.17	0.49	0.2
1	380	260	270	12.00	736.14	2.09	1.9
7	390	270	140	8.00	-349.38	2.23	3.5
8	400	270	280	8.00	-120.96	0.77	0.6
0	410	290	280	8.00	87.22	0.56	0.3
3	420	260	290	8.00	160.84	1.03	1.0
2	430	290	300	10.00	-619.36	2.53	4.1
5	440	255	300	12.00	155.41	0.44	0.1
3	445	255	250	12.00	844.59	2.40	3.0
3	450	300	310	10.00	-3.69	0.02	0.0
0	460	240	310	6.00	-63.15	0.72	0.7
3	470	310	320	10.00	-226.00	0.92	0.6
4	480	320	325	12.00	-78.26	0.22	0.0
4	485	325	230	12.00	701.74	1.99	2.1
5	490	320	330	6.00	37.27	0.42	0.2
8							

Link Results: (continued)

Link	Start Node	End Node	Diameter in	Flow gpm	Velocity fps	Headloss /1000ft
500	220	330	6.00	-59.35	0.67	0.65
510	330	340	6.00	-22.08	0.25	0.10
520	340	350	8.00	-91.38	0.58	0.36
530	350	320	12.00	185.01	0.52	0.18
540	350	355	8.00	38.76	0.25	0.07
545	355	360	8.00	162.31	1.04	1.03
546	355	2430	8.00	-123.55	0.79	0.62
550	360	310	8.00	111.73	0.71	0.52
560	2430	370	8.00	77.71	0.50	0.26
570	370	300	12.00	338.47	0.96	0.47
572	370	305	8.00	71.21	0.45	0.23
574	305	300	8.00	121.79	0.78	0.61
576	360	305	8.00	50.58	0.32	0.10
580	370	375	10.00	751.31	3.07	4.98
583	375	380	10.00	751.31	3.07	4.98
585	2450	370	12.00	472.84	1.34	0.64
590	380	290	12.00	-295.18	0.84	0.36
600	380	390	12.00	968.11	2.75	3.90
610	390	280	8.00	-403.77	2.58	3.98
620	2545	390	12.00	-978.57	2.78	3.34
625	400	2545	12.00	-916.30	2.60	2.96
630	280	410	8.00	562.49	3.59	10.30
640	410	420	8.00	186.05	1.19	1.3

3							
	650	420	400	8.00	-150.62	0.96	0.7
5							
	660	400	430	12.00	815.16	2.31	2.3
8							
	665	2570	430	12.00	-97.86	0.28	0.0
5							
	670	430	440	12.00	102.63	0.29	0.0
5							
	675	430	2585	12.00	245.00	0.70	0.2
6							
	680	420	440	6.00	99.37	1.13	1.6
9							
	690	410	450	6.00	22.83	0.26	0.1
1							
	700	450	460	12.00	22.83	0.06	0.0
0							
	710	460	470	12.00	422.29	1.20	0.6
0							
	720	505	500	6.00	700.00	7.94	62.6
7							
	725	7540	505	8.00	700.00	4.47	15.4
4							
	730	500	510	4.00	196.55	5.02	43.0
6							
	740	510	520	4.00	87.35	2.23	9.6
1							
	745	270	460	12.00	1206.48	3.42	3.6
1							
	750	520	530	6.00	87.35	0.99	1.3
3							
	760	530	540	6.00	153.88	1.75	3.8
0							
	770	530	550	12.00	-66.52	0.19	0.0
3							
	775	7520	550	8.00	500.00	3.19	8.2
8							
	780	550	560	12.00	433.48	1.23	0.8
8							
	785	7560	565	10.00	700.00	2.86	5.2
1							
	790	500	540	6.00	503.45	5.71	34.0
6							
	810	7510	570	8.00	120.00	0.77	0.5
9							
	820	570	580	12.00	-60.42	0.17	0.0
2							

		Start	End	Diameter	Flow	Velocity	Headlos
s	Link	Node	Node	in	gpm	fps	/1000f
t							
-	830	580	540	6.00	48.77	0.55	0.4
5	840	120	600	12.00	-642.88	1.82	1.8
3	850	570	600	8.00	180.42	1.15	1.2
6	860	600	610	8.00	-567.89	3.62	10.4
8	870	610	620	8.00	92.11	0.59	0.3
6	880	7580	610	8.00	660.00	4.21	13.8
5	890	580	630	6.00	-109.20	1.24	2.0
2	900	630	510	4.00	-109.20	2.79	14.5
2	910	600	640	6.00	105.44	1.20	1.8
9	920	640	650	6.00	73.61	0.84	0.9
7	930	650	620	6.00	-254.21	2.88	9.6
2	940	620	565	8.00	-162.10	1.03	1.0
3	945	565	560	8.00	537.90	3.43	9.4
8	950	560	660	8.00	507.68	3.24	8.5
2	960	650	660	8.00	-323.83	2.07	3.7
1	970	640	670	6.00	31.82	0.36	0.2
1	980	670	680	6.00	31.82	0.36	0.2
1	990	650	680	6.00	-54.44	0.62	0.5
6	1000	680	690	6.00	-22.62	0.26	0.1
1	1010	690	700	6.00	-22.62	0.26	0.1
1	1020	700	660	6.00	-183.84	2.09	5.2
8	1030	700	710	6.00	161.22	1.83	4.1
4	1040	560	710	6.00	463.70	5.26	29.2
5	1050	710	715	10.00	548.92	2.24	3.3
2	1052	715	7570	10.00	-500.00	2.04	2.7

9	1055	715	720	10.00	1048.92	4.28	11.0
0	1060	720	730	10.00	1123.43	4.59	12.4
9	1070	730	740	10.00	687.76	2.81	5.0
4	1080	740	755	8.00	594.18	3.79	11.4
0	1090	720	760	10.00	-74.51	0.30	0.0
8	1100	760	770	10.00	955.49	3.90	9.2
6	1110	770	740	12.00	-93.59	0.27	0.0
5	1120	770	780	10.00	704.08	2.88	5.2
6	1130	7530	760	12.00	1030.00	2.92	4.3
8	1140	7550	780	12.00	1025.00	2.91	4.3
4	1150	730	790	6.00	435.66	4.94	26.0
6	1160	790	805	6.00	-183.04	2.08	5.2
4	1170	750	800	12.00	-772.35	2.19	1.8
3	1175	755	805	6.00	183.04	2.08	3.7
4	1180	800	810	12.00	-772.35	2.19	1.8
3	1190	810	820	12.00	-1300.35	3.69	4.8
1	1200	820	830	12.00	-1159.10	3.29	3.8
9	1210	830	840	12.00	-1427.35	4.05	5.7
1	1220	840	850	12.00	-1017.35	2.89	3.0
5	1230	850	860	8.00	40.24	0.26	0.0
6	1240	860	870	8.00	-269.16	1.72	1.8
8							

Link Results: (continued)

	Start	End	Diameter	Flow	Velocity	Headlos
-						
s						
Link	Node	Node	in	gpm	fps	/1000f
t						

-	1250	850	870	12.00	-1057.59	3.00	3.2
8	1260	870	880	12.00	-1411.75	4.00	5.6
0	1270	7180	880	12.00	855.00	2.43	2.2
1	1280	7190	840	12.00	410.00	1.16	0.5
7	1290	820	890	12.00	-141.25	0.40	0.0
8	1300	830	890	8.00	268.25	1.71	1.8
7	1310	810	900	8.00	528.00	3.37	6.5
4	1320	900	910	8.00	-246.57	1.57	1.6
0	1330	750	910	12.00	1183.49	3.36	4.0
4	1335	755	750	8.00	411.14	2.62	4.1
2	1340	910	920	12.00	936.92	2.66	2.6
2	1350	920	930	12.00	1179.76	3.35	4.0
2	1360	930	940	12.00	-318.97	0.90	0.3
6	1370	900	940	8.00	318.97	2.04	2.5
7	1380	780	950	12.00	1729.08	4.91	8.1
5	1390	950	960	12.00	1224.23	3.47	4.3
0	1400	960	920	12.00	242.84	0.69	0.2
2	1410	960	970	8.00	362.59	2.31	3.2
6	1420	930	980	12.00	228.64	0.65	0.1
9	1430	980	970	12.00	-152.35	0.43	0.0
9	1440	970	990	12.00	210.25	0.60	0.1
7	1450	470	1000	12.00	1308.29	3.71	4.8
6	1460	1000	1010	12.00	1309.44	3.71	4.8
7	1470	950	1000	12.00	504.85	1.43	0.8
4	1480	1010	990	12.00	959.53	2.72	2.7
4	1490	1000	1020	12.00	503.70	1.43	0.8
3	1500	1040	440	12.00	-202.00	0.57	0.1

```
5
     1510     1040      460      12.00    -807.02        2.29        1.9
9
     1520     1040     1050      12.00     999.02        2.83        2.9
5
     1530     1050     1060      12.00     799.02        2.27        1.9
5
     1540     2585     2625      12.00    1226.34        3.48        4.3
2
     1542      430     2585      12.00     267.27        0.76        0.2
6
     1545     2625     1070      12.00    1046.18        2.97        3.2
2
     1550     1070     1060      12.00    -656.19        1.86        1.3
6
     1560     1060     1020      12.00      -4.16        0.01        0.0
0
     1570     1020     1030       8.00     499.54        3.19        5.9
0
     1580     1030     1010       8.00    -271.91        1.74        1.9
2
     1590     1030     1080       8.00     227.44        1.45        1.3
8
     1600     1080      990      12.00    -127.67        0.36        0.0
7
     1610     1080     1090      12.00     355.12        1.01        0.4
4
     1620     1090     1100      12.00     -22.15        0.06        0.0
0
     1630     1070     2690      12.00    1182.71        3.36        4.0
4
     1635     2690     1100      12.00     874.66        2.48        2.3
1
     1640     1100     1110      12.00    1045.44        2.97        3.2
1
     1650     1110     1120      12.00      29.46        0.08        0.0
0
     1660     1090     1120       8.00     322.67        2.06        2.6
3

  Page 20            CLOVIS WATER MASTER PLAN - HYDRAULIC MODEL


  Link Results: (continued)
  --------------------------------------------------------------
-
              Start      End  Diameter       Flow  Velocity  Headlos
s
      Link     Node     Node        in        gpm       fps   /1000f
t
  --------------------------------------------------------------
-
      1670      990     1130     12.00    1033.70      2.93      3.1
5
```

1680	1130	1140	12.00	971.86	2.76	2.8
1690	1140	1120	12.00	378.87	1.07	0.4
1700	980	1150	8.00	380.98	2.43	3.5
1710	1150	1130	8.00	-61.85	0.39	0.1
1720	1150	1160	8.00	143.63	0.92	0.5
1730	1140	1160	12.00	-543.29	1.54	0.9
1740	930	1170	12.00	1008.29	2.86	3.0
1750	1170	1180	12.00	1022.46	2.90	3.0
1760	1160	1170	12.00	556.34	1.58	1.0
1770	1170	1200	8.00	280.37	1.79	1.7
1780	1200	1180	8.00	280.37	1.79	1.7
1790	1180	1190	12.00	1041.03	2.95	2.7
1800	1190	1210	12.00	358.57	1.02	0.3
1810	1190	1260	12.00	682.47	1.94	1.2
1820	1110	2830	12.00	876.67	2.49	2.0
1822	2830	1230	12.00	684.54	1.94	1.2
1830	1140	1240	12.00	908.47	2.58	2.1
1840	1240	1250	12.00	905.87	2.57	2.1
1850	1230	1250	12.00	277.59	0.79	0.2
1860	1250	1210	12.00	-48.51	0.14	0.0
1870	7130	1240	12.00	300.00	0.85	0.2
1880	1210	1260	12.00	310.06	0.88	0.2
1900	1250	9100	12.00	21.57	0.06	0.0
1905	9100	9102	12.00	0.00		Close
1910	9102	9090	12.00	-0.00	0.00	0.0
1920	9090	9200	12.00	368.28	1.04	0.4
1930	1222	9200	16.00	-0.00	0.00	0.0
1935	1220	1222	16.00	0.00		Close

```
d
     1940     1220     1260      16.00     183.87       0.29      0.0
3
     2000      350     2400      12.00    -384.45       1.09      0.5
9
     2010     2400     3000      12.00     567.48       1.61      1.2
2
     2020      370     2450       8.00    -137.60       0.88      0.6
4
     2025      370     2450      12.00    -472.84       1.34      0.6
4
     2030     2450     3140       8.00     152.05       0.97      0.7
7
     2035     2450     3115      12.00     523.07       1.48      0.7
7
     2040     3140     3120       8.00      38.46       0.25      0.0
6
     2050     2500      380      12.00     -34.39       0.10      0.0
1
     2060     2500     3110      12.00    -697.03       1.98      1.7
8
     2070     2500     2510       8.00     212.72       1.36      1.4
3
     2080     2510     2520       6.00     212.72       2.41      5.8
0
     2090      390     2520      12.00     214.42       0.61      0.2
0
     2100     2520     3280      12.00     427.14       1.21      0.7
2
     2110      390     2530       8.00     178.89       1.14      1.2
4
     2120     2530     3280       8.00    -101.32       0.65      0.4
3
     2130     2530     2540       8.00     280.21       1.79      2.8
4

  Page 21           CLOVIS WATER MASTER PLAN - HYDRAULIC MODEL


  Link Results: (continued)
  ---------------------------------------------------------------
-
            Start      End  Diameter      Flow  Velocity  Headlos
s
     Link    Node     Node        in       gpm       fps   /1000f
t
  ---------------------------------------------------------------
-
     2140     2540     3270       6.00     -39.39       0.45      0.3
1
     2150     2545     2550       8.00      62.26       0.40      0.1
8
     2160      400     2550       8.00     -73.47       0.47      0.2
4
```

2170	2550	3260	8.00	-11.21	0.07	0.0
1						
2180	2570	2560	8.00	97.86	0.62	0.4
1						
2190	2560	3460	8.00	16.36	0.10	0.0
1						
2200	2560	2580	6.00	9.50	0.11	0.0
2						
2210	430	2580	8.00	102.40	0.65	0.4
4						
2220	2580	3450	8.00	111.90	0.71	0.5
2						
2230	2585	2600	8.00	285.92	1.82	2.9
5						
2240	2600	2590	8.00	-122.92	0.78	0.6
2						
2250	2590	3440	8.00	-279.27	1.78	2.8
2						
2260	2590	2595	8.00	156.35	1.00	0.9
6						
2270	2595	3430	8.00	-168.38	1.07	1.1
1						
2280	2595	2630	8.00	324.73	2.07	3.7
3						
2290	2630	2640	8.00	318.34	2.03	3.5
9						
2300	2610	2630	8.00	-6.39	0.04	0.0
0						
2305	2600	2610	8.00	266.84	1.70	2.1
7						
2310	2610	2620	8.00	101.03	0.64	0.4
3						
2320	2625	2620	8.00	180.17	1.15	1.2
5						
2340	2620	2660	8.00	281.20	1.79	2.8
6						
2350	1070	2660	12.00	414.66	1.18	0.6
8						
2360	2660	2650	12.00	695.86	1.97	1.7
8						
2370	2650	2640	12.00	262.56	0.74	0.2
9						
2380	2640	3420	12.00	580.90	1.65	1.2
7						
2390	2650	2670	8.00	433.29	2.77	5.3
3						
2400	2690	2680	8.00	308.04	1.97	2.8
3						
2403	2400	2420	10.00	201.26	0.82	0.4
4						
2405	2410	2420	10.00	0.00	0.00	0.0
0						
2407	2420	2430	12.00	201.26	0.57	0.1
8						
2410	2680	2670	8.00	184.87	1.18	1.1

0

2420	2670	3610	8.00	108.16	0.69	0.4	1
2430	2680	2700	8.00	123.18	0.79	0.5	2
2431	2700	2701	8.00	85.54	0.55	0.2	6
2440	2700	3600	12.00	675.38	1.92	1.4	3
2450	2700	2710	12.00	-637.74	1.81	1.2	9
2460	2710	1100	12.00	192.93	0.55	0.1	4
2470	2701	2720	6.00	157.75	1.79	2.8	4
2471	2710	2701	6.00	72.21	0.82	0.6	7
2480	2720	3600	8.00	-132.15	0.84	0.5	0
2490	2720	2740	8.00	289.89	1.85	2.1	6
2500	2710	2750	8.00	427.12	2.73	4.4	2
2510	2750	2730	8.00	-227.11	1.45	1.3	7
2520	2730	2740	8.00	-413.01	2.64	4.1	5
2530	2740	3590	8.00	-123.12	0.79	0.4	4
2540	2730	2770	8.00	185.90	1.19	0.9	5

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Link Results: (continued)

Link	Start Node	End Node	Diameter in	Flow gpm	Velocity fps	Headlos /1000f	s t
2550	2750	2780	8.00	134.03	0.86	0.5	2
2560	2750	2760	8.00	-0.00	0.00	0.0	0
2570	1110	2790	12.00	86.82	0.25	0.0	3
2580	2790	2780	12.00	-285.85	0.81	0.2	9
2590	2780	2770	12.00	-151.82	0.43	0.0	9

	2600	2770	3580	12.00	-207.57	0.59	0.1
6	2700	2770	2800	8.00	241.65	1.54	1.5
4	2710	2800	3800	8.00	-59.89	0.38	0.1
2	2720	2800	2810	8.00	301.55	1.92	2.3
2	2730	2790	2810	8.00	372.67	2.38	3.4
3	2740	2810	2820	8.00	-36.39	0.23	0.0
5	2750	2820	2830	8.00	-58.13	0.37	0.1
1	2760	2820	2840	8.00	21.74	0.14	0.0
2	2770	2840	3790	8.00	-173.46	1.11	0.8
3	2780	2840	2850	8.00	195.20	1.25	1.0
4	2800	1230	2850	12.00	-334.72	0.95	0.3
9	2810	2850	3770	12.00	-662.96	1.88	1.1
9	2820	2850	2860	12.00	523.44	1.48	0.7
7	2830	2860	2870	12.00	-429.95	1.22	0.5
4	2840	3770	2870	12.00	711.63	2.02	1.3
6	2850	2870	2880	12.00	281.68	0.80	0.2
4	2860	2860	2890	12.00	-223.00	0.63	0.1
6	2870	2890	2880	12.00	-237.24	0.67	0.1
8	3000	3000	3010	12.00	695.49	1.97	1.3
0	3010	3010	3020	12.00	474.99	1.35	0.6
4	3020	3020	3030	12.00	474.99	1.35	0.7
5	3030	3030	3040	12.00	-202.54	0.57	0.1
5	3040	3040	3050	12.00	1047.46	2.97	3.2
2	3050	3050	3060	12.00	785.03	2.23	1.8
9	3055	3055	3060	12.00	-0.56	0.00	0.0
0	3056	3055	3060	8.00	0.56	0.00	0.0
0	3060	3060	3070	12.00	293.03	0.83	0.3
1	3065	3060	3070	8.00	92.38	0.59	0.3

1							
	3070	3070	3080	12.00	-621.98	1.76	1.4
4							
	3080	3080	3090	12.00	-1560.78	4.43	7.9
2							
	3090	3090	3100	12.00	-2571.10	7.29	19.9
4							
	3092	3101	7620	12.00	-571.11	1.62	0.9
1							
	3093	3102	3100	12.00	571.11	1.62	0.9
1 CV							
	3096	3101	3104	12.00	0.00		Close
d							
	3097	3104	3100	12.00	-0.00	0.00	0.0
0							
	3100	3110	3090	12.00	-1010.32	2.87	3.5
4							
	3110	3120	3115	10.00	14.44	0.06	0.0
0							
	3112	3115	3110	10.00	537.51	2.20	2.6
8							
	3120	3120	3130	10.00	137.61	0.56	0.2
2							
	3130	3130	3135	10.00	-101.79	0.42	0.1
2							
	3132	3135	3000	12.00	223.21	0.63	0.1
8							

Link Results: (continued)

	Link	Start Node	End Node	Diameter in	Flow gpm	Velocity fps	Headlos /1000f
- s t -							
	3134	3135	3136	10.00	-325.00	1.33	1.0
6							
	3136	3136	3137	10.00	-325.00	1.33	1.0
6							
	3140	3120	3140	12.00	-113.60	0.32	0.0
6							
	3150	3070	3150	12.00	1289.75	3.66	5.5
6							
	3160	3150	3160	12.00	909.46	2.58	2.9
2							
	3170	3160	3170	12.00	42.01	0.12	0.0
1							
	3180	3170	3180	12.00	42.01	0.12	0.0
1							

