VACAVILLE

City of Vacaville

1990 WATER SYSTEM MASTER PLAN

Prepared by:

NOLTE and ASSOCIATES

Sacramento

May 22, 1990
2008-84-03

Mr. David Tompkins
Assistant Director of Public Works
City of Vacaville
650 Merchant Street
Vacaville, CA 95688

SUBJECT: 1990 WATER SYSTEM MASTER PLAN

Dear Dave:

Transmitted herewith is our 1990 Water System Master Plan. As you are aware, the City is currently undergoing a revision to the General Plan and such revision could substantially impact the Master Plan. However, the General Plan work will not be complete for some time, and we feel that it is very important to prepare this report and provide the City with a current working document.

After the General Plan work is complete we would suggest revision to the Water System Master Plan. Revisions should include modification of the study area boundary to conform with the new General Plan, updating the water demand projections for the service area, and updating the capital improvements program. At this time it would also be appropriate to update the computer model.

If you have any questions on the Master Plan, do not hesitate to call.

Very truly yours,

NOLTE AND ASSOCIATES

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1990 DRAFT

WATER SYSTEM MASTER PLAN

May 1990

Prepared for

CITY OF VACAVILLE

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TABLE OF CONTENTS

Section		Page
1.	INTRODUCTION	1-1
	A. BACKGROUND	1-1
	B. SCOPE OF WORK	1-1
	C. OVERVIEW OF THE WATER SYSTEM	1-2
2.	STUDY AREA	2-1
	A. STUDY AREA BOUNDARIES	2-1
	B. LAND USE	2-1
	C. TOPOGRAPHY	2-5
	D. CLIMATE	2-5
3.	WATER DEMANDS AND WATER SUPPLY	3-1
	A. EXISTING WATER DEMANDS	3-1
	B. PROJECTED WATER DEMANDS	3-3
	C. GROUND WATER SUPPLY	3-6
	D. SURFACE WATER SUPPLIES	3-10
	E. SUPPLY DEMAND ANALYSIS	3-12
4.	WATER SYSTEM DESIGN CRITERIA	4-1
	A. UPPER AND LOWER ELEVATION LIMITS OF A PRESSURE ZONE	4-1
	B. MINIMUM CONDITIONS FOR FIRE PROTECTION PLANNING	4-2
	C. DISTRIBUTION SYSTEM PIPELINE SIZING	4-3
	D. DISTRIBUTION SYSTEM STORAGE	4-4
	E. PUMPING FACILITIES	4-5
	F. ALTERNATE CRITERIA FOR SECONDARY ZONES	4-6
	G. PIPELINE MATERIALS OF CONSTRUCTION	4-7
	H. MISCELLANEOUS CRITERIA	4-7
5.	TREATMENT FACILITIES	5-1
	A. DIATOMACEOUS EARTH TREATMENT PLANT AND BOOSTER PUMPING STATION	5-1
	B. RECOMMENDED IMPROVEMENTS AT THE DE PLANT	5-4
	C. WATER SYSTEM SCADA FACILITIES	5-7
	D. NORTH BAY REGIONAL WATER TREATMENT PLANT	5-8
	E. RECOMMENDED ADDITIONAL TREATMENT CAPACITY	5-8
6.	DISTRIBUTION SYSTEM STORAGE	6-1
	A. EXISTING FACILITIES (AND FACILITIES UNDER CONSTRUCTION)	6-1
	B. ZONE I RECOMMENDATIONS	6-2

TABLE OF CONTENTS
(Continued)

Section	Page
7. **DISTRIBUTION SYSTEM**	7-1
A. EXISTING FACILITIES	7-1
B. DEVELOPMENT AND APPLICATION OF THE HYDRAULIC MODEL	7-4
C. DISTRIBUTION SYSTEM ANALYSIS PROCEDURES	7-6
D. ANALYSIS OF THE DISTRIBUTION SYSTEM	7-7
E. RECOMMENDED TRANSMISSION IMPROVEMENTS	7-10
F. RECOMMENDED LOCAL IMPROVEMENTS	7-12
G. RECOMMENDED DOWNTOWN IMPROVEMENTS	7-13
H. WELL FIELD PIPING	7-14
8. **HIGHER ELEVATION PRESSURE ZONES**	8-1
A. GENERAL CONSIDERATIONS FOR HIGHER ELEVATION ZONES	8-1
B. VINE STREET WATER SYSTEM	8-1
C. HIDDEN VALLEY WATER SYSTEM	8-5
D. WYKOFF WATER SYSTEM	8-7
E. PROPOSED ZONE 2 SYSTEM	8-10
F. TRANQUILITY LANE SYSTEM	8-12
9. **CAPITAL IMPROVEMENTS PROGRAM**	9-1
REFERENCES	
APPENDICES	
A. GROUNDWATER RESOURCES OF THE VACAVILLE AREA	
B. URBAN WATER MANAGEMENT	
C. NOTES ON CAPITAL IMPROVEMENT COSTS	

LIST OF TABLES

TABLE		PAGE
1-1	Existing Water Supply Facilities in Vacaville	1-4
1-2	Existing Water Storage Reservoirs in Vacaville	1-5
1-3	Existing Pumping Facilities (excluding wells) in Vacaville	1-6
2-1	Land Uses in Vacaville	2-3
3-1	Monthly Water Production in Vacaville In Acre-Feet	3-2
3-2	Maximum Day Peaking Factors in Vacaville	3-3
3-3	Projected Development in Vacaville	3-4
3-4	Actual and Projected Water Demands	3-5
3-5	Water Production History (AF/YR)	3-8
3-6	Available Future Water Entitlements	3-11
3-7	Summary of Annual Water Supplies and Demands	3-16
4-1	Fire Flows and Durations	4-3
6-1	Distribution System Storage Facilities	6-3
6-2	Required Storage Volume in the Zone 1 Distribution System	6-5
7-1	Distribution System Performance Guidelines	7-8
9-1	Summary of Capital Improvements	9-3
9-2	Summary of "Local" Pipeline Capital Improvements	9-7
9-3	Summary of Capital Improvements at the DE Water Treatment Plant	9-14

LIST OF FIGURES

FIGURES		PAGE
1-1	Water System Vertical Schematic	1-7
2-1	Study Area Boundary	2-2
3-1	Well Locations	3-7
3-2	Maximum Day Water Demand Versus Production Capacity	3-14
3-3	Annual Water Demand Versus Water Supply	3-15
5-1	Booster Pumping Facilities at the DE Treatment Plant	5-3
7-1	Distribution System Functional Areas	7-2
8-1	Planned Vine Street Service Area	8-3
8-2	Planned Vine Street Water System Schematic	8-4
8-3	Existing and Recommended Water Facilities in Northwest Vacaville	8-8

ABBREVIATIONS

mgd	million gallons per day
MG	million gallons
AF	acre feet
gpm	gallons per minute
gpd	gallon per day
DE	diatomaceous earth
SID	Solano Irrigation District
NBA	North Bay Aqueduct
NBR	North Bay Regional
psi	pounds per square inch
NTU	nephelometric turbidity unit
SCWA	Solano County Water Agency

1. INTRODUCTION

A. BACKGROUND

In 1986, Nolte and Associates prepared a Draft Water System Master Plan for the City of Vacaville, but a final version was not prepared and the draft has continued to serve as a guide for the City in planning for water system improvements. Between 1986 and 1990 substantial improvements have been made to the water system, and development in the City has proceeded at a moderate to fast pace. Current development trends indicate that substantial rates of growth may be sustained in the future which will necessitate continued expansion of water system facilities and water supplies.

Because of changes since 1986 and anticipated future growth, Nolte and Associates has been retained to update the 1986 Draft and prepare this new version of the master plan.

B. SCOPE OF WORK

The scope of work is intended to update and expand the 1986 Draft but not to reevaluate basic distribution system analyses in the original report. For this reason the description of our scope of work provided below includes the scope for the 1986 Draft and the additional scope for the current version.

1. Scope of Work for 1986 Draft

Initial tasks in the 1986 Draft included determination of the study area boundaries and existing land use and evaluation of the water demands and water supply. Existing water demands were estimated from land use and typical water demand factors for each type of land use and were compared with actual city records for verification. Future demands were estimated based on projected development as predicted by the City planning staff. Water supply was evaluated in terms of the ground water aquifer characteristics, the well field production capacities, and the sources and quantity of available surface water. Once defined, the available supply and future demands were compared to ascertain possible deficiencies in the quantity of water available to Vacaville.

Water treatment facilities were addressed in terms of treatment capacity and management strategies relative to supply and demand. Consideration was not given to identifying specific unit processes needs within the treatment facilities.

A major focus of the 1986 Draft was on evaluation of the distribution system including pumping facilities, the pipe network and storage reservoirs. Each of these system components was analyzed in terms of their existing and future needs relative to the projected water demands. To aid in this analysis, a computer model was employed which simulates distribution system operation. The model was used under both existing and future demand conditions to evaluate the system deficiencies.

Various alternative improvements to the water system were then tested using the model to determine their effectiveness.

Storage reservoirs were evaluated in terms of their ability to meet fire, emergency and operational needs. Required storage volumes for future levels of water demands were determined, and alternate sites for new reservoirs were identified.

Based on the analysis of existing and future deficiencies, a capital improvement program was developed. For each improvement, an estimated capital cost, and a recommended time of construction were identified.

2. Scope of Work for the 1990 Master Plan

A significant portion of the work for the 1990 Master Plan includes updating the capital improvements program in the 1986 Draft. Since completion of the 1986 Draft, several of the recommended improvements have been completed including the Ulatis Drive well (No. 3), the Fallen Leaf Drive Well (No. 9), and major pipelines on Leisure Town Road, Elmira Road, Peabody Road and in the downtown area. Additionally, several other major projects are in the design and construction stages.

Updating costs is another important aspect of the 1990 Master Plan. Many of the recent construction projects have provided a solid basis for estimating costs which are now reflected in the capital improvements program.

The scope of work for the 1990 Master Plan also includes updating water demand projections and the supply versus demand analysis. This effort is required as a result of growth rates which have exceeded 1986 projections and as a result of finite water supplies.

Other tasks for the 1990 Master Plan include 1) an update of the groundwater report prepared by Dr. John Mann, 2) addition of a section in the report addressing water system design criteria, and 3) addition of discussion on the elevated pressure zones in the City.

C. OVERVIEW OF THE WATER SYSTEM

The Vacaville water system consists of surface water treatment facilities, wells, pumping facilities, distribution and transmission pipelines and storage reservoirs. A list of the existing facilities making up the water system is provided in Tables 1-1 to 1-3. A schematic representation of the water system is shown in Figure 1-1. This figure illustrates the functional components and elevation relationships of the facilities. The locations of the facilities in the City are shown on Map 1, contained in the back pocket of this report.

Surface water from Putah South Canal is provided by contract between the U.S. Bureau of Reclamation and the Solano County Water Agency and delivered by the Solano Irrigation District. This water is currently treated at a 10 mgd (million gallons per day) diatomaceous earth filter treatment plant (DE Plant), and the treated water empties into a 1 MG (million gallon) ground level reservoir as shown in Figure 1-1. Wells 1, 4 and 6 also supply water directly to this 1 MG reservoir. From the reservoir a booster pump station lifts the water into the Zone 1 distribution system. The remaining wells (2, 3, 5, 7, 8 and 9) supply water directly to the Zone 1 distribution system.

In 1990, the North Bay Regional (NBR) water treatment plant is expected to come on line with a capacity of 13.3 mgd for Vacaville. This water plant will supply water directly to the Zone 1 distribution system. The water supply source for the NBR Plant will be the North Bay Aqueduct and Putah South Canal. Vacaville will treat entitlements from both sources at the NBR plant.

The Zone 1 distribution system serves most of the demands in Vacaville, including all development up to an elevation of 220 ft and a few isolated areas at slightly higher elevations. The Zone 1 system incorporates two reservoirs on Butcher Road and one reservoir on Buck Street for a combined volume of 8.0 MG. In 1990 a new 5.0 MG reservoir will be completed near Vaca Valley Parkway, referred to as the Browns Valley Reservoir.

In the northwest sector of Vacaville, there are several small, higher elevation pressure zones. In each case, a booster pump station draws water from the Zone 1 distribution system and pumps it up to an elevated reservoir as illustrated in Figure 1-1. The Tranquility Lane service area, which is relatively small, is supplied from the Wykoff reservoir. Tranquility Lane facilities include a small pump station and hydropneumatic tank.

TABLE 1-1

EXISTING WATER SUPPLY FACILITIES

WATER SUPPLY

Item	Approx. Capacity (gpm / mgd)	Service Location
DE Filter Treatment Plant	6,940/10.0	To ground level clearwell at plant site which is pumped into the main distribution system
Well No. 1	230/ 0.33	To ground level clearwell at DE plant site
Well No. 2	1,100/ 1.58	To the main distribution system
Well No. 3	1,470/ 2.12	To the main distribution system
Well No. 4	1,340/ 1.93	To ground level clearwell at DE plant site
Well No. 5	1,300/ 1.87	To the main distribution system
Well No. 6	1,140/ 1.64	To ground level clearwell at DE plant site
Well No. 7	1,110/ 1.60	To the main distribution system
Well No. 8	1,520/ 2.19	To the main distribution system
Well No. 9	1,400/ 2.02	To the main distribution system
SUBTOTAL - WELLS	10,610/15.28	
TOTAL	17,550/ 25.28	

TABLE 1-2

EXISTING WATER STORAGE RESERVOIRS

Item	Capacity (Gallons)	Type of Service	Service Location
DE Plant Clearwell	1,000,000	Ground level	Pumped into the main distribution system
Butcher No. 1	2,000,000	Elevated	Main distribution system
Butcher No. 2	4,000,000	Elevated	Main distribution system
Buck	2,000,000	Elevated	Main distribution system
Browns Valley (1990)	5,000,000	Elevated	Main distribution system
Wykoff	120,000	Elevated	Wykoff distribution system
Hidden Valley	73,000	Elevated	Hidden Valley distribution system
PLANNED IN 1990:			
Vine	620,000	Elevated	Vine distribution system
PLANNED TO BE ABANDONED IN 1990:			
Vine No. 1	374,000	Ground level	Out of service, but would serve Vine Street
Vine No. 2	56,000	Elevated	Vine distribution system

TABLE 1-3

EXISTING PUMPING FACILITIES (EXCLUDING WELLS)

Item	Approx. Capacity (gpm)	Service Location
DE Plant Booster Station	10,800[1]	From DE Plant reservoir to main distribution system
Wykoff Pump Station	2 pumps at 500 gpm ea[2]	From main distribution system to Wykoff distribution system
Tranquility Lane Pump Station	2 pumps at 140 gpm ea	From Wykoff reservoir to Tranquility Lane service area
Hidden Valley Pump Station	2 pumps at 520 gpm ea	From main distribution system to Hidden Valley distribution system.
PLANNED IN 1990:		
Vine Pump Station	2 pumps at 580 gpm ea	From main distribution system to Vine distribution system
PLANNED TO BE ABANDONED IN 1990:		
Vine Pump Station	2 pumps at 125 gpm ea[3]	From main distribution system to Vine distribution system

1. Total, not firm capacity. Based on 5 existing centrifugal pumps plus 1 existing vertical pump, and a system pressure of 98 psi.
2. 500 gpm is design point on pump curve, but actual conditions may vary.
3. Pump curves not available.

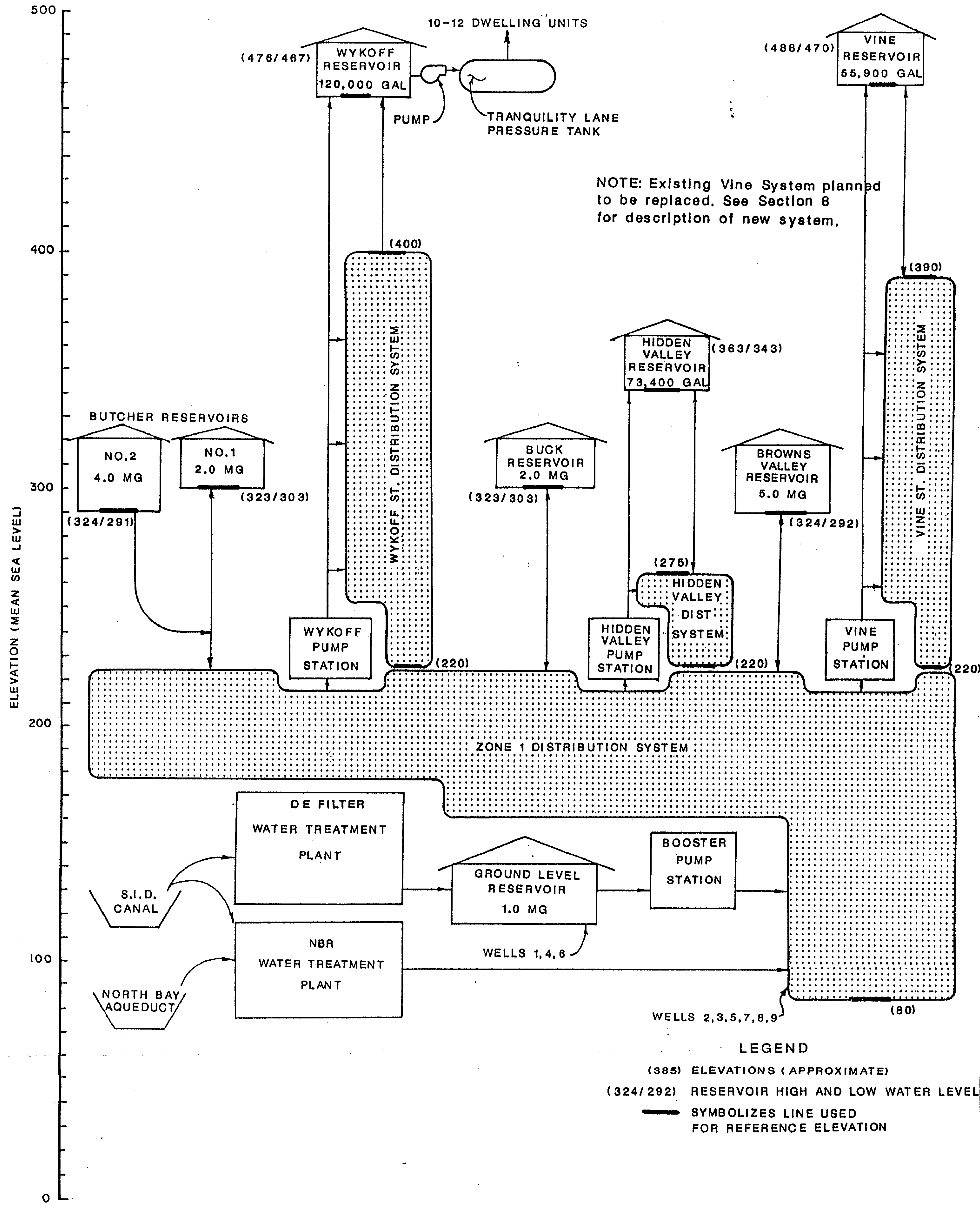


FIGURE 1-1

VACAVILLE WATER SYSTEM
VERTICAL SCHEMATIC

2. STUDY AREA

A. STUDY AREA BOUNDARIES

The study area boundary is shown in Figure 2-1. The boundary is based on the existing city limits plus some additional areas with future development potential.

In the southeast portion of Vacaville, additional land outside the city limits but within the study area boundary includes the Cooper School area (west of Leisure Town road), a large parcel east of Leisure Town Road and south of Elmira Road, and another large parcel south of Alamo Drive and east of Peabody Road. Most of this land is assumed to be developed as low density residential.

In the northeast, a parcel outside the city limits approximately 880 acres in size (Chevron property) east of Interstate 5 and north of Vaca Valley Parkway is also included in the study area. This land is planned for industrial facilities.

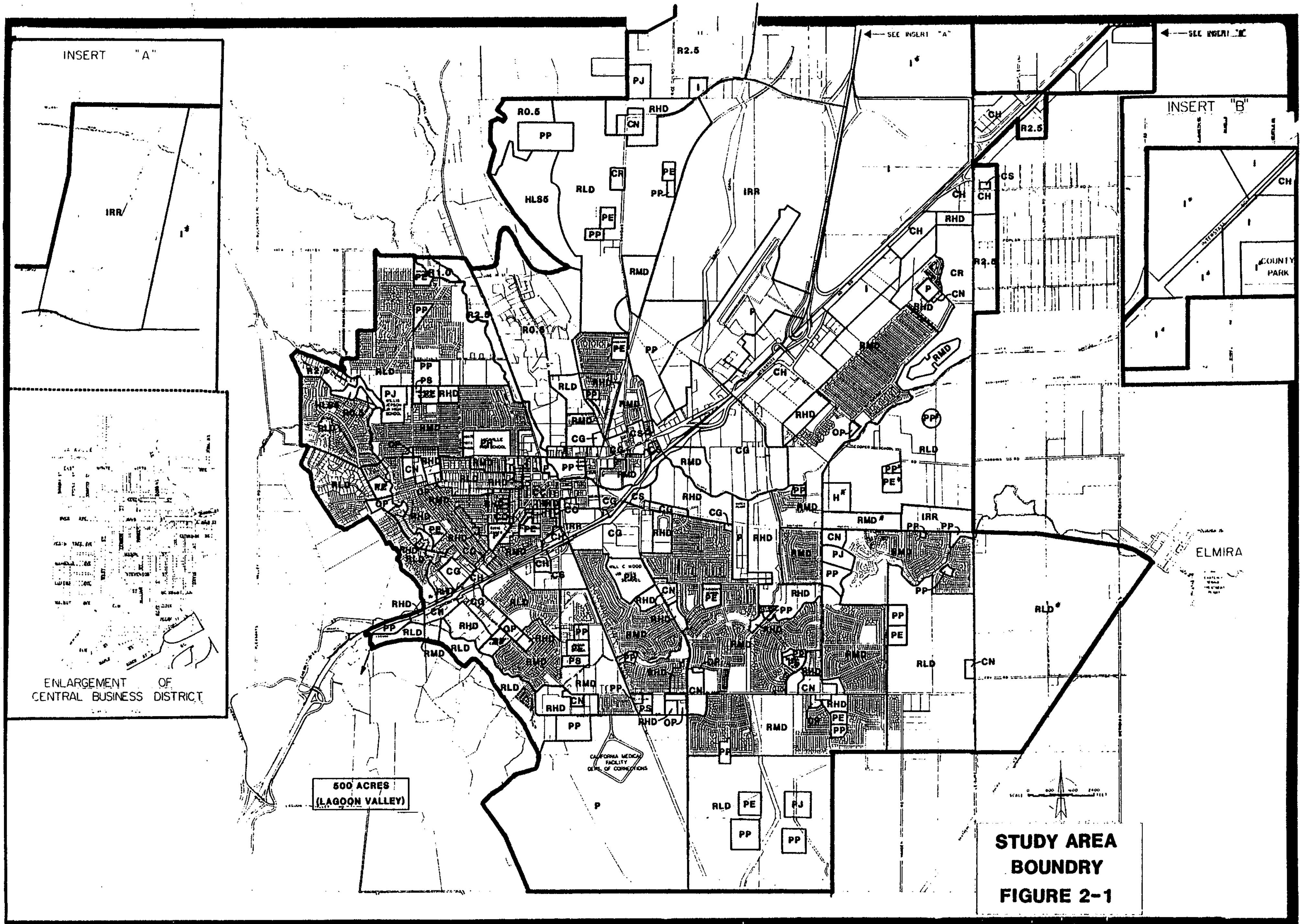
Lagoon Valley, to the southwest of Vacaville, has been included in the water demand projections. However, facilities to serve this area have not been planned.