709200060665169677777453988

3190	3180	3190	12.00	-198.29	0.56	0.1
3200	3190	3200	12.00	-310.35	0.88	0.4
3240	3200	3260	12.00	-1529.29	4.34	6.4
3250	3260	3270	12.00	-277.45	0.79	0.3
3255	3270	3260	8.00	73.84	0.47	0.2
3260	3260	3280	12.00	-800.15	2.27	2.3
3265	3280	3270	8.00	390.68	2.49	4.4
3270	3280	3110	10.00	-850.80	3.48	6.2
3280	3150	3290	6.00	130.40	1.48	2.8
3290	3290	3300	6.00	49.38	0.56	0.4
3300	3300	3310	6.00	-325.09	3.69	15.1
3310	3310	3280	6.00	14.21	0.16	0.0
3320	3310	3080	8.00	-424.30	2.71	6.1
3330	3290	3320	6.00	81.01	0.92	1.1
3340	3320	3180	8.00	81.01	0.52	0.2
3350	3300	3330	6.00	54.87	0.62	0.5
3360	3330	3260	6.00	-110.94	1.26	2.0
3370	3330	3190	8.00	165.81	1.06	0.7
3380	3160	3340	12.00	867.45	2.46	2.2
3390	3340	3350	12.00	1037.59	2.94	3.1
3400	3350	3360	12.00	1037.59	2.94	3.1
3402	3360	3365	6.00	-248.71	2.82	9.2
3405	3365	3475	6.00	101.29	1.15	1.7
3406	3475	3485	6.00	-40.98	0.47	0.3
3407	3475	3395	6.00	142.27	1.61	3.2
3410	3360	3370	12.00	1105.34	3.14	4.1
3420	3370	3375	12.00	606.48	1.72	1.3
3425	3375	3380	12.00	1324.54	3.76	5.8

```
5
     3430     3380     3390       12.00       178.81        0.51        0.1
4
     3440     3390     3395       12.00       -18.50        0.05        0.0
0
     3445     3395     3415       12.00       123.77        0.35        0.0
7
     3450     3415     3410       12.00       315.06        0.89        0.4
1
     3460     3410     3420       10.00       -86.94        0.36        0.0
9
     3470     3420     3430       10.00      -822.46        3.36        5.8
8
     3475     3400     3420       10.00       -75.60        0.31        0.0
7
     3480     3430     3440        8.00      -337.34        2.15        2.8
5
     3485     3440     3435       12.00       893.51        2.53        2.8
2
     3486     3435     3430       12.00       893.51        2.53        2.8
2
     3490     3440     3450       12.00      -846.89        2.40        2.1
8
```

Page 24   CLOVIS WATER MASTER PLAN - HYDRAULIC MODEL

Link Results: (continued)

```
  ---------------------------------------------------------------------------
-
               Start      End   Diameter        Flow   Velocity   Headlos
s
       Link     Node     Node         in         gpm        fps    /1000f
t
  ---------------------------------------------------------------------------
-
       3500     3450     3460      12.00     -734.99       2.09       1.6
7
       3505     3460     3440      12.00      695.57       1.97       1.7
8
       3510     3460     3200      12.00    -1218.94       3.46       5.0
1
       3520     3180     3470       8.00      117.32       0.75       0.4
0
       3530     3470     3480       6.00      199.93       2.27       5.1
7
       3535     3480     3485       6.00      232.27       2.64       6.8
3
       3540     3485     3415       6.00      191.29       2.17       4.7
7
       3550     3480     3440       6.00      -32.35       0.37       0.1
8
       3560     3190     3490       8.00      277.87       1.77       1.9
9
```

3570	3490	3470	8.00	82.61	0.53	0.2	1
3580	3490	3460	8.00	195.26	1.25	1.0	4
3590	3380	3500	12.00	1145.72	3.25	3.8	1
3600	3500	3510	12.00	531.86	1.51	0.9	2
3610	3510	3520	12.00	735.74	2.09	1.6	8
3620	3520	3530	12.00	893.15	2.53	2.4	0
3630	3530	3540	12.00	876.43	2.49	2.3	2
3640	3540	3550	12.00	657.09	1.86	1.3	6
3650	3550	3560	12.00	-279.44	0.79	0.2	8
3660	3560	3570	12.00	-134.40	0.38	0.0	7
3665	3570	3575	12.00	-388.61	1.10	0.5	1
3670	3575	3580	12.00	111.39	0.32	0.0	5
3680	3580	3590	12.00	-812.18	2.30	2.0	1
3690	3590	3600	12.00	-935.30	2.65	2.6	1
3693	3600	3605	12.00	229.57	0.65	0.1	7
3695	3605	3700	12.00	700.06	1.99	1.5	3
3700	3600	3610	12.00	-621.64	1.76	1.2	3
3710	3610	3620	12.00	-513.48	1.46	0.8	6
3720	3620	3630	12.00	-983.96	2.79	2.8	7
3730	3630	3420	12.00	-1240.83	3.52	4.4	1
3740	3390	3640	6.00	197.31	2.24	6.0	2
3745	3640	4170	6.00	-262.09	2.97	10.1	8
3750	3640	3650	8.00	375.52	2.40	3.4	8
3760	3650	3510	8.00	203.88	1.30	1.1	2
3770	3640	3660	8.00	-89.51	0.57	0.2	5
3780	3660	3410	8.00	-402.00	2.57	3.9	5
3790	3650	3670	8.00	171.63	1.10	0.8	2
3800	3670	3680	8.00	42.65	0.27	0.0	

6							
2	3810	3680	3630	8.00	-256.87	1.64	1.7
8	3820	3660	3680	8.00	312.49	1.99	2.4
9	3830	3520	3690	12.00	-285.91	0.81	0.2
2	3840	3690	3700	12.00	-563.71	1.60	1.0
3	3860	3620	3605	12.00	470.48	1.33	0.7
8	3870	3670	3700	8.00	128.99	0.82	0.4
9	3880	3690	3720	8.00	277.79	1.77	1.9
2	3890	3720	3730	8.00	294.51	1.88	2.2
	3900	3700	3730	8.00	265.33	1.69	1.8
3							

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Link Results: (continued)

-		Start	End	Diameter	Flow	Velocity	Headlos
s	Link	Node	Node	in	gpm	fps	/1000f
t							
-							
0	3910	3730	3560	8.00	145.04	0.93	0.6
7	3920	3550	3740	12.00	594.01	1.69	0.9
2	3930	3740	3750	12.00	424.69	1.20	0.5
0	3940	3750	3760	12.00	17.23	0.05	0.0
9	3950	3760	3770	12.00	-160.49	0.46	0.0
1	3960	3770	3780	12.00	-373.30	1.06	0.4
6	3970	3780	3790	12.00	-482.64	1.37	0.6
7	3980	3790	3800	12.00	-656.10	1.86	1.1
8	3990	3800	3580	12.00	-716.00	2.03	1.3
9	4000	3740	3810	8.00	169.32	1.08	0.6
	4010	3810	3820	8.00	-363.56	2.32	2.8
3							

4020	3820	3780	8.00	-109.34	0.70	0.31
4030	3570	3820	8.00	254.22	1.62	1.46
4040	3810	3760	8.00	-177.72	1.13	0.75
4050	3030	3830	12.00	677.53	1.92	1.44
4060	3830	3840	12.00	470.84	1.34	0.73
4070	3840	3850	12.00	483.68	1.37	0.77
4080	3850	3860	12.00	543.93	1.54	0.96
4090	3860	3870	12.00	376.72	1.07	0.49
4100	3870	3880	12.00	210.51	0.60	0.17
4110	3880	3890	8.00	-149.50	0.95	0.63
4120	3890	3060	8.00	-231.62	1.48	1.42
4130	3050	3900	8.00	262.43	1.68	1.79
4140	3900	3910	8.00	125.36	0.80	0.46
4150	3910	3830	8.00	-206.69	1.32	1.15
4160	3910	3920	8.00	243.66	1.56	1.56
4170	3920	3850	8.00	60.25	0.38	0.12
4180	3920	3930	8.00	183.40	1.17	0.92
4190	3930	3870	8.00	-114.21	0.73	0.38
4200	3900	3940	8.00	137.07	0.87	0.54
4210	3940	3950	6.00	137.07	1.56	2.19
4220	3950	3890	6.00	-82.12	0.93	0.85
4230	3950	3930	8.00	219.18	1.40	1.29
4240	3880	3960	12.00	443.83	1.26	0.66
4250	3960	3970	10.00	443.83	1.81	1.88
4260	3970	3980	12.00	-378.05	1.07	0.57
4270	3980	3070	12.00	-697.65	1.98	1.79
4280	3970	3990	10.00	681.13	2.78	4.15
4290	3990	4000	10.00	446.77	1.83	1.9

0

4300	4000	4010	12.00	428.11	1.21	0.6	2
4310	4010	4020	12.00	-160.63	0.46	0.1	0
4320	4020	4030	12.00	-367.69	1.04	0.4	6
4330	4030	3340	12.00	-679.25	1.93	1.4	5
4340	3340	4035	8.00	-849.39	5.42	18.5	1
4345	4035	4040	8.00	350.61	2.24	3.6	0
4350	4040	4050	8.00	155.04	0.99	0.8	0

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Link Results: (continued)

Link	Start Node	End Node	Diameter in	Flow gpm	Velocity fps	Headlos s /1000f t	
4360	3150	4050	8.00	239.39	1.53	1.7	8
4370	4050	4060	8.00	394.43	2.52	4.4	8
4380	4060	4070	8.00	49.02	0.31	0.0	9
4390	4060	4080	8.00	213.10	1.36	1.4	3
4400	4080	4030	12.00	121.46	0.34	0.0	7
4410	4040	4080	6.00	195.57	2.22	5.9	2
4420	4080	4070	6.00	-83.39	0.95	1.2	2
4430	4070	3990	8.00	-34.37	0.22	0.0	5
4440	4000	4090	8.00	326.69	2.09	2.6	9
4450	4090	4100	12.00	1483.33	4.21	6.1	4
4460	4100	4110	12.00	1090.88	3.09	3.4	8
4470	4110	4120	12.00	949.15	2.69	2.6	9
4480	4120	4130	12.00	597.24	1.69	1.1	4

	4481	4120	4130	8.00	171.71	1.10	1.1
5							
	4490	4130	4140	10.00	237.91	0.97	0.5
9							
	4491	4130	4140	12.00	454.36	1.29	0.5
9							
	4500	4140	4150	10.00	96.10	0.39	0.1
1							
	4501	4140	4150	12.00	183.54	0.52	0.1
1							
	4510	4150	4160	8.00	96.97	0.62	0.3
3							
	4511	4150	4160	12.00	333.22	0.95	0.3
3							
	4520	4160	4170	6.00	9.19	0.10	0.0
0							
	4521	4160	4170	12.00	25.96	0.07	0.0
0							
	4530	4170	3375	8.00	-226.94	1.45	1.3
7							
	4540	4030	4180	8.00	241.03	1.54	1.5
3							
	4550	4180	4190	8.00	430.20	2.75	4.4
8							
	4560	4190	4200	8.00	471.16	3.01	5.3
0							
	4570	4200	4150	8.00	161.05	1.03	0.7
3							
	4580	3370	4210	12.00	498.86	1.42	0.8
2							
	4590	4210	4160	12.00	598.17	1.70	1.1
4							
	4620	4210	4200	8.00	-99.31	0.63	0.3
0							
	4630	4200	4120	8.90	-180.20	0.93	0.6
3							
	4640	4110	4250	8.00	53.53	0.34	0.0
9							
	4650	4190	3360	6.00	-40.96	0.46	0.2
8							
	4660	4020	4260	8.00	207.06	1.32	1.1
6							
	4670	4010	4270	12.00	588.75	1.67	1.1
1							
	4680	4180	4260	10.00	-189.17	0.77	0.3
9							
	4690	4260	4270	10.00	17.89	0.07	0.0
0							
	4700	4270	4090	12.00	1156.63	3.28	3.8
7							
	4710	4160	4280	12.00	993.21	2.82	2.9
2							
	4720	4280	4290	12.00	1347.84	3.82	5.1
4							
	4730	4290	4300	12.00	878.21	2.49	2.3

3

4740 4300 4310 12.00 654.14 1.86 1.3

5

4750 4310 4320 12.00 -276.32 0.78 0.2

7

4760 4320 4330 12.00 -578.67 1.64 1.0

8

4770 4330 4340 12.00 -128.50 0.36 0.0

7

4780 4340 3520 8.00 -128.50 0.82 0.4

8

Link Results: (continued)

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Start End Diameter Flow Velocity Headlos

s

Link Node Node in gpm fps /1000f

t

\-

4790 4340 4350 12.00 -0.00 0.00 0.0

0

4800 3500 4360 8.00 199.06 1.27 1.0

8

4810 4360 4330 12.00 450.17 1.28 0.6

8

4820 4360 4290 8.00 -251.10 1.60 1.6

5

4830 4140 4280 8.00 412.63 2.63 4.1

4

4840 4130 4380 10.00 26.35 0.11 0.0

1

4841 4130 4380 12.00 50.33 0.14 0.0

1

4850 4380 4290 8.00 375.54 2.40 3.4

8

4860 4290 4390 8.00 289.57 1.85 2.1

5

4865 4390 4300 8.00 289.57 1.85 2.1

5

4870 4300 4400 12.00 806.85 2.29 1.7

2

4880 4400 4410 12.00 559.90 1.59 0.8

7

4890 7630 4420 20.00 -0.00 0.00 0.0

0

4891 4425 4410 20.00 -0.00 0.00 0.0

0 CV

4895 4420 4410 20.00 0.00 Close

d

4900	4410	4430	18.00	559.89	0.71	0.1	2
4910	4430	4440	12.00	624.42	1.77	1.0	7
4920	4440	4450	12.00	242.75	0.69	0.1	9
4930	4450	4460	12.00	297.58	0.84	0.2	7
4940	4460	4470	12.00	121.60	0.34	0.0	5
4950	4470	4480	12.00	180.64	0.51	0.1	1
4960	4480	3550	12.00	-192.92	0.55	0.1	2
4970	4320	4490	8.00	302.36	1.93	2.3	3
4980	4490	4500	8.00	-160.30	1.02	0.7	2
4990	4500	4470	8.00	59.04	0.38	0.1	1
5000	3720	3530	8.00	-16.72	0.11	0.0	1
5010	3540	4500	8.00	219.35	1.40	1.2	9
5020	4310	4510	12.00	930.46	2.64	2.5	9
5030	4510	4520	12.00	751.36	2.13	1.7	4
5040	4520	4450	12.00	540.82	1.53	0.9	5
5050	4510	4530	8.00	153.90	0.98	0.5	8
5060	4530	4400	8.00	-246.95	1.58	1.3	8
5070	4490	4520	8.00	-210.54	1.34	1.1	9
5080	4530	4440	8.00	176.45	1.13	0.7	4
5090	4440	4445	8.00	133.12	0.85	0.4	4
5095	4540	4445	8.00	-133.12	0.85	0.4	4
5100	4540	4550	8.00	159.23	1.02	0.6	1
5110	4550	4560	8.00	335.21	2.14	2.4	3
5120	4560	4570	8.00	-102.02	0.65	0.2	7
5130	4570	4580	8.00	-99.06	0.63	0.2	6
5140	4580	4590	8.00	-171.59	1.10	0.7	0
5150	4480	4590	10.00	373.56	1.53	1.0	0
5160	4590	4560	8.00	201.97	1.29	0.9	

5							
	5170	4460	4550	8.00	175.98	1.12	0.7
4							
	5180	4450	4540	12.00	485.98	1.38	0.6
7							
	5190	4540	4600	12.00	459.88	1.30	0.6
1							

Link Results: (continued)

		Start	End	Diameter	Flow	Velocity	Headlos
-							
s	Link	Node	Node	in	gpm	fps	/1000f
t							
-							
	5200	4600	4610	12.00	220.53	0.63	0.1
6							
	5210	4610	4620	12.00	217.58	0.62	0.1
5							
	5220	4620	3750	12.00	290.11	0.82	0.2
6							
	5230	4570	4610	8.00	-2.95	0.02	0.0
0							
	5240	4580	4620	8.00	72.53	0.46	0.1
4							
	5250	3840	4630	12.00	-12.84	0.04	0.0
0							
	5260	4630	3860	8.00	101.39	0.65	0.3
1							
	5270	3880	4640	12.00	-83.82	0.24	0.0
3							
	5280	4640	4650	12.00	-287.60	0.82	0.3
0							
	5290	4650	4660	12.00	872.40	2.47	2.3
0							
	5300	4660	4670	12.00	658.09	1.87	1.3
6							
	5310	4680	4690	14.00	-1557.43	3.25	3.1
7							
	5320	4690	4700	12.00	-62.43	0.18	0.0
2							
	5330	4700	4710	12.00	-69.12	0.20	0.0
2							
	5340	4710	3970	12.00	-140.74	0.40	0.0
8							
	5350	4660	4700	8.00	214.30	1.37	1.2
3							
	5360	4640	4710	8.00	203.78	1.30	1.3
2							

5370	4000	4720	10.00	-308.04	1.26	0.9	6
5380	4100	4730	12.00	392.45	1.11	0.5	2
5390	4730	4740	12.00	68.43	0.19	0.0	2
5400	4740	4750	12.00	-188.04	0.53	0.1	3
5410	4730	4760	8.00	324.02	2.07	2.6	5
5420	4760	4250	8.00	196.62	1.25	1.0	5
5430	4250	4770	8.00	250.14	1.60	1.6	4
5435	4755	4760	8.00	190.73	1.22	0.9	9
5440	4740	4760	8.00	256.47	1.64	1.7	2
5450	4380	4770	10.00	-103.88	0.42	0.1	3
5451	4380	4770	12.00	-198.38	0.56	0.1	3
5460	4770	4780	10.00	-69.46	0.28	0.0	6
5461	4770	4780	12.00	-132.66	0.38	0.0	6
5470	4430	5865	18.00	-64.53	0.08	0.0	0
5480	4600	4800	12.00	239.35	0.68	0.1	8
5600	4630	5000	12.00	885.78	2.51	2.0	4
5610	5000	5040	12.00	516.33	1.46	0.7	5
5620	5040	5010	12.00	349.39	0.99	0.3	6
5630	5010	5090	12.00	-373.87	1.06	0.4	1
5640	5090	5100	12.00	556.13	1.58	0.8	6
5650	5100	5110	12.00	185.10	0.53	0.1	1
5660	5110	5020	12.00	238.02	0.68	0.1	8
5670	5020	5080	12.00	-819.76	2.33	1.7	7
5680	5080	5030	12.00	313.26	0.89	0.3	0
5710	5040	5050	8.00	78.54	0.50	0.1	7
5720	5110	5050	8.00	-52.92	0.34	0.0	8
5730	5080	5050	8.00	276.98	1.77	1.7	1
5750	5020	5025	12.00	116.65	0.33	0.0	

5

5780	5025	5200	12.00	1246.65	3.54	3.84

Link Results: (continued)

Link	Start Node	End Node	Diameter in	Flow gpm	Velocity fps	Headloss /1000ft
5790	5200	5210	12.00	1327.88	3.77	4.31
5800	5210	5500	12.00	-727.94	2.07	1.42
5810	5500	4680	12.00	-1414.74	4.01	4.85
5820	4680	4670	12.00	-762.80	2.16	1.55
5830	4670	5030	12.00	-272.70	0.77	0.23
5840	4680	4725	12.00	905.49	2.57	2.46
5850	4725	4685	12.00	433.65	1.23	0.63
5860	4725	4720	10.00	465.54	1.90	1.51
5900	5100	5120	8.00	371.04	2.37	2.94
5910	5120	5130	8.00	-115.16	0.74	0.34
5920	5130	5140	12.00	124.76	0.35	0.05
5930	5140	5150	12.00	-201.64	0.57	0.13
5940	5150	5300	12.00	701.22	1.99	1.32
5950	5150	5240	12.00	533.17	1.51	0.80
5960	5240	5230	12.00	579.64	1.64	0.93
5970	5230	5220	12.00	1776.24	5.04	7.39
5980	5220	5310	14.00	-510.18	1.06	0.35
5990	5300	5310	12.00	579.71	1.64	0.93
6000	5220	5215	14.00	58.67	0.12	0.01