B. LAND USE

Land uses in Vacaville are summarized in Table 2-1. The table includes the estimated number of total acres in each category of use. Total acres were determined by planimeter and are therefore approximate. Also shown in Table 2-1 are the water demand factors for each type of land use. The water demand factors are discussed in more detail in Section 3.

Vacaville contains significant portions of residential, commercial and industrial development. Residential development is spread throughout the city. Most of the industrial development is centered north of I-80 and east of Browns Valley Road while commercial development predominates near the downtown area and along the I-80 corridor. In general, public and governmental facilities are less significant in terms of total water demand, with the exception of the Correctional Medical Facility (CMF). CMF is located south of Alamo Drive and west of Peabody Road.

A land use map has been prepared for this study and is contained in the back pocket of this report. The land use map was developed from General Plan land uses and recommendations of the city staff. The primary purposes of the land use map are to serve as a basis for rational generation of water demands throughout the city, and to illustrate development trends.



STUDY AREA BOUNDRY
FIGURE 2-1

TABLE 2-1

LAND USES IN VACAVILLE

Land Use		Estimated Area (acres)	Water Demand Factor[2] (gpd/ac)
RESIDENTIAL LAND USES			
R-2.5	Rural Residential - 2.5 acre minimum	473	600
R-1	Rural Residential - 1 acre minimum	16	300
R-0.5	Rural Residential - 0.5 acre average	475	1,000
RLD	Low Density - 3 to 5 dwellings per acre	3,748	1,500
RMD	Medium Density - 6 to 12 dwellings per acre	2,970	2,300
RHD	High Density - 12 to 24 dwellings per acre	751	3,400
COMMERCIAL LAND USES			
CC	Central Commercial	83	1,400
CG	General Commercial	570	1,400
CO	Office Commercial	26	1,400
CN	Neighborhood Commercial	113	1,400
CS	Commercial Service	80	1,400
CH	Highway Commercial	498	1,400
CR	Recreation Commercial	170	1,400
OP	Office Professional	28	1,400
CL	Lagoon Valley[1]	500	1,400
INDUSTRIAL LAND USES			
I	Industrial	2,560	1,500
IRR	Industrial - Rail Restricted	1,882	1,500

(Continued)

TABLE 2-1

LAND USES IN VACAVILLE
(Continued)

Land Use		Estimated Area (acres)	Water Demand Factor (gpd/ac)
PUBLIC LAND USES			
PE	Elementary School	152	1,000
PJ	Junior High School	107	1,000
PH	High School	81	1,000
PS	Other Schools	20	1,000
P	Other Public Land Uses	1,758	500
H	Hospital[1]	25	3,000
OPEN SPACE LAND USES			
PP	Parks and Recreation Centers	485	300
HLS5	Hazardous Land	171	0

[1] Not a general land use designation.
[2] Water demand factors are calibrated with actual conditions in 1984, and therefore do not incorporate a factor of safety.

C. TOPOGRAPHY

The topography in the eastern portion of Vacaville is generally flat. Ground surface elevations on the east boundary typically range from 80 to 90 ft, and the terrain gently slopes up toward the west where it reaches about 150 ft near the center of town. On the west side of town, the English Hills dominate the landscape and elevations rise considerably in that direction. In the main pressure zone (Zone 1), the maximum building pad elevations are generally near 220 ft, but some reach 230 ft. In higher pressure zones, some building pads are over 400 ft. The topography and general terrain in Vacaville are very important as they directly influence the pressures in water mains throughout the City.

D. CLIMATE

The climate in Vacaville is characterized by mild winters and hot summers. Almost all of the precipitation occurs during the late fall, winter and early spring. Temperatures during the winter usually drop into the forties at night and occasionally drop below the freezing point. Snow is rare. In the summer, temperatures occasionally rise above 100°. The days are typically hottest between four and five p.m. and temperatures cool off noticeably in the evenings.

The climate has significant influence on the water demands in Vacaville, as the winters are characterized by relatively low demands, while the summers have substantially higher demands. Lawn watering in the summer is a major contributor to the higher summer demands.

3. WATER DEMANDS AND WATER SUPPLY

A. EXISTING WATER DEMANDS

Water use in Vacaville has increased rapidly in the last decade in accordance with rapid population growth during this period.

Residential, commercial and industrial developments are the primary water users in Vacaville, but schools and other public land facilities are also significant consumers. The type of development is an important factor influencing the magnitude of the water demands. High density residential developments typically have relatively low per capita water use, while low density developments have substantially higher per capita use as a result of greater lawn watering and increased number of appliances in each home. Commercial and industrial water use can be much greater or much less than typical residential demands as they are highly dependent on the type of business. Industrial demands can be substantial for food processing and other "wet" industries but can be minor for activities such as warehousing.

In addition to the type of development, factors such as water conservation practices, climate, and water pressure significantly affect the demands in Vacaville. Water conservation practices are discussed in detail in the water conservation plan prepared for Vacaville in 1986. The variable climate in Vacaville is responsible for low winter and high summer demands. High water pressure in some parts of the City is expected to cause high water use because under increased pressure conditions, the flow rate from faucets, showers, sprinklers and pipe leaks increases. For example, the water use rate has been known to increase by as much as 30% for a 20 psi change in line pressure (Clark, Viessman and Hammer, 1977).

Important characteristics of water demands include annual, monthly, average day, maximum day and peak hour flow. In Table 3-1, monthly water production records for 1985, 1986, 1987, 1988 and 1989 are presented. It is apparent that monthly demands vary significantly from the winter to the summer. Typically, the maximum month will have a demand which is 50% greater than the annual average.

Table 3-1 also shows annual average water production rates in terms of mgd. It is apparent that annual average water demand has steadily increased in recent years. Between 1982 and 1989 annual average demands have increased from 8.34 mgd to 11.94 mgd which corresponds to an increase of nearly 5.3%/yr.

TABLE 3-1

MONTHLY WATER PRODUCTION

IN ACRE-FEET[1,2]

Month	1985	1986	1987	1988	1989
January	561	571	646	710	660
February	548	516	602	764	615
March	620	676	735	956	675
April	850	862	1,092	985	969
May	1,046	1,255	1,371	1,160	1,393
June	1,358	1,416	1,528	1,359	1,614
July	1,526	1,600	1,610	1,696	1,815
August	1,455	1,619	1,580	1,573	1,747
September	1,059	1,080	1,336	1,400	1,279
October	919	1,045	1,074	1,088	987
November	661	886	694	683	857
December	608	665	700	708	771
Total (Acre-Feet)	11,212	12,191	12,968	13,083	13,382
Total (mgd)	10.01	10.88	11.58	11.68	11.94

[1] Production in the years 1982, 1983, and 1984 were 8.34 mgd, 8.72 mgd and 10.00 mgd. respectively.

[2] City of Vacaville Production Records

Maximum day demands are critical in sizing certain water system facilities. Table 3-2 presents maximum day peaking factors based on the historical record of water use in Vacaville. These peaking factors are defined as the ratio of maximum day demands to average day demands. In Table 3-2 it is evident that the peaking factor is typically under 2.0. For planning and design, a daily peaking factor of 2.0 is recommended.

TABLE 3-2

MAXIMUM DAY PEAKING FACTORS IN VACAVILLE

Year	Average Day Flow mgd	Average Day Flow %increase	Maximum Day Flow, mgd	Peaking Factor
1984	10.00	-	18.60	1.86
1985	10.01	0.1	16.77	1.67
1986	10.88	8.7	18.44	1.69
1987	11.58	6.4	20.14	1.69
1988	11.68	0.9	20.37	1.74
1989	11.94	2.2	22.38	1.87

Peak hour demands are also important in sizing certain facilities. However, less data is available for determination of peak hourly flow than for peak day flow. In 1984 peak hourly flow was estimated from the maximum rate of flow from the storage reservoirs plus the flow from all supply pumps to the system. The maximum rate of reservoir depletion was estimated from the slope on continuous depth recording charts at the reservoirs. From this analysis, the peak hour flow in 1984 was estimated as 35 mgd, which corresponds to an hourly peaking factor of 3.5. Herein, an hourly peaking factor of 4.0 is recommended for design and analysis (4 times average day flow).

B. PROJECTED WATER DEMANDS

Water consumption through the year 2010 is projected based on anticipated development in Vacaville. Development assumptions are given in Table 3-3 and projections are given in Table 3-4. The number of single and multiple family homes and the square footage of commercial and industrial developments are based on projections prepared by city staff. Unit water demands for each type of development are based on the following rationale:

TABLE 3-3

PROJECTED DEVELOPMENT IN VACAVILLE

Growth Scenario	Residential: Single Family Units	Residential: Multiple Family Units	Industrial ($1,000\ ft^2$)	Commercial ($1,000\ ft^2$)
Low	700	200	150	150
Moderate	950	300	240	240
High	1,200	400	340	500

Single Family Dwelling Units: Records of actual water use by single family homes in Vacaville in 1984 indicate a per dwelling unit demand of 413 gpd, so for master planning work a value of 420 gpd is used. (More recent checks on the 420 gpd value have been confirming.)

Multiple Family Dwelling Units: Records of actual water use for multiple family and mobile homes from the City of Vacaville in 1984 indicate a demand of 282 gpd/unit on an annual average basis. For master planning work a value of 320 gpd/unit is assumed.

Commercial Development: Commercial development is assigned a demand value of 1,400 gpd/acre including the building and lawn areas. It is assumed that the building area covers 25% of the land area, so the corresponding demand on a basis of building area is 129 gpd/1,000 ft .

Industrial Development: Industrial development is assigned a demand value of 1,500 gpd/acre including the building and lawn areas. It is assumed that the building covers 40% of the land area, so the corresponding demand on a basis of building area is 86 gpd/1,000 ft .

With each of the types of development, unit demand factors are used which will predict water use which is typical of existing conditions. If "wet" industries such as food processing develop in Vacaville, demands considerably larger than predicted could occur.

The impacts of conservation efforts on future demands are not considered in the projections which adds a conservative element to sizing of facilities in the master plan. If water conservation is effective, the result will be to delay the time when capital improvements are needed. The impacts of water conservation efforts are further analyzed in The Urban Water Management Plan for the City of Vacaville.

TABLE 3-4

ACTUAL AND PROJECTED WATER DEMANDS

YEAR	AVERAGE DAY WATER DEMAND (MGD)			MAXIMUM DAY WATER DEMAND (MGD)		
ACTUAL VALUES						
1984		10.00			18.60	
1985		10.01			16.77	
1986		10.88			18.44	
1987		11.58			20.14	
1988		11.68			20.37	
1989		11.94			22.38	
PROJECTED VALUES						
	HIGH	MEDIUM	LOW	HIGH	MEDIUM	LOW
1990	12.66	12.49	12.33	25.32	24.97	24.66
1991	13.38	13.03	12.72	26.76	26.07	25.44
1992	14.10	13.58	13.11	28.19	27.16	26.22
1993	14.82	14.13	13.50	29.63	28.25	27.00
1994	15.53	14.67	13.89	31.07	29.35	27.78
1995	16.25	15.22	14.28	32.51	30.44	28.56
1996	16.97	15.77	14.67	33.94	31.53	29.34
1997	17.69	16.31	15.06	35.38	32.63	30.12
1998	18.41	16.86	15.45	36.82	33.72	30.90
1999	19.13	17.41	15.84	38.26	34.81	31.69
2000	19.85	17.95	16.23	39.69	35.91	32.47
2001	20.57	18.50	16.62	41.13	37.00	33.25
2002	21.29	19.05	17.01	42.57	38.09	34.03
2003	22.00	19.59	17.40	44.01	39.18	34.81
2004	22.72	20.14	17.79	45.45	40.28	35.59
2005	23.44	20.69	18.18	46.88	41.37	36.37
2006	24.16	21.23	18.57	48.32	42.46	37.15
2007	24.88	21.78	18.96	49.76	43.56	37.93
2008	25.60	22.33	19.35	51.20	44.65	38.71
2009	26.32	22.87	19.75	52.63	45.74	39.49
2010	27.04	23.42	20.14	54.07	46.84	40.27

C. GROUNDWATER SUPPLY

1. Description of the Aquifer

The well field along Elmira Road withdraws ground water from an underlying deep confined aquifer which contains high quality water. The geologic conditions of the aquifer are summarized in a report prepared in 1985 by John F. Mann titled "Ground Water Resources of the Vacaville Area." Excerpts of the report are provided below and the complete report is provided in Appendix A.

> "The main aquifer east of Vacaville consists of gravels near the base of the Tehama formation...The Tehama formation is inclined 15 to 25° from the horizontal so that the formation becomes deeper to the east...The base of the Tehama formation is at the ground surface just west of the Water Treatment Plant. At Nut Tree Road (Well 2) the main gravel zone is between depths of 340 and 710 ft. Farther east, at Well 4, the main Tehama gravels are found between depths of 560 and 865 ft. At Leisure Town Road, the base of the Tehama gravels may be as deep as 1,200 ft.
>
> The basal gravel zone of the Tehama formation is a pressure or confined aquifer. It is overlain by a shallow (unconfined) aquifer with a true water table. The pumping of the deep aquifer at the Elmira Road well field has no effect on the shallow water table. When the deep aquifer is pumped, a cone of depression is created in the pressure (also called the piezometric or potentiometric) surface. This cone will grow and deepen until the flow in the confined aquifer equals the amount of water being pumped from the well field."

2. Well Field

Currently, 9 wells withdraw water from the deep aquifer. The locations of the wells and their years of construction are shown in Figure 3-1. Wells 1, 4, and 6 have been in operation since the 1950's, while the other wells are considerably newer. Approximate well capacities at typical operating heads are given in Table 1-1. With the exception of the small capacity of Well No. 1, production capacities for all wells are within the range of 1,100 to 1,520 gpm. Specific capacity varies from one well to the next but typically it will range from 10 to 20 gpm/ft.

As shown in Table 3-5, water withdrawal from the well field has increased appreciably. Annual production records vary from 2,862 AF/yr in 1968 to 8,156 AF/yr in 1983. During this same period static water levels in the wells declined from about 130 ft to over 200 ft. In 1984, water production was reduced in response to the declining levels, and since that time it appears that water levels have reached a dynamic equilibrium with the pumping rates. (However, preliminary data from 1989 indicates that water levels may be lower than in 1988). Additional discussion on this matter is provided in Appendix A.

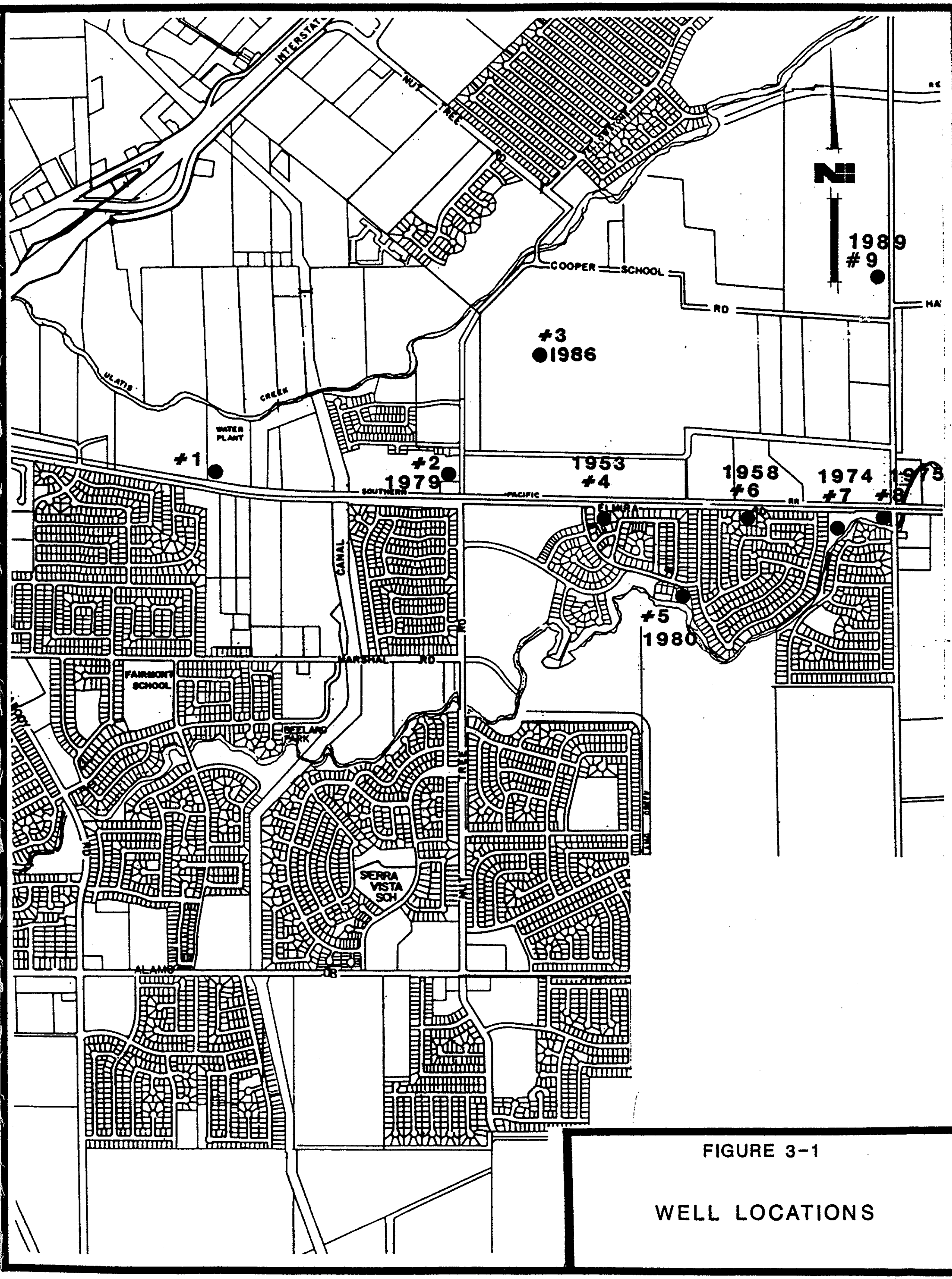


FIGURE 3-1

WELL LOCATIONS

TABLE 3-5

WATER PRODUCTION HISTORY (AF/YR)

Year	Well Production	Surface Water Production	Total Production
1968	2,862	968	3,830
1969	3,046	1,234	4,280
1970	2,871	1,262	4,133
1971	3,196	1,334	4,540
1972	3,255	1,448	4,703
1973	3,125	1,791	4,916
1974	3,316	1,921	5,237
1975	3,970	1,732	5,702
1976	4,965	1,897	6,862
1977	5,076	1,190	6,266
1978	5,707	1,551	7,258
1979	6,236	1,728	7,964
1980	7,043	1,786	8,829
1981	7,740	1,588	9,328
1982	7,684	1,658	9,342
1983	8,156	1,616	9,772
1984	6,063	5,137	11,200
1985	5,853	5,359	11,212
1986	5,829	6,363	12,191
1987	6,267	6,702	12,969
1988	5,419	7,664	13,083
1989	6,180	7,202	13,382

3. Recommendations

The ground water supply system is an important source of water for Vacaville in that it is a reliable source of high quality water. However, the capacity of the aquifer is limited, so expansion of this supply to meet increasing annual demands in Vacaville may not be practical. In the long term, the role of the ground water resources should be to meet peak demands, and use practices should be aimed at maintaining the integrity of the aquifer. Recommendations for future management practices of the ground water supply system are discussed below.

In the 1986 Draft Water System Master Plan it was recommended that additional data be obtained to quantify the recharge characteristics of the aquifer.

Since that time additional data has been collected on the static water levels. This data was reviewed by our consulting geologist and hydrologist, Dr. John Mann in conjunction with this 1990 Master Plan. Based on the 1986 through 1988 pumping rates (5,829, 6,267, and 5,419 acre-feet per year) Dr. Mann has concluded that "there is no evidence of overdraft, and it would appear that the safe yield of the system has not yet been reached. It is quite possible that a larger amount of seasonal pumping will also result in a dynamic equilibrium -- but with somewhat lower static and pumping levels." (Preliminary review of water levels in 1989 by city staff have indicated somewhat lower levels than in 1988).

With the 9 existing wells and with the NBR plant on-line, additional capacity from the well field is not needed to meet water demands in the next few years, but it would be beneficial to take advantage of this water supply to the fullest extent possible. In addition to increasing well field capacity, new wells will be needed as old ones become inefficient or fail. Currently, Well No. 4 (installed in 1953) is having a new hole drilled at the site for relocation of the pump. Furthermore, Well No. 2 has had problems in recent months with drawdown of the water in the well and may require rehabilitation.

In replacing the wells it will be possible to use existing sites for drilling the new wells in certain instances. However, where existing sites are confined, the replacement wells may need to be located elsewhere. When selecting new locations, consideration should be given to expanding the areal extent of the well field in order to minimize interferences in the cones of depression between wells.