6010	5215	5210	14.00	-988.53	2.06	1.1	8
6020	5240	5250	8.00	-165.47	1.06	0.6	6
6030	5250	5160	8.00	-165.47	1.06	0.6	6
6040	5150	5160	12.00	-436.03	1.24	0.5	5
6045	5160	5170	12.00	-601.50	1.71	1.0	0
6050	5170	5020	12.00	-941.12	2.67	2.2	8
6060	5170	5190	8.00	42.72	0.27	0.0	5
6070	5190	5200	8.00	81.22	0.52	0.1	8
6080	5190	5180	8.00	-38.51	0.25	0.0	4
6090	5170	5180	8.00	38.51	0.25	0.0	4
6100	5220	5400	12.00	742.19	2.11	1.4	7
6110	5400	5410	8.00	137.03	0.87	0.4	7
6120	5410	5430	8.00	-253.50	1.62	1.4	5
6130	5430	5210	12.00	-938.09	2.66	2.2	7
6140	5430	5440	12.00	664.19	1.88	1.2	0
6150	5410	5420	8.00	125.33	0.80	0.3	9
6155	5420	8210	12.00	410.55	1.16	0.4	9
6160	5420	5440	12.00	-285.22	0.81	0.2	5
6170	5630	5450	12.00	-962.25	2.73	2.3	8
6180	5450	5440	12.00	-317.25	0.90	0.3	0
6190	5440	5600	12.00	61.72	0.18	0.0	1
6200	5600	5610	12.00	-340.51	0.97	0.3	5
6210	5610	4750	12.00	-15.51	0.04	0.0	0
6220	5600	5620	8.00	402.23	2.57	3.4	1
6230	5620	5630	8.00	-122.48	0.78	0.3	8
6240	5620	5670	8.00	194.91	1.24	0.8	9
6250	5670	5650	8.00	194.91	1.24	0.8	9

Link Results: (continued)

Link	Start Node	End Node	Diameter in	Flow gpm	Velocity fps	Headloss /1000ft
6260	5630	5640	12.00	839.76	2.38	1.85
6270	5640	5770	12.00	-154.81	0.44	0.08
6280	5650	5640	12.00	-138.88	0.39	0.07
6290	5650	5660	12.00	333.79	0.95	0.34
6300	5660	5700	12.00	40.71	0.12	0.01
6310	4750	4755	14.00	1222.10	2.55	1.75
6320	4755	4780	12.00	888.57	2.52	2.38
6330	5700	4780	12.00	409.23	1.16	0.49
6340	5700	5710	10.00	511.48	2.09	1.79
6350	4780	5720	12.00	1083.09	3.07	2.96
6360	5710	5720	12.00	576.50	1.64	0.92
6370	5660	5680	8.00	293.08	1.87	1.90
6380	5680	5690	8.00	-57.12	0.36	0.09
6390	5640	5690	12.00	855.69	2.43	1.91
6500	5690	5730	12.00	560.58	1.59	0.87
6510	5730	5740	12.00	-589.05	1.67	0.96
6520	5740	5750	12.00	-877.47	2.49	2.00
6530	5740	5780	8.00	288.42	1.84	1.84
6540	5780	5770	8.00	-405.18	2.59	3.46
6550	5770	5760	12.00	-559.99	1.59	0.87
6600	5752	5830	12.00	930.31	2.64	2.23

6610	5830	5840	12.00	562.26	1.60	0.8	8
6620	5830	5990	12.00	491.48	1.39	0.6	9
6625	5990	6100	12.00	1228.93	3.49	3.7	4
6630	5830	5820	12.00	258.21	0.73	0.2	1
6640	5820	5810	12.00	57.61	0.16	0.0	1
6650	5810	5800	12.00	-282.39	0.80	0.2	5
6660	5800	5850	14.00	-881.44	1.84	0.9	5
6670	5850	5866	14.00	-904.54	1.89	1.0	0
6672	5866	5865	12.00	0.00		Close	d
6680	5860	5720	12.00	-1609.59	4.57	6.1	5
6690	5865	5860	12.00	-840.38	2.38	1.8	5
6700	5865	5870	12.00	775.85	2.20	1.6	0
6710	5870	5880	12.00	775.85	2.20	1.6	0
6730	5880	4800	12.00	357.65	1.01	0.3	8
6740	4800	5890	12.00	487.00	1.38	0.6	7
6750	5890	5900	12.00	391.00	1.11	0.4	5
7000	5010	8000	12.00	723.26	2.05	1.4	0
7010	8000	5130	12.00	239.93	0.68	0.1	8
7020	8000	8010	12.00	734.37	2.08	1.4	4
7030	8010	5300	12.00	-164.91	0.47	0.0	9
7040	8010	8020	12.00	899.28	2.55	2.1	0
7050	8020	8090	12.00	226.02	0.64	0.1	6
7060	8020	8030	12.00	305.20	0.87	0.2	8
7070	8030	8080	12.00	-176.30	0.50	0.1	0
7080	8030	8040	12.00	481.50	1.37	0.6	6

Link Results: (continued)

Link	Start Node	End Node	Diameter in	Flow gpm	Velocity fps	Headloss /1000ft
7090	8040	8070	12.00	199.19	0.57	0.13
7100	8040	8050	12.00	-85.75	0.24	0.03
7110	8050	8060	12.00	914.25	2.59	2.16
7120	8060	8130	12.00	148.04	0.42	0.07
7130	8070	8060	12.00	601.85	1.71	1.00
7140	8120	8130	12.00	-148.04	0.42	0.07
7150	8070	8120	12.00	927.24	2.63	2.22
7160	8070	8080	12.00	-329.90	0.94	0.33
7170	8110	8120	12.00	292.78	0.83	0.26
7180	8080	8110	12.00	562.86	1.60	0.88
7190	8080	8090	12.00	-69.06	0.20	0.02
7200	8100	8110	12.00	584.58	1.66	0.95
7210	8090	8100	14.00	1113.56	2.32	1.47
7220	5300	8090	12.00	956.59	2.71	2.35
7230	5310	8100	12.00	1069.54	3.03	2.89
7300	5400	8200	16.00	2341.76	3.74	3.03
7310	8200	5760	18.00	1566.40	1.97	0.81
7320	8200	8210	12.00	-410.55	1.16	0.49
7330	5750	5760	18.00	-941.85	1.19	0.32
7331	5750	5754	24.00	0.00	0.00	0.00
CV 7332	5754	5751	20.00	0.00		Closed
7333	7640	5751	24.00	4500.00	3.19	1.41
7350	5710	5730	12.00	-660.03	1.87	1.18

	7360	5730	5800	12.00	0.00		Close
d							
	7400	5000	5030	12.00	137.44	0.39	0.0
6							
	7410	4685	4750	12.00	1425.65	4.04	4.9
2							
	7420	4300	5860	12.00	-293.21	0.83	0.2
6							
	7500	5840	6000	24.00	3012.85	2.14	0.6
7							
	7510	5830	5840	20.00	2450.60	2.50	1.1
1							
	7520	5830	5752	20.00	-3569.69	3.65	2.2
3							
	7530	5220	5400	12.00	736.60	2.09	1.4
5							
	8000	5800	6010	12.00	410.05	1.16	0.4
9							
	8010	6000	6010	12.00	-322.42	0.91	0.3
1							
	8020	6010	6020	12.00	-282.96	0.80	0.2
5							
	8030	6000	6120	16.00	1966.75	3.14	2.2
0							
	8040	6120	6150	16.00	-554.01	0.88	0.2
1							
	8050	6120	6130	12.00	-862.46	2.45	1.9
4							
	8060	6130	6140	12.00	844.01	2.39	1.8
6							
	8070	6140	6150	12.00	-203.93	0.58	0.1
3							
	8080	6020	6130	24.00	3312.50	2.35	0.8
0							
	8090	6130	6150	36.00	12122.55	3.82	1.2
2							
	8100	6000	6100	14.00	1368.52	2.85	2.1
5							
	8110	6100	6110	14.00	1341.13	2.80	2.0
7							
	8120	6110	6120	12.00	-897.77	2.55	2.0
9							
	8130	6110	6170	14.00	562.79	1.17	0.4
2							
	8140	6170	6180	14.00	-336.16	0.70	0.1
6							

Link Results: (continued)

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	Start	End	Diameter	Flow	Velocity	Headlos

s								
		Link	Node	Node	in	gpm	fps	/1000f
t								
-								
		8150	6150	6160	18.00	3402.00	4.29	3.4
1								
		8160	6160	6170	14.00	777.15	1.62	0.7
6								
		8170	6160	6180	18.00	948.75	1.20	0.3
2								
		8180	6150	6200	30.00	7962.60	3.61	1.3
7								
		8190	6200	6204	30.00	0.00		Close
d	CV							
		8200	6190	6300	24.00	8000.00	5.67	4.0
9								
		8201	6180	6188	24.00	0.00		Close
d	CV							
		8202	6188	6190	24.00	8000.00	5.67	4.0
9								
		8203	6180	6182	16.00	0.00		Close
d								
		8204	6182	6184	16.00	-0.00	0.00	0.0
0	CV							
		8210	6180	6200	16.00	-761.50	1.22	0.3
8								
		8220	6300	6280	24.00	7461.00	5.29	3.5
9								
		8230	6280	6290	16.00	2045.31	3.26	2.3
6								
		8240	6270	6280	16.00	-2719.19	4.34	4.0
0								
		8250	6210	6290	16.00	1213.27	1.94	0.9
0								
		8260	6250	6280	18.00	-2696.50	3.40	2.2
2								
		8270	6260	6270	16.00	539.39	0.86	0.2
0								
		8280	6204	6210	24.00	5573.32	3.95	2.0
9								
		8290	6210	6250	20.00	1988.51	2.03	0.7
6								
		8300	6250	6260	18.00	2244.15	2.83	1.5
8								
		8310	6210	6220	16.00	2371.54	3.78	3.1
1								
		8320	6230	6250	18.00	-2440.87	3.08	1.8
5								
		8330	6240	6260	16.00	-1704.75	2.72	1.6
9								
		8340	6220	6230	16.00	-887.04	1.42	0.5
0								
		8350	6230	6240	16.00	1553.83	2.48	1.4
2								

8400	8100	8220	18.00	230.46	0.29	0.0	2
8405	8200	8220	18.00	1185.91	1.50	0.4	9
8410	8220	8225	16.00	132.86	0.21	0.0	2
8415	8225	8230	16.00	197.42	0.32	0.0	3
8420	8230	5990	16.00	0.00		Close	d
8425	8225	8235	12.00	-64.56	0.18	0.0	2
8430	8235	5760	12.00	-64.56	0.18	0.0	2
8435	8230	8240	12.00	197.42	0.56	0.1	3
8440	8240	5750	12.00	197.42	0.56	0.1	3
8445	5990	8245	14.00	-737.45	1.54	0.6	9
8450	8245	5830	14.00	-737.45	1.54	0.6	9
9000	7700	7701	36.00	20315.37	6.40	3.1	8
9001	7701	7702	36.00	20315.37	6.40	3.1	8
9050	6020	6030	24.00	-3595.46	2.55	0.9	3
9051	6030	5866	20.00	904.54	0.92	0.1	8
9054	7630	6050	20.00	4500.00	4.60	3.4	3
9080	6130	4810	36.00	-10516.52	3.31	0.9	4
9100	4810	4820	36.00	-10516.52	3.31	0.9	4
9150	4820	4830	36.00	-10516.52	3.31	0.9	4
9200	3750	4831	12.00	479.16	1.36	0.6	5
9210	4830	4831	12.00	0.00		Close	d

Page 33 CLOVIS WATER MASTER PLAN - HYDRAULIC MODEL

Link Results: (continued)

-

	Start	End	Diameter	Flow	Velocity	Headlos
						s
Link	Node	Node	in	gpm	fps	/1000f
						t

-

9230	4830	4840	36.00	-10781.72	3.40	0.9	9
9260	2880	4831	16.00	-479.16	0.76	0.1	6
9280	4840	4850	36.00	-10781.72	3.40	0.9	9
9300	2882	3773	24.00	1161.65	0.82	0.1	2
9310	4850	2882	24.00	1161.65	0.82	0.1	2
9320	4850	4860	36.00	-13257.07	4.18	1.4	4
9340	2890	2900	12.00	-55.76	0.16	0.0	1
9350	1230	2900	12.00	218.07	0.62	0.1	5
9380	4860	9009	30.00	7058.30	3.20	1.0	9
9390	2900	2902	12.00	0.00		Close	d
9400	2902	9000	12.00	-0.00	0.00	0.0	0
9420	9000	9010	12.00	-1286.46	3.65	4.0	7
9440	9010	9020	18.00	3048.92	3.84	2.7	9
9460	9020	9030	18.00	1873.14	2.36	1.1	3
9480	9030	9040	18.00	594.03	0.75	0.1	4
9500	2900	9100	12.00	162.30	0.46	0.0	9
9540	9000	9090	12.00	522.40	1.48	0.7	7
9570	9010	9080	16.00	2649.12	4.23	3.8	1
9590	9020	9070	12.00	792.24	2.25	1.6	6
9600	9030	9060	12.00	705.31	2.00	1.3	4
9610	9040	9050	12.00	594.03	1.69	0.9	7
9620	9090	9080	12.00	-609.94	1.73	1.0	2
9640	9070	9080	12.00	-470.50	1.33	0.6	3
9660	9060	9070	12.00	-537.91	1.53	0.8	1
9680	9050	9060	12.00	271.60	0.77	0.2	3
9700	1220	9100	12.00	-183.87	0.52	0.1	1
9720	9080	9210	16.00	1304.62	2.08	1.0	3

9740	9070	9220	12.00	460.77	1.31	0.61	
9760	9060	9230	12.00	367.22	1.04	0.40	
9780	9050	9240	12.00	322.44	0.91	0.31	
9820	9200	9210	16.00	-779.32	1.24	0.40	
9840	9210	9220	12.00	261.24	0.74	0.21	
9860	9220	9230	12.00	457.95	1.30	0.60	
9880	9230	9240	12.00	-322.44	0.91	0.31	
3094	3101	3102		571.11	6 hp	-40.43	Pump
4892	4420	4425		0.00		Closed	Pump
7334	5751	5756		4500.00	250 hp	-219.78	Pump
7336	5750	5758		0.00		Closed	Pump
8182	6200	6202		5573.32	25 hp	-17.75	Pump
8206	6184	6186		8000.00	100 hp	-49.45	Pump
9052	6050	6040		4500.00	250 hp	-219.78	Pump
7335	5756	5752	24.00	4500.00	3.19	20.69	FCV
7337	5758	5752	24.00	-0.00	0.00	0.00	FCV
8184	6202	6204	30.00	5573.32	2.53	0.00	PSV
8207	6186	6188	24.00	8000.00	5.67	15.73	FCV
9002	7702	4860	30.00	20315.37	9.22	0.00	FCV

Link Results: (continued)

Link	Start Node	End Node	Diameter in	Flow gpm	Velocity fps	Headloss /1000ft	
9053	6040	6030	20.00	4500.00	4.60	21.83	FCV
9301	3773	3770	20.00	1161.65	1.19	120.0	

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7   PRV
       9381      9009      9010        24.00   7058.30        5.01       74.3
7   PRV
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APPENDIX B

Regulations Affecting
Clovis Water Treatment Facility

Water Treatment Regulatory Requirements

The first national standards for drinking water quality were established by the U.S. Public Health Service in 1914. The standards were revised in 1925, 1942, 1946, and 1962. In 1974, the Safe Drinking Water Act (SDWA) transferred responsibility for public water supplies to the U.S. Environmental Protection Agency (EPA). EPA has recently revised the SDWA to include a broad spectrum of contaminants not previously regulated.

A. Current Regulations

1. The Safe Drinking Water Act of 1974

The Safe Drinking Water Act of 1974 mandated that Primary Drinking Water Regulations be established for a number of chemical, physical, and biological constituents. These regulations consisted of maximum contaminant levels (MCLs) for individual contaminants and identified treatment technologies that could be used to comply with the MCLs. Following passage of this law, EPA promulgated National Interim Primary Drinking Water Regulations (NIPDWR), which went into effect in June 1977. These regulations established MCLs for ten inorganic chemicals, six organic chemicals, two categories of radioactive contaminants, turbidity, and coliform organisms.

2. Total Trihalomethanes Regulation

In 1979, EPA promulgated a final rule for the control of four trihalomethane compounds (chloroform, dibromochloromethane, bromodichloromethane, and bromoform). The MCL for the sum of these four contaminants (collectively referred to as total trihalomethanes (TTHMs)) was set at 0.10 mg/L, measured as a running annual average of the last four quarterly monitoring samples.

3. Fluoride

In April 1986, EPA promulgated an MCL for fluoride of 4.0 mg/L, and a Secondary MCL (SMCL) of 2.0 mg/L. While the SMCL is not a federally enforceable standard, individual state regulatory agencies are free to make the SMCL mandatory for public water supplies. However, EPA requires water systems which exceed the SMCL to notify their consumers.

4. The 1986 Safe Drinking Water Act Amendments

Amendments to the Safe Drinking Water Act of 1974 became law in June 1986. In passing these amendments, Congress determined that it is the responsibility of the Federal Government to determine what constitutes "safe" drinking water. The 1986 Amendments empowered EPA to set enforceable standards for contaminants in drinking water based upon the level of removal that can be achieved using the "best available" technology, and removed EPA's discretion in determining whether to set standards for contaminants in drinking water. The 1986 Amendments also gave EPA the power to enforce standards by issuing administrative enforcement orders, rather than the time-consuming (and largely ineffective) process of obtaining court orders to correct system deficiencies.

The 1986 Amendments required EPA to develop regulations and exercise stricter control of trace contaminants, many of which were relatively unknown when the original SDWA was passed, and mandated the development of standards for 83 specific contaminants or contaminant groups by mid-1989, with additional contaminants to be added every three years thereafter. (The 1996 Amendments modified the requirement for regulation of additional contaminants, as discussed later in this section). EPA was also directed to develop regulations to require all drinking water systems to disinfect their water, and criteria under which systems that use surface water supplies would be required to provide filtration. Other requirements included limitations on the use of lead in the installation and repair of water distribution facilities, a revised MCL for lead, monitoring requirements for various "unregulated contaminants", and revised criteria for coliforms in treated water. These specific requirements are discussed in more detail below. Provisions for public notification of violations of water quality regulations were also expanded under the revised SDWA, as summarized in Table 1.

a. Regulation of Initial 83 Contaminants. The 83 contaminants identified for regulation in the 1986 SDWA Amendments are summarized in Table 2. At present, new MCLs or treatment techniques have been promulgated for 76 of the 83 contaminants or contaminant groups (14 volatile organic contaminants, 40 synthetic organic contaminants, 16 inorganic contaminants, 5 microbial contaminants, and turbidity). Promulgation of new MCLs for five radionuclides, sulfate, and arsenic has been delayed several times, and is not expected to occur within the next several years. Current Primary Drinking Water Standards, including the new MCLs and treatment techniques, are summarized in Table 3.

Table 1 SDWA Public Notification Requirements			
Violation	Description	Notification	Schedule
Tier 1	MCL, Trmt. Technique, Variance/Exemption Schedule Violations	Radio/Television*	Within 72 Hours
		Newspaper	Within 14 Days
		Direct Mailing Hand Delivery**	Within 45 Days, Quarterly Repeat
Tier 2	Monitoring, Testing Violations, Variance/ Exemption Issued	Newspaper	Within 3 Months
		Direct Mailing Hand Delivery**	Quarterly Repeat

* "Acute" health risk conditions only (as determined by State).
** State may waive if violation corrected within stated period.

b. The Surface Water Treatment Rule. EPA published its proposed "Surface Water Treatment Rule" (SWTR) on November 3, 1987. The primary purpose of the rule is to protect the public from waterborne diseases. The SWTR was finalized on June 29, 1989, and specifies mandatory performance requirements for filtration and disinfection of surface water supplies. Principal requirements of the rule are summarized below.

(1) Turbidity Removal. Under the SWTR, the MCL for filtered water turbidity has been reduced to 0.5 NTU, and 95 percent of all samples analyzed must meet the revised criteria. The reduced MCL is based on the desire to maximize removal of microbial contaminants such as *Giardia* cysts and enteric (intestinal) viruses. The maximum allowable turbidity sampling interval is 4 hours.

The SWTR includes provisions for state regulatory agencies to specify a turbidity MCL as high as 1.0 NTU. This determination could be based on analysis of design and operating conditions (treatment adequacy prior to filtration, overall turbidity removal through the plant, stringency of disinfection, etc.), and/or performance relative to specific water quality characteristics (filtered water microbiological characteristics, particle size ranges). Under this option, the state could consider such factors as source water quality and system size in determining appropriate analysis procedures.