We also recommend that consideration be given to adding ammonia feed facilities to the wells to provide disinfection by chloramination. This method of disinfection will reduce the potential for formation of trihalomethanes. The NBR plant will disinfect with chloramines and there may be benefits in standardizing the type of disinfection for the entire water supply.

D. SURFACE WATER SUPPLIES

1. Sources

Currently, surface water is supplied to Vacaville by the Solano County Water Agency. The water originates from Lake Berryessa behind Monticello Dam and is conveyed to Vacaville via Putah Creek, Lake Solano and then the Putah South Canal. Putah South Canal is operated and maintained by Solano Irrigation District (SID). Vacaville is currently entitled to 5,600 AF of this water annually. Generally, the water is good quality water which can be treated most of the year by diatomaceous earth filters without prior clarification. However, in the winter months turbidities can exceed 20 NTU in which case the water cannot be effectively treated. A water quality study showed that there were 39 days where turbidity exceeded 20 NTU in 1982 (Black and Veatch, 1983).

The other source of surface water available to Vacaville is the North Bay Aqueduct (NBA). Entitlements of NBA water to Vacaville commenced in 1986 with an annual volume of 100 AF and increase to a maximum value of 6,100 AF in 1996. The schedule for NBA entitlements to Vacaville is given in Table 3-6.

Another source of surface water for Vacaville is additional SID entitlements which will become available as industrial land in the northeast sector of Vacaville is developed. The entitlements will be in proportion to the water demands, and there are approximately 2,350 acres in this area.

For planning purposes it is assumed that 80% of all industrial growth and 20% of all commercial growth occurs in this north industrial area generating new water entitlements in direct proportion to their demands. It is also assumed that 500 AF/yr is currently utilized and that new water demands correspond with the per acre values given in Table 2-1 and the moderate growth projections given in Table 3-3. Based on these assumptions, entitlements are expected to increase by about 25 AF/yr as given in Table 3-6.

To date, none of the northern industrial area entitlements have been acquired, but acquisition of such water by the City will be very important to the long-term water supply needs of the City.

TABLE 3-6

AVAILABLE FUTURE WATER ENTITLEMENTS
(acre-feet per year)

Year	North Bay Aqueduct	Northern Industrial SID[1]
1990	1,000	500[2]
1991	1,000	525
1992	1,500	550
1993	2,000	575
1994	2,500	600
1995	3,000	625
1996	6,100	650
1997	6,100	675
1998	6,100	700
1999	6,100	725
2000	6,100	750
2001	6,100	775
2002	6,100	800
2003	6,100	825
2004	6,100	850
2005	6,100	875

[1] Based on 80% of industrial and 20% of commercial moderate growth projections and water demand factors from Table 2-1.

[2] Approximate current utilization.

2. Recommendations

As water demands increase and sources of production capacity are expanded in the future, the utilization of each source of production will shift. It is recommended that at the beginning of each year goals be established for utilization of each source (i.e. NBA water, SID water and groundwater). The goals should take into account the following recommendations.

- The NBR plant should be used as a base (year-round, constant) production source.

- The DE plant should function as a peaking supply during the summer, but should be kept ready for emergency use at other times. (pending viability in light of future regulations).

- Wells should be used for peak summer demands with reduced use during the winter, but extreme pumping in the summer should be avoided to limit draw down of the water levels.

It is also recommended that the City actively pursue acquisition of SID entitlements for development of the northern industrial area and other surface water sources which could be made available. As discussed in the following subsection, new sources of water are needed to meet current and future water demands.

E. SUPPLY DEMAND ANALYSIS

The following section presents an analysis of the water supply and demand situation based on two critical conditions. The first condition is the ability of the existing wells and treatment facilities to produce sufficient water. The second condition is the quantity of water available to meet annual demands.

1. Maximum Day Production Capacity Versus Demand

In order to meet peak demands in the summer months, production capacity must equal or exceed the maximum day demands. The ability of existing wells and treatment facilities to meet projected maximum day demands is illustrated in Figure 3-2. In this figure two levels of production capacity are presented. The first level represents the condition where all surface treatment facilities are operational and the largest well is out of service, i.e the firm capacity. As discussed in Section 5,E, Recommended Additional Treatment Capacity, a 10% to 20% reserve firm capacity is desired. Therefore, a second level has been shown as a band which represents the condition where there would be 10% to 20% reserve capacity.

In Figure 3-2 it is apparent that existing maximum day demands are very close to existing firm production capabilities. This situation may be critical in 1990 if the NBR plant is not on line before the summer.

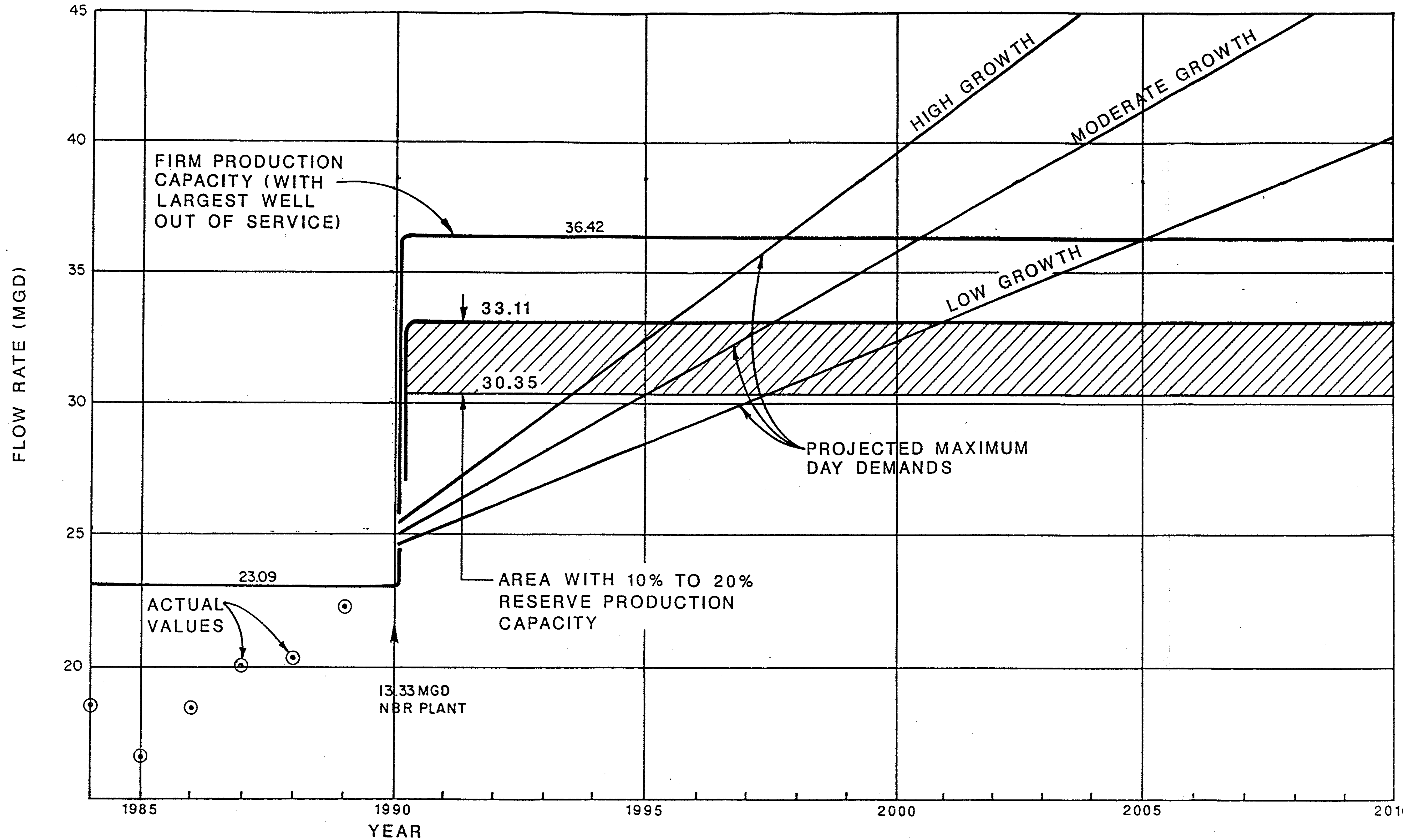
However, once the NBR plant is on line the production capabilities will easily meet the demands for several years.

As demands increase beyond the year 1990 they will once again approach the production capacity and additional capacity will be required. From our analysis it appears that demands will exceed firm production capacity some time between 1997 and 2005, but to maintain the 10% to 20% reserve, expansion will be needed before that period. (See Section 5,E for the recommended time for expansion.)

2. <u>Annual Water Supply Versus Demand</u>

The comparison of water supply and demand on an annual basis is depicted in Figure 3-3 and given in Table 3-7. In Figure 3-3, two levels of available supply are presented. The first level is the quantity of Putah South Canal water currently available (5,600 AF/yr) plus NBA entitlements and 6,000 AF/yr of groundwater. The second level includes additional SID water entitlements resulting from development of the north industrial area.

In Figure 3-3 and Table 3-7 it is apparent that the water demands will likely exceed the supply through most of the planning horizon regardless of the growth scenario. To some degree this deficit can be made up by withdrawing some additional water from the well field, perfection of water rights from development in the northern industrial area, and if possible, short term utilization of SID and NBA supplies in excess of entitlements. Also, acquisition of new entitlements from the Central Valley Project's proposed Tehema Colusa Canal Extension is a possibility.

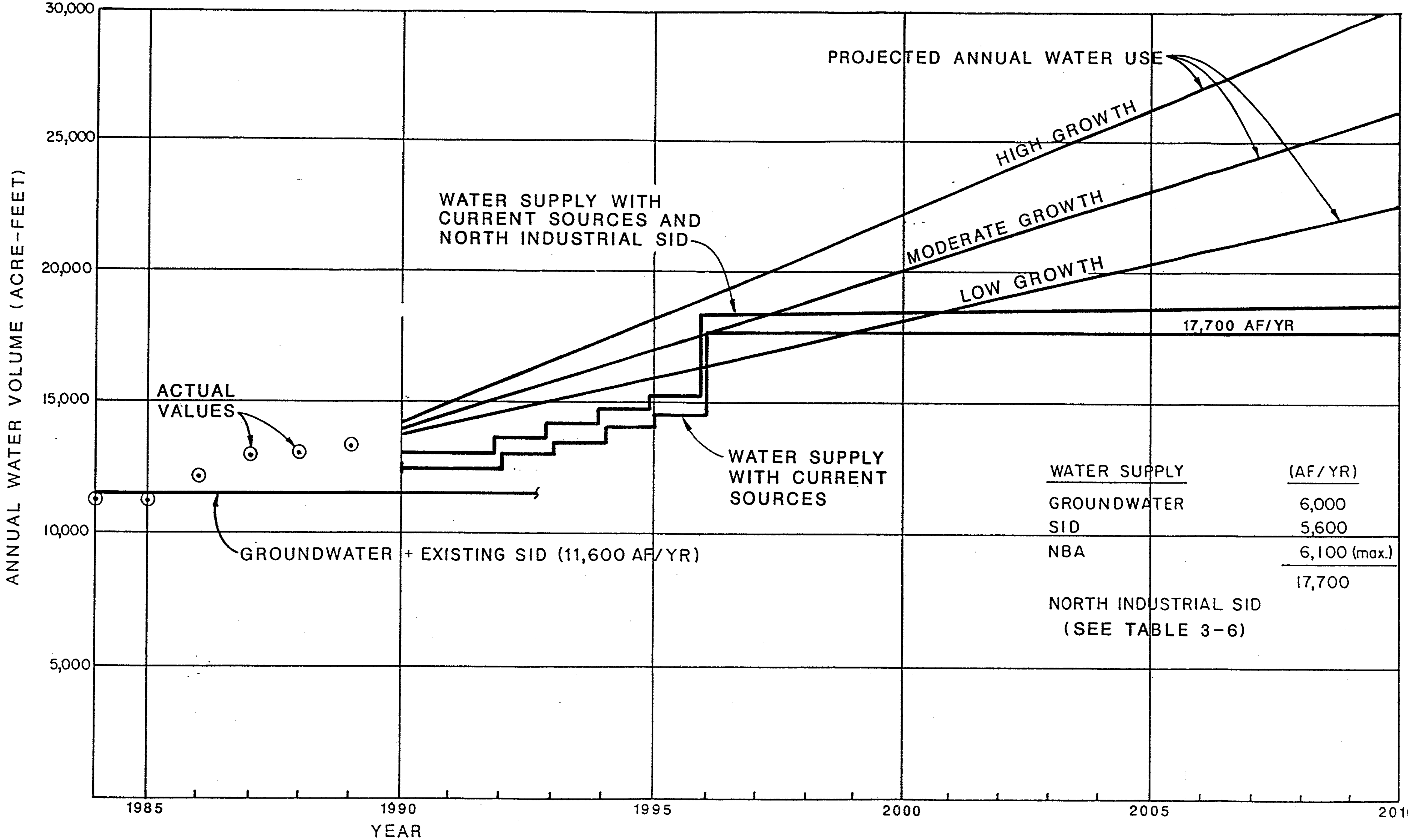


GROWTH ASSUMPTIONS

	SINGLE FAMILY UNITS	MULTIPLE FAMILY UNITS	INDUSTRIAL (1000 S.F.)	COMMERCIAL (1000 S.F.)
LOW	700	200	150	150
MODERATE	950	300	240	240
HIGH	1200	400	340	500

FIGURE 3-2

MAXIMUM DAY WATER DEMAND VERSUS PRODUCTION CAPACITY



GROWTH ASSUMPTIONS

	SINGLE FAMILY UNITS	MULTIPLE FAMILY UNITS	INDUSTRIAL (1000 S.F.)	COMMERCIAL (1000 S.F.)
LOW	700	200	150	150
MODERATE	950	300	240	240
HIGH	1200	400	340	500

FIGURE 3-3

ANNUAL WATER DEMAND VERSUS WATER SUPPLY

TABLE 3-7

SUMMARY OF ANNUAL WATER SUPPLIES AND DEMANDS
(acre-feet)

	Water Supply						
Year	Groundwater	Basic SID	North Industrial SID[1]	NBA[1]	Total	Water Demands[2]	Net Requirement
1990	6,000	5,600	500	1,000	13,100	13,989	889
1991	6,000	5,600	525	1,000	13,125	14,593	1,468
1992	6,000	5,600	550	1,500	13,650	15,210	1,560
1993	6,000	5,600	575	2,000	14,175	15,826	1,651
1994	6,000	5,600	600	2,500	14,700	16,430	1,730
1995	6,000	5,600	625	3,000	15,225	17,046	1,821
1996	6,000	5,600	650	6,100	18,350	17,662	-688
1997	6,000	5,600	675	6,100	18,375	18,267	-108
1998	6,000	5,600	700	6,100	18,400	18,883	483
1999	6,000	5,600	725	6,100	18,425	19,499	1,074
2000	6,000	5,600	750	6,100	18,450	20,104	1,654
2001	6,000	5,600	775	6,100	18,475	20,720	2,245
2002	6,000	5,600	800	6,100	18,500	21,336	2,836
2003	6,000	5,600	825	6,100	18,525	21,940	3,415
2004	6,000	5,600	850	6,100	18,550	22,557	4,007
2005	6,000	5,600	875	6,100	18,575	23,173	4,598

[1] See Table 3-6
[2] Moderate Growth Rate, See Table 3-4, 1 mgd = 1,120 AF/yr

4. WATER SYSTEM DESIGN CRITERIA

The following section presents our recommended design criteria for water system facilities. Consideration is given to selecting building pad elevations, sizing water mains, storage reservoir volumes, pump station capacity, and materials of construction for pipelines.

In some cases, criteria differ between "Primary Pressure Zones" and "Secondary Pressure Zones". Primary zones include Zone 1 and planned Zone 2. Secondary zones include all other pressure zones such as the Vine Street, Hidden Valley and Wykoff zones.

A. UPPER AND LOWER ELEVATION LIMITS OF A PRESSURE ZONE

Appropriate selection of building pad elevations in a pressure zone is important in that it will, to a large extent, control the water pressures at the building pad. One very important factor in selecting allowable elevations is the vertical distance between the building pad and the water level in the storage reservoirs. During average and low water demand conditions, the pressure at the building pad should be close to the static head (or vertical distance) between the reservoir water level and the building pad.

Under high demand situations, or in a system with undersized pipelines, the hydraulics of the water system between the reservoirs and the building pad becomes a more significant factor due to headlosses in the system. These losses will result in temporary lowered pressures at the building pads.

It is possible to use a performance criteria for selecting allowable building pad elevations taking into account both the static head and the characteristics of the distribution system, but we recommend that the City not adopt such a policy. Establishing a performance criteria will necessitate engineering analyses to prove satisfactory performance prior to development. Such analyses will likely lead to debates on acceptable building pad elevations.

Instead of the performance criteria, we recommend using fixed elevations for each pressure zone in the water distribution system. The elevations should be set to account for a moderate amount of headloss under high demand conditions, and emphasis should be placed on providing adequate pipeline facilities so that the system headlosses do not become excessive.

The recommended criteria are based on the following conditions:

- The highest building pad should be no less than 80 ft. below the low water level in the water storage reservoir. This will assure a minimum static pressure (neglecting system losses) of 35 psi.

- The lowest building pad should be no more than 204 feet below the high water level in the water storage reservoir. This will assure a maximum static pressure of 88 psi.

- Therefore, with a 24 foot high water tank, the range of elevations in any pressure zone should be limited to 100 feet.

The low water level in the Zone 1 reservoirs is assumed as 300 feet (although certain reservoirs extend down to approximately 292 feet). **Therefore, the maximum recommended building pad elevation in the Zone 1 system is 220 feet.**

The high water level in the Zone 1 reservoirs is 324 feet. Therefore, the minimum recommended building pad elevation is 120 feet. Certain areas in Zone 1 are currently developed at elevations below 120 feet, and high pressures are realized. There is little that can be done to remedy this situation economically, so pressure reducing devices are needed to mitigate the high pressures.

The criteria identified above should be applicable to both one and two story buildings. Single story buildings should not be allowed at a higher elevation because of the potential for building additions or reconstruction in the future.

As discussed in Section 8, a new Zone 2 is planned for northwest Vacaville (with possible expansion to the West Valleys). The recommended range in elevations for Zone 2 is 215 to 315 feet. Anything higher than 315 feet would require a new zone.

B. MINIMUM CONDITIONS FOR FIRE PROTECTION PLANNING

Following are the minimum fire protection criteria to be used in planning and sizing new facilities in the water system. The Vacaville Fire Department will continue to determine actual fire flow requirements at the site based on building construction and will specify appropriate fire protection mitigation for the buildings.

1. Minimum Test Conditions

In testing the distribution system, the following two sets of criteria shall be used:

Test 1:

- Fire flow at the most critical point in the distribution system.
- Maximum day water demand.
- A residual pressure of 20 psi.
- The most critical component of the water system is out of service.
- In Zone 1, the water level in all reservoirs is 305 ft. In all other zones the water level is equal to that with all operational and emergency volumes depleted.

Test 2:

- Peak hour water demand.
- A residual pressure of 30 psi.
- The most critical component of the water system is out of service.
- In Zone 1, the water level in all reservoirs is 305 feet. In all other zones the water level is equal to that with the operational and emergency volumes depleted.

2. Minimum Fire Flow and Storage Requirements

The fire flows and durations in Table 4-1 shall be used to plan and size new distribution system and storage facilities. In all cases, the most severe condition in any given area shall prevail.

TABLE 4-1

FIRE FLOWS AND DURATIONS

Type of Development[1]	Flow (gpm)	Duration (hours)
Single Family Resid.	1,500	2
Medium Dens. Resid. and Schools	3,000	3
High Dens. Resid., Industrial, and Commercial	4,500	4

[1] See Table 2-1 for land use definitions. Single family residential is R-2.5, R-1, R-0.5 and RLD.

3. Calculations and Modeling

See Section H.4.

C. DISTRIBUTION SYSTEM PIPELINE SIZING

Pipeline sizing in the distribution system shall be based on anticipated water demands, fire flows and hydraulic characteristics of the distribution system. Following are a set of recommendations for selecting alignments and pipeline sizes in the distribution system. The City Standard Specifications are referenced for additional design detail.

1. Pipeline Network

Single pipelines supplying a given area should be avoided whenever possible, and "looped" systems should be required whenever possible. A looped system will provide two substantial benefits: 1) the supply to a given area cannot be cut off by a failure of only one pipeline, and 2) during a fire or when other high localized demands occur, the looped system will allow water to be supplied from two or more directions thereby minimizing system headloss between the points of supply and demand.

2. Major Transmission Mains

Major transmission mains which are 12 in. and larger should be selected and aligned based on a hydraulic analysis of the distribution system.

3. Local Distribution System Pipelines

Pipelines which distribute water from the transmission mains to local service areas must meet the following minimum requirements: 1) In single family residential areas pipelines shall not be less than 8 inches; 2) In medium and high density residential areas, school locations and in commercial and industrial areas, pipelines shall not be less than 12 inches Also, minimum sizes are conditional on meeting the fire flow criteria specified herein.

In all cases minimum sizes shall be subject to approval by the City, and larger sizes may be required at the City's discretion.

4. Water Valve Location

Water valves should be strategically located on 8 in. and larger pipelines so that if a pipeline fails it can be isolated on both sides of the failure at the nearest tees or crosses with other 8 in. and larger pipelines.

D. DISTRIBUTION SYSTEM STORAGE

The volume of water in storage must be sufficient to provide the sum of operational, fire and emergency needs. A brief definition for each of the storage components is provided below.

Operational Storage: This is the volume required to equalize the diurnal fluctuations in water system supply and demand. For example, during a hot summer day when water demands are high the pumping and water production capacity cannot meet the demands, so water is withdrawn from the reservoirs until the demands subside. The amount of water withdrawn is part of the operational storage. The other part of operational storage is that needed as a result of not sequencing booster pumps and other production facilities at maximum efficiency.

Fire Storage: Fire storage is the amount of water reserved for fire protection. It is calculated from a specified flow rate and a duration.

Emergency Storage: Emergency storage is the amount of water reserved for use in the case of a power outage, loss of a critical water supply source or other emergency.