Table 2
83 Contaminants Scheduled For Regulation
Under 1986 SDWA Amendments

Inorganics

Antimony	Beryllium	Cyanide	Nickel	Sulfate
Arsenic	Cadmium	Fluoride	Nitrate	Thallium
Asbestos	Chromium	Lead	Nitrite	
Barium	Copper	Mercury	Selenium	

Organics

Acrylamide	Endrin	Pichloram
Adipates	Epichlorohydrin	Simazine
Alachlor	Ethylbenzene	Styrene
Aldicarb	Ethylene dibromide	Toluene
Aldicarb sulfone	Glyphosate	Toxaphene
Aldicarb sulfoxide	Heptachlor	Vydate
Atrazine	Heptachlor epoxide	Xylene
Carbofuran		1,1,2-Trichloroethane
Chlordane	Lindane	1,2-Dichloropropane
Dalapon	Methoxychlor	2,3,7,8-TCDD (Dioxin)
	PAHs	2,4-D
Dinoseb	PCBs	2,4,5-TP
Diquat	Pentachlorophenol	
Endothall	Phthalates	

Volatile Organic Chemicals

Benzene	Trichloroethylene
Carbon tetrachloride	Vinyl chloride
Chlorobenzene	cis-1,2-Dichloroethylene
Dichlorobenzene	trans-1,2-Dichloroethylene
Methylene chloride	1,1-Dichloroethylene
Tetrachloroethylene	1,1,1-Trichloroethane
Trichlorobenzene	1,2-Dichloroethane

Radionuclides

Beta particle and photon radioactivity	Radium 226 and 228 Uranium
Gross alpha particle activity	Radon

Microbiological and Turbidity

Giardia Lamblia	Standard plate	Turbidity
Legionella	Total coliforms	Viruses

Table 3
Current Primary Drinking Water Standards

Inorganic Contaminants	MCL
Antimony	0.006 mg/L
Arsenic	0.05 mg/L
Asbestos	7 million fibers/L
Barium	2 mg/L
Beryllium	0.004 mg/L
Cadmium	0.005 mg/L
Chromium	0.1 mg/L
Copper	Treatment Technique
Cyanide	0.2 mg/L
Fluoride	4 mg/L
Lead	Treatment Technique
Mercury	0.002 mg/L
Nickel	0.1 mg/L
Nitrate	10 mg/L as N
Nitrite	1 mg/L as N
Nitrate + Nitrite	10 mg/L as N
Selenium	0.05 mg/L
Silver	0.05 mg/L
Thallium	0.002 mg/L
Organic Contaminants	**MCL**
Acrylamide	Treatment Technique
Alachlor	0.002 mg/L
Aldicarb	0.003 mg/L
Aldicarb Sulfoxide	0.004 mg/L
Aldicarb Sulfone	0.002 mg/L
Atrazine	0.003 mg/L
Benzene	0.005 mg/L

Table 3 (continued) Current Primary Drinking Water Standards	
Organic Contaminants	MCL
Benzo(a)pyrene	0.0002 mg/L
Carbofuran	0.04 mg/L
Carbon Tetrachloride	0.005 mg/L
cis-1,2-Dichloroethylene	0.07 mg/L
Chlordane	0.002 mg/L
Dalapon	0.2 mg/L
Di(2-ethylhexyl)adipate	0.4 mg/L
Di(2-ethylhexyl)phthalate	0.006 mg/L
Dibromochloropropane	0.0002 mg/L
Dichloromethane	0.005 mg/L
Dinoseb	0.007 mg/L
Diquat	0.02 mg/L
Endothall	0.1 mg/L
Endrin	0.002 mg/L
Epichlorohydrin	Treatment Technique
Ethylbenzene	0.7 mg/L
Ethylene Dibromide	0.00005 mg/L
Glyphosate	0.7 mg/L
Heptachlor	0.0004 mg/L
Heptachlor Epoxide	0.0002 mg/L
Hexachlorobenzene	0.001 mg/L
Hexachlorocyclopentadine	0.05 mg/L
Lindane	0.0002 mg/L
Methoxychlor	0.04 mg/L
Monochlorobenzene	0.1 mg/L
o-Dichlorobenzene	0.6 mg/L
Oxamyl(Vydate)	0.2 mg/L

Table 3 (continued)
Current Primary Drinking Water Standards

Organic Contaminants	MCL
PCBs	0.0005 mg/L
p-Dichlorobenzene	0.075 mg/L
Pentachlorophenol	0.001 mg/L
Picloram	0.5 mg/L
Simazine	0.004 mg/L
Styrene	0.1 mg/L
Tetrachloroethylene	0.005 mg/L
Toluene	1 mg/L
Total Trihalomethanes	0.10 mg/L
Toxaphene	0.003 mg/L
trans-1,2-Dichloroethylene	0.1 mg/L
Trichloroethylene	0.005 mg/L
Vinyl Chloride	0.002 mg/L
Xylene (Total)	10 mg/L
1,1-Dichloroethylene	0.007 mg/L
1,1,1-Trichloroethane	0.20 mg/L
1,1,2-Trichloroethane	0.005 mg/L
1,2-Dichloroethane	0.005 mg/L
1,2-Dichloropropane	0.005 mg/L
1,2,4-Trichlorobenzene	0.07 mg/L
2,3,7,8-TCDD (Dioxin)	3×10^{-8} mg/L
2,4-D	0.07 mg/L
2,4,5-TP (Silvex)	0.05 mg/L
Radionuclides	**MCL**
Beta/Photon Activity	4 mrem/yr
Gross Alpha	15 pCi/L
Radium-226, -228	5 pCi/L

Table 3 (continued) Current Primary Drinking Water Standards	
Microbiological/Turbidity	MCL
Giardia Lamblia	Treatment Technique
Heterotrophic Bacteria	Treatment Technique
Legionella	Treatment Technique
Total Coliforms	Absent in minimum of 95 percent of monthly samples
Turbidity	0.5 NTU or less in minimum of 95 percent of samples
Viruses	Treatment Technique

The SWTR "Guidance Manual for Compliance with the Filtration and Disinfection Requirements for Public Water Systems Using Surface Water Sources" (the "Guidance Manual") provides additional guidance to the states for determining when a higher turbidity limit might be appropriate.

It is emphasized that the SWTR addresses turbidity of the "filtered" water. Subsequent addition of chemicals for corrosion/pH control and/or fluoridation which may increase turbidity above 0.5 NTU is therefore permissible to the extent that the treated water turbidity does not exceed 5 NTU at any time.

(2) Disinfection. The 1986 SDWA Amendments directed EPA to establish new criteria for regulation of five microbial contaminants in drinking water derived from surface supplies: *Giardia lamblia* cysts *(Giardia),* enteric (intestinal) viruses, *Legionella,* heterotrophic bacteria (HPC), and coliforms. EPA recognized that it is neither economical nor technologically feasible to measure the levels of these contaminants on a regular basis. The Agency therefore promulgated treatment techniques which will result in removal and/or inactivation of these microbial contaminants, with primary focus on controlling *Giardia* cysts and enteric viruses. When these two contaminants are effectively inactivated, the remaining three are also reduced to acceptable levels. The treatment techniques for control of these microbial contaminants are specified in the SWTR, such that a minimum of 99.9 percent and 99.99 percent removal and/or inactivation is achieved for *Giardia* cysts and enteric viruses, respectively. For utilities which filter, disinfection is required to maintain a minimum disinfectant residual of 0.2 mg/L for water entering the distribution system at all times. The SWTR also requires that a "detectable" disinfectant residual be maintained within the distribution system for a minimum of 95 percent of all samples analyzed (on a monthly basis). Where no residual is detected, and a heterotrophic plate count (HPC) analysis indicates less than 500 colonies per mL, the sample will be considered acceptable. Sampling frequencies and locations must be the same as required by the Coliform Rule.

EPA recommends disinfection treatment criteria in the SWTR "Guidance Manual" which establishes disinfection residuals and contact times to be maintained to inactivate *Giardia* cysts and enteric viruses. Disinfection efficiency is to be evaluated through the use of CT values. CT values are the product of the disinfectant concentration, C, and the contact time, T, at the point of residual measurement. The CT values have been developed within controlled laboratory environments for a wide range of temperature, pH, and disinfectant residual conditions. CT values for disinfection with free chlorine are dependent upon water temperature, pH, and the chlorine residual. For disinfection with ozone, chlorine dioxide, and/or monochloramine, CT values are dependent only upon water temperature when pH is between 6 and 9. CT values increase as water temperatures drop (and, for free chlorine, as pH values increase). Disinfectant contact times used in calculating the achieved degree of disinfection are to be determined by field studies using tracer compounds. CT values for inactivation of *Giardia* cysts and enteric viruses by monochloramine are high enough to limit future use of this compound strictly to secondary disinfection and/or maintenance of disinfectant residuals within distribution systems.

The use of CT values for determining disinfection efficiency is required for systems that do not filter. Use of CT values for systems which filter is not specifically required by the SWTR. EPA indicates that individual state regulatory agency discretion will be allowed in determining appropriate disinfection criteria. Systems must still meet the minimum required 99.9 percent *Giardia* cyst/99.99 percent enteric virus removal/inactivation criteria, but may not be required to monitor CT values if other state-specified disinfection criteria are met. Systems which do not monitor CT values will, in all probability, be required to demonstrate through pilot studies that the minimum disinfection criteria proposed under the 1986 SDWA Amendments are being met.

EPA has recognized that *Giardia* cysts are readily removed by efficiently-operated conventional treatment facilities using granular media filtration, and therefore credit for 99.7 percent (2.5-log) cyst removal by filtration is to be allowed. Likewise, credit for 99 percent (2-log) removal of viruses by filtration is allowed. Provisions for a minimum additional 68 percent (0.5-log) inactivation of cysts and 99 percent (2-log) inactivation of viruses must therefore be made by disinfection to achieve the minimum required 99.9 percent (3-log) cyst and 99.99 percent (4-log) virus removal and/or inactivation. Virus inactivation well in excess of 99.99 percent is typically achieved when conditions for 99.9 percent removal and/or inactivation of *Giardia* cysts are maintained.

In the SWTR "Guidance Manual," EPA recommends specific minimum *Giardia* cyst removal/inactivation levels in the 3-log to 5-log range, depending upon the expected degree of cyst contamination in the source water.

c. Coliform Control. On June 29, 1989, EPA promulgated revisions to the current regulation governing total coliform levels in water distribution systems. The revised rule expands current coliform monitoring requirements and specifies new MCLs. Principal requirements of the revised rule are as follows:

- Compliance with the revised MCLs is to be based on presence/absence of total coliforms, rather than specific coliform density levels.
- Up to 5 percent of the monthly samples analyzed may be coliform-positive.
- Limits for heterotrophic bacteria (HPC) have been established, based on potential HPC interference during coliform analysis.
- Fecal or *Escherichia* coliform levels are to be monitored for each sample where the presence of total coliforms is indicated.
- Public notification by electronic media (TV or radio) is required within 72 hours if a positive result indicates the presence of either fecal or *Escherichia* coliforms.

EPA subsequently modified the Total Coliform Rule to allow states to use a variance procedure for utilities encountering nonfecal biofilm problems in their distribution systems. Some coliform species, which are not classified as fecal, produce positive analytical results in total coliform and fecal coliform tests. Under the revised rule, states are allowed to disregard any coliform-positive analytical results that are speciated and found not to be of fecal origin.

d. Lead and Copper Control. In August 1988, EPA proposed a new set of standards for the control of lead and copper in drinking water. In the proposed rule, an MCL and a treatment technique were specified. Revised MCLs of 0.005 mg/L for lead and 1.3 mg/L for copper were proposed for water entering the distribution system. A new treatment technique to optimize corrosion control was also proposed based on quarterly monitoring of samples drawn from the first water that flows from the cold water kitchen tap in the morning.

The primary source of lead at the consumer's tap is from lead-solder joints and brass fixtures in household plumbing. The water utility therefore cannot rely on controlling lead strictly by removal at the treatment plant alone, but must also control the corrosivity of the treated water to reduce the potential for lead dissolution from household plumbing and fixtures.

The final Lead and Copper Rule, promulgated during May 1991, differs significantly from the regulation proposed in 1988. Under the final Rule, all systems serving more than 50,000 consumers were required to perform diagnostic monitoring and to conduct corrosion control studies (regardless of the results of the diagnostic monitoring). The Rule establishes "Action Levels" for both lead and copper. Based on first-draw samples collected at taps within the distribution system, lead and copper concentrations must be less than 0.015 mg/L and 1.3 mg/L in ninety percent of the samples, respectively. Each utility was required to complete a materials survey for its distribution system in order to identify a pool of targeted sampling sites. The selected sites were to consist of single-family residences which contain copper pipes with lead solder installed after 1982, which contain lead pipes, or which are served by a lead service line. Initial monitoring of tap samples was to be conducted over two six-month periods. The results of the diagnostic monitoring were then used to determine the need for a public education program. Monitoring data and corrosion control study results were to be submitted to the state regulatory agency, which then designates the "optimal" treatment required. Optimal treatment, as defined within the Rule, may consist of (1) alkalinity and/or pH adjustment, (2) calcium hardness adjustment, (3) use of a phosphate or silicate based corrosion inhibitor, or (4) a combination of two or more of these three approaches. Following implementation of the state-specified treatment, follow-up monitoring is required. If the results of the follow-up monitoring indicate that the system is in compliance with the lead and copper action levels, the state may eventually reduce the annual monitoring requirements. Should follow-up monitoring indicate noncompliance, the utility is required to initiate a public education program, collect additional water quality samples, and possibly begin a program of replacing lead service lines. Key dates for compliance with the Rule for systems serving more than 50,000 consumers are as follows:

Complete Material Survey and Submit Sampling Plan to State	January 1, 1992
Results of Diagnostic Monitoring Submitted to State	
First 6-Month Period	July 11, 1992
Second 6-Month Period	January 11, 1993
Results from Corrosion Study Submitted to State	July 1, 1994
State Approves or Designates "Optimal" Treatment	January 1, 1995
Installation and Operation of Treatment Completed	January 1, 1997
Complete Follow-Up Monitoring for Treatment Performance	January 11, 1998
State Review of Data and Designation of Operating Conditions for Compliance Determinations	July 1, 1998
Complete Follow-Up Monitoring for Compliance with State-Specified Water Quality Parameters	July 11, 1999

e. Phase II, Phase V SOC, IOC Regulations. The Phase II regulation for Synthetic Organic Chemicals (SOCs) and Inorganic Chemicals (IOCs) established MCLs and Maximum Contaminant Level Goals (MCLGs) for 30 SOCs and 9 IOCs. Promulgation of MCLs for three of the Phase II SOCs (aldicarb, aldicarb sulfone, and aldicarb sulfoxide) has since been delayed. The Phase V regulation established MCLs and MCLGs for an additional 23 contaminants. (Contaminants regulated under the Phase II and Phase V regulations are primarily volatile organic compounds and pesticides/herbicides.)

f. Radionuclides. A proposed rule for radionuclides was release in 1991, but never finalized. The proposed rule includes new standards for radon and uranium, and revised standards for radium-226, radium-228, and gross beta/gross alpha activity. The proposed radionuclide MCLs are as follows:

Radon	300 pCi/L
Radium-226, -228	20 pCi/L
Uranium	20 pCi/L
Beta Emitters	4 mrem ede/yr
Adjusted Gross Alpha	15 pCi/L

Considerable controversy has surrounded the level at which the MCL for radon should be set. AWWA has recommended an MCL of 1,000 pCi/L, while the EPA Science Advisory Board has suggested a range of 1,000 to 3,000 pCi/L. Congress therefore delayed promulgation of a final radon standard, but indicated that EPA could proceed with promulgation of standards for the remaining radionuclides. Resource constraints within EPA have prevented promulgation of standards for these contaminants, however. Because the proposed limits for radium-226 and -228 are actually higher than the current MCL, EPA and some states are not enforcing the current regulation pending finalization of revised MCLs. EPA recently withdrew the 1991 proposed radon standard of 300 pCi/L, and the new MCL is expected to be much higher, i.e., probably on the order of 1,000 pCi/L.

5. Information Collection Rule

The Information Collection Rule (ICR) was proposed during February 1994, and finalized during May 1996 after a 23-month delay caused by technical and administrative problems. The purpose of the ICR is to collect data to be used in the development of future disinfection by-product and microbial contaminant control regulations. While this regulation will not directly affect the current treatment practices of water utilities, it will provide EPA with occurrence and water treatment data which will be used in formulating future drinking water regulations.

The ICR consists of three major components: 1) microbial contaminant monitoring requirements, 2) monitoring of disinfection by-products and related parameters, and 3) bench scale and/or pilot scale testing requirements for utilities serving more than 500,000 consumers. Surface water systems serving more than 100,000 consumers must conduct monthly source-water monitoring for microbial contaminants (*Giardia*, *Cryptosporidium*, total/fecal coliforms, and viruses); this testing must continue for 18 consecutive months, beginning in July 1997. The rule also includes requirements for monitoring of microbial contaminants in the treated water, if these contaminants are identified in the source water at concentrations exceeding 1 per liter for viruses and 10 per liter for the remaining pathogens. Utilities serving more than 100,000 consumers must also conduct quarterly monitoring for a wide range of DBPs at the plant discharge and within the distribution system over 18 consecutive months beginning in July 1997. The DBP monitoring data are intended to provide information regarding relationships between DBP formation and source water characteristics, the resulting concentrations of DBPs, and the most cost-effective methods for future DBP monitoring efforts. Utilities serving more than 500,000 consumers, and whose running annual average source water total organic carbon (TOC) concentration exceeds 4.0 mg/L (as determined by 12 months of testing beginning in August 1996) must also conduct bench-scale and/or pilot-scale studies of disinfection by-product precursor removal beginning no later than April 1998.

6. The 1996 Safe Drinking Water Act Amendments

The 1996 Safe Drinking Water Act Amendments were signed into law on August 6, 1996. The 1996 Amendments represent a significant change in the manner in which regulations are to be developed and implemented. A brief summary of the major provisions of the 1996 Amendments is presented below.

a. Standard-Setting Process Changes. The requirement that EPA regulate an additional 25 contaminants every 3 years (a provision of the 1986 Amendments) has been eliminated. EPA must instead conduct a review of at least 5 contaminants every 5 years, and must then decide whether to regulate a contaminant or not based on the following three criteria: 1) the contaminant adversely affects human health, 2) the contaminant is known or substantially likely to occur in public water systems with a frequency and at levels of public health concern, and 3) regulation of the contaminant presents a meaningful opportunity for health risk reduction. Contaminant occurrence, relative risk, and cost-benefit considerations will therefore be the primary factors in determining which contaminants should be regulated. EPA must conduct a thorough cost-benefit analysis for all future drinking water standards, and provide comprehensive, informative, and readily-understandable information to the public. EPA must determine whether the costs of a new standard would be justified by the benefits. If not, EPA may then adjust an MCL to a level that "maximizes health risk reduction benefits at a cost that is justified by the benefits".

On February 6, 1998, EPA finalized the first Drinking Water Contaminant Candidate List (DWCCL), which will be used to set regulatory, research, and occurrence-investigation priorities. In the list, EPA identifies 19 chemicals and 1 microbial which the Agency considers as "high priority" with respect to determination of the need to regulate. As discussed above, EPA must select at least five contaminants from this list and decide, by August 2001, whether to regulate them. The first DWCCL is presented in Table 4.

b. Compliance Time Frames. Under the 1986 Amendments, utilities typically were allowed 18 months to comply with new regulations following promulgation. The 1996 Amendments extend the compliance period following promulgation to three years; EPA or individual states may grant an additional 2 years if necessary to implement significant capital improvements.