The recommended criteria for storage volumes are separated into primary zone requirements and secondary zone requirements.

1. <u>Primary Zone Storage (Zone 1 and Planned Zone 2)</u>

   a. Operational storage should be equal to 25 percent of the total water demand on the maximum day.

   b. Fire storage should be equal to the most critical combination of flow rate and duration in the pressure zone. Zone 1 fire storage should be equal to 4,500 gpm for a duration of 4 hours which is equal to 1,080,000 gallons.

   c. Emergency storage should be equal to 12 hours of maximum day demands in the most critical emergency condition which can be reasonably expected. Reference is made to Section 6 for further discussion on emergency conditions.

   d. In calculating the volume of storage needed in Zone 1, fire volumes in planned Zone 2 can be included if such elevated sources can be directly released back into Zone 1 via a zone isolation system. The zone isolation system shall include two isolation valves surrounding a blow off valve located in a vault.

2. <u>Secondary Zone Storage</u>

   a. Operational storage should be equal to 25% of the total water demand on the maximum day.

   b. Emergency storage must be equal to 75% of the total water demand on the maximum day. This requirement is more stringent than that for the primary zones because there is less operational control in the secondary zones.

   c. Fire storage should be equal to the most critical combination of flow rate and duration encountered in the zone.

E. PUMPING FACILITIES

1. <u>Primary Zone Pumping Capacity (Zone 1 and Planned Zone 2)</u>

Firm pumping capacity shall be equal to or greater than the maximum day plus fire volumes pumped over a 24 hour period. The firm capacity in Zone 1 shall be calculated as the sum of the individual firm capacities from each of the following facilities: 1) the NBR plant once it is on line, 2) the DE plant, and 3) the well field excluding wells 1,4, and 6.

2. <u>Secondary Zone Pumping</u>

The firm capacity must be sufficient to fill the storage reservoir (i.e., one maximum day demand plus fire storage) in a period of 18 hours.

3. <u>Firm Capacity</u>

Multiple pumps are required at each pumping station and the firm capacity must be calculated with the largest pump out of service.

4. <u>Reservoir Maintenance Provisions</u>

Where there is only one reservoir in a service area, provisions must be made for taking the reservoir out of service without interrupting water supplies to the customers.

F. ALTERNATE CRITERIA FOR SECONDARY ZONES

If it is not technically feasible to construct all of the recommended elevated storage in Section D, then alternate criteria can be applied provided that 1) emergency power supplies are constructed at the pumping station with capacity equal to the firm pumping capacity, 2) the supply pipeline into the booster pumping station is not subject to service interruptions, and 3) the Director of Public Works has declared that the system will provide reliable water service and is acceptable.

If elevated storage is provided, but at less than recommended volumes, then the relationship between pumping capacity and storage volume is provided below. In this case, the storage volume must be at least equal to the operational requirement, and should be as large as technically feasible.

$$P = \frac{OSR + ESR + FS}{1080} + \frac{FSD}{Duration}$$

Where:

P	=	firm pumping capacity, gpm
OSR	=	operational storage requirement, gallons
ESR	=	emergency storage requirement, gallons
FS	=	actual fire storage, gallons
FSD	=	fire storage deficit, gallons (recommended tank volume minus actual tank volume with a maximum equal to the recommended fire storage volume)
1080	=	minutes per 18 hr
Duration	=	fire duration in minutes

If it is not possible to provide any elevated storage and a hydropneumatic system is necessary, the pumps must be sized for peak hour times 1.5 plus fire flow.

G. PIPELINE MATERIALS OF CONSTRUCTION

1. 6 in. to 12 in. Pipelines

We recommend that ductile iron pipe be allowed in all applications throughout the City. As a minimum, ductile iron pipe should be class 50, and should be wrapped in a polyethylene film in accordance with AWWA C-105. Additional strength and cathodic protection should be provided as necessary in the opinion of the Department of Public Works.

We also recommend that PVC C-900 be added to the City standards. As a minimum, the pipe should be class 150 PVC. This pipe should be allowed for local distribution through subdivisions and other areas, however, it should not be used for critical transmission mains, reservoir tie-ins and applications subject to high pressures and hydraulic surges.

2. Pipelines Larger than 12 in.

Pipelines larger than 12 in. should be ductile iron or alternate materials as specifically designed and approved on a case by case basis.

3. Other

The City Standard Specifications are referenced for additional detail.

H. MISCELLANEOUS CRITERIA

1. Access Roads to Reservoirs and Mechanical Facilities

An access road shall be provided to all reservoirs and mechanical facilities in the water system. This road shall be paved with asphalt and shall not be constructed at slopes greater than 15% unless approved otherwise by the City.

2. Cross Country Pipelines

Cross country pipelines shall be avoided whenever possible. If approved by the City, they shall be constructed on slopes less than 20% and an all weather access road shall be provided. The road shall be asphalt paved on all sections with a slope greater than 10%. Portions of the road at less than 10% may require pavement as a result of site specific conditions.

3. Right of Ways

All water system facilities shall be in City property. All distribution system facilities shall be within City street right of ways. However, with approval by the Director of Public Works, distribution system facilities may be placed in dedicated easements.

4. Hydraulic Calculations

Additions to the water system shall be contingent on submission to the City hydraulic calculations based on fire flow test results confirming that the design criteria are met. The City's computer model shall be used except in simple, non-looped conditions such as a residential cul-de-sac where manual calculations will suffice.

5. Reservoir Materials of Construction

New reservoirs should be of concrete construction because a concrete tank can be partially buried to reduce its visibility and keep the water cool to prevent chlorine dissipation. Concrete tanks will also have lower maintenance requirements. Steel water storage reservoirs cause problems due to aesthetics and the need to periodically repair the coating system.

5. TREATMENT FACILITIES

The following section addresses surface water treatment facilities and associated booster pumping facilities. Currently, the City operates a 10 mgd diatomaceous earth filtration water treatment plant located on Elmira Road, referred to as the DE plant. An additional 13.3 mgd of treatment capacity will be available in 1990 from a new gravity filtration plant which will be jointly owned and operated by Fairfield and Vacaville. This plant is referred to as the North Bay Regional or NBR plant.

When the NBR plant is completed, the need to use the DE plant will initially be limited to summer months when peak water production is required. However, each year after 1990 the need for the DE plant will be greater as new development increases the city wide water demands. This increasing need for the DE plant will only be offset by an expansion of the NBR plant or acquisition of other water production facilities. In summary, the DE plant cannot be abandoned without replacing its capacity, and therefore should be maintained if it can economically meet upcoming water treatment regulations. A study is currently underway to assess the potential impacts from upcoming regulations on the DE plant, and it should be complete in the fall of 1990.

A. DIATOMACEOUS EARTH TREATMENT PLANT AND BOOSTER PUMPING STATION

The existing facilities are briefly described, and some present deficiencies are identified below.

1. Existing DE Treatment Plant Facilities

The DE plant consists of eight diatomaceous earth filters, each with a capacity of 1.25 mgd yielding a total capacity of 10.0 mgd. The DE plant has been in operation since 1963, but four of the eight filters were added in 1983. The treated water enters a 1 MG ground level reservoir. This reservoir also receives water from wells 1, 4 and 6. From the reservoir, the water is pumped into a 30 in. main which feeds into the distribution system.

Although the DE plant is designed with a capacity of 10.0 mgd, average production rates can vary depending upon how the plant controls are set, and how closely the filter runs are monitored. The plant production will also depend on the condition of the raw water. Relatively clean water may be treated at a rate greater than 10.0 mgd; but as noted in the Water Treatment Master Plan (Black and Veatch, 1983), "operating experience to date has indicated that costs are excessive when turbidities reach 30 or more in the canal." These higher costs can be attributed to shorter filter runs and a greater rate of diatomaceous earth utilization.

High turbidities normally occur in the winter and cleaner, less turbid water is normally present in the summer. Thus, the DE plant will function better in the summer when it is most needed. During the winter months temporary shutdown of the DE plant may be necessary when turbidities are high, but such shutdowns should not cause a problem in meeting demands.

Historically, winter demands have been small enough to be met with ground water alone and after 1990 the NBR plant will be able to meet most, if not all of the winter demands.

2. Existing Booster Pumping Station

A schematic of the existing booster pumping station is shown in Figure 5-1. Currently five pumps in the Booster Pump Building withdraw water from the ground level reservoir. Each of these five pumps is limited in the ability to draw down the clearwell as a result of suction head requirements. Booster pumps 1, 2, 3 and 4 are centrifugal pumps with typical production rates between 1,200 and 1,800 gpm each depending on the operating head. Pumps 1 and 2 are equipped with dual drives and standby engines. Pump 5 is a larger centrifugal pump with typical production rates between 2,000 and 2,500 gpm depending on the operating head. With all five pumps operating together and a pressure of 98 psi on the discharge side of the pumps, approximately 8,200 gpm are pumped.

In 1985, the booster pumping capacity was expanded by installing a vertical turbine pump rated at 2,600 gpm. This pump is situated outside of the booster pump building along with four additional sumps for future vertical turbine pumps. The intake to these sumps is in the bottom of the existing storage reservoir allowing complete drawdown of the reservoir without suction lift problems. A 30 in. blind flange is situated on the pipe leading from the reservoir to the sumps in anticipation of a future connection to a reservoir west of the existing one.

The suggested design criteria for this booster pumping station is to match the maximum production capacity and use some of the reservoir storage volume with one pump out of service. The maximum production capacity of the DE plant is assumed to be 10 mgd (6,940 gpm), but this pumping station must also meet the production capacity of wells 1 (230 gpm), 4 (1,340 gpm), and 6 (1,140 gpm). Furthermore, the pumping capacity must also be able to use part of the capacity in the ground level reservoir to meet peak demands. If 500,000 gallons is pumped from the reservoir in 12 hours, the corresponding average flow rate is equal to 700 gpm. Thus, the total "firm" pumping capacity needed is equal to 10,350 gpm. The current total pumping capacity is about 10,800 gpm; but with the largest pump (2,600 gpm) out of service, only 8,200 gpm can be expected.

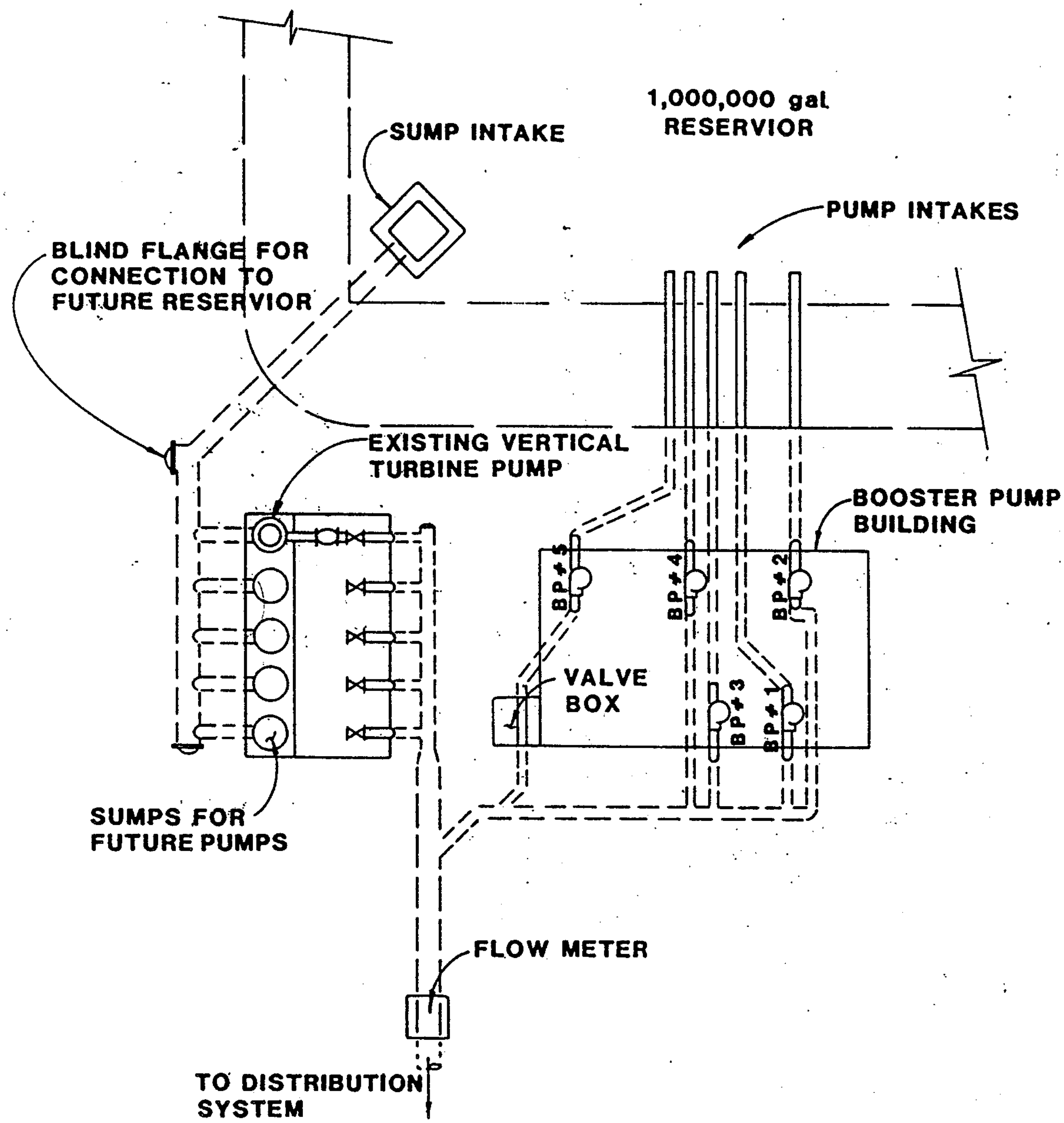


FIGURE 5-1

BOOSTER PUMPING FACILITIES
AT THE D. E. TREATMENT PLANT

B. RECOMMENDED IMPROVEMENTS AT THE DE PLANT

No expansions of treatment capacity at the DE plant are recommended in the future, as additional treatment capacity will be obtained at the NBR plant. However, several improvements are recommended at the site as identified below. These improvements were identified in conjunction with the Corporation Yard Master Plan and refined in the Water Plant and Water System Improvements Preliminary Design Report (Nolte, 1989). All of these improvements are contingent on the results of the 1990 Disinfection and Operations Study which will address the impacts of future water treatment regulations.

1. Booster Pump Station Upgrade

Currently, only one vertical turbine pump is installed and the remaining four (4) pumps still need to be added. The pumping capacity provided by the additional vertical turbine pumps will permit the existing centrifugal pumps to be taken out of service. The existing pump building is constructed of metal, and replacement with a smaller block building is recommended for housing the electrical controls. The new pump controls should be designed to operate with the DE plant standby generator under emergency conditions and should be designed in accordance with the Water System SCADA Master Plan (Nolte, 1989).

Installation of a surge tank on the discharge header from the vertical turbine pumps is recommended to prevent excessive water hammer during a power outage when several of the vertical turbine pumps are operating simultaneously. During design of the vertical turbine pump structure in 1984 it was decided to delay analysis and installation of the surge control facilities until installation of the remaining pumps because of the high cost of this equipment.

The existing 30 in. propeller flow meter is located under a future soundwall and will have to be relocated. As part of the relocation, it is recommended that the propeller meter be replaced with a pressure differential type meter.

2. Water Treatment Plant Building Addition

Expansion to the DE plant building was initially discussed in the Water Treatment Master Plan prepared by Black and Veatch in 1983. In this plan, the building was to be expanded to the north and to the south. We recommend that the plan be revised by expanding primarily to the south. This will allow the employee facilities (offices, lockers, assembly room) to be separated from the sources of noise at the plant. It would be possible to reduce the size of the expansion and provide employee facilities for the water supply staff at the administration building. However, it is considered important to have the chief plant operator and staff at a single location which is proximate to the treatment facilities.

The expansion to the south should include space to house future water system SCADA equipment and a new laboratory. The laboratory should be equipped with pumped samples of raw, filtered, and dosed waters. Other new rooms in the expansion should include an office for the chief plant operator, mens and women's locker rooms, and an assembly area.

Once the above improvements are made, the trailer currently used by the staff as an assembly area can be removed, and the storage shed and chief plant operator's office located west of the trailer can be abandoned.

3. Chemical Addition Facilities

Substantial improvements to the chemical addition facilities at the DE plant are needed including relocation, rehabilitation and upgrading the chlorination facilities, relocation and upgrading of the fluoridation facilities, and possibly addition of all new ammonia facilities. A new chemical addition building located northwest of the DE plant should be constructed to house the chlorine, fluoride, and ammonia feed facilities.

Once the new chemical addition building is constructed, the existing chlorine building should be converted into a storage building for chemicals used at the City's wells. The existing two rooms will be suitable for storing ammonia and 150 pound chlorine cylinders once the building hardware and HVAC equipment are replaced. One new room will have to be added to the existing structure to store the 55 gal drums of fluoride.

a. Chlorination Feed Facilities:

Currently, chlorine is applied with three manually controlled chlorinators dedicated to pre-chlorination and post-chlorination of the DE plant and the supply line from wells 1, 4 and 6. It is recommended that chlorinator controls for the DE plant water be upgraded from manual to automatic modes. Pre-chlorination should be flow paced, and post-chlorination should be controlled with a compound loop system using both flow and chlorine residuals. Manual control for chlorination of wells 1, 4 and 6 is considered acceptable because the flow remains relatively constant.

b. Fluoride Feed Facilities:

The existing fluoridation facilities are located out doors in the southwest corner of the corporation yard and need to be relocated in the new chemical addition building. The fluoride feed system should also be upgraded to more automated control.

c. Ammonia Feed Facilities:

Once the NBR plant is on line, it may be necessary to install ammonia feed facilities to convert the disinfection process from chlorination to chloramination in order to prevent the occurrences of taste and odor problems. New ammonia facilities would be located in the chemical addition building.

d. Chlorine Residual Monitoring

It is anticipated that new state regulations will be adopted in 1990 which will require continuous monitoring of chlorine residual at the point of application to the distribution system from surface water supplies. As a result of this requirement residual monitoring should be installed at the DE plant.

4. New Waste Diatomaceous Earth Drying Facilities

The existing DE dewatering ponds need to be replaced due to the inefficiency and expense involved in operating them. It is recommended that new facilities be located in the vacant lot to the east of the DE plant. The new facilities will consist of sunken concrete basins with an access ramp for a front end loader. The bottoms of the basins should be sloped to a sump at one end where the water will be decanted and drained to the sewer system.

5. Filter to Waste Capability

The DE plant is not equipped with the capability to "waste" treated water and it is anticipated that this provision will be required by the new 1990 state regulations. This provision is needed for periods when the system is being tested and the water should not be directed to the reservoir and during startup of the filtration process when the plant effluent quality can be substandard.

6. Second Ground Level Reservoir

In the 1983 Water Treatment Master Plan prepared by Black and Veatch, a new 1 MG clearwell is recommended west of the existing clearwell. This reservoir would not serve the same function as elevated storage in the distribution system because it will not equalize distribution system pressures and its ability to supply water will be limited by the pumping capacity. The primary benefits of this reservoir are that it would increase the operational flexibility of the DE Plant and the booster pumps. Also, it would allow one of the resulting two reservoirs to be taken out of service when repairs are required or if the reservoir becomes contaminated. However, when the NBR plant is on line in 1990 it will be easier to take the existing reservoir out of service. If chloramination is implemented at the DE plant, longer contact times could be required to achieve sufficient disinfection. If this is the case, the second ground level reservoir could be very important.

This reservoir is not included in the Capital Improvements Program.

7. DE Plant Controls

The DE plant is run with an outdated control system which has little flexibility. A new system is needed which will allow system flexibility and will be compatible with the future SCADA system. This can be accomplished with the use of programmable logic controllers (PLCs) which will communicate directly with the SCADA equipment and will allow the modes of operation at the plant to be quickly modified and adapted to changing water quality and water treatment requirements.

8. <u>Body Feed System</u>

Currently, diatomaceous earth is fed to the filters continuously during each filter run with positive displacement pumps. These pumps have had maintenance problems in the past, and new pumps with more abrasion resistance are needed.

9. <u>Air Supply System</u>

Certain filters at the plant utilize an air scour system for cleaning the filter septums, and current air supplies are only sufficient to allow one air scour cycle during filter cleaning. Two air scour cycles are needed to effectively clean the filters so more air supply capacity is needed. Also, the existing air compressors are located out-of-doors and an enclosure is needed to facilitate maintenance. These compressors are critical components of the water plant.

10. <u>Main Switchboard Improvements</u>

The existing electrical equipment associated with the main switchboard (MSB) is inadequate for handling additional loads from new equipment. To remedy the situation it is proposed to continue to use the existing MSB for the booster pump station and install a second MSB for the filtration building.

C. WATER SYSTEM SCADA FACILITIES

A supervisory control and data acquisition (SCADA) master plan has been prepared for the water system (Nolte, 1989). The plan calls for a microcomputer based central control facility which will be used to monitor and provide operational control over facilities in the system such as booster pumps, wells and reservoirs. These functions will be performed at the master station computer which is proposed to be located in the expansion to the DE plant building.

The control philosophy at the SCADA master plan is classified as "distributed local automatic control with centralized supervisory control and data acquisition". Local automatic control will be retained at each of the individual facilities in the water system so that they can function independently without reliance on the SCADA system.

The selected system configuration was determined through consideration of the City's functional requirements, the type and location of water system facilities, and the control philosophy. The recommended system will include the following primary elements:

- Programmable controllers located at pump stations and wells.

- Data acquisition equipment at reservoirs to transmit level and altitude valve data to the associated pump station controller.

- A central SCADA Master Station Computer, with operator workstation, data storage devices and printer(s).

- A communications facility to allow the above equipment to share data and respond to supervisory commands.

Reference is made to the water system SCADA master plan for additional detail.

D. NORTH BAY REGIONAL WATER TREATMENT PLANT

The following text provides a brief description of new facilities at the North Bay Regional (NBR) Water Treatment Plant.