Table 4
Drinking Water Contaminant Candidate List

Chemicals: 1,1,2,2-tetrachloroethane* 1,2,4-trimethylbenzene* 1,1-dichloroethane* 1,1-dichloropropene 1,2-diphenylhydrazine 1,3-dichloropropane 1,3-dichloropropene* 2,4,6-trichlorophenol 2,2-dichloropropane* 2,4-dichlorophenol 2,4-dinitrophenol 2,4-dinitrotoluene 2,6-dinitrotoluene 2-methylphenol Acetochlor Alachlor ESA (and other degradation products of acetanilide pesticides) Aldrin* Aluminum Boron* Bromobenzene* DCPA mono-acid degradate DCPA di-acid degradate DDE Diazinon Dieldrin* Disulfoton Diuron EPTC Fonofos Hexachlorobutadiene* p-isopropyltoluene* Linuron Manganese*	Methyl bromide Metolachlor* Metribuzin* Molinate MTBE Naphthalene* Nitrobenzene Organotins* Perchlorate Prometon RDX Sodium Sulfate* Terbacil Terbufos Triazines* (and degradation products, including but not limited to cyanazine and atrazine-desethly) Vanadium* Microbials: Acanthamoeba* (guidance for contact lens wearers) Adenoviruses *Aeromonas hydrophila* Cyanobacteria (blue-green algae), other freshwater algae and their toxins Caliciviruses Coxsackieviruses Echoviruses *Helicobacter pylori* *Microsporidia* (*enterocytozoon* and *septata*) *Mycobacterium avium intracellulare* (MAC)

*EPA identifies as "high priority" for regulatory consideration.

c. Regulation Promulgation Schedule Changes. The 1996 Amendments require EPA to promulgate new requirements for control of disinfection by-products, and an "enhanced" version of the current Surface Water Treatment Rule. These new rules will also be subject to the extended compliance time interval discussed above (i.e., compliance will normally be required three years after promulgation, and five years after promulgation with EPA/state approval based on the need for significant capital expenditures). These rules are discussed in detail in the "Pending Regulations" section below, and revised schedules for promulgation of these regulations are presented in Table 6.

d. Arsenic. EPA must conduct additional research on the health effects of arsenic, particularly at low levels of exposure. EPA must propose a new regulation for arsenic not later than January 1, 2000, and issue a final regulations twelve months later.

e. Radon. EPA must arrange for a radon risk assessment to be conducted by the National Academy of Sciences (NAS), issue a cost-benefit analysis for radon within 30 months of promulgation (i.e., prior to February 1999), and issue a proposed regulation within 36 months (i.e., prior to August 1999).

f. Sulfate. EPA must conduct, jointly with the Centers for Disease Control and Prevention, a dose-response study for sulfate within 30 months (i.e, prior to February 1999). Sulfate will thereafter be considered in the first round of the new contaminant selection process discussed above.

g. Monitoring of Unregulated Contaminants. EPA must issue regulations establishing criteria for monitoring of unregulated contaminants. Within three years of enactment (i.e., by August 1999), and every 5 years thereafter, EPA must issue a list of no more than 30 such contaminants for which monitoring is required.

h. State Revolving Loan Fund. A new Drinking Water State Revolving Loan Fund (SRLF) will be established to provide loans to public water systems to "facilitate compliance" or "significantly further" the rule's health protection objectives. This fund authorizes $1 billion per year from 1995 through 2003 for capitalization grants to primacy states. (States must match the grants at a 20 percent level.) Setasides are provided for administration, capacity development, disadvantaged communities, source water protection, and USEPA health effects research.

i. Operator Certification. EPA has 30 months to provide guidance to states specifying minimum standards for certification of water system operators. States which currently administer operator certification programs can continue to use them if EPA determines that the existing programs are "substantially equivalent" to its program guidelines.

j. Groundwater Disinfection. EPA must adopt a rule requiring disinfection by certain groundwater systems and provide guidance on determining which systems must disinfect; this must be accomplished no earlier than August 1999 and no later than the date that EPA adopts the Stage 2 Disinfectants/Disinfection By-Products Rule.

B. Pending Regulations

Although not in final form at this time, several rules are scheduled to be implemented within the next several years. Because the regulations discussed in this section are not yet final (and in many cases, not yet formally proposed), the information contained in this section should be regarded as preliminary in nature, and subject to change.

1. Microbial Control / Disinfection By-Products Regulations

The Microbial /Disinfection By-Product (M/DBP) cluster of rules is a term applied collectively applied to the impending regulations discussed in this section. Because of the recent signing of an Agreement in Principal (June 1997) by the stakeholders charged with developing these rules in concert with EPA, the overall direction of these rules is fairly firm. The rules are interrelated and are expected to by published concurrently, so a provision discussed below under one rule may be actually be promulgated under a companion rule.

a. Disinfectants / Disinfection By-Products Rule (Stage 1). The Stage 1 Disinfectants/Disinfection By-Products Rule (D/DBPR) was proposed in July of 1994, and has been the topic of much discussion since then. The proposed rule contained provisions that would:

- Set MCLs and MCL Goals (MCLGs) for several DBPs, including total trihalomethanes, total haloacetic acids (referred to as HAA5, as five of the nine known haloacetic acid compounds would be regulated), bromate (a by-product of disinfection using ozone), and chlorite (a by-product of disinfection using chlorine dioxide).

- Set Maximum Residual Disinfectant Levels (MRDLs) of 4 mg/L for free chlorine and monochloramine, and 0.8 mg/L for chlorine dioxide
- Establish a treatment technique requiring surface water systems using conventional treatment to operate in either an enhanced coagulation or enhanced softening mode to achieve specified total organic carbon (TOC) percent removals. TOC removal was to be accomplished prior to the continuous application of a disinfectant (this provision has since been remanded, as discussed below).

The Stage 1 proposed rule includes three "triggers" for requiring enhanced coagulation:

- A TOC concentration greater than 2 mg/L at the point of initial addition of disinfectant.
- Formation of TTHMs exceeding 0.040 mg/L (40 ug/L) on an annual "running average" basis using free chlorine.
- Formation of HAA5s exceeding 0.030 mg/L (30 ug/L) on an annual "running average" basis using free chlorine.

For enhanced coagulation, the required level of TOC reduction which must be achieved is a function of the initial source water TOC concentration and alkalinity. Enhanced softening is by definition achieved when lime softening processes are operated to remove more than 10 mg/L of magnesium hardness (as $CaCO_3$). (The proposed rule also includes provisions for obtaining a waiver from the enhanced coagulation/enhanced softening requirements, should the water supply be determined to not be amenable to enhanced coagulation treatment.)

The recently-signed Agreement in Principle provides some additional insight into the probable content of the final Stage 1 D/DBPR, and includes the following provisions:

- MCLs for three DBPs will remain at the originally-proposed levels, i.e., 0.080 mg/L for TTHMs, 0.060 mg/L for HAA5, and 0.010 mg/L for bromate.
- The Agreement is silent regarding the originally-proposed MCL for chlorite of 1.0 mg/L. The stakeholders committee did not reach a consensus regarding an MCL for this contaminant, so EPA must make the final determination. It is expected that if the MCL for chlorite is revised, it will not be modified until the Stage 2 D/DBPR is promulgated.
- The proposed "3x3 matrix" which specifies the levels of TOC removal required was revised, but will apply to systems that practice enhanced softening as well as those which practice enhanced coagulation. (The revised matrix is shown in

Table 5.)

Table 5 TOC Removal Requirements for Enhanced Coagulation/Enhanced Softening			
Source Water	Percent TOC Removal Required at Indicated Source Water Alkalinity		
TOC, mg/L	0 - 60 mg/L	>60 - 120 mg/L	>120 mg/L
>2 - 4	35	25	15
>4 - 8	45	35	25
>8	50	40	30

- Specific UV absorbance (SUVA, defined as the ratio of the water's ultraviolet absorbance at 254 nm (UV_{254}) to its dissolved organic carbon (DOC) concentration) will be added as a criteria for determining if systems will be required to practice enhanced coagulation or enhanced softening. For softening plants such as the City's, enhanced softening would not be required if the raw water has an SUVA <2.0 liter/(mg)(m).

As directed by the 1996 SDWA Amendments, the final rule must be promulgated by November 1998, and effective during November 2001.

b. Disinfectants/Disinfection By-Products Rule (Stage 2). At this time, it appears that the primary thrust of the Stage 2 D/DBPR will be to lower the limits for certain DBPs beyond those promulgated in the Stage 1 Rule. The MCLs for Stage 2 discussed in the proposed D/DBPR (i.e., 0.040 mg/L for TTHMs, 0.030 mg/L for HAA5) are not firm, and are subject to negotiation with the stakeholders once again. EPA currently plans to initiate a regulatory negotiation for the Stage 2 DBP rule in mid-1999. It is expected that the Stage 2 rule will be proposed during November 2000, and finalized during May 2002.

c. Interim Enhanced Surface Water Treatment Rule. The primary aspects of the pending Interim Enhanced Surface Water Treatment Rule (IESWTR) are as follows:

- Allowable finished water turbidity will be reduced from the present 0.5 NTU to 0.3 NTU. This standard applies to the combined filtered water, and a minimum of 95 percent of the monthly turbidity measurements must meet the revised turbidity criteria. The turbidity of the combined filter effluent cannot exceed 1 NTU at any time (the current Surface Water Treatment Rule allows for a maximum combined filter effluent turbidity of 5 NTU).

- Disinfection credit will continue to be allowed for a disinfectant applied at any point in the treatment process. (The proposed D/DBPR included provisions that would not allow disinfection CT credit to be claimed until after enhanced coagulation/enhanced softening treatment.)
- Surface water systems that filter and serve more than 10,000 people must achieve at least a 2-log (99%) removal of *Cryptosporidium*. (Systems utilizing granular media filtration and meeting the revised turbidity removal criteria discussed above are assumed to achieve at least a 2-log removal of Cryptosporidium.)
- Water systems with DBP levels exceeding or approaching the new MCLs for total trihalomethanes and total haloacetic acids (expected to be 0.080 mg/L and 0.060 mg/L, respectively, as discussed above for the Stage 1 D/DBPR) may consider changing their disinfection practices in order to comply with the new limits. In an effort to avoid increasing the risk from microbial pathogens while attempting to lower DBPs, EPA will require systems which have DBPs within 80% of the new MCLs (i.e., >0.064 mg/L for TTHMs or >0.048 mg/L for HAA5) to prepare a "disinfection profile" for state review prior to altering disinfection practices. Three years of daily operating data will be used to develop the disinfection profile. If the State does not approve changes in disinfection, systems must develop alternate ways of reducing DBPs to meet the new MCLs.
- For those water systems that do not have four quarters of distribution system HAA5 monitoring data available within 90 days of the promulgation of the IESWTR, HAA5 monitoring must be conducted for four quarters.
- If a PWS uses surface water and serves more than 10,000 people, continuous turbidity monitoring is required for each filter. Specific performance criteria will apply to each filter.

The IESWTR will only apply to systems serving 10,000 or more consumers. Under the provisions of the 1996 SDWA Amendments, the IESWTR must be promulgated by November 1998 and be effective three years later, i.e., by November 2001 at latest.

Under the IESWTR, EPA proposes to amend the existing SWTR to require that all systems using surface water supplies conduct a periodic sanitary survey, regardless of whether they filter or not. Each utility would be responsible for ensuring that the sanitary survey is completed. Only the State or an agent approved by the State would be allowed to conduct the sanitary survey. Sanitary surveys would be conducted every three to five years (the Agency has requested comment on this frequency interval). The initial survey would need to be completed within 5 years of promulgation of the ESWTR. EPA defines the sanitary survey process as "an on-site review of the water source, facilities, equipment, operation and maintenance of a public water system for the purpose of evaluating the adequacy of such sources, facilities, equipment, operation and maintenance for producing and distributing safe drinking water." EPA has stated that its intent in requiring sanitary surveys is to focus more attention on watersheds and watershed protection activities to enhance and maintain the quality of both surface waters and ground waters as sources of drinking water.

d. Long-Term Enhanced Surface Water Treatment Rule.

A long-term Enhanced Surface Water Treatment Rule which will extend the IESWTR to systems serving less than 10,000 consumers is under development, and is expected to be promulgated during November 2000. This regulation (currently being referred to by EPA as the LT1ESWTR) is also expected to address recycling of filter backwash water within the treatment process and possibly other issues affecting all system sizes. EPA is planning to initiate a regulatory negotiation process for a long-term Stage 2 ESWTR (currently referred to as the LT2ESWTR) in mid-1999, and a proposed LT2ESWTR is scheduled for November 2000, with promulgation expected in May 2002.

While the overall direction of the IESWTR is fairly firm, specific provisions which may be promulgated under the long-term ESWTR cannot be predicted with any certainty at this time. However, because turbidity and disinfection practices are already being revised under the IESWTR and Stage 1 of the D/DBPR, the LT2ESWTR is expected to still only address turbidity and disinfection as treatment techniques to protect the public health from infectious microorganisms. Using information gathered under the Information Collection Rule, the LT2ESWTR will likely be the vehicle EPA uses to set a treatment technique aimed at protecting the public from infection due to *Cryptosporidium* in drinking water. An MCLG of zero can be expected for this microbe.

Most of the turbidity changes that EPA intends to make will be included in the IESWTR, and therefore no major revisions are expected when the LT2ESWTR is promulgated. However, there is the possibility that the ICR data will support lowering the turbidity standard slightly.

Promulgation of a treatment technique for *Cryptosporidium*, however, could have more serious consequences with respect to future treatment requirements. Although a firm removal/inactivation requirement for *Cryptosporidium* cannot be predicted with any certainty at this point, it appears likely that at least a 3-log (99.9%) removal/inactivation will be required. *Cryptosporidium* oocysts have been shown to be a major public health hazard if live oocysts penetrate through the water treatment process. Because a 3-log removal/inactivation standard for *Giardia* currently exists, it is considered unlikely that EPA would propose anything less as a treatment level for *Cryptosporidium*. As discussed above, a 2-log removal credit will be granted for well-operated plants that filter, and it is presumed that EPA will allow individual plants to present operating data as part of a petition to their state regulatory agency for removal credit beyond 2-logs. However, some level of inactivation of Cryptosporidium by disinfection will likely be necessary in most cases to achieve the total removal/inactivation requirement.

Cryptosporidium organisms have been shown to be much more difficult to inactivate than *Giardia* cysts, and are very resistant to disinfection using chlorine. Research is still underway to determine the effectiveness of various disinfectants against viable *Cryptosporidium* oocysts. EPA will evaluate all available information when promulgating the LT2ESWTR before deciding on required disinfection CT criteria. Recent research suggests that sequential disinfection of *Cryptosporidium* using different disinfectants (such as free chlorine followed by monochloramine) is more effective than that indicated by the effectiveness of each disinfectant from independent studies (i.e., chlorine followed by monochloramine disinfection produces "synergistic" *Cryptosporidium* inactivation effects). While this synergistic effect has only been observed in bench-scale laboratory studies under controlled conditions, these findings suggest that new strategies for inactivation of chlorine-resistant microbial contaminants such as *Cryptosporidium* may be developed in the near future. However, significant additional research and full-scale evaluation will be required to assess the effectiveness of this approach as compared to use of alternative disinfectants such as chlorine dioxide and/or ozone.

2. Consumer Confidence Reports Rule

As directed by the 1996 SDWA Amendments, all Public Water Systems serving more than 500 consumers will need to prepare annual reports to advise their users of the quality of the distributed water. The reports must contain a specific list of material such as information on the source water, an explanation of terms such as MCLs and MCLGs, data on specific contaminants, and information regarding potential health effects of the contaminants. Guidance on the Consumer Confidence Reports is under development, and AWWA is preparing a "mock report" to assist systems in complying with this regulation. A draft rule is expected during early 1998, with the final regulation currently scheduled for promulgation in August 1998.

3. Source Water Protection

The 1996 SDWA Amendments require states to adopt a source water protection program, and will assist in providing funding for this endeavor through the recently-established Drinking Water State Revolving Fund (DWSRF). Guidance for this program was recently released from EPA to the states. The Rule will require each state to have an EPA-approved program which will include the development of comprehensive Source Water Assessment Programs (SWAPs) that will delineate source water areas of public water systems and assess the susceptibility of these sources to contamination.

4. Filter Backwash Water Rule

Recycling of filter backwash and/or sludge dewatering process decant streams to the head of the treatment process is a relatively common practice. However, residual recycle practices have recently come under increased scrutiny due to concerns regarding the potential for return of *Giardia* cysts and/or *Cryptosporidium* oocysts to the head of the treatment process. Recycling of filter backwash and/or clarification sludge flows containing these microbial contaminants would increase their concentration within the raw water, thereby providing increased opportunities for the cysts to pass through the treatment process and into the finished water.

The 1996 SDWA Amendments require EPA to promulgate a regulation governing the recycling of filter backwash water within the treatment process of public water systems by August 2000. EPA recently indicated that it intends to address backwash recycling in the LT1ESWTR. While specific provisions of this rule cannot be predicted with any certainty at this time, EPA's initial thinking on this issue was expressed in a February 1994 internal memorandum from the Director of the Office of Ground Water and Drinking Water:

> "In the interest of public health, systems should either run backwash waters to waste or treat these waters before reuse. Treatment may consist of coagulation and settling, disinfection, or both. As an additional measure, a system may also decide to monitor the source water for *Cryptosporidium* and avoid recycling the backwash water when the density of *Cryptosporidium* oocysts in the source water exceeds a particular value (e.g., the Severn-Trent Water Authority in England uses a value of five oocysts/L)....."

In some cases, this regulation may require treatment or separate disposal of recycled filter backwash flows. The ICR will provide the first detailed data regarding backwash water recycling and the impacts of the recycled water on the stability and efficiency of the treatment process.

5. Groundwater Disinfection Rule

A rule to regulate the disinfection of ground water supplies is being developed, and is currently scheduled to be proposed in January 1999 and finalized in January 2001. EPA established disinfection requirements for groundwaters under the direct influence of surface water in its 1989 Surface Water Treatment Rule. However, in order to fulfill the amended SDWA mandate that disinfection requirements be imposed on all public water systems, EPA must also promulgate regulations governing disinfection of groundwater not under the direct influence of surface water. A draft Groundwater Disinfection Rule (GWDR) was made available for public comment during July 1992. The draft rule presented possible regulatory requirements and the rationale behind the rule, in addition to requesting comment on issues related to development of the rule. EPA's intention was to formally propose the GWDR during August 1995; however, proposal of this regulation was delayed due to resource limitations within the Agency and the current emphasis within EPA on the development of the Disinfectants/Disinfection By-Products Rule. Provisions included in the draft GWDR are summarized below; it is emphasized, however, that the final GWDR may differ significantly from the draft rule.

The GWDR will apply to all community water systems. Potential provisions of the rule include requirements for disinfection of source water, distribution system disinfection, use of qualified plant operators, treatment techniques for control of microbial contaminants, maximum contaminant level goals (MCLGs), and provisions for variances and exemptions. A treatment technique will probably be specified for viruses, heterotrophic bacteria, and *Legionella*, rather than specific maximum contaminant levels (disinfection will likely be proposed as the treatment technique). EPA has selected viruses as the target organism for this rule, as pathogens such as *Giardia* and *Cryptosporidium* are not normally found in groundwaters not under the direct influence of surface water. The minimum level of virus inactivation required has not yet been decided. However, it is expected that the level of inactivation to be required will not exceed the value specified in the Surface Water Treatment Rule (99.99 percent, or 4-log), and may in fact be lower (2-log or 3-log inactivation), based on removal of viruses by "natural disinfection" processes during passage of the water through subsurface strata.

EPA intends to provide guidance to state regulatory agencies for specifying design and operating conditions for systems using groundwater supplies. The Agency plans to include the application of the CT concept (as developed for the SWTR) in this guidance, but is also considering other methods that would also indicate adequacy of the disinfection provided. Unlike systems treating surface water supplies, the use of ultraviolet light (UV) for disinfection will probably be allowed for systems treating groundwater not under the influence of surface water. A discussion of UV disinfection requirements (light intensities, need for equipment redundancy, and factors that impact the overall process efficiency) is presented in the draft GWDR.

The draft rule also discusses the concept of "natural disinfection". A wellfield or well that is not vulnerable to virus contamination would be considered to meet the criteria for "natural disinfection", and may therefore be eligible to receive an exemption from (or a reduction in) the minimum disinfection requirements.

C. Future Regulations

1. General

In addition to the pending regulations discussed in the previous section, there are several additional regulations that will eventually be promulgated under the current SDWA agenda. These rules will come under the procedures established by the 1996 Amendments to the SDWA, meaning that EPA will no longer establish an MCL for a contaminant based solely on projected health related issues. The Amendments require the use of sound science, and allow for consideration of other factors such as cost, benefits, and competing risks.

2. Arsenic

Under the 1996 SDWA Amendments, EPA must develop "a comprehensive plan for study in support of drinking water rule making to reduce the uncertainty in assessing health risks associated with exposure to low levels of arsenic" and publish a proposed revised MCL for arsenic by January 2000 and a final MCL by January 2001. EPA is reported to be "considering" an MCL in the range of 0.002 to 0.020 mg/L (2 to 20 ug/L), although this may be modified based on results of ongoing studies regarding the health risks associated with exposure to low levels of arsenic. While an MCL in the 0.010 to 0.020 mg/L range would appear to strike a reasonable balance between risks to public health and increased treatment costs, compliance with an MCL significantly less than 0.010 mg/L would likely be problematic for many utilities.

3. Sulfate

Alternative MCLs of 400 mg/L and 500 mg/L were proposed for sulfate under the Phase V Rule in July 1990. Final promulgation of these MCLs was deferred, and a revised MCL of 500 mg/L was proposed in December 1994, with an allowance for an alternative compliance option to centralized treatment. Under the 1996 SDWA Amendments, EPA is to conduct a joint study with the Centers for Disease Control and Prevention (CDC) to assess the adverse health effects of exposure to high levels of sulfate in drinking water; this study is currently underway and must be completed by February 1999. EPA is also required to include sulfate in the first five or more contaminants for which a determination to regulate is to be made not later than August 2001.

4. Radionuclides

Under the 1996 SDWA Amendments, EPA agreed to either finalize the MCLs proposed in 1991 for radium, alpha emitters and beta and photon emitters by November 2000 or provide justification as to why revision is not necessary. EPA also agreed to promulgate an MCL for uranium by November 2000. The 1991 proposed rule included raising the MCLs for radium-226 and radium-228 to 20 pCi/L (the current MCL is 5 pCi/L for combined radium-226 and -228). However, EPA recently indicated that it is unlikely that the radium MCL will be increased above current levels, as this would result in a greater risk than that actually being achieved by the current 5 pCi/L MCL.

In accordance with the 1996 SDWA Amendments, EPA must promulgate a regulation for radon. Prior to proposing an MCL, EPA is to arrange for the National Academy of Science to conduct a risk assessment for radon in drinking water; this assessment is currently underway and is to be completed in July 1998. A proposed rule is to published by August 1999, and a final rule promulgated by August 2000.

5. Other Rules

There are additional rules likely to be proposed by EPA, but these will primarily address administrative issues such as the reformatting of drinking water amendments, streamlining of public notification requirements, and analytical methods updates. EPA presently plans to defer action on regulation of contaminants such as nickel, atrazine, aldicarb, aldicarb sulfone, and aldicarb sulfoxide.

D. Implementation Schedule

EPA's current regulatory promulgation schedule is summarized in Table 6.

Table 6
Schedule for Promulgation of SDWA Regulations (as of 02/98)

Regulation	Proposed	Final	Effective
Fluoride	11/85	04/86	10/87
8 VOCs (Phase I)	11/85	07/87	01/89
Surface Water Treatment Rule	11/87	06/89	06/93
Coliform Rule	11/87	06/89	12/90
Lead & Copper	08/88	06/91	01/92[1]
26 Synthetic Organic Contaminants, 7 Inorganic Contaminants (Phase II)	05/89	01/91[2]	07/92
MCLs for barium, pentachlorophenol (Phase II)	01/91	07/91	01/93
Phase V Organics, Inorganics	07/90	07/92	01/94
Radionuclides (Phase III) Radon	07/91 08/99	11/2000 08/2000[3]	11/2003 08/2003[4]
Sulfate	To be included on first Drinking Water Contaminant Candidate List		
MCLs for aldicarb, aldicarb sulfoxide, aldicarb sulfone	08/2003	02/2005	02/2008[4]
Disinfectants / Disinfection By-Products Stage 1 Stage 2	 07/94 07/94	 11/98[3] 05/2002	 11/2001[4,6,7] 05/2005
Information Collection Rule	02/94	05/96	07/97
Enhanced Surface Water Treatment Rule Interim Long-Term (Stage 1) Long-Term (Stage 2)	 07/94 11/99 11/2000	 11/98[3] 11/2000 05/2002	 11/2001[4,6] 11/2003[4] 05/2005[4]
Consumer Confidence Reports Rule	01/98	08/98	09/98
Groundwater Disinfection	01/99	01/2001	01/2003
Filter Backwash Rule	08/99	08/2000	08/2003[4]
Source Water Protection Program (Guidance[5])	08/97	-	-
Arsenic	01/2000	01/2001	01/2004[4]

[1]Start date for tap monitoring; systems serving more than 50,000 consumers.
[2]MCL, MCLG for atrazine to be reconsidered.
[3]Date mandated by District Court.
[4]Assumes regulation in effect 3 years after final promulgation.
[5]Program required as part of 1996 Amendments.
[6]For systems serving more than 10,000 consumers.
[7]Effective 2003 for systems serving less than 10,000 consumers.

APPENDIX C

Water Treatment Plant:
Process and Site Requirements

Technical Memorandum
Water Treatment Alternatives

Clovis, California
Water System Master Plan

B&V Project 34404.104
March 5, 1998
Revised March 13, 1998

Prepared By: Bruce Corwin, Ron Henderson, and Doug Elder

Introduction

A. Background

This technical memorandum is one of several special studies being conducted as part of the development of a Water System Master Plan for the City of Clovis. The Master Plan addresses the development of a surface water supply system to augment the City's existing groundwater supply. The raw water supply will be the Kings River, delivered to the treatment plant site through the Enterprise Canal, an unlined canal which serves Clovis and the northern portions of the City of Fresno. Initial treatment plant capacity is projected to be 10 mgd, with provisions for expansion to an ultimate capacity of 30 mgd.

B. Purpose

The purposes of this memorandum are: (1) to present the results of a preliminary screening of treatment process alternatives for a new surface water treatment facility, and (2) to identify treatment alternatives which warrant additional evaluation prior to initiating design of the new treatment facilities, and their associated probable construction costs.

Treatment Objectives

The new treatment facility must be designed to comply with both current and anticipated future water quality and treatment requirements. (Regulatory requirements are summarized in the February 10, 1998 Technical Memorandum "Water Quality and Regulatory Requirements". In addition to meeting all applicable federal and state water quality criteria, the following supplemental criteria should be addressed in the design of the treatment facilities:

- Ability to accommodate rapid variations in raw water turbidity during periods of localized runoff, while maintaining filtered water turbidities of 0.1 NTU or lower. Processes which cannot accomodate periodic high turbidities must be easily stoppped and re-started, allowing the plant to be "turned off" for a day or so during each winter event.
- Ability to produce a treated water which is chemically-compatible with the City's current groundwater supply.
- Provisions for reliable removal of *Giardia* cysts and *Cryptosporidium* oocysts through highly-efficient filtration.
- Flexibility to facilitate future expansion and/or construction of modifications to meet increasingly-stringent future water quality regulations.
- Ability to remove algae and other nuisance organisms without physical accumulation in flumes, basins, or other areas and without significant reduction in filter productivities.
- Provisions for control of undesirable tastes and odors resulting from localized runoff and/or algae activity within the Enterprise Canal.
- Provisions for removal of agricultural chemicals using powdered activated carbon or post-filter granular activated carbon contactors.

Treatment Process Alternatives

The probable impacts of raw water quality for the Enterprise Canal and current/impending regulations on treatment requirements were discussed in the Technical Memorandum "Water Quality & Regulatory Requirements" dated February 10, 1998. The following summarizes potential treatment process options.

A. Conventional Treatment

1. Process Overview

Conventional treatment consists of chemical coagulation, flocculation, sedimentation, and filtration. As the majority of the suspended solids present in the raw water supply and/or formed through coagulation and flocculation are removed by gravity settling prior to filtration, filter run times between backwashes are maximized. Sedimentation also reduces the potential for accumulation of nuisance organisms, such as algae, within the filters. The residence time within a conventional treatment process (typically 4 to 5 hours at the design flow rate) allows for oxidation/adsorption of disinfection by-product precursors, color, odors, and other contaminants prior to filtration and post-disinfection.

Conventional treatment provides multiple barriers for the removal of particulate material through the treatment process, and consequently minimizes disinfection requirements. Further, conventional treatment allows optimizing of coagulation conditions for the removal of disinfection by-product precursor compounds, with minimal impact on filter run times and productivity. Disadvantages include the construction costs associated with the installation of large sedimentation basins (although this impact can be reduced to some extent through use of high-rate sedimentation technologies, as discussed below), and production of greater quantities of sludge than other treatment alternatives. Also, as algae are very difficult to remove through conventional sedimentation, the settled water may contain high concentrations of algae cells during periods when algae are present in the Enterprise Canal water, which could reduce filter run times between backwashes.

When coupled with an advanced oxidant/disinfectant such as ozone, benefits of the combined process include: (1) maximum disinfection effectiveness and ability to meet potential future requirements for inactivation of *Cryptosporidium*, (2) ability to oxidize iron, manganese, and taste and odor-causing compounds, and (3) the capability, particularly when provisions for supplemental addition of hydrogen peroxide are included, to oxidize synthetic organic chemicals, should they be detected in the Enterprise Canal water in the future.

2. Unit Process Alternatives

Efficient flocculation (i.e., agglomeration of non-settleable particles and colloidal materials into settleable and/or filterable floc particles) is required for successful removal of turbidity, color, and disinfection by-product precursor compounds. Current design practice includes provisions for "tapering" of flocculation energy as flow proceeds through the basin. This reduces "shearing" of floc particles and permits operators to optimize the flocculation process. While both horizontal paddle and vertical shaft turbine flocculators have been widely used, current design practice tends to favor turbine-type units, based on absence of submerged bearings and reduced susceptibility to corrosion.

Conventional sedimentation basins are typically designed using surface loading rates of 500 to 800 gallons per day per square foot of surface area. Equivalent settling efficiency can be achieved in smaller basins through installation of inclined plates. This equipment consists of a series of stainless steel or FRP plates inclined at 55 degrees from horizontal and typically spaced 2 inches apart. Flow through the plates is upward. The use of inclined-plate equipment typically permits sedimentation basin sizes to be reduced by factors of 5 to 8 (as compared to conventional sedimentation basins), an important consideration where available site areas are limited. Where space is sufficient, the use of inclined plates is usually not cost effective.

Filtration is required for final polishing of chemically coagulated waters before distribution to consumers. While filtration has historically been used primarily to improve the aesthetic quality of the water through removal of turbidity, it has recently been recognized as a critical process in the removal of microbial contaminants such as *Giardia* cysts and *Cryptosporidium* oocysts. Filter media typically consists of conventional dual media (anthracite over sand) with total depths of 30 to 36 inches, or newer deep-bed monomedium designs consisting of 4 to 6 feet of 1.2 to 1.5 mm anthracite or granular activated carbon. Advantages of the deep-bed configurations include ability to operate at higher hydraulic loading rates, increased run times between backwashes attributable to increased solids storage capacity, and superior performance when algae are present in the raw water supply. (As mentioned above, the presence of algae in the settled water sometimes results in substantial reductions in run times for conventional dual-media and mixed-media filters.)

The current California Surface Water Treatment Rule (SWTR) requires that surface water treatment facilities provide multi-barrier treatment, consisting of both filtration and disinfection, to achieve a minimum 3.0-log removal/inactivation of *Giardia* cysts and a minimum 4.0-log removal/inactivation of enteric viruses. DHS typically credits facilities practicing conventional treatment with a 2.5-log Giardia removal and a 2.0-log virus removal. Therefore, an additional 0.5-log inactivation of *Giardia*, and an additional 2.0-log inactivation of viruses must be achieved through disinfection to comply with current minimum SWTR requirements. These required levels of inactivation are used to establish a necessary, "CT" value, which dictates the volume of the disinfected contact basin.

B. Direct Filtration

Direct filtration consists of chemical coagulation, flocculation, and filtration. All suspended solids present in the raw water and/or formed as a result of chemical coagulation are removed during the filtration process. Elimination of the sedimentation process (as utilized in conventional treatment) reduces required plant site area, and may result in construction cost savings of 20 to 30 percent. Where treatment of low turbidity/low color waters is required, direct filtration can produce treated water of quality similar to that produced using treatment processes incorporating conventional sedimentation. Production of a small, filterable floc, rather than a large, rapidly settling floc, is required for efficient filter operation. Chemical coagulant dosage requirements are therefore typically less than for conventional treatment processes, and sludge solids production is reduced. Disadvantages of direct filtration include shorter filter runs between backwashes than for conventional treatment, reduced operator "reaction time" to changes in raw water quality, and the inability to readily accommodate large variations in raw water turbidity and suspended solids. Also, when algae is present in the raw water supply, use of direct filtration may lead to unacceptably-short filter run times between backwashes.

Typical direct filtration process designs include flocculation detention periods of 15 to 20 minutes, at mixing intensity levels generally 50 to 75 percent higher than for conventional treatment processes. Filters are commonly dual- or mixed-media with hydraulic loading rates in the 4 to 5 gpm/sq ft range. However, recent full-scale operating experience has demonstrated that deep-bed monomedium filters, when preceded by ozonation, are capable of operating in a direct filtration mode at loading rates of 10 to 15 gpm/sq ft with no degradation in filtered water quality. Several direct filtration facilities utilizing high-rate monomedium filters are currently in operation.

Based on pilot- and full-scale operating experience, DHS has credited direct filtration with a 2.0-log removal of *Giardia* cysts and a 2.0-log removal of viruses. Therefore, an additional 1.0-log inactivation of *Giardia*, and an additional 2.0-log inactivation of viruses must be achieved through disinfection to comply with current minimum Surface Water Treatment Rule requirements.

C. Microfiltration

Microfiltration (MF) is a physical treatment process in which colloidal particles are removed from the water supply by straining through a porous medium. MF provides exceptional removal of turbidity (most operating facilities routinely produce treated water with turbidities of less than 0.05 to 0.1 NTU). Most MF membranes used for treatment of surface water supplies are hollow-fiber polypropylene with a nominal pore size of 0.2 microns. As this pore size is significantly smaller than *Cryptosporidium* oocysts (2 to 5 microns) and *Giardia* cysts (7 to 10 microns), MF also provides excellent removal of these microbial contaminants (pilot studies have demonstrated *Giardia* cyst removals of up to 6.0-logs). Based on pilot- and full-scale operating experience, DHS has credited

MF with a 3-log removal of *Giardia* cysts and a 0.5-log removal of viruses. Therefore, an additional 3.5-log inactivation of viruses must be achieved through disinfection to comply with current minimum Surface Water Treatment Rule requirements (this inactivation can be easily achieved through post-MF disinfection using free chlorine). These required levels of inactivation are used to establish a necessary, "CT" value, which dictates the volume of the disinfected contact basin.

MF exhibits minimal pretreatment requirements. The raw water is typically passed through 300 to 500 micron continuous-cleaning strainers to remove large particles which could rapidly foul the membranes. Typical design MF loading rates are 0.5 to 0.6 gpd/m^2, and typical "average" feedwater pressure is 15 to 20 psi. Backwashing of the membrane modules is typically initiated every 18 to 20 minutes, and the backwash cycle typically lasts for approximately 2.5 minutes. A combination of air and raw water is used to backwash the membrane surface (the air dislodges particles from the membrane surfaces, and raw water is used to flush the particles from the modules). Backwashing typically uses approximately 5 to 7 percent of the raw water pumped to the MF system; however, recycling of the backwash flow to the plant influent following treatment to remove settleable solids can reduce overall losses to 1 to 2 percent of plant production.

While air/water backwashing is effective in removing most of the solids deposited on the membrane surfaces, a small percentage of the particles remain after backwashing. This accumulation of material on the membrane surface eventually leads to increases in required membrane operating pressures. When differential pressures across the membrane system (i.e., "transmembrane pressures") following backwashing routinely reach 18 to 20 psi, chemical cleaning with caustic and proprietary detergent solutions is initiated to restore system production capacity. Typically, membrane cleaning is required every 10 to 15 days. Disposal of the spent cleaning solutions typically is accomplished through discharge to the sanitary sewer system.

MF-treated water exhibits extremely low turbidities which are difficult to monitor consistently; provisions for continuous monitoring of treated water particle counts are required to ensure that the membranes are operating properly. In addition, DHS requires that an air integrity test be conducted at least once per day to ensure that the membranes and associated gaskets/seals are functioning properly. The air integrity test is typically automated (no operator attention is required), and lasts for only several minutes.

As the MF process does not remove organic compounds, and has not demonstrated the ability to affect any significant removal of tastes and odors, provisions for short-term addition of powdered activated carbon (PAC) prior to MF, or post-MF granular activated carbon contactors would be required. For addition of PAC, a mixed contact basin upstream of the MF units would be used to provide approximately 20 minutes of contact time, which would be adequate for reduction of tastes/odors and for removal of most of the regulated synthetic organic chemicals. The PAC present in the contact basin discharge would be removed by the MF process. (While experience in MF treatment of waters containing high concentrations of PAC is limited, preliminary results suggest that operation at applied PAC dosages of up to 20 mg/L has no detrimental impact on process performance.) When raw water quality does not require PAC, the mixing basin would be idle. Carbon feed could be required in Clovis for 30 to 60 days per year.

Advantages of microfiltration over conventional treatment processes are: (1) little or no chemical addition is typically required, with correspondingly lower sludge production and the sludge produced is easily handled and disposed, (2) wide variations in raw water turbidities have relatively little impact on the MF-treated turbidities, (3) compact size; modular construction facilitates plant expansion, (4) MF provide a positive barrier to *Giardia* and *Cryptosporidium*, thereby reducing disinfection requirements, and (5) ease of operation. Disadvantages include: (1) poor removal of organics and taste/odors, (2) limited ability to remove color, and (3) higher electrical power costs than conventional processes.

D. Contact Adsorption Clarification Process

The contact adsorption clarification (CAC) process combines coagulation, flocculation, and clarification processes within a single upflow adsorption clarifier which utilizes the contact flocculation/adsorption phenomenon to remove turbidity and color. Chemically-coagulated water is introduced at the bottom of the adsorption clarifier, and passes upward through the adsorption media. The media consists of either buoyant plastic beads retained within the clarifier by a screen (U.S Filter "Trident" process) or smaller, non-buoyant media (Infilco Degremont "Advent" process, Roberts Filter "Pacer II" process). Flocculation is accomplished by turbulence imparted as the water flows upward through the media. Solids formed as a result of flocculation adhere to the media, and subsequently enhance the removal of newly-formed floc particles. As formation of a large, rapidly-settling floc is not required for efficient process operation, required coagulant dosages may be somewhat less than for conventional flocculation/sedimentation processes.

When the adsorption clarifier effluent quality degrades to unacceptable levels, or when headloss across the clarifier reaches design levels, the clarifier is cleaned using upflow hydraulic flushing. Air is introduced at the bottom of the clarifier through a distribution system. For the non-buoyant media system, the air is used solely to scour accumulated solids from the media, while the buoyant media system uses air to reduce the buoyancy of the media, thereby allowing it to expand downward. The dislodged solids are flushed from the clarifier using the contactor influent flow, and are discharged to waste. Typical contactor operating times between wash cycles are 4 to 8 hours. Adsorption contactors are typically designed to provide a media loading rate of 10 gpm/sq ft. The clarifiers are available alone or coupled with conventional dual- or mixed-media filters in a modular configuration.

As contact time within the adsorption clarifier portion of the process is relatively short (typically less than 1.5 minutes at design flow rates), application of powdered activated carbon (PAC) at the clarifier inlet may be ineffective in removing taste and odors. A separate PAC contact basin prior to the adsorption clarifiers (as discussed above for the microfiltration process), or provisions for post-filtration granular activated carbon contactors would therefore be required to ensure positive control of tastes and odors and removal of synthetic organic contaminants. The PAC present in the contact basin discharge would be removed by the adsorption clarifiers. However, experience with operation at high applied PAC dosages (i.e., above 5 to 10 mg/L) at the adsorption clarifier inlet is extremely limited; pilot testing would therefore be required to ensure that problems with PAC carryover through the adsorption clarifiers to the filters at high dosages are not experienced. Also, while full-scale operating experience using raw water supplies with significant levels of algae is limited, the CAC process should be superior to conventional treatment with respect to ability to remove algae prior to filtration.

DHS has credited the contact adsorption/filtration process with a 2.5-log *Giardia* removal and a 2.0-log virus removal. Therefore, an additional 0.5-log inactivation of *Giardia*, and an additional 2.0-log inactivation of viruses must be achieved through disinfection to comply with current minimum SWTR requirements.

E. Ballasted Flocculation Process

The ballasted flocculation process (BFP) is a relatively new treatment innovation marketed by two firms (Kruger, Inc. "ACTIFLO" process, Microsep Systems "BFR" process). In the BFP, floc particles formed through addition of a metal-salt coagulant (alum or ferric) to the raw water supply are attached to an inert particle carrier (IPC) through addition of polymer. The IPC acts as a weighting agent, and facilitates removal of the combined particles in a clarifier downstream of the mixing/flocculation zone.

(Micro-sand with an effective size of 50 to 100 microns is used as the weighting agent by the current primary manufacturer of this process.) The high settling rates achieved for the combined floc and micro-sand particles allows operation of the settling process at rates 10 to 40 times higher than for conventional sedimentation basins. Total detention time through the mixing, flocculation, and settling basins is typically only about 12 minutes at the design flow rate. The micro-sand is separated from the settled sludge by pumping through a high-shear pump and a small cyclone separator. The micro-sand is then recycled to the treatment process, while the floc particles removed from the micro-sand are conveyed in a liquid sidestream to the sludge disposal facilities. Typical floc/micro-sand recycle rates are approximately 3% of the process flow rate. (Some loss of micro-sand occurs during the recycle/solids separation process, and supplemental addition equal to approximately 8 to 10 pounds per million gallons of water treated is typically required.) Settled water from the high-rate clarifier is directed to filtration for final polishing prior to distribution. While full-scale operating experience using raw water supplies with significant levels of algae is limited, results of pilot-scale testing suggest that the BFP process is superior to conventional treatment with respect to ability to remove algae prior to filtration.