The NBR water treatment plant is designed to treat water from the North Bay Aqueduct and Putah South Canal for both Fairfield and Vacaville. The initial capacity of the plant is 40 mgd with 26.7 mgd for Fairfield and 13.3 mgd for Vacaville. The plant is designed to facilitate future expansion in anticipation of increasing water demands in both cities. The ultimate treatment capacity is 60 mgd and the ultimate hydraulic capacity is 90 mgd.

The NBR plant is a gravity filtration plant with disinfection by ozonation and chloramination. Water from both Putah South Canal and the North Bay Aqueduct is pumped into the pre-ozonation contact basin for initial disinfection. This process is followed by coagulation, flocculation and sedimentation. Effluent from the sedimentation basin is directed to dual media gravity filters which utilize granular activated carbon and sand. The filtered water is disinfected for a final time by ozonation and/or chloramination before being pumped into the distribution system. Provisions are also designed for fluoride addition.

The booster pumping station will have three 5 mgd pumps and two 2.5 mgd pumps dedicated to Vacaville. This will provide a firm capacity of 15 mgd.

E. RECOMMENDED ADDITIONAL TREATMENT CAPACITY

The recommended time for expansion of Vacaville's capacity beyond 13.3 mgd in the NBR Plant is based on the projected peak day demands presented in Section 3 (E), Supply Demand Analysis and shown in Figure 3-2. In this section it was projected that demands will exceed firm production capacity between 1997 and 2005. However, it is recommended that firm production capacity be at least 1.1 to 1.2 times the maximum day demand (i.e. 10% to 20% reserve capacity). This would indicate that an expansion would be needed about 1995 depending upon the growth rate. Herein we have assumed that a 6.7 mgd increment is constructed in 1995.

After the proposed 1995 NBR expansion, additional treatment capacity will be needed as the City continues to grow and expand its service area. It is estimated that the next expansion would be needed by the year 2000 assuming a moderate growth rate. A 10 mgd expansion is assumed.

It is possible that all additional capacity may be made available at the NBR plant, but other options may have to be considered depending upon the location of new sources of water that could become available to the City.

If the recommendations in the 1990 Disinfection and Operation Study call for abandoning the DE plant then this capacity should be replaced immediately.

The 10% to 20% reserve capacity is recommended for several reasons. First of all, there can be substantial lead times for funding, design and construction of new facilities, and it is desirable to have sufficient time to perform these tasks properly. Also, growth projections are approximate in nature and actual growth patterns can change quickly. If planning and design are started for a plant which is needed in four years based on a moderate rate of growth, and high growth occurs and/or several larger water users enter the service area, then the reserve capacity would be needed to accommodate these conditions. Generally, if the growth rate is lower, then 10% reserve capacity should be sufficient, but in higher growth conditions the 20% is more appropriate.

6. DISTRIBUTION SYSTEM STORAGE

The main functions of storage are to equalize the distribution system pressure, to hold water for fire or other emergency conditions and to provide water supply during periods when demands exceed pumping capacity. The following chapter addresses the ability of the storage facilities to perform these functions. Existing facilities are identified first, and then the future requirements are estimated.

A. EXISTING FACILITIES (AND FACILITIES UNDER CONSTRUCTION)

The main focus in this section is on the Zone 1 reservoirs. The reservoirs in the higher pressure zones are discussed more fully in Section 8. A list of the existing storage facilities and those under construction is given in Table 6-1 along with pertinent data for each reservoir. Locations of the reservoirs are identified in Map 1 contained in the back pocket of this report, and a schematic representation of the reservoir relationships to the distribution system is shown in Figure 1-1.

1. Zone I Storage

On Butcher Road two reservoirs provide a combined capacity of 6.0 MG. The larger 4.0 MG reservoir is connected to a 24 in. main and the smaller 2.0 MG reservoir is connected to an 18 in. main. Before feeding into the distribution system, the 24 and 18 in. mains are connected, so both reservoirs essentially function as a single larger reservoir.

As discussed further in Section 7, the Butcher Road reservoirs are not tied into a strong portion of the distribution system, and water from the reservoirs cannot easily flow to the central or northeast portions of the city. Therefore, these reservoirs are somewhat limited in their ability to help meet peak demands. However, the volume in these reservoirs is very important in terms of meeting emergency needs.

In contrast to the Butcher reservoirs, the 2.0 MG reservoir on Buck Street is tied into a strong portion of the system. This reservoir connects into an 18 in. pipe which is directly tied into the center of town and the water production facilities on Elmira Road. As evidenced by observations from the city staff and confirmed by the computer model, the Buck reservoir typically fills and empties more quickly than the Butcher reservoirs. Therefore, the Buck reservoir is better at meeting operational storage needs.

Currently under construction is a 5 MG reservoir referred to as the Browns Valley Reservoir. It is located on the north side of Vaca Valley Parkway to the west of Browns Valley Road. This reservoir will be well tied into the distribution system with a 24 in. pipe between the reservoir and the central portion of the City.

2. <u>Elevated Pressure Zone Storage</u>

As shown in Table 6-1, there are three main elevated pressure zones in Vacaville referred to as Wykoff, Vine and Hidden Valley pressure zones. The existing Vine, Wykoff and Hidden Valley reservoirs have substandard storage volumes but the Vine reservoir proposed to be constructed in 1990 will meet the criteria listed in Section 4. The facilities in the elevated pressure zones are discussed in greater detail in Section 8.

B. ZONE I RECOMMENDATIONS

In the following paragraphs the required volume of storage in the Zone 1 system is estimated using projected 20 year water demands. The deficit between the needed and existing volumes is determined, and recommended dates for addition of new reservoirs are presented.

1. <u>Required Storage Volume</u>

As discussed in Section 4, distribution system storage must be able to meet operational, fire and emergency needs. The total capacity of all reservoirs in the Zone 1 system must equal the sum of volumes required for each of these three categories of use.

The function of operational storage is to supply or receive water when production (pumping) cannot or does not match demands. It is often advantageous to use operational storage to meet peak hour demands because the required storage volume is usually less expensive than the firm pumping capacity to meet these peaks. Normally, firm production (pumping) capacity is designed to meet the maximum day flow rate, and storage handles the diurnal fluctuations on that day. However, with this operating scheme it is imperative that the production (pumping) capacity does not fall below maximum day demands as demands increase from year to year. Otherwise, an "artificial" storage deficit may be created. This occurs when the reservoirs cannot be filled overnight following near maximum day demands. With several such consecutive days, the reservoir level will cycle at continually lower and lower levels creating an "artificial" storage deficit. It is considered an "artificial" deficit because it is the production (pumping) capacity which is deficient and not the storage volume.

The volume of operational storage needed to meet peak hour demands will theoretically be about 15% of the maximum day demand. However, it should be recognized that the theoretical value will only be adequate if: 1) there are no inefficiencies in supply pump operation and sequencing and 2) the reservoirs are completely full when the peak demands begin. In most cases 20% or more will actually be needed. Twenty-five percent (25%) has been specified in Section 4 as the recommended design criteria.

TABLE 6-1

DISTRIBUTION SYSTEM STORAGE FACILITIES

Reservoir	Bottom Elevation (feet)	High Water Level Elevation (feet)	Capacity (MG)
ZONE 1			
Butcher No. 1	302.5	323.0	2.0
Butcher No. 2	291.0	324.0	4.0
Buck	303.2	323.2	2.0
Browns Valley (to be constructed in 1990)	292.0	324.0	5.0
ELEVATED PRESSURE ZONES			
Wykoff	466.5	476	0.12
Tranquility Pressure Tank	--	--	--
Vine (planned to be constructed in 1990)	511.0	535.0	0.62
Vine No. 1 (planned to be taken out of service in 1990)	249	285	0.374
Vine No. 2 (planned to be taken out of service in 1990)	470	488	0.056
Hidden Valley	343.0	363.0	0.073

The amount of water needed during any fire should be available in storage at all times. As discussed in Section 4, the maximum requirement is assumed to be 4,500 gpm for a duration of 4 hours. Thus, the amount of fire storage needed is 1.08 million gallons. This volume should remain constant in the future unless large buildings with greater fire flow requirements are constructed.

Emergency storage is needed to provide water during periods when the normal supply is cut off (i.e., wells, treatment plants). Such conditions may be a power failure, loss of raw water supply, pumping equipment or pipeline failure, or the need to take facilities out of service for repair. The amount of emergency storage required should be determined from the most severe emergency condition which is expected.

The amount of emergency storage available varies greatly from one city to the next. For example, Fairfield has a goal of more than four average days of total storage while the goal in Vallejo is about 25% of a maximum day. Alameda County Water District has about 2.2 average days storage and Sacramento has about 0.6 average days.

It is expected that the worst emergency condition would be a power outage. PG&E records for Solano County between 1980 and 1985 indicate an average of 1.27 outages per year with an average duration of 103 minutes. Normal maximum durations are about 4 hours, but in extreme cases a duration of 12 hours is reasonable to expect (Huemmer, 1985). Currently, a loss of raw water supply from Putah South Canal could create a critical emergency condition. However, when new surface water supplies from the North Bay Aqueduct become available, the risk of having a cutoff of all surface water supplies will be greatly reduced.

In order to determine the amount of emergency storage required we have assumed that power is out for 12 hours under maximum day demand conditions at the NBR plant and in Vacaville, and that only facilities with standby power can supply water. Under this condition the DE plant could supply water at a rate of 5.0 mgd and well no. 8 could supply at a rate of 2.2 mgd. Therefore, during the 12 hour period, a total of 3.6 MG could be produced.

The total operational, fire and emergency storage requirements are given in Table 6-2 based upon the moderate growth projections shown in Table 3-4. Also shown is the existing capacity and the deficit between existing and needed capacity. Based on the information displayed in Table 6-2, it is evident that the existing facilities including the Browns Valley Reservoir result in a deficit of about 3.21 MG.

We feel that an appropriate size for Zone 1 reservoirs is between 5 and 10 MG because significant economies of scale will be realized in this range. Actual sizes should be determined during preliminary design of such facilities taking into consideration site constraints, aestethics and availability of funds. Herein we have assumed that 5 MG tanks are constructed every 5 years beginning in 1992.

2. Reservoir Location

In selecting future reservoir sites, appropriate land elevations are a primary consideration. In order that all of the Zone 1 reservoirs function together, their overflow elevations should be similar. Other important considerations are visibility, location within the distribution system pipe network, and cost. Cost differences between sites will be primarily be a function of the pipe length required to connect with the distribution system, site grading requirements and access.

TABLE 6-2

REQUIRED STORAGE VOLUME IN THE ZONE 1 DISTRIBUTION SYSTEM
(Million Gallons)

Year	Operational[1]	Fire[2]	Emergency,[3]	Total	Existing	Deficit
1990	6.24	1.08	8.89	16.21	13.00	3.21
1995	7.61	1.08	11.62	20.31	13.00	7.31
2000	8.98	1.08	14.35	24.41	13.00	11.41
2005	10.34	1.08	17.09	28.51	13.00	15.51
2010	11.71	1.08	19.82	32.61	13.00	19.61

[1] Operational = 25% of maximum day demand
[2] Fire = 4,500 gpm for 240 min
[3] Emergency = 50% maximum day demand, minus 3.6 MG
[4] Moderate growth rate assumed.

Two of the reservoir sites identified in the 1986 Draft Master plan (herein referred to as sites A and B) will offer distinct advantages however, several other possibilities are also available. Site A is located near Foothill Drive west of the City limits. Site B is located in northern Vacaville to the north of the Browns Valley Reservoir. Other areas for consideration will include the hills west of Pleasants Valley Road, and the hills in southwest Vacaville.

Reservoir site A will be particularly useful if the North Orchard Avenue and Fruitvale Road area is not converted to higher pressure zone (Zone 2), because a reservoir at site A could help to moderate Zone 1 pressure drops under high demand conditions. However, in order to make a reservoir at site A function properly, substantial pipeline improvements would be necessary to make a strong connection with the distribution system.

Although reservoir site B is relatively close to the Browns Valley Reservoir it is not a bad location because substantial storage volumes can be utilized by developing areas in the northern portion of the City. Northern Vacaville is hydraulically and physically remote from the sources of water supply thereby making the flow equalization function of storage reservoirs even more important than in other areas of the City.

3. Existing Reservoir Inlet/Outlet Piping

The existing piping on the two Butcher Road reservoirs and the Buck reservoir is arranged with both inflow and outflow through a single pipe. The problem with this arrangement is that it does not promote mixing of the tank contents, and if there is not good mixing, there is a risk of having zones in the tank where the water warms up and loses chlorine residual. Therefore, it is suggested that the inlet and outlet piping be modified to promote good mixing. Specifically, the outlet should be from the tank bottom and the inlet should be in the upper portion of the tank on the opposite side from the outlet.

In order to make the suggested modifications, structural and mechanical improvements will be needed within the valve vaults, and pipe will have to be extended to the back of the reservoir and through the tank shell.

7. DISTRIBUTION SYSTEM

The following section addresses the adequacy of the Zone 1 distribution system to meet existing and future water demand conditions. The first section describes the layout of the existing pipe network and presents some of the overall conceptual deficiencies of the network. Next, the hydraulic model used to simulate conditions in the pipe network is described in terms of its principle of operation and the specific inputs used to model Vacaville's water system. The remainder of the chapter is dedicated to formulation of specific recommended improvements.

A. EXISTING FACILITIES

The water distribution system is depicted on Map 1 contained in the back pocket of this report. The system shown on this map consists of 10 in. and larger pipes and some smaller pipes which function in critical loops in the distribution system.

The distribution system has been divided into four functional sectors as defined in Figure 7-1. The pipes which connect the sectors together are very important in terms of operation of the system as whole because they can limit the ability to convey water across town. In the following paragraphs, the overall characteristics and inherent problems in each sector are discussed.

1. Production Area

The water production area is located around Elmira Road east of Interstate 80. Water supply from all wells and the DE plant originates in this area. The DE plant feeds into a 30 in. pipeline on Elmira Road, which runs west toward I-80, then reduces to a 24 in. and continues toward the center of town. This pipeline is one of the strongest portions of the distribution system.

The well field lies in the eastern half of the production area. (See Figure 3-1). Well Nos. 5, 7, and 8 are all tied together by a common 18 in. pipe, and Well Nos. 2 and 3 and the Fallen Leaf Well are closely tied to the area.

2. Northeast Sector

The Northeast Sector contains the largest land area of any of the sectors. This area consists of relatively flat, lower elevation (typically 80-150 ft) land which is designated in the General Plan with a predominance of industrial and commercial development. Much of the land is undeveloped, so current demands are relatively small, but future demands are potentially very large.

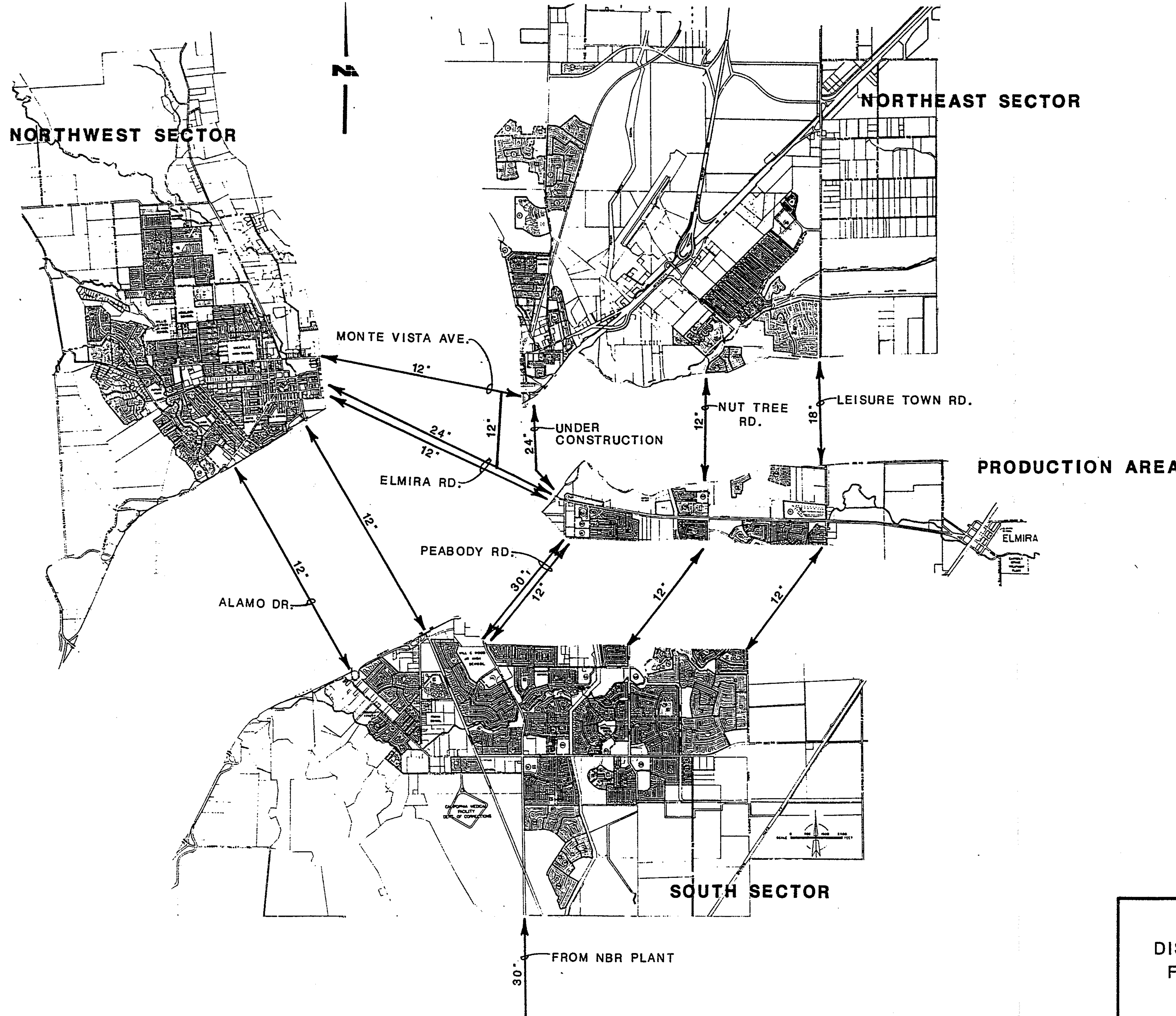


FIGURE 7-1

DISTRIBUTION SYSTEM FUNCTIONAL AREAS

In Figure 7-1, it is apparent that the Northeast Sector is marginally connected to the production area. The only connections to the northeast sector are the 12 in. pipeline on Nut Tree Road, the 12 in. pipeline on Monte Vista Avenue and the 18 in. pipeline on Leisure Town Road. A 24 in. pipeline on Browns Valley Road is currently being constructed which will extend from the intersection of Elmira Road and Peabody Road. This pipeline will greatly increase the transmission capacity into the northeast sector.

Within the Northeast Sector, 12 in. pipelines form the majority of the loops in the system. In general, the existing network in the northeast sector can meet existing demands, but substantial improvements will be needed as development occurs.

3. Northwest Sector

The Northwest Sector contains the central business district and residential development at the base of the English Hills. The Northwest Sector includes areas with relatively high building pad elevations making portions of this area subject to low pressure problems. Particularly important with respect to pressure is the Fruitvale Road and North Orchard Avenue area where building pads are as high as 230 ft in Zone 1. Because the water level in Zone 1 storage reservoirs can drop to around 300 ft, the typical static pressures can be as low as 30 psi. With headloss in the system, pressure can drop well below 30 psi.

All of the higher pressure zones above Zone 1 are located in the Northwest Sector including the Vine, Hidden Valley and Wykoff systems. These systems are described in Section 8.

In Figure 7-1 it is apparent that the Northwest Sector is tied to the production area by 24 in. and a 12 in. pipelines on Elmira Road. The capacity in the 24 in. pipe is relatively large, so transmission into the northwest sector is not a problem. In the central business district, the 24 in. pipeline reduces to 18 in. and heads west providing a strong leg supplying this sector. The Northwest Sector is tied to the south sector with two 12 in. pipes and is tied to the northeast sector with a 12 in. pipe on Monte Vista Avenue.

An important component of the distribution system in the northwest sector is Buck reservoir. It is important because during peak hour demand conditions, the reservoir will function as temporary water supply to the system. Thus, short-term (i.e. hourly) transmission requirements from the production area to the Northwest Sector are reduced.

4. South Sector

The South Sector is mainly composed of residential development on relatively flat, lower elevation land (typically 80-150 ft). The Correctional Medical Facility (CMF) is also located in the south sector. Historically, this facility has exerted a large water demand but construction of their own treatment plant has reduce demands on the city water system significantly, although demands up to 1 mgd from CMF will still occur during the summer months.

The South Sector is currently connected to the production area with three 12 in. pipes (Figure 7-1) and a 30 in. pipe from the NBR treatment plant. These connections provide good transfer capabilities between the production area and the south sector.

The South Sector contains the two Butcher reservoirs. As in the Northwest Sector, the reservoirs serve as a supply source during short-term (i.e., hourly) peak demands.

B. DEVELOPMENT AND APPLICATION OF THE HYDRAULIC MODEL

The ability of the distribution system to meet the existing and future water demands has been evaluated by computer simulation of the water supply pumps, reservoirs and distribution pipelines. The model was used extensively in preparation of the 1986 Draft Water System Master Plan, but has not been employed for subsequent use in this 1990 Master Plan. The analysis for the 1986 Draft is current enough to provided an understanding of the distribution system behavior and to plan improvements herein. However, we do recommend that the model be updated as soon as the 1990 General Plan Revisions are complete. Critical tasks in updating the model will include calculation of water demands throughout the system and addition of new pipelines and other facilities.

Originally, the model was set up on a main frame computer system using the HYNET program. However, in 1987 we adopted the model to use on personal computers with the KYPIPE program.

1. Principle of Operation

In the hydraulic model, flow, headloss and pressure are simulated in the distribution system. At each pipe junction (node) a mean sea level elevation and a water demand are assigned. Reservoirs are represented in the system as nodes with constant head, and pumps are modeled with equations representative of their pump curves.

The headloss in each pipe is computed as a function of flow rate with the Hazen-Williams pipe flow formula listed below.

$$Q = 0.281\ C\ D^{2.63}\ S^{0.54}$$

Where:

Q = flow (gpm)
C = Hazen-Williams coefficient
D = pipe diameter (in.)
S = hydraulic grade (ft/ft)

The result of the program is values for flow rate in each pipe and pressure at each node in the system at one instant in time. Dynamic situations can also be evaluated with the KYPIPE version of the model.