Advantages of the BFP (as compared to conventional flocculation/sedimentation basins) include: (1) significantly lower site area requirements (approximate total site area requirements for two parallel 5 mgd treatment trains would be only 41 ft x 34 ft, plus filters, (2) lower settled turbidity/suspended solids concentrations, (3) potential savings in chemical costs, as overall coagulant dosages are typically lower than for conventional treatment, (4) rapid process startup and stabilization, and (5) stable operation/performance during short-term influent turbidity "spikes". Disadvantages include the need for continuous addition of both a metal coagulant and polymer (loss of either coagulant typically results in rapid degradation of settled water quality), and the current lack of full-scale U.S. operating experience (essentially all of the existing installations are located in Canada and Europe; the first operating installation began service in 1991). DHS has not specified microbial removal credits for the BFP, but it is expected that credits equal to those for conventional treatment could be obtained following demonstration testing.

F. Dissolved Air Flotation

Dissolved air flotation (DAF) is an emerging technology which has shown promise in the treatment of water supplies with low turbidity and/or algae problems. It has been used successfully for more than 20 years in Europe, and has become the preferred method for clarification of surface water supplies in England, the Netherlands, and Belgium. DAF is very effective for removal of coagulated low-density suspended particles, such as algae and colloidal turbidity, from water because it is easier to "float" these particles than to form large floc particles that will settle.

Both direct filtration and DAF/filtration utilize rapid mixing, coagulation, and flocculation. However, in the DAF process, the flocculated water is discharged to a flotation basin. The flotation action is produced by recycling a portion of the clarified water which has been saturated with air under pressure. The cloud of small bubbles produced by the discharge of this water into to inlet of the flotation chamber carries the coagulated particles to the top of the chamber, where they accumulate and are periodically skimmed off and directed to a waste handling system.

The DAF process utilizes smaller basins for flocculation and flotation than required for conventional sedimentation processes, which results in lower construction costs. Significant savings in chemical costs are possible as well, as less coagulant is used for flocculation. When the water contains algae, DAF typically produces a more filterable water than conventional sedimentation, which results in longer filter runs. The solids concentration of the residuals generated (commonly referred to as "float") is also typically higher than that produced by conventional sedimentation (typically about 3 percent, vs. 0.25 to 1 percent for conventional treatment).

The primary disadvantage of DAF/filtration is the electric power costs associated with pumping of the recycle stream and operation of the air saturation system, which produce the bubbles required for flotation. However, higher power costs are often partially offset by the reduction in the costs of coagulants and flocculant aids, as well as in the reduced size of facilities for residuals treatment and disposal.

G. Residuals Handling & Disposal

Treatment of surface water supplies, such as water from the Enterprise Canal, produces waste streams containing both natural solids (silt, clay) and chemical constituents resulting from addition of treatment compounds such as coagulants and powdered activated carbon. Waste streams for each of the treatment alternatives discussed above are summarized in Table 1. A listing of applicable filter backwash and sludge treatment/disposal options is presented below.

Table 1 Residuals Produced by Water Treatment Process Alternatives	
Treatment Process	Residual(s) Produced
Conventional Treatment	(1) Chemical sludge from sedimentation process (2) Filter backwash / Filter-to-waste flows
Direct Filtration	(1) Filter backwash / Filter-to-waste flows
Microfiltration	(1) Membrane backwash
Contact Adsorption Process	(1) Adsorption clarifier backwash (2) Filter backwash / Filter-to-waste flows
Ballasted Flocculation Process	(1) Chemical sludge from sedimentation process (2) Filter backwash / Filter-to-waste flows
Dissolved Air Flotation	(1) Float from DAF clarifiers (2) Filter backwash / Filter-to-waste flows

1. Filter Backwash Disposal Options

Treatment

- Onsite settling ponds
- High-rate clarifiers
- Microfiltration

Disposal of Treated Backwash

- Recycle to treatment process
- Discharge to groundwater recharge basins
- Discharge to Enterprise Canal
- Discharge to sanitary sewer

2. Clarification Sludge Dewatering & Disposal Options

Sludge Dewatering

- Onsite Temporary Lagooning
- Permanent Lagoons
- Vacuum-Assisted Drying Beds
- Mechanical Dewatering

Ultimate Sludge Disposal

- Landfill Disposal
- Land Application
- Discharge to Municipal Wastewater Treatment Facility
- Contract Disposal

For purposes of planning level design, we have assumed that filter backwash water will be recylced after treatment, sedimentation sludges will be disposed of by lagoon, and membrane process waste will be disposed of by lagoon.

Treatment Process Evaluation

Preliminary discussions with DHS suggest that approval of a direct filtration process for treatment of Enterprise Canal water is unlikely, based on concerns regarding limited treatment flexibility and difficulties in accommodating periodic high raw water turbidities. Also, limited full-scale U.S. operating experience with the ballasted flocculation process, and the absence of operating facilities in California, may result in the need to conduct extensive pilot-scale testing before DHS approval could be obtained. (However, based on the potential for both cost and site area savings, further discussions with DHS regarding the feasibility of this process and probable approval requirements should be considered prior to implementing any pilot-scale testing program.) Dissolved air flotation would be considered only if problems with short filter runs attributable to the presence of significant levels of algae in the settled water are experienced during initial pilot-scale testing. Therefore, three treatment processes (conventional treatment, microfiltration, and contact adsorption clarification) were selected for evaluation of site area requirements and probable costs. A brief description of each of these processes is presented below, and preliminary design parameters used in the development of probable construction and annual operating and maintenance costs are summarized at the end of this memorandum.

A. General

Facilities and design considerations which would be common to two or more of the treatment process alternatives are discussed below.

1. Capacity

Design treatment capacity for the initial facilities would be 10 mgd, with provisions to facilitate future expansion to an ultimate capacity of 30 mgd on the selected plant site. For the conventional treatment and contact adsorption clarification alternatives, each component would be sized to handle up to 150 percent of its design treatment capacity without overtopping of structure walls; however, under hydraulic overload conditions, overall treatment performance will be diminished. For reliability and operational flexibility, pretreatment unit processes (the treatment processes preceding filtration) will be designed as two or more separate treatment trains suitable for independent and parallel operation to enable direct comparison and optimization of chemical feed rates, energy inputs, and other process variables.

For cost development purposes, it is assumed that the plant would operate in a "baseload" flow condition (i.e., at full design treatment capacity) 11 months per year, and be shut down for routine maintenance of the raw water supply system during the remaining month.

2. Operations Building

All of the treatment alternatives include an operations building which will house offices, administrative and personnel areas, maintenance facilities, required laboratory facilities, and chemical feed areas. The facility would be sized based on projected requirements at the full 30 mgd ultimate capacity.

3. Raw Water Intake & Pumping, High Service Pumping

It is assumed that water would be taken from the Enterprise Canal and that a direct connection would be made for intake facilities. This connection to the canal would consist of screening and control facilities placed in the canal. The design of such facilities will need to consider items such as horizontal and vertical location relative to the canal top and bottom and protection. Actual construction details would need to be coordinated with the Fresno Irrigation District.

Raw water pumping would take place at the water plant site. For sizing purposes this facility would include redundancy in terms of pumping capacity so that one pump could be down for service and/or maintenance. The intent of such facilities would be to lift the raw water and provide enough head to move water through the water treatment plant. Initial firm capacity would be 10 mgd with space allocated for future pumps to 30 mgd.

Pumping to the distribution system from the water treatment plant would take water out of disinfectant contact tank/reservoir and deliver it to the water distribution system. Pumps would be provided to ensure proper head and velocity to the service connection point. The facility would include redundancy and provide for capability to have one pump out of service.

4. Disinfection

Free chlorine would be used for primary disinfection and to maintain a disinfectant residual within the distribution system. For site layout and cost development purposes, it is assumed that DHS may require that a higher level of disinfection than current SWTR minimum values be provided, based on results of a sanitary survey for the Enterprise Canal supply which would be completed prior to plant design. Provisions for a total 4-log removal/inactivation of *Giardia* cysts (1-log greater than the current minimum SWTR requirement of 3-log removal/inactivation) are therefore assumed.

Results of limited trihalomethane formation potential testing conducted on water from the Enterprise Canal suggest that use free chlorine for disinfection should not present any significant problems with respect to compliance with current and anticipated future disinfection by-product regulations. However, additional disinfection by-product testing (seasonally, as a minimum) is recommended prior to initiating design of the new treatment facilities to verify feasibility of using free chlorine for disinfection and distribution system residual maintenance.

For the conventional treatment and contact adsorption clarification alternatives, plant facilities design and layout should include space allocation for installation of ozone disinfection capability, should this be required by DHS in response to impending microbial contaminant control regulations or to address concerns with the quality of the raw water from the Enterprise Canal. In addition to providing positive inactivation of chlorine-resistant microbial contaminants such as *Cryptosporidium* and *Giardia*, additional benefits of ozonation would include control of undesirable tastes and odors, and oxidation of synthetic organic contaminants, such as pesticides & herbicides. Ozone would be added between the sedimentation basins and the filters for the conventional treatment alternative, and between the CAC units and filters for the contact adsorption clarification alternative.

5. Sludge Handling / Disposal

For cost development purposes, and to provide a conservative estimate of maximum plant site area requirements, use of onsite lagoons for dewatering and temporary storage of sludge produced by the treatment process is assumed. A minimum of three lagoons would be provided, each with sufficient capacity to store approximately 2 years of solids production at an average settled solids concentration of 5 percent. Decant from the lagoons would be suitable for discharge back to the Enterprise Canal, for discharge to the sanitary sewer system, or for discharge to groundwater recharge basins. Following settling/decanting of a cell, the sludge would be allowed to air dry for up to one year, and then would be removed and transported to an ultimate disposal site (additional dewatering may be required prior to disposal if landfilling is to be used as the means of ultimate disposal). Use of mechanical dewatering would also be a viable alternative; however, both initial construction and annual operating costs would be considerably higher than for temporary lagoon dewatering. Construction of mechanical dewatering facilities may be appropriate, however, during future plant expansions, when increases in sludge production result in the need for additional lagoons and more frequent removal of thickened sludge.

6. Chemical Feed Facilities

Chemical feed systems would be designed to deliver the maximum expected dosages at the plant design capacity. Storage facilities would be designed to provide a minimum of 30 days storage at average dosages and flow rates. Feed systems would be designed to utilize liquid chemicals to the maximum practical extent in order to reduce feed system complexity and demands for operator attention. Sodium hypochlorite would be used for disinfection, based on safety and handling concerns associated with storage and feeding of gaseous chlorine.

7. Treated Water Stability Adjustment

As discussed in the February 10th Technical Memorandum "Water Quality and Regulatory Requirements", adjustment of treated water alkalinity and pH will be required to ensure the compatibility of the City's existing groundwater supply with the new surface water supply. For cost development and site layout purposes, it is assumed that this would require addition of hydrated lime to increase the treated water's alkalinity, and addition of carbon dioxide to maintain the pH of the resulting lime-treated water below 8.5. Lime would be added first in a chamber equipped with a mechanical mixer, and carbon dioxide would be diffused into a second chamber.

B. Conventional Treatment Alternative

A weir structure or rate·controllers would divide the total incoming flow from the Enterprise Canal equally between two flocculation/sedimentation basin trains. Each train would be equipped with a tw stage rapid mix chamber for dispersion of chemical coagulants into the process stream. (For cost development and site layout purposes, use of conventional two-cell mechanically-mixed chambers is assumed.) Provisions for occasionally feeding powdered activated carbon (PAC) at the rapid mix to remove tastes and odors and for adsorption of synthetic organic contaminants would be included. The chemically-treated water would then flow to two parallel rectangular multi-cell flocculation basins. Flocculation cells would be equipped with vertical turbine-type mixers, with provisions for reducing energy levels imparted to the process flow as it progresses through the basin. A theoretical flocculation detention time of approximately 30 minutes has been assumed. Flocculated water would enter the sedimentation basins through slotted baffle walls. Sedimentation basins would be designed with surface loading rate of 0.5 gpm/sq ft. Sludge form the sedimentation basins would discharge to onsite lagoons, as discussed above.

Settled water would flow to four dual-media filters. Design filter hydraulic loading would be approximately 4.5 gpm/sq ft, and filter media would consist of 30 inches of 1.0 mm effective size anthracite over 10 to 12 inches of fine sand. The filters would be equipped with provisions for air scouring and filter-to-waste, and for addition of polymer at the filter influent as a filter aid to maximize turbidity removal and to reduce the duration and intensity of turbidity spikes following return of a backwashed filter to service. Filter backwash would discharge by gravity to a holding basin, and be pumped back to the treatment process through a high-rate treatment basin equipped with inclined plates, which would maximize removal of suspended solids from the washwater prior to recycle.

Filtered water would flow to two parallel chlorine contact basins, with each basin treating one-half of the design flow. The contact basins would be designed to provide a minimum 1.5-log inactivation of *Giardia* cysts under anticipated "worst-case" temperature and pH conditions, and would be equipped with baffles to provide an effective (T_{10}) disinfectant contact time equal to 60 percent of the basin's theoretical contact time. (DHS could specify a lower minimum *Giardia* inactivation requirement of 0.5-log, which would allow significant reductions in the size of the chlorine contact basins.)

Following disinfection, the treated water would flow to a stabilization basin, where hydrated lime would be added to increase alkalinity to 40-50 mg/L, and carbon dioxide would be added to maintain the pH of the finished water at 8.5 or below. (Sodium hydroxide could be added as an alternative to lime, although chemical costs would be significantly higher. Other alternatives to lime/CO_2 addition should be evaluated during preliminary plant design phases.) The stabilized water would then flow to onsite storage facilities prior to pumping to the distribution system.

C. Microfiltration Treatment Alternative

A weir structure or rate controllers would divide the total incoming flow from the Enterprise Canal equally between two parallel powdered activated carbon (PAC) treatment basins, with a total volume sufficient to provide 20 minutes of contact time at the plant design flow. PAC would occasionally be fed at the basin inlet to control undesirable tastes and odors, and to adsorb synthetic organic contaminants (herbicides/pesticides). The contact basins would be divided into two cells, each equipped with a turbine-type mixer to maintain the PAC in suspension. (Results of limited full-scale testing indicate that PAC can be fed at low dosages at the microfilter influent without significant degradation of microfiltration performance. However, experience at high applied PAC dosages is limited, and pilot-scale testing would therefore be recommended prior to initiating design to confirm the feasibility and effectiveness of the proposed PAC pretreatment system.) The PAC-treated water would be pumped to five 2 mgd microfiltration trains. Each train would consist of 2 parallel microfiltration units, each with 90 microfilter modules. Average microfilter flux rate assumed for cost development and site layout purposes is approximately 0.54 gpm per square meter. Assumed average backwash interval and duration are 25 minutes and 2.5 minutes, respectively. The projected average backwash water requirement with the plant operating at full 10 mgd production capacity would be approximately 0.6 mgd; required raw water pumping capacity is therefore 10.6 mgd. Microfilter backwash flows would discharge to onsite lagoons.

Microfilter effluent would discharge to two parallel chlorine contact basins, with each basin treating one-half of the design flow. The contact basins would be designed to provide a minimum 1.0-log inactivation of *Giardia* cysts under anticipated "worst-case" temperature and pH conditions, and would be equipped with baffles to provide an effective (T_{10}) disinfectant contact time equal to 60 percent of the basin's theoretical contact time. (Based on results of a sanitary survey of the Enterprise Canal supply, and microfiltration's demonstrated ability to achieve a minimum 3-log *Giardia* removal, DHS could elect to require disinfection treatment only for inactivation of viruses, which would allow substantial reductions in the size of the chlorine contact basins.) The disinfected water would flow to stabilization basins (as discussed above for the conventional treatment altern^tive), and then to onsite treated water storage prior to being pumped to the distribution system.

D. Contact Adsorption Clarification Alternative

Raw water would be split to two parallel PAC treatment basins, as discussed above for the microfiltration treatment alternative. The PAC-treated water would then be directed to the upflow contact adsorption clarifiers. (The number of CAC trains to be provided would be determined during preliminary plant design phases; however, for cost development purposes, use of 4 parallel CAC trains was assumed.) Alum or ferric sulfate (and coagulant aid polymer, if necessary) would be added at the CAC influent using in-line static mixers to disperse the coagulants into the process stream. Design CAC hydraulic loading rate would be 10 gpm/sq ft. CAC media would be either buoyant plastic beneath a retaining screen, or unrestrained, nonbuoyant (i.e., garnet) media. Both types of media would require provisions for air scouring during the media flush cycle. The effluent and flush water for each CAC unit would be collected in concrete troughs above the media, and can be routed to waste during the flush cycle or to the filter influent when the unit is on-line. The CAC flush water (typically about 1% of total plant production) would discharge to onsite lagoons.

CAC effluent flow would be routed to a common header, with provisions for future connection to an ozonation facility. The CAC effluent would be split to four dual-media filters (recommended filter configuration and backwash handling systems would be identical to that for the conventional treatment alternative discussed above). The filtered water would then discharge to disinfection contact chambers and to stabilization facilities, as discussed for the conventional treatment alternative.

E. Probable Costs

Conventional Treatment Costs

Component	Cost x $1,000
Mobilization	500
Raw Water Pump Station	400
Chemical Feed Facilities	500
Coagulation/Flocculation	600
Filter Complex	1,200
2.0 MG Disinfectant Contact Stabilization Basin	1,500
Finished Water Pump Station	375
Wash Water Recovery and Treatment	200
Solids Lagoons	500
Electrical	750
Instrumentation and Controls	600
Sitework	750
Control Building - 2,000 sf - 2 2X14 offices Control Room Lunch/Meeting Room 2 Locker/Restrooms Operators Lab Furnishings	400
20 % Contingencies	1,645
Total	**$9,890** $0.99 / gallon of capacity

Microfiltration Costs

Component	Cost x $1,000
Mobilization	550
Raw Water Pump Station	400
Chemical Feed Facilities	300
Microfiltration	4,400
2.0 MG Disinfectant Contact Stabilization Basin	1,500
Finished Water Pump Station	375
Reject Holding Pond	300
Electrical	500
Instrumentation and Controls	250
Sitework	400
Control Building - 2,000 sf - 2 2X14 offices Control Room Lunch/Meeting Room 2 Locker/Restrooms Operators Lab Furnishings	400
20 % Contingencies	1,875
Total	**$11,250** $1.13/ gallon of capacity

Contact Absorption Costs

Component	Cost x $1,000
Mobilization	475
Raw Water Pump Station	400
Chemical Feed Facilities	400
Contact Adsorption Clarifier Basins	550
Filter Complex	1,200
2.0 MG Disinfectant Contact Stabilization Basin	1,500
Finished Water Pump Station	375
Wash Water Recovery and Treatment	200
Solids Lagoons	500
Electrical	750
Instrumentation and Controls	500
Sitework	625
Control Building - 2,000 sf - 2 2X14 offices Control Room Lunch/Meeting Room 2 Locker/Restrooms Operators Lab Furnishings	400
20 % Contingencies	1,535
Total	**$9,410** $0.94 / gallon of capacity

Table 2
Annual Operational Cost
for 10 MGD WTP

Item	Conventional	Membrane	CAC
Labor - Supervision - Maintenance - Operation - Sludge Handling	$500,000	$500,000	$500,000
Equipment - Maintenance - Replacements	$150,000	$230,000	$200,000
Power (Pumping)	$150,000	270,000	200,000
Chemicals - PAC - Lime - Cl2 - Polymer - Flocculant - WW Recovery/Trmt	$260,000	$140,000	$230,000

Total O&M

Conventional	$1,060,000
Membrane	$1,140,000
CAC	$1,130,000

Table 3
Life Cycle Cost

Plant Type	Costs
Conventional	
Plant Cost	$9,890,000
O&M	$1,060,000
Land	$1,500,000
Cost per Year - 8% @ 20 years	$2,134,000/year
Membrane	
Plant Cost	$11,250,000
O&M	$1,140,000
Land	$1,100,000
Cost per Year - 8% @ 20 years	$2,305,000/year
Contact Absorption	
Plant Cost	$9,410,000
O&M	$1,130,000
Land	$1,300,000
Cost per Year - 8% @ 20 years	$2,310,000/year

Attachment A

Preliminary Design Criteria For Water Treatment Alternatives (Initial 10 mgd Plant Capacity)

Table A-1
Preliminary Design Criteria for Conventional Treatment

Parameter	Value
Rapid Mixing	
No. of basins	2
No. of cells per basin	2
Detention time per cell, seconds	15
Mixing velocity gradient ("G"), sec^{-1}	750
Flocculation	
No. of basins	2
Mixing zones per basin	3
Detention times	
Per basin, minutes	30
Per zone, minutes	10
Maximum velocity gradient ("G"), sec^{-1}	
Zone 1	75
Zone 2	50
Zone 3	30
Sedimentation	
No. of basins	2
Surface loading rate, gpm/sq ft	0.5
Hydraulic detention time, hours	3.0
Filtration	
No. of filters	4
Hydraulic loading rate	
With 4 filters in service, gpm/sq ft	4.5
With 3 filters in service, gpm/sq ft	6.0
Media	
Anthracite (1.5 mm ES), inches	30
Fine sand (0.45 - 0.55 mm ES), inches	10 - 12
Backwash	Upflow water w/air scour
Max. backwash rate, gpm/sq ft	20
Chlorine Contact Basins	
No. of basins	2
Design flow per basin, mgd	5.0
Log *Giardia* inactivation req'd*	1.5
Min. water temperature, degrees C	8
Max. pH	7.0
Max. free chlorine residual, mg/L	1.0
CT required, mg-min/L	64
T_{10}/DT ratio	0.6
Min. volume/basin, MG	0.37

Table A-1 (continued)
Preliminary Design Criteria for Conventional Treatment

Parameter	Value
<u>Post-Filtration Stabilization</u>	
Lime mix chamber	
Detention time, seconds	30
Mixing velocity gradient ("G"), sec^{-1}	300
CO_2 diffusion chamber	
Detention time, minutes	2
<u>Washwater Recovery</u>	
Surge basin capacity, gallons	200,000
Plate settler loading, gpm/sq ft	0.5
<u>Sludge Lagoons</u>	
Projected solids production, lbs/MG	141
No. of cells	3
Max. settled sludge depth, ft	5
Side slopes	1:3
Solids storage capacity per cell at 5% sludge concentration, years	2
Req'd storage volume per cell, cu ft	293,000
Approx. total lagoon site area req'd, acres	6.5
<u>Chemical Feed Systems</u>	
Average / maximum dosages, mg/L	
Alum	20 / 50
Coagulant aid polymer	1.0 / 5.0
Filter aid polymer	0.05 / 0.10
Sodium hydroxide	3.5 / 10
Powdered activated carbon	3 / 20
Hydrated Lime	33 / 40
Carbon dioxide	35.5 / 40
Sodium hypochlorite (as available chlorine)	4 / 10
Hydrofluosilicic acid (as F ion added)	0.8 / 1.0

*Assumes min. 4-log Giardia removal/inactivation req'd by DHS; plant receives 2.5-log physical removal credit for conventional treatment.