2. Distribution System Map

The distribution system map (Map 1), as presented earlier in this report and contained in the back pocket of this report, was developed for use with the model. By no means is the map all inclusive of the distribution system pipes, as it is considered a "skeleton" of the distribution system. The map includes 10 in. and larger and some 8 in. and 6 in. pipes which form important links in the network.

In certain cases the map does not exactly resemble the existing conditions. At some locations the relative positions of one pipe to the next have been changed in order to visually simplify the situations. On Elmira Road such locations include the area near the well field and the treatment plant. Some pipes in the central business district have also been shifted for the purpose of simplifying the node map. Therefore, it is important that as-built drawings of the water system be used for design of improvements.

Pipelines in the elevated pressure zones have not been shown on the distribution system map. In these cases, demands from the elevated pressure zones are assumed to be exerted where the booster pumps withdraw water from the Zone 1 system.

3. Water Demands in the Distribution System

The following paragraphs present our methodology for estimating water demands and distributing these demands to nodes throughout the system. Map 3 presents the land uses which were used to estimate demands.

Each type of land use was assigned a water demand factor (gpd/acre) which represents annual average demand. Water demand factors were determined by comparison of demands in other areas in California and standard per capita use rates in conjunction with an evaluation of the water use environment in Vacaville and existing city records. The demand factors used herein are given in Table 2-1. These factors represent 100% levels of development including any land needed for roads etc.

With the demand factors in Table 2-1, it is very important to recognize that only rough estimates of actual demands are provided. Over large areas, differences from actual conditions are expected to become insignificant, so for purposes of the distribution system model, they are appropriate. However, when estimating demands from smaller areas, they should be used with caution and consideration must be given to site specific conditions.

The categories of land use in Table 2-1 correspond to General Plan land uses except in two cases. Development in Lagoon Valley is given the symbol "CL", and the hospital north of Elmira Road is given the symbol "H."

In most locations in Vacaville, the current level of development is less than 100%. The actual level of development was estimated by overlaying the land use map on an aerial photograph of the city and estimating the fraction of land occupied.

Water demands from each land use area were determined from the land use type, the fraction of land occupied, and the land acreage. For example, the residential area (RMD) north and west of Vacaville High School has an area of 219 acres. For RMD type development the demand factor is 2,300 gpd/acre and the estimated level of development is 100%. Thus the demand for this zone is 503,700 gpd (219 x 2300 x 1.00).

Demands from each land use area were apportioned to the nearest nodes in the distribution system and then total demands at each node in the network were determined.

To estimate future demands for the model, the levels of development were increased in each land use area throughout the city. For any particular zone, the increase in water demands was proportional to the difference between the existing and build-out levels of developments. Therefore, a zone which is 90% developed will have small increases in demands, while a zone at 20% development will accrue demands at a much faster rate.

C. DISTRIBUTION SYSTEM ANALYSIS PROCEDURES

The following paragraphs address calibration of the model and criteria used to evaluate system deficiencies.

1. Calibration of the Model

Before analysis of the distribution system, the model was checked for accuracy. This task was achieved by modeling the existing flow rates and comparing the results with actual system operation. For the calibration, a total demand of about 20.2 MGD was used. The results yielded pressures in the northwest portion of Vacaville near North Orchard Avenue from 30 to 40 psi, while pressures to the southeast of the water treatment plant were estimated in the range of 95 to 115 psi. The results also indicated that Buck reservoir received water preferentially to the Butcher Road reservoirs.

Actual system pressures are similar to those predicted by the model. In the northwest part of town, maximum day pressures near 40 psi are reported with values as low as 25 psi during shorter time intervals. Also, pressures near 110 psi are known to exist to the southeast of the treatment plant. Furthermore, the Buck Street reservoir does provide water to the system, preferentially to the Butcher Road reservoirs. Based on these and other comparisons, the model was determined to correlate well with the existing conditions.

2. Guidelines for System Performance

Adequacy of the distribution system was evaluated in terms of residual pressure at all nodes and headloss in the pipes. It is important that pressures are high enough to effectively supply water to the users, but not so high as to cause excessive water loss from the system and maintenance problems with service plumbing. Headlosses in pipes should be kept low so that energy requirements for pumping are not excessive. Minimizing pipeline flow velocities is also important in controlling surges in the system. Guidelines are listed in Table 7-1.

D. ANALYSIS OF THE DISTRIBUTION SYSTEM

Following is a summary of the results from our analysis of the distribution system based on the computer modeling work performed in conjunction with the 1986 Draft Water System Master Plan.

In our analysis, peak hour demands and maximum day demands with fire flow were the most common conditions which were tested. Peak hour demands represent the largest demand situation and are therefore very useful in evaluation of overall system deficiencies, such as the ability of the water supply to distribute itself toward the extremities of the pipe network, and the reservoir fill and draw characteristics. Maximum day demands with fire flow are very useful in the evaluation of localized system deficiencies.

1. 1986 Analysis

Demand conditions for 1986, were evaluated for maximum day flows with all of the well field in operation and the DE plant at nearly full capacity. The results of the hydraulic model showed very high pressures (in excess of 105 psi) in the vicinity of the well field. Part of the reason for the high pressures is the low ground elevation in the vicinity of the well field, but piping restrictions resulting in "backpressure," were also a contributing factor. Since 1986 some of the restrictions have been eliminated, and there is now a less severe restriction problem.

The hydraulic model also pointed out potential problems in the Northwest Sector of the city in the vicinity of Fruitvale Road and North Orchard Avenue. On the northern end of North Orchard Avenue pressures dropped below 30 psi under peak hour conditions. Pressures in this area are expected to be low, as a result of the predominant elevations, but headloss in the non-looped 12 in. pipe on North Orchard Avenue was also a contributing factor.

The 1986 analysis included evaluations of the loss of power condition and the recovery period. Under the loss of power condition, Well No. 8 and one small booster pump at the DE plant were assumed to be on during maximum day demands. The results of the hydraulic model showed a

TABLE 7-1

DISTRIBUTION SYSTEM PERFORMANCE GUIDELINES

CONDITION		COMMENT
RESIDUAL WATER PRESSURE		
psi	feet	
<20	<46	Not acceptable under any conditions
20-30	46-69	Acceptable in fire or emergency conditions only
30-88	69-204	Recommended pressure range
>88	>204	High Pressure
HEADLOSS feet per 1,000 feet		
<5		Low Losses
5-10		Moderate Losses
>10		High Losses
PIPE VELOCITY feet per second		
>5		High Velocity

significantly greater flow rate out of Buck reservoir than out of the Butcher reservoirs. Also, pressures at nodes to the north, south and east extremities of the system were lower than under normal operating conditions. This trend is explained by the relatively high head losses in pipes leading from the reservoirs, and the low input from the production area.

Under the recovery condition, enough wells and supply pumps were on to meet maximum day demands in 1986, but only average day demands were exerted on the system and the reservoirs received the excess water. The results of the hydraulic model showed greater flow into Buck reservoir than the Butcher reservoirs. Pressures on the east side of town were very high with reported values greater than 110 psi in many cases. Pressures on the west side of town were generally good.

2. 1995 Analysis

The 1995 analysis indicated the following deficiencies in the existing distribution system:

- The connection between the DE plant and the well field is inadequate. In particular, the 12 in. pipe on Elmira Road, west of Nut Tree Road should be paralleled with an 18 in. pipe. (This work is currently being done)

- Fire flows and peak hour demands could not be met on the northern end of North Orchard Avenue.

- The Butcher reservoirs and Buck reservoir do not function simultaneously as water preferentially flows in and out of Buck Reservoir.

- The northeast sector is weekly tied into the production area and new transmission pipelines are needed into the northeast sector. (Improvements to reduce this problem are currently underway)

3. 2005 Analysis

Once improvements needed to accommodate 1995 conditions were input into the computer model, water demands were scaled up to the year 2005 condition, and the distribution system was tested again. This analysis predicted that the following deficiencies will occur.

- Residual pressures in the northeast sector will be very low under peak hour conditions and additional transmission capacity into this area will be needed as the large amount of undeveloped land in this sector is built out.

- In the southeast portion of the city, velocities and headloss in the pipelines will be high, but residual pressures will still be acceptable as a result of the lower ground surface elevations in this area.

4. Post 2005 Analysis

As a final analysis condition improvements were made to the system to meet 1995 needs, and the demands were once again increased. (The demands were equated to 80% of the buildout.)

Under maximum day demand conditions there were no apparent deficiencies. Residual pressures at all locations were acceptable, and headlosses in the pipes were not high. However, when the peak hour demands were tested, the deficiencies with transmission in the Northeast Sector of the city were apparent.

E. RECOMMENDED TRANSMISSION IMPROVEMENTS

Following is a discussion of the recommended "transmission" improvements to the distribution system. A transmission improvement is loosely defined as one which increases the ability to convey water from one region of the City to another. In contrast, a "local" improvement primarily functions within an individual region.

1. Changes in Elmira Road

The connection between the DE plant and the well field should be strengthened. In particular, the 12 in. pipe on Elmira Road, west of Nut Tree Road should be paralleled with an 18 in. pipe. With this improvement, the piping connections in Elmira Road, adjacent to the DE plant, should also be changed because the current configuration has (1) minimal ability to isolate flows, (2) undesirable ties between the main distribution system and wells 4 and 6 and (3) non-operational valves.

Construction of these improvements is currently complete.

2. North Orchard Avenue Area

One of the most critical areas in Vacaville in terms of the ability to meet fire flows and peak hour demands is on the north end of Orchard Avenue. Alternatives to help solve the problem by creating pipeline loops were suggested in the 1986 Draft Water System Master Plan. These improvements were referred to as the "Fruitvale Loops". The current concept is to create a new pressure zone (Zone 2) to improve service to the North Orchard Avenue area. This concept is discussed in more detail in Section 8, E, Proposed Zone 2 System.

It is recognized that creation of a new pressure zone in the North Orchard Avenue area is an expensive proposition and may not be feasible unless improvements can also be used to serve development in the West Valleys. If a new pressure zone is not created, then pipeline improvements to add loops to this area should be made. An abbreviated description of the loop alternatives is provided below.

Pipeline improvements to serve the North Orchard Avenue area (in absence of Zone 2) should include extension of a new 18 in. pipe up to Fruitvale Road. Two alignments are possible but the first is preferred:

a. From the junction of Fruitvale Road and North Orchard Avenue, extend the pipe east to Gibson Canyon Road then south past Monte Vista Avenue to the 18 in. pipe on Kendal Street. This new pipe should also be tied in with the existing 12 in. pipe on the corner of Monte Vista Avenue and Gibson Canyon Road.

b. From the junction of Fruitvale Road and North Orchard Avenue, extend the pipe east about half way to Gibson Canyon Road and then south on Stinson Avenue, east on Monte Vista Avenue and south on Chestnut Street to the existing 18 in. pipe on Kendal Street.

The first alignment, although more expensive, is preferred to the second in terms of system operation as it will make a stronger loop.

Improvements in the North Orchard Avenue area should also include closing of the loop on Fruitvale Road and North Alamo Drive with a 12 in. connection. Completion of this loop will not alleviate the pressure problems on North Orchard Avenue as well as the Gibson Canyon-Fruitvale loop, but it will strengthen the system and produce an additional 1 to 2 psi on the north end of North Orchard Avenue under peak hour conditions.

To meet the fire flow conditions in the North Orchard Avenue area, further improvements will still be needed even with the proposed loops. Additional improvements should include a parallel 12 in. pipe on North Orchard Avenue to the north of Fruitvale Road. The existing 12 in. pipe in this reach is adequate to convey peak hour demands but not a fire flow.

3. Browns Valley Road

It is recommended that a 24 in. pipe be constructed from the junction of Elmira and Peabody roads north to Browns Valley Road and to the new Browns Valley Reservoir. This pipe will effectively link the production area to the Northeast Sector of the city and will allow the new Browns Valley Reservoir to be fully utilized.

Design of this improvement is currently complete and construction should be complete in 1990.

4. North Browns Valley Road

Transmission capacity into the northern industrial area will need to be improved. Therefore, it is recommended that an 18 in. pipe be extended from Browns Valley Road at Glen Eagle Way, north, then east on Vaca Valley Parkway to Eubanks Drive.

5. Peabody Road

The pipe from the NBR plant to the intersection of Peabody Road and Elmira Road has a section which is only 24 in. This section extends south from California Drive for about 3,500 ft. The 24 in. size is acceptable with the first phase of construction at the NBR plant but will need to be strengthened before the second phase of improvements at the NBR plant. A 36 in. pipe is recommended at this time.

6. North Leisure Town Road

Recently an 18 in. pipe was constructed in Leisure Town Road from Elmira Road to Yellowstone Drive. This improvement allows water be to conveyed out of the production area; but where the 18 in. ties into the 12 in. at Yellowstone Drive, the smaller pipe begins to limit transmission further to the north. This limitation will have increasingly greater impacts as demands increase in the northwest sector. Therefore, it is recommended that a parallel 18 in. pipe be placed along Leisure Town Road from Yellowstone Drive to Vaca Valley Parkway.

7. Monte Vista Avenue

Transmission between the functional sectors in the city is very important. Therefore, it is suggested that the Northeast and Northwest sectors be connected with a parallel 12 in. pipe on Monte Vista Avenue between the proposed 24 in. Browns Valley pipe (see Improvement No. 3) and Gibson Canyon Road. This improvement will provide benefit during periods when one sector is stressed and the other is not (e.g., a large fire).

If Zone 2 is implemented in the North Orchard Avenue area, the importance of this improvement needs to be re-evaluated with the computer model.

8. Nut Tree Road and Interstate 505

Additional transmission capacity into the Northeast Sector will be needed to meet future demands. It is recommended that a new 18 in. pipe be placed along Nut Tree Road and Interstate 505 between Elmira Road and Vaca Valley Parkway.

9. Parallel NBR Pipeline

The existing 30 in. pipeline from the NBR plant will handle part but not all of the proposed 1995 expansion of the NBR plant (13.3 mgd to 20.0 mgd). Therefore, a parallel pipeline will be needed within a few years after the 1995 plant expansion.

10. Pipelines for the West Valleys (Lagoon Valley, Pleasant Valley)

This report does not address pipelines needed to convey water for the West Valleys. If development in these areas occurs, pipelines will be needed from the extremities of the existing system to and throughout the West Valleys. However, water demands in these areas will also necessitate strengthening the pipelines in the existing system. Such improvements might include a new transmission main beginning at Peabody Road and Alamo Drive, and following Alamo Drive to Buck Avenue.

F. RECOMMENDED LOCAL IMPROVEMENTS

Following is a discussion of several recommended "local" improvements to the distribution system. These improvements generally function to provide water service with individual regions of the City and are not intended as major "cross-town" transmission facilities. Additional local improvements may be added to the list as development plans for new areas are made.

By their nature, local improvements will only be needed as development proceeds in the vicinity of the improvements. Therefore, it is difficult to predict an exact date when such improvements will be necessary. We suggest that these improvements be implemented with the first phases of development in a given area and in conjunction with street improvements to the extent possible.

1. <u>Gonsalves-Locke Area</u>

In 1987, Nolte and Associates used the computer model to identify appropriate sizes for pipelines in the Gonsalves-Locke Property located south of Alamo Drive and east of Peabody Road. The recommended improvements include a southern extension of the existing 12 in. pipeline on Nut Tree Road plus two east-west 12 in. pipelines through the property.

A portion of these improvements have already been made.

2. <u>Akerly Drive</u>

In the northeast sector of the city, south of Vaca Valley Parkway a 12 in. pipeline is recommended for completion of the loop on Akerly Drive.

3. <u>Leisure Town, Midway, Eubanks Loop</u>

As industrial development proceeds in the northeast sector a 12 in. loop will be needed, north along Leisure Town Road to Midway Road, then west across I-505 to Eubanks Drive, then south to the existing 12 in. pipeline. Larger pipelines along Eubanks Drive would be needed to provide fire flows on an interim basis if the loops are not completed and this area develops first.

4. <u>Midway Road Pipeline</u>

The extreme northeastern portion of the study area is currently served by a single 12 in. main. A single supply pipeline is not a preferred arrangement, and this situation should be improved by constructing a 12 in. pipeline on Midway Road from Leisure Town Road (connected to future Leisure Town, Midway, Eubank Loop) to Meridian Road.

5. <u>Allison Drive</u>

Allison Drive is a proposed road to the east of and parallel to Browns Valley Road between Vaca Valley Parkway and Monte Vista Avenue. When this road is constructed 12 in. pipeline should be installed to serve future industrial demands.

6. <u>Other Local Improvements</u>

As specific developments are proposed, the need for additional local improvements will be determined.

G. RECOMMENDED DOWNTOWN IMPROVEMENTS

In the downtown portion of Vacaville, smaller pipes currently play an important role in the system operation, and improvements in this area are necessary. The following paragraphs address a few of the more important needs. The importance of the improvements is largely based on input of the city staff, and selection of which improvements are to be done first is left to the city's discretion.

Recently, an improvement project was completed to correct some of the deficiencies in the downtown area. This project includes 12 in. pipe on Main Street plus 8 in. pipe in portions of Kentucky Street, Merchant Street, Peach Tree Avenue, Magnolia Avenue and Luzena Avenue.

Between Monte Vista Avenue and Main Street, much of the distribution system consists of 4 in. pipe and some 6 in. pipe. Increased capacity in this area is needed. One possible improvement scenario is as follows. First, increase the pipe capacity on Monte Vista Avenue. As a minimum, the 6 in. pipe between West and Chestnut streets could be replaced with a 10 in. pipe, but a more positive improvement would be to place a 12 in. pipe between Dobbins Street and Orchard Avenue. Second, place a 10 in. pipe on Kendal and Neil Streets between Elizabeth Street and Eldridge Avenue. Third, tie the parallel pipelines on Monte Vista Avenue, Kendal Street and Main Street together.

Elsewhere in the downtown area, 4 in. and 6 in. pipes will have to be replaced with larger sizes over the next 20 years primarily because they cannot provide adequate fire flow. Furthermore, many of the pipes in the downtown area will have to be replaced as they become old and fail structurally. The city staff has noted particular problems with wrapped steel pipe. Additional corrections in the downtown area will also be needed to improve connections between pipes which are not "clean" or have non-working valves.

H. WELL FIELD PIPING

The 1990 Disinfection and Operations Study will address the need to chloraminate water from the wells. The result of this study could lead to bringing water from several or all of the wells to a central location for disinfection. If this is done, substantial pipeline improvements will be needed along with the disinfection facilities at the central location.

8. HIGHER ELEVATION PRESSURE ZONES

A. GENERAL CONSIDERATIONS FOR HIGHER ELEVATION ZONES

In general, serving development in higher elevation pressure zones (proposed Zone 2 and secondary zones) requires relatively high construction costs, and greater operation and maintenance requirements than the same amount of development in the Zone 1 system. Reservoirs in these pressure zones are particularly expensive items because each pressure zone must have fire and emergency storage volumes independently from the Zone 1 system. Furthermore, small reservoirs typically used in these pressure zones are more costly than larger Zone 1 reservoirs in terms of dollars per gallon because of economies of scale. The booster pumping facilities also result in high costs as a result of initial capital costs, annual pumping energy costs, and man hours required for operation and maintenance.

As growth proceeds in Vacaville and the demand to open up new land for development increases, new pressure zones above Zone 1 may be needed despite the higher cost. If developed, such pressure zones must meet the design requirements given in Section 4. Also, the City should consider the additional O&M costs before allowing new zones to be created. Combining several smaller zones into a single zone will result in economics of scale, and should be considered where feasible.

B. VINE STREET WATER SYSTEM

The following description presents the Vine Street water system facilities which have been designed and are planned to be constructed in 1990. For a description of the facilities being replaced or the alternative plans considered before reaching the final design, reference is made to the Vine Street Water System Preliminary Design Report (Nolte, 1989) and the Vine Street and Boulder Park Ranch Water Supply Study (Nolte, 1986).

Plans for the Vine Stret water system were formulated prior to establishment of the criteria in Section 4 of this report. The plans deviate somewhat from the criteria as discussed in the following paragraphs. However, deviations are not considered to be great enough to warrant redesign at this point in time.

1. <u>Service Area</u>

The Vine Street service area includes the land above 220 ft and below 431 ft in elevation as shown in Figure 8-1. The service area is divided into upper and lower zones. Lots between 340 ft and 431 ft in elevation will be served by the upper zone. All lots between 220 ft and 340 ft will be served by the lower zone. One exception is the fire station on the north end of Vine Street. The fire station has an elevation of about 300 ft, and will be served by the upper zone. It is evident that the elevation range in the lower zone exceeds the 100 ft maximum range specified in Section 4. However, this has been allowed due to the difficulty in economically accommodating the existing development in this area. Also, the improvements will reduce maximum service pressure over existing conditions.

As shown in Figure 8-1 the lower zone has been divided into 2 areas designated as "B" and "C". Area B includes land in close proximity to existing and currently planned lower zone pipelines. Land in area C will require construction of additional pipeline and/or pressure reducing facilities in order to make water service available.

2. Reservoir

The reservoir has been sized to store the maximum day demands of 1,200 gpd/unit for maximum buildout of 318 units plus a fire flow of 2,000 gpm sustained for 120 minutes. Therefore, storage capacity for maximum buildout is approximately 620,000 gal. The current fire criteria only require 1,500 gpm for 2 hours, so there is some additional capacity in the reservoir. The tank has a diameter of 68 ft and a height of nearly 30 ft from base to peak. The low water level of the reservoir is 511 ft with a high water level of 535 ft.

The pipeline that supplies the reservoir branches into inlet and outlet pipes as it enters a valve pit. The 12 in. outlet pipe has a check valve to prevent water from entering the reservoir via this pipe. The 8 in. inlet pipe has an altitude valve to prevent overflows in the reservoir.