Table A-2
Preliminary Design Criteria for Microfiltration Treatment

Parameter	Value
<u>Carbon Contact Basin</u>	
No. of basins	2
No. of cells per basin	2
Total detention time, minutes	20
Mixing velocity gradient ("G"), sec^{-1}	50
Total basin volume, MG	0.14
<u>Microfiltration System</u>[1]	
No. of trains	5
Capacity per train, mgd	2.0
Microfilter units per train	2
Microfilter modules per unit	90
Area per module, m^2	15
Reference microfilter module	Memtec 90M10C
Backwash interval, minutes	25
Backwash duration, minutes	2.5
Total backwash flow, mgd	0.59
Total feedwater requirement, mgd	10.59
<u>Chlorine Contact Basins</u>	
No. of basins	2
Design flow per basin, mgd	5.0
Log *Giardia* inactivation req'd[2]	1.0
Min. water temperature, degrees C	8
Max. pH	7.0
Max. free chlorine residual, mg/L	1.0
CT required, mg-min/L	42
T_{10}/DT ratio	0.6
Min. volume/basin, MG	0.25
<u>Post-Filtration Stabilization</u>	
Lime mix chamber	
Detention time, seconds	30
Mixing velocity gradient ("G"), sec^{-1}	300
CO_2 diffusion chamber	
Detention time, minutes	2

[1]Design parameters shown are considered preliminary, and must be confirmed through pilot testing prior to design.
[2]Assumes min. 4-log Giardia removal/inactivation req'd by DHS; plant receives 3.0-log physical removal credit for microfiltration treatment.

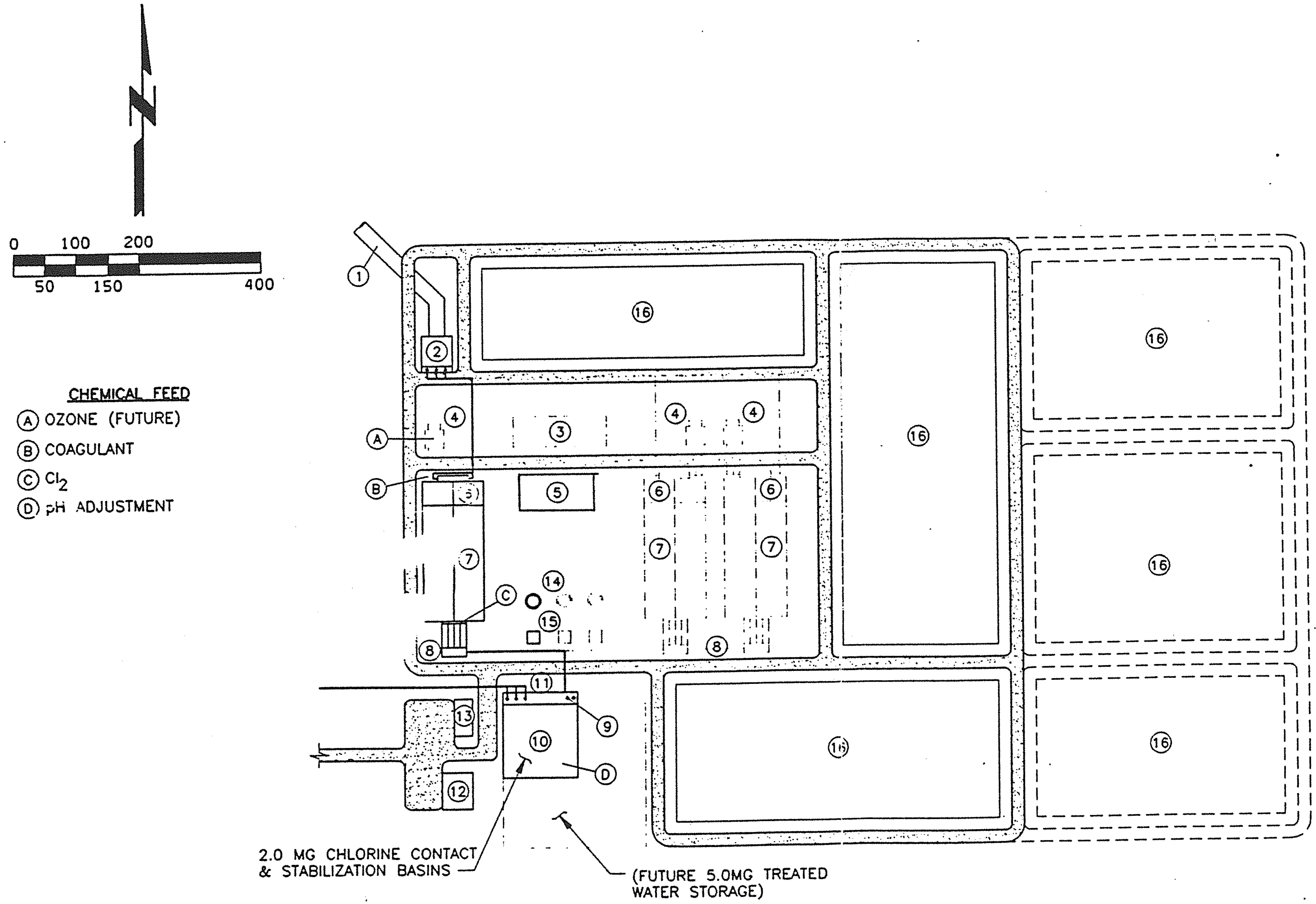
Table A-2 (continued)
Preliminary Design Criteria for Microfiltration Treatment

Parameter	Value
<u>Sludge Lagoons</u>	
Projected solids production, lbs/MG	60
No. of cells	3
Max. settled sludge depth, ft	5
Side slopes	1:3
Solids storage capacity per cell at 5% sludge concentration, years	2
Req'd storage volume per cell, cu ft	125,000
Approx. total lagoon site area req'd, acres	3.5 - 4.0
<u>Chemical Feed Systems</u>	
Average / maximum dosages, mg/L	
Powdered activated carbon	3 / 20
Hydrated Lime	33 / 40
Carbon dioxide	35.5 / 40
Sodium hypochlorite (as available chlorine)	4 / 10
Hydrofluosilicic acid (as F ion added)	0.8 / 1.0

Table A-3
Preliminary Design Criteria for Contact Adsorption Clarification

Parameter	Value
<u>Carbon Contact Basin</u>	
No. of basins	2
No. of cells per basin	2
Total detention time, minutes	20
Mixing velocity gradient ("G"), sec^{-1}	50
Total basin volume, MG	0.14
<u>CAC System</u>	
No. of trains	4
Capacity per train, mgd	2.5
Hydraulic loading, gpm/sq ft	10
Average CAC run time, hours	6 - 8
Contactor BW requirement, gals/sq ft per BW	40
<u>Filtration</u>	
No. of filters	4
Hydraulic loading rate	
With 4 filters in service, gpm/sq ft	4.5
With 3 filters in service, gpm/sq ft	6.0
Media	
Anthracite (1.5 mm ES), inches	30
Fine sand (0.45 - 0.55 mm ES), inches	10 - 12
Backwash	Upflow water w/air scour
Max. backwash rate, gpm/sq ft	20
<u>Chlorine Contact Basins</u>	
No. of basins	2
Design flow per basin, mgd	5.0
Log *Giardia* inactivation req'd*	1.5
Min. water temperature, degrees C	8
Max. pH	7.0
Max. free chlorine residual, mg/L	1.0
CT required, mg-min/L	64
T_{10}/DT ratio	0.6
Min. volume/basin, MG	0.37

*Assumes min. 4-log Giardia removal/inactivation req'd by DHS; plant receives 1.5-log physical removal credit for CAC/filtration treatment.



FACILITIES

1. RAW WATER SUPPLY
2. RAW WATER PUMP STATION
3. OZONE GENERATION BUILDING (FUTURE)
4. OZONE CONTACT BASIN (FUTURE)
5. CHEMICAL STORAGE AREA
6. COAGULATION/FLOCCULATION BASINS
7. SEDIMENTATION BASINS (FUTURE)
8. FILTERS
9. STABILIZATION
10. Cl_2 CONTACT & STABILIZATION
11. TREATED WATER PUMP STATION
12. ADMINISTRATION/OPERATIONS
13. MAINTENANCE
14. WASH WATER RECOVERY
15. WASH WATER TREATMENT
16. SLUDGE DRYING BEDS

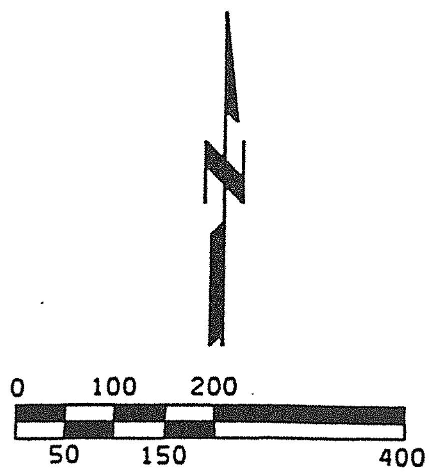
TOTAL SITE SIZE= 35 ACRES

TREATMENT PLANT= 10 ACRES

SOLIDS HANDLING= 25 ACRES

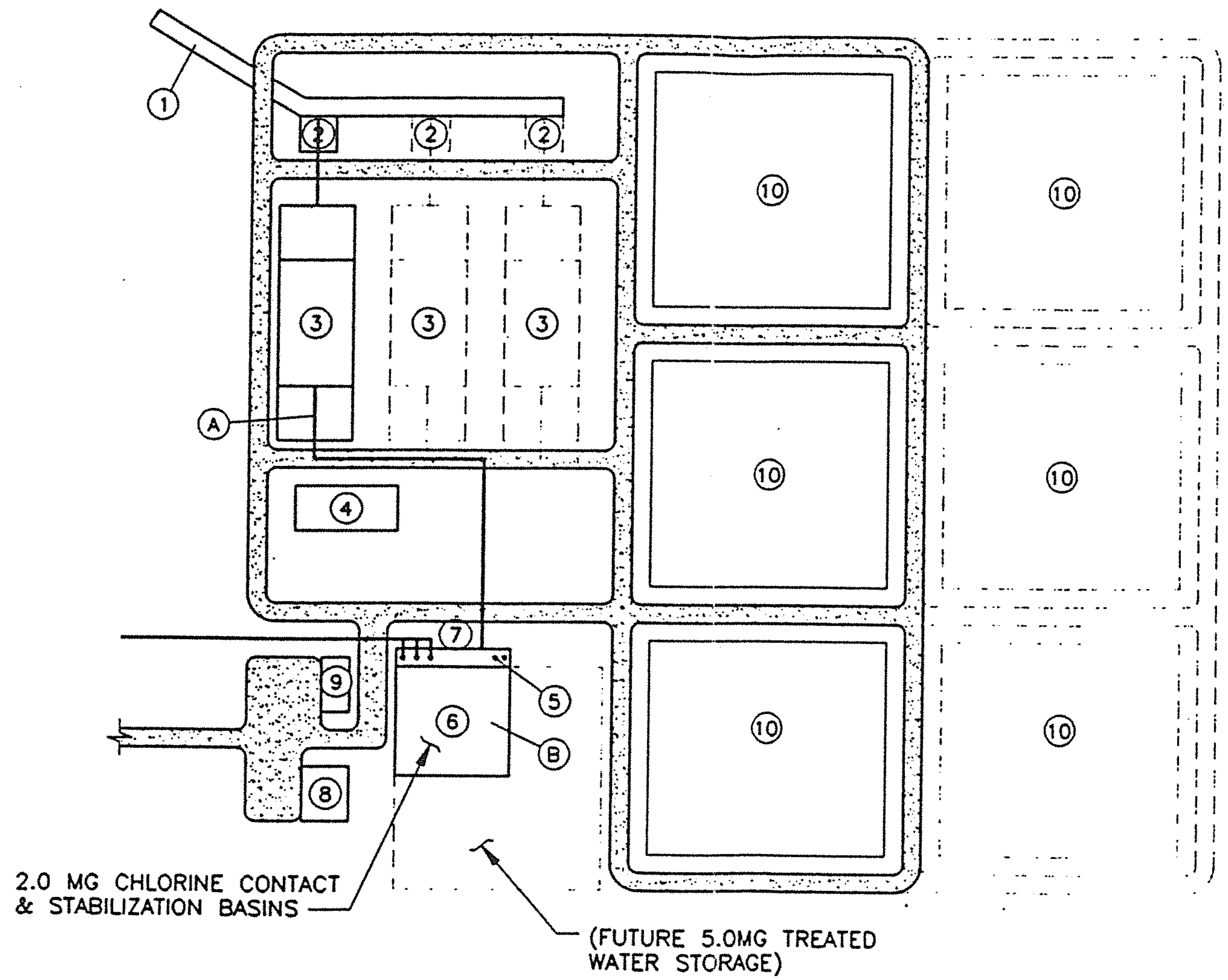
CONCEPTUAL SITE LAYOUT
CONVENTIONAL PROCESS ALTERNATIVE

34404-104-000-FIG1



CHEMICAL FEED

- (A) Cl_2
- (B) ph ADJUSTMENT



FACILITIES

1. RAW WATER SUPPLY
2. CARBON CONTACT
3. MICROFILTRATION
4. CHEMICAL STORAGE AREA
5. STABILIZATION
6. Cl_2 CONTACT & STABILIZATION
7. TREATED WATER PUMP STATION
8. ADMINISTRATION/OPERATIONS
9. MAINTENANCE
10. REJECT HOLDING POND

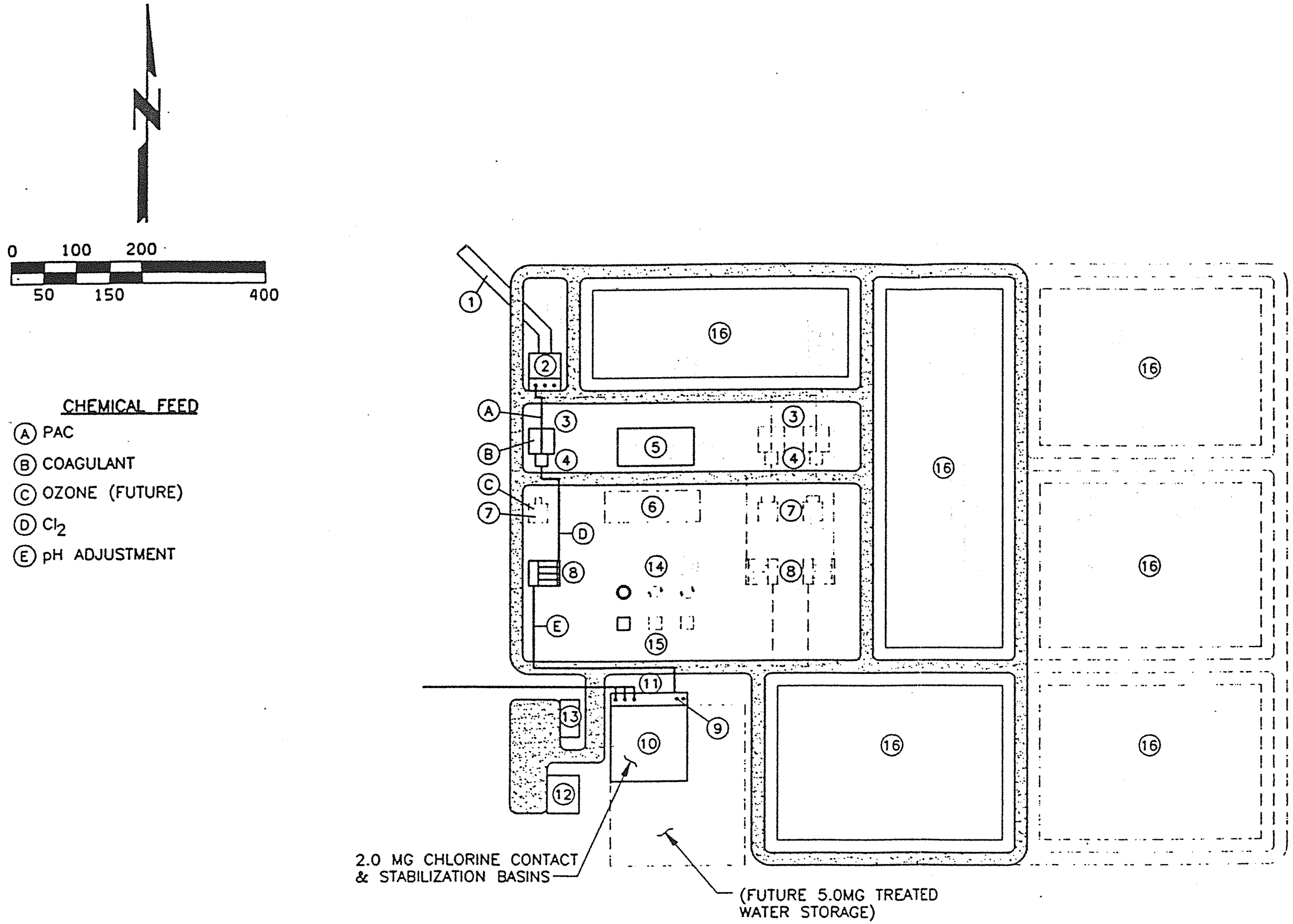
TOTAL SITE SIZE= 25 ACRES

TREATMENT PLANT= 10 ACRES

SOLIDS HANDLING= 15 ACRES

CONCEPTUAL SITE LAYOUT
MICROFILTRATION ALTERNATIVE

34404-104-000-FIG2



CHEMICAL FEED

- (A) PAC
- (B) COAGULANT
- (C) OZONE (FUTURE)
- (D) Cl_2
- (E) pH ADJUSTMENT

FACILITIES

1. RAW WATER SUPPLY
2. RAW WATER PUMP STATION
3. CARBON CONTACT
4. CONTACT ABSORPTION CLARIFIER
5. CHEMICAL STORAGE AREA
6. OZONE GENERATION BUILDING (FUTURE)
7. OZONE CONTACT BASIN (FUTURE)
8. FILTERS
9. STABILIZATION
10. Cl_2 CONTACT & STABILIZATION
11. TREATED WATER PUMP STATION
12. ADMINISTRATION/OPERATIONS
13. MAINTENANCE
14. WASH WATER RECOVERY
15. WASH WATER TREATMENT
16. SLUDGE DRYING BEDS

TOTAL SITE SIZE= 30 ACRES
TREATMENT PLANT= 11 ACRES
SOLIDS HANDLING= 19 ACRES

CONCEPTUAL SITE LAYOUT
CONTACT ADSORPTION CLARIFIER ALTERNATIVE