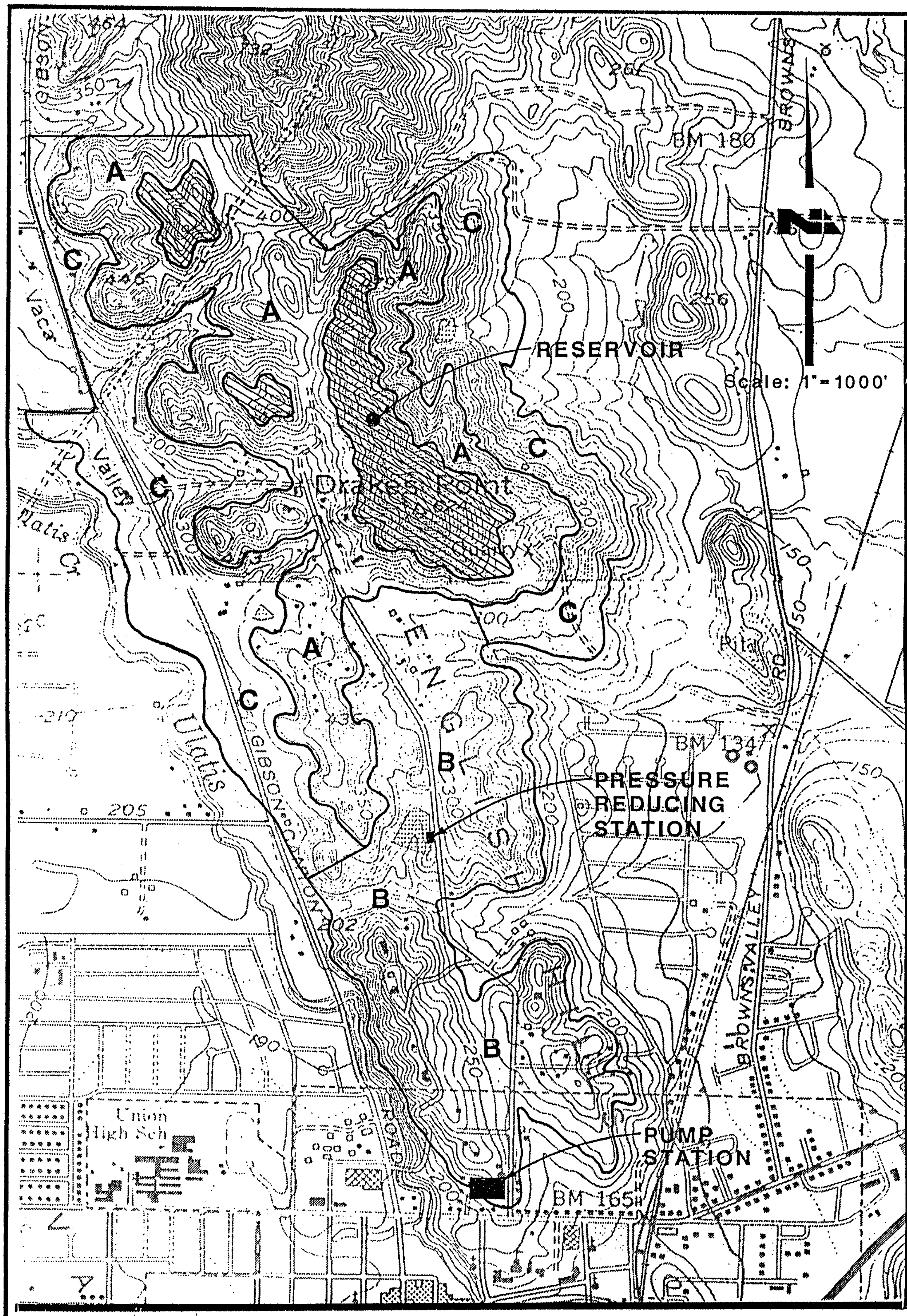
A control system is included which operates the pump and reservoir system utilizing a pressure switch at the reservoir, a telephone telemetry system to the pump station and controls at the pump station. A schematic representation of the water system if given in Figure 8-2.

A backup control system is also provided which relies on the altitude value. During filling of the reservoir, the altitude valve remains open. When the high water level is reached, a 1-1/2 in. overflow pipe fills and exerts a static pressure on the altitude valve which in turn causes it to close. Pressure in the transmission main then rises due to the action of the booster pumps and a pressure sensor at the pump station then signals the pumps to turn off. The 1-1/2 in. overflow pipe slowly drains and after draining, the altitude valve is free to open again.

3. Transmission Pipelines

The water system contains both upper zone and lower zone transmission pipelines as shown in Figure 8-2.

Between the booster pump station and the pressure reducing station 8 in. pipe is used on the upper zone side of the system. In this reach, 12 in. pipe is not necessary because it must only accommodate the design pumped flow rate of 580 gpm and not fire flow. If there were a fire in the southern portion of Vine Street, the water would flow from the reservoir, through the pressure reducing station and through the lower zone's 8 in. pipeline. With this arrangement, a fire flow of 1,500 gpm would be available at the southern end of Vine street. Between the pressure



LEGEND

AREA ABOVE MAXIMUM SERVICE ELEVATION

A UPPER VINE ZONE
Building Pads from 340 to 431 ft.

B LOWER VINE ZONE
Building Pads from 220 to 340 ft.

C EXPANDED LOWER VINE ZONE*
Building pads from 220 to 340 ft.

*WILL REQUIRE ADDITIONAL WATER MAINS AND PRESSURE REDUCING FACILITIES

FIGURE 8-1

PLANNED VINE STREET SERVICE AREA

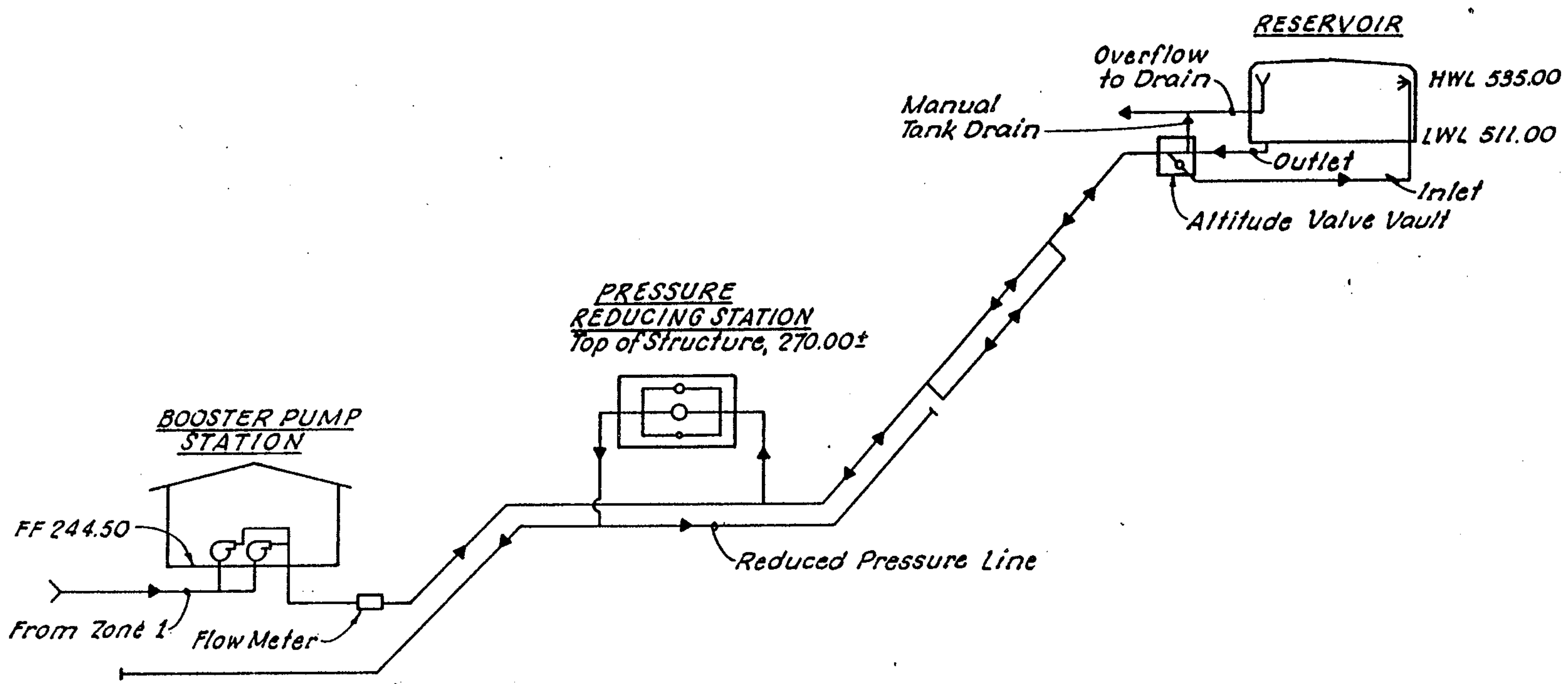


FIGURE 8-2

PLANNED
VINE STREET WATER SYSTEM
SCHEMATIC

reducing station and the reservoir, 12 in. and parallel 8 in. pipelines provide transmission in the upper zone.

The Vacaville Fire Department Standards for hillside development require fire hydrants installed at 300 ft intervals, and static pressure in the hydrants which do not exceed 110 psi. In the southern portions of the service area, the upper zone pipelines could not be used for fire protection because static pressures would be too high for the fire department to handle. Because of this pressure limitation, all fire hydrants below an elevation of 290 ft MSL should be connected to the existing 8 in. lower zone pipeline. Above 290 ft MSL, hydrants may be connected either the upper or lower zone pipelines.

4. Pressure Reducing Station

A pressure reducing station prevents pressure in the water system from becoming excessive due to the high water level at the proposed reservoir site. This station is designed so that connections in the service area will not have pressures in excess of 100 psi.

The pressure reducing station is located as shown in Figure 8-1 at an elevation of approximately 270 ft in the vicinity of the SID reservoir. The station consists of 1-1/2 in., 3 in. and 8 in. pressure reducing valves. The valves are set to open at the following pressures; 1-1/2 in. - 80 psi; 3 in. - 76 psi; 8 in. - 72 psi.

5. Pumping Station

The booster pump station consists of a new building and 2 pumps. Each pump can supply 580 gpm which is in accordance with the design criteria in Section 4.

C. HIDDEN VALLEY WATER SYSTEM

The Hidden Valley water system is located on the northern end of Alamo Drive in northwest Vacaville as shown on Map 1 attached to this report. The system includes a pump station, reservoir and distribution system pipelines. A study of the Hidden Valley water system was conducted (Nolte, 1986) which included an analysis of the existing facilities and a set of recommendations for improvement to the system. However, recommended improvements from this study will not be valid if the Hidden Valley system is incorporated into the planned Zone 2.

1. Service Area

The Hidden Valley service area consists of about 41 acres, but much of the land is undevelopable because of steep slopes and only 26 acres of land is used for actual development. The development consists of 31 single family homes on lots up to about and acre in size. Building pads in the service area have elevations between about 225 and 275 ft.

2. Existing Reservoir

The storage reservoir for the Hidden Valley system has a low water level of 343 ft. The reservoir has a maximum water depth of 20 ft and a capacity of 73,430 gallons.

The volume of the storage reservoir is considered inadequate. With the top 5 ft of the storage allocated for operational needs, the remaining volume in the reservoir could sustain a 1,000 gpm fire and peak day demands for only 55 minutes. This fire protection capability is less than required, and it is based on the assumption that there is no allocation in the reservoir for other emergency conditions.

The elevation of the Hidden Valley Reservoir is also not appropriate. With a low water level of 343 ft, the elevation differential to the highest building pad in the service area is only about 61 ft which results in a residual pressure of about 29 psi. Headloss in the system will cause pressures to be even lower in actual practice.

3. Existing Pumping Station

The booster pump station for Hidden Valley is equipped with two booster pumps each with a capacity of about 500 gpm depending upon actual system head conditions. When the capacity of the pumps are compared with the peak hour demands in Hidden Valley (86 gpm), it is apparent that one pump alone will produce much more water than needed. Therefore, when one pump is on, the reservoir is filled very quickly. Once the reservoir is filled, the pump is shut off until the reservoir is depleted to a pre-set minimum level. This sequencing cycle results in a relatively short period with the pump on, and a relatively long period with the reservoir functioning as the supply source for the distribution system. Operation in this manner is not considered a problem.

The pumps have been oversized for peak hour demands because they are intended to supply water during fire conditions. However, the actual ability to provide reliable fire protection is limited by small pipelines in the distribution system and the lack of standby power a the pump station. Available fire flows are estimated to range from 670 gpm to 1,000 gpm depending upon actual hydraulic conditions.

4. Existing Pipelines

The distribution pipes in the Hidden Valley system are typically 8 in. in diameter. From the pump station, an 8 in. pipe runs north on Alamo Drive. This pipe has 8 in. extensions running west to the end of Hidden Valley Lane and east to the end of Hidden Glen Court. On Hidden Glen Court an 8 in. branch extends to the reservoir. The non-looped 8 in. pipelines which exist in this system cannot convey the recommended fire flows.

5. Recommended Improvements

As discussed in the subsection E, "Proposed Zone 2 System" it is recommended that the reservoir and pumping station in the Hidden Valley system be abandoned and that the service area be incorporated into Zone 2.

If the Zone 2 plan is not implemented then improvements to the Hidden Valley system should be made before allowing any new development in the service area. Such improvements should address the problems of inadequate storage, fire fighting capability and pipeline size. For specific details, refer to the 1986 Hidden Valley Water Supply Study.

D. WYKOFF WATER SYSTEM

The Wykoff water system is located on the western edge of the City in the vicinity of Alamo Drive, Foothill Drive, Buck Avenue and Wykoff Drive. The system includes a booster pumping station, reservoir and distribution system pipelines.

Following is a brief description of the existing facilities. Discussion on recommended improvements is provided in subsection E in conjunction with the proposed Zone 2 system. The existing water system is depicted in Figure 8-3, along with the proposed Zone 2 plan.

1. Existing Facilities

a. Service Area

The existing service area consists of 233 single family lots with building pad elevation between 220 and 400 feet. Most of the lots are west of Alamo Drive and Foothill Drive in the vicinity of Wykoff Drive and Buck Avenue.

b. Reservoir

The existing reservoir is located on Tranquility Lane near Wykoff Drive. It has a storage volume of 120,000 gals with a diameter of 46 ft and a sidewall depth of about 10 ft. The high water level is located at an elevation of approximately 476 ft.

The existing reservoir is clearly undersized. The actual operational volume currently being utilized is about 48,000 gals. Therefore, only 72,000 gals are available for emergency and fire conditions. With a fire flow of 1,500 gpm, the reservoir could be depleted in as little as 48 minutes.

The elevation of the reservoir is also not appropriate for the service area. As noted earlier, the existing reservoir serves development with building pad elevations ranging from approximately 220 to 400 ft. Based on the elevation criteria given in Section 4, the recommended

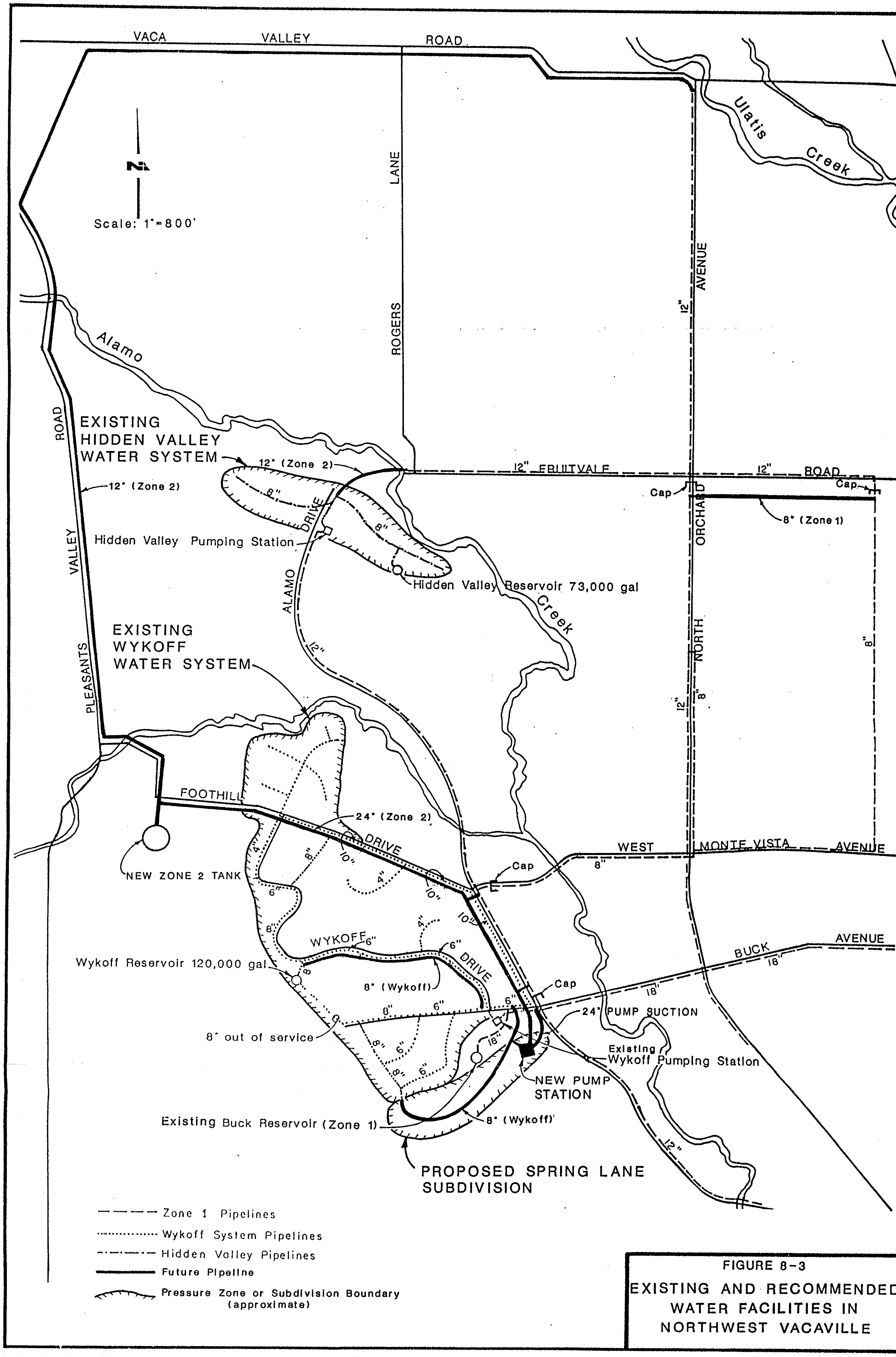


FIGURE 8-3
EXISTING AND RECOMMENDED WATER FACILITIES IN NORTHWEST VACAVILLE

building pad elevations are from 272 to 386 ft. Therefore, the actual elevations slightly exceed the criteria in both the high and low directions. Static pressures in the existing service area will be in the range of 29 to 111 psi.

c. Pumping Station

The pumping station is located near the intersection of Buck Avenue and Wykoff Drive. The station is equipped with two pumps and 60 hp motors. The design point on the pump curve indicates 500 gpm at 350 ft. However, based on our estimates of actual flow rate from reservoir filling records and on calculations of system head, we estimate that the actual operating point is about 900 gpm at 190 ft.

The existing booster pumps are adequately sized for the Wykoff system only if the system is provided with adequate reservoir capacity. However, the reservoir is undersized and does not contain adequate fire storage volumes, so the pumps should provide the fire protection. The existing pumps cannot deliver the required fire protection and the pumps are not provided with a standby power source. Therefore, the booster pump station is not adequate given the existing reservoir capacity, but it would be adequate if reservoir capacity were increased.

d. Distribution System Pipelines

The largest pipeline in the existing Wykoff service area is a 10 in. pipeline on Alamo Drive and Foothill Drive. From the intersection of Foothill Drive and Fern Way, an 8 in. pipeline extends to the reservoir. A 6 in. pipeline on Wykoff Drive completes a loop through the service area. The remaining pipelines in the existing service area are predominately 6 in. with some 8 in. and 4 in. portions.

Many of the pipelines in the existing service area do not meet the recommended criteria. The pipelines do provide adequate service for normal water demands, but are inadequate under fire conditions. Many of the fire hydrants are connected to 6 in. pipelines, which is not recommended even with a looped system.

One particular problem in the distribution system is that the dwelling units in Montgomery Estates on the west end of Buck Avenue are "hydraulically remote" from the reservoir, which means that the water must travel a relatively long distance in relatively small pipelines from the reservoir to the services. This condition could be critical under fire conditions where high headlosses could prevent adequate residual pressures from being maintained at the fire hydrants. Also, service into Montgomery Estates is only provided through a single 6 in. pipeline.

E. PROPOSED ZONE 2 SYSTEM

Following is a description of the proposed plan to create a new Zone 2 pressure zone. The improvements needed for this plan are depicted in Figure 8-3.

Several of the reasons for developing Zone 2 are identified below:

- Building pads in the North Orchard Avenue area are above the recommended limit of 220 ft in Zone 1 and as a result, pressures are low.

- The Hidden Valley system does not meet recommended design criteria.

- The Wykoff system does not meet recommended design criteria.

- Development in the West Valleys (Pleasants Valley and Lagoon Valley) may occur, and would require a pressure zone above Zone 1. Development in the West Valleys is currently under discussion in conjunction with the update of the General Plan.

Much of the developable land in the West Valleys lies between elevations 215 ft and 315 ft. The Hidden Valley system as well as much of the North Orchard Avenue area are also between 215 ft and 315 ft. This range could be ideally served with a reservoir having high and low water levels of 419 ft and 395 ft, respectively. Unfortunately, the existing Wykoff reservoir is substantially higher with a high water level of 476 ft, and if the existing Wykoff reservoir elevation were used in the West Valleys, the pressures for much of the development would be higher than recommended. Furthermore, the higher reservoir elevation would result in increased and unnecessary pumping energy costs. Because of these pressure and cost considerations, the following concept is proposed:

- Keep the existing Wykoff reservoir in service, but reduce it's service area to the extent possible, and provide pumping capacity to meet fire demands in the reduced service area. This reduced Wykoff service area would be limited to existing homes on the west side of Alamo Drive and south of Foothill Drive. An addition of several homes in the proposed Spring Lane subdivision south of Buck Avenue and west of Alamo Drive is suggested because looping pipelines through the development can benefit the system.

- Construct a new reservoir with high and low water levels of 419 ft and 395 ft to provide service to the remainder of the Wykoff service area, the Hidden Valley service area, and to the North Orchard Avenue area.

- Reservoir and pumping facilities will initially be sized to exclude development in the West Valleys. However, the system will be planned for expansion to serve the West Valleys if and when they are eventually developed.

Specific details of this alternative are described below:

A single new pumping station would be used to fill the existing Wykoff Reservoir and to fill the new Zone 2 reservoir. Although pumps would have to be dedicated to each system because of different head conditions, economies of scale in building and site construction cost would be realized.

The pumping capacity for the reduced Wykoff service area is based on the alternate design criteria given in Section 4 which results in a firm capacity of about 1,600 gpm. The required pumping capacity for Zone 2 excluding the West Valleys would be about 2,300 gpm (assuming that it is a secondary zone on an interim basis). Standby power would be required for the pumps dedicated to the Wykoff service area.

The pumping station would be relatively large, so the existing pump station site could not be utilized. The preferred location is in the vicinity of Buck Avenue and Alamo Drive. A lot on one of the easterly parcels of the proposed Spring Lane subdivision would serve this purpose very well.

The Northwest Zone 2 reservoir should have a capacity of 2.6 million gallons to serve existing development areas in northwest Vacaville. This volume would not cover any of the requirements for the West Valleys. The reservoir might be located on the undeveloped land south of Foothill Drive just west of the existing city limits, but availability of land could require that the reservoir be located west of Pleasants Valley Road.

Substantial pipeline improvements will be needed with this alternative. Such improvements are identified below.

- New 24 in. pipeline from the intersection of Alamo Drive and Buck Avenue to the new pumping station. (Sized for West Valleys)

- New pipeline from the new pumping station to the reduced Wykoff service area. Assuming that the pump station is located in the proposed Spring Lane subdivision, pipelines should extend in two directions from the pump station. An 8 in. branch should extend to the intersection of Alamo Drive and Buck Avenue, and another branch should be looped through the proposed Spring Lane subdivision to Montgomery Estates.

- New 8 in. pipelines from the Wykoff Reservoir, along Wykoff Drive to Buck Avenue.

- New 24 in. pipeline from the new pumping station to the new reservoir. (Sized for West Valleys)

- New 12 in. pipeline crossing Alamo Creek connecting existing pipelines on Alamo Drive and Fruitvale Road.

- New 8 in. pipeline on Fruitvale Road between North Orchard Avenue and Stinson Avenue.

- Miscellaneous improvements to isolate Zone 1 from the Zone 2 service area.

- New 12 in. pipeline on Vaca Valley Road and Pleasants Valley Road to complete a loop to the North Orchard Avenue area. (This pipeline can be delayed until development in the West Valleys, but ultimately is recommended regardless of the status of development in the West Valleys)

Inherent in this plan for Zone 2 are several issues which are important to recognize. These are identified below:

- Although the size of the Wykoff service area is reduced, pumping capacity is increased, and pipeline improvements are made, the system will still be deficient and will not meet the recommended design criteria. One of the most important deficiencies will be high pressures. Certain homes near to the 220 ft elevation will remain on the system which will experience pressures near 110 psi. Also, because it is not practical to replace all of the pipelines in the system, several 6 in. and 4 in. pipelines will remain in service.

- The proposed Spring Lane subdivision will be added to the Wykoff system despite the fact that the Wykoff system will continue to have high pressures. However, this arrangement is justified because the Spring Valley subdivision can help to improve the system overall by providing an important pipeline loop from the pump station westward through the subdivision into the Montgomery Estates subdivision.

F. TRANQUILITY LANE SYSTEM

The Tranquility Lane system serves eleven parcels near to the existing Wykoff Reservoir which have building pad elevations too high to be served by the existing reservoir. The system consists of two 5 HP booster pumps and a hydropneumatic tank located at the reservoir site. Although a pump curve is not available, we estimate that the pumps supply about 140 gpm. The pumps withdraw water from the reservoir and discharge into the 525 gal hydropneumatic tank. About 200 gal of the total volume is actual working volume. The tank has an operating pressure range of about 25 to 50 psi. The controls at the pumping station do not allow both pumps to operate simultaneously. One pump is normally in operation while the other remains in standby mode. Switching pumps from operational to standby made is a completely manual operation.

The nominal pipeline size in the Tranquility Lane distribution system is 4 in., which is clearly too small to convey fire flows. The booster pumps and hydropneumatic tank are also too small to convey fire flows, so the houses on the Tranquility Lane system remain unprotected against fire except for fire hydrants in the Wykoff zone which have pressures below that required for the Tranquility Lane homes.

Within the last year, operational problems with the Tranquility Lane facilities have developed. The problems have been caused by pumps which do not start on occasion, even though power supplies are adequate. The result is that the hydropneumatic tank and pipelines in the pumping station are completely drained by water system demands. When this happens, the system must be manually purged of the air and the proper air/water balance in the hydropneumatic tank must be set before the system will function properly. If the pumps are turned on with air in the pipelines, the motors can burn out.

Recently, the pumps have been replaced and the new pumps do not have the same starting problem as the previous pumps. However, pump replacement alone is not a complete solution to the system's problems.

Water demands in the Tranquility Lane system can vary from near zero at night to well over 100 gpm during heavy use periods. Actual peak demands are difficult to predict because specific characteristics of individual lawn sprinkler systems will influence the peaks dramatically.

Minimum pump cycle times will occur when demands are equal to one-half of the pumping capacity. With a pumping capacity of 140 gpm, a demand of 70 gpm and a working volume in the hydropneumatic tank of 200 gal., the cycle time would be just under 6 minutes. This cycle time is considered appropriate.

If water demands ever exceed pumping capacity for a prolonged period of time, serious problems could result. In this case, the water level in the hydropneumatic tank would continually drop, even though the pumps were simultaneously supplying water to the tank. With sufficient time, the tank and pumping station pipelines could be completely drained in a manner similar (but slower) to the case where the pumps don't start at all. Given the size of the existing facilities and the potential demands, this problem is a real concern.

Providing fire protection capabilities for the Tranquility Lane system would be very expensive. Completely new pumping and distribution system improvements would be required. Although we recommend that all homes are provided with adequate fire protection, the cost of the improvements may be hard to justify in this case.

We do recommend modification of the existing pump controls. We suggest that the new controls provide a lead pump/lag pump mode of operation. The lead pump would provide normal service and the lag pump would provide automatic backup service. The lag pump would come on if either 1) The lead pump failed to come on or 2) The water level in the hydropneumatic tank falls below the normal low water level. These control modifications would effectively increase system capacity and reliability and would greatly reduce the chances of the hydropneumatic tank being drained. Control equipment could be provided by reusing the equipment from the existing Vine Street pump station once the planned pump station replacement is made.

Currently, there are also plans to replace the existing hydropneumatic tank with a larger tank which is now available from the easterly wastewater treatment plant.

9. CAPITAL IMPROVEMENTS PROGRAM

Specific recommendations for the well field, treatment and pumping facilities, storage reservoirs and distribution system piping are discussed in detail in their respective chapters. The purpose of this section is to summarize the recommended improvements and to present capital costs for each improvement.

In Tables 9-1 through 9-3, all of the recommended capital improvements through the year 2005 are summarized. The improvements have been subdivided into several categories identified below.

- Water Production Improvements (9-1)

  This category includes improvements associated with the sources of water production including the wells, the DE Plant and the NBR Plant.

- Water Transmission and Distribution Improvements (9-2)

  This category includes improvements associated with the distribution system such as pipelines, reservoirs and distribution system pump stations.

- Local Pipeline Improvements (9-3)

  This category includes pipelines used to serve local distribution needs. This is a separate category because the time for construction of these improvements is contingent upon development of specific areas and is therefore difficult to estimate. Dates of construction have not been assigned to improvements in this category.

Each table provides the recommended beginning of construction date and the estimated capital cost. We have also included columns on the right hand side of the table for keeping track of the City's budget allocations for each project.

The selected dates of construction are based in part on the water demands predicted earlier in Section 3. Therefore, if the demands grow at a different rate than expected, the time of construction may also change.

Capital costs have been escalated from 1990 dollars to future dollars at a rate of 5% per year. An exception is Table 9-3 where dates of construction are not specified and all costs are in 1990 dollars.

The costs in Table 9-1 are comprehensive budget level costs and include the following items:

ESTIMATED CONSTRUCTION COST

OTHER COSTS (40% of construction costs)
- 10% Contingency
- Design Engineering
- City Administration
- Construction Management
- Surveying
- Geotechnical

Appendix C provides notes on the cost estimate calculations for each improvement.

Map 2 (contained in the back pocket of this report) shows the recommended improvements to the distribution system.

TABLE 9-1

SUMMARY OF WATER PRODUCTION CAPITAL IMPROVEMENTS

Beginning of Construction Year	Description of Improvement	Actual or Estimated Cost ($'s)	City Budget Work Order Number or Funding Year	Budget Amount	Estimated Completion Year
1989	1. Water Plant Clearwell Reroof	190,000			1990
1989	2. NBR Plant Construction, 13.3 mgd:	14,440,000			1990
1990	3. Well Field Study: Development of a well management program. Costs to be determined.	NA			1991
1990	4. NBR Plant CIP: Annual CIP's for plant improvements.	150,000 200,000 200,000 200,000 250,000			Ongoing
1990	5. Well Rehabilitation: Rehabilitation of existing wells per the Well Management Program. Cap and close old Well No. 6, install building around Well No. 7, and install landscaping at sites. Cost to be determined during well field study.	NA			1995

TABLE 9-1

SUMMARY OF WATER PRODUCTION CAPITAL IMPROVEMENTS
(Continued)

Beginning of Construction Year	Description of Improvement	Actual or Estimated Cost ($'s)	City Budget Work Order Number or Funding Year	Budget Amount	Estimated Completion Year
1990	6. Disinfection and Operations Study: Study to analyze impact of new water quality regulations on DE plant and wells.	130,000			1990
1990	7. Replacement Well: Replacement of existing Well No. 4 at the original site.	750,000			1990
	8. D.E. Plant Improvements: Improvements are based on the DE Plant Preliminary Design Report plus clearwell baffling.				
1991	Booster Pump Station Upgrade	690,000			1992
1991	Water Plant Building Addition	580,000			1992
1991	Chemical Addition Facilities (Chlorine, Fluoride, Ammonia)	960,000			1992
1991	Waste DE Drying Beds	260,000			1992

TABLE 9-1

SUMMARY OF WATER PRODUCTION CAPITAL IMPROVEMENTS
(Continued)

Beginning of Construction Year	Description of Improvement	Actual or Estimated Cost ($'s)	City Budget Work Order Number or Funding Year	Budget Amount	Estimated Completion Year
1991	Filter to Waste Capability	74,000			1992
1991	DE Plant Controls (Convert to PLCs)	116,000			1992
1991	Body Feed System	19,000			1992
1991	Air Supply System	100,000			1992
1991	Main Switchboard Improvements	82,000			1992
1992	Clearwell Baffling	100,000			1993
1991	9. Well Field Chloramination: Chloramination facilities for the well field. Method and cost to be determined.	NA			1993
	10. Well Field Development: Test well development and land acquisition for 3 new well sites. Construct new wells to expand raw water sources from 6,000 AF/yr to 7,000/AF/yr.				

TABLE 9-1

SUMMARY OF WATER PRODUCTION CAPITAL IMPROVEMENTS
(Continued)

Beginning of Construction Year	Description of Improvement	Actual or Estimated Cost ($'s)	City Budget Work Order Number or Funding Year	Budget Amount	Estimated Completion Year
1991	Engineering/Land Acquisition	350,000			1991
1992	New Well	1,280,000			1992
1993	New Well	1,340,000			1993
1994	New Well	1,410,000			1994
1995	11. NBR Plant Expansion, 13.3 to 20.0 mgd:	3,900,000			1996
1999	12. Replacement Well: Replacement or rehabilitation of an older well.	1,800,000			2000
2000	13. NBR Plant Expansion, 20.0 to 30.0 mgd:	7,300,000			2001
2004	14. Replacement Well: Replacement or rehabilitation of an older well.	2,300,000			2005

TABLE 9-2

SUMMARY OF WATER TRANSMISSION AND DISTRIBUTION CAPITAL IMPROVEMENTS

Beginning of Construction Year	Description of Improvement	Actual or Estimated Cost ($'s)	City Budget Work Order Number or Funding Year	Budget Amount	Estimated Completion Year
1989	1. Downtown Water Main Improvements: Replacement of several old and undersized pipes in the downtown area.	1,725,000			1989
1989	2. Nut Tree Road Pipeline: New 18" pipe placed in conjunction with road improvements.	600,000			1989
1989	3. CMF-North Water Meter Station: Removal and replacement of existing unit.	15,000			1990
1989	4. Elmira Road Pipeline: New 18" pipe between well field and 30" pipe on Elmira Road.	390,000			1990
1989	5. Browns Valley Road Pipeline: New 24" pipe from Elmira Road and Peabody Road to Glen Eagle Way.	2,110,000			1990
1989	6. Browns Valley Reservoir: 5.0 MG reservoir north of Vaca Valley Parkway and east of Browns Valley Road.	2,050,000			1990

TABLE 9-2

SUMMARY OF WATER TRANSMISSION AND DISTRIBUTION CAPITAL IMPROVEMENTS
(Continued)

Beginning of Construction Year	Description of Improvement	Actual or Estimated Cost ($'s)	City Budget Work Order Number or Funding Year	Budget Amount	Estimated Completion Year
	7. Water Main Capacity Program: Replacement of old and undersized pipelines, plus upsizing of facilities installed privately with new development. Costs to be determined.				Ongoing
1990	See Description Above	200,000			1991
1991	See Description Above	300,000			1992
1992	See Description Above	300,000			1993
1993	See Description Above	300,000			1994
	8. Installation of SCADA for Water System:				
1990	Phase 1 SCADA (minimum master station, Well No. 9, Vine Street system)	248,000			1990
1991	Additional SCADA Equipment	84,000			1991

TABLE 9-2

SUMMARY OF WATER TRANSMISSION AND DISTRIBUTION CAPITAL IMPROVEMENTS
(Continued)

Beginning of Construction Year	Description of Improvement	Actual or Estimated Cost ($'s)	City Budget Work Order Number or Funding Year	Budget Amount	Estimated Completion Year
1992	Additional SCADA Equipment	88,000			1992
1993	Additional SCADA Equipment	93,000			1993
1994	Additional SCADA Equipment	97,000			1994
1995	Additional SCADA Equipment	102,000			1995
1991	9. Reservoir Rehabilitation: Modification of inlet/outlet piping at reservoirs, corrosion prevention and misc site improvements. Costs to be determined.	NA			1993
1991	10. Tranquility Pump Station Improvements: New pump station equipment and building to meet state codes.	50,000			1991
1991	11. ACAD Implementation: Fund accumulation to convert the water system drawings to Autocadd. Cost to be determined.	NA			1993

TABLE 9-2

SUMMARY OF WATER TRANSMISSION AND DISTRIBUTION CAPITAL IMPROVEMENTS
(Continued)

Beginning of Construction Year	Description of Improvement	Actual or Estimated Cost ($'s)	City Budget Work Order Number or Funding Year	Budget Amount	Estimated Completion Year
	12. Wykoff/Zone 2 Improvements, Phase 1:				
1991	New pump building and site, plus "Wykoff" pumps, generator, mechanical and electrical improvements, and the 24" suction pipe.	540,000			1991
1991	New 8" pipes to strengthen the tie between new pump station and existing reservoir.	250,000			1991
1992	13. Noonan Reservoir: Fund accumulation for construction of joint use reservoir. Costs to be determined.	NA			1993
1992	14. New Zone 1 Reservoir: 5 MG capacity at an unspecified location.	3,600,000			1993

TABLE 9-2

SUMMARY OF WATER TRANSMISSION AND DISTRIBUTION CAPITAL IMPROVEMENTS
(Continued)

Beginning of Construction Year	Description of Improvement	Actual or Estimated Cost ($'s)	City Budget Work Order Number or Funding Year	Budget Amount	Estimated Completion Year
	15. Wykoff/Zone 2 Improvements, Phase 2:				
1993	Addition of Zone 2 pumps and mechanical and electrical improvements.	146,000			1994
1993	New 2.6 MG Zone 2 reservoir.	3,000,000			1994
1993	New 24" Zone 2 pipeline on Alamo Drive and Foothill Drive connecting the pump station to the reservoir.	1,110,000			1994
1993	New pipes to form Zone 2, including 12" pipe across Alamo Creek at west end of Fruitvale Avenue and 8" pipe on Fruitvale Road.	250,000			1994
1994	16. North Browns Valley Road Pipeline: New 18" pipe on North Brown's Valley Road and Vaca Valley Parkway.	1,180,000			1995
1995	17. Peabody Road Pipeline: New 36" pipe on Peabody Road from California Drive south 3,500 LF.	1,070,000			1996

TABLE 9-2

SUMMARY OF WATER TRANSMISSION AND DISTRIBUTION CAPITAL IMPROVEMENTS
(Continued)

Beginning of Construction Year	Description of Improvement	Actual or Estimated Cost ($'s)	City Budget Work Order Number or Funding Year	Budget Amount	Estimated Completion Year
	18. Wykoff/Zone 2 Improvements, Phase 3:				
1996	New 12" pipe connecting the new reservoir to the North Orchard Avenue area via Pleasants Valley Road.	1,310,000			1997
1996	19. North Leisure Town Road Pipeline: New 18" pipe on Leisure Town Road from Sequoia Drive to Vaca Valley Parkway.	1,150,000			1997
1997	20. Monte Vista Avenue Improvements: New 12" pipe on Monte Vista Avenue from Depot Street to Gibson Canyon Road.	270,000			1998
1997	21. New Zone 1 Reservoir: 5 MG capacity at an unspecified location.	4,600,000			1998
1998	22. Nut Tree/I-505 Pipeline: New 18" pipe on Nut Tree Road from Elmira Road to I-505 to Vaca Valley Parkway.	3,000,000			1999
1999	23. NBR Pipeline: New 30" pipe from the NBR plant.	4,000,000			2000

TABLE 9-2

SUMMARY OF WATER TRANSMISSION AND DISTRIBUTION CAPITAL IMPROVEMENTS
(Continued)

Beginning of Construction Year	Description of Improvement	Actual or Estimated Cost ($'s)	City Budget Work Order Number or Funding Year	Budget Amount	Estimated Completion Year
2003	24. New Zone 1 Reservoir: 5 MG capacity at unspecified location.	6,200,000			2004

TABLE 9-3

SUMMARY OF "LOCAL" PIPELINE CAPITAL IMPROVEMENTS

Beginning of Construction Year	Description of Improvement	Estimated Cost (1990 $'s)	City Budget: Work Order Number or Funding Year	City Budget: Budget Amount	City Budget: Estimated Completion Year
NA	Gonsalves Lockie Pipelines: New 12" pipe in the project.	NA			
NA	Akerly Drive Pipeline: New 12" pipe to complete loop on Akerly Drive south of Vaca Valley Parkway.	340,000			
NA	Leisure Town, Midway, Eubanks Pipeline: New 12" pipe, north on Leisure Town Road, and west on Midway Road to Eubanks Drive.	1,240,000			
NA	Midway Road Pipeline: New 12" pipe on Midway Road between Leisure Town Road and Meridian Road.	400,000			
NA	Allison Drive Pipeline: New 12" pipeline on Allison Drive from Monte Vista Avenue to Vaca Valley Parkway.	950,000			

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APPENDIX A

GROUNDWATER RESOURCES OF THE VACAVILLE AREA

Following is the 1985 report prepared by Dr. John F. Mann, Consulting Geologist and Hydrologist. Also included is a supplement prepared in May, 1989 based on a review of well data gathered since the 1985 report. Note that the recommendations in the supplement were made prior to the summer of 1989 when one of the wells began to have serious drawdown problems. Therefore, the recommendations may not be wholly appropriate in light of this new information.

SUPPLEMENT TO GROUND WATER RESOURCES OF THE VACAVILLE AREA

John F. Mann, Jr.
Consulting Geologist and Hydrologist

May 1989

Introduction

The present report is intended as a supplement to my report of April 1985 in which the ground water hydrology of the Elmira Road Well Field was discussed. A major recommendation of the April 1985 report was that water levels be measured systematically so that the pumping effects could be evaluated. That recommendation has been effectively implemented and numerous water-level measurements (both static and pumping) have been recorded since the latter part of 1986. Thus, for two calendar years (1987 and 1988) the measurements are adequate.

Responses of Water Levels to Pumping

The Tehama aquifers in the Elmira Road Well Field respond to pumping like a large conduit rather than the typical ground water basin which is alternately emptied and filled. As water is removed from the Tehama aquifers by the City's wells, there is induced within those aquifers a horizontal flow (mainly from the east) into the pumping hole. If the amount of this flow (on a seasonal basis) is equal to the amount removed by the wells, the pumping hole tends to stabilize and produces a dynamic equilibrium. If the inflow is unable to keep up with the pumping, the hole continues to deepen and grow in area.

During the last three calendar years, the amounts of water pumped from the Elmira Road Well Field were:

1986 - 5829 acre-feet

1987 - 6267 acre-feet

1988 - 5420 acre-feet

An analysis of static water levels in Wells #2 - #8 indicates complete recovery following periods of heavy pumping, and the establishment of a dynamic equilibrium. There is no evidence of overdraft, and it would appear that the safe yield of the system has not yet been reached. It is quite possible that a larger amount of seasonal pumping will also result in a dynamic equilibrium -- but with somewhat lower static and pumping levels.

<u>Recommendations</u>

1. If operationally convenient, pumping during 1989 should be about 7000 acre-feet.
2. Well operation should be intermittent -- as in the past -- to allow recovery in nonpumping periods.
3. Daily water-level measurements (whether the well is idle or pumping) should be continued.

GROUND WATER RESOURCES

OF THE

VACAVILLE AREA

John F. Mann, Jr.

Consulting Geologist and Hydrologist

April 1985

Introduction

My previous general report on ground water for the City of Vacaville (with VTN Engineers) was dated May 17, 1974, almost eleven years ago. At that time, Wells 3, 6, 7, and 8 (old numbers) were active. Information on those wells and on Wells 2 and 4 had appeared in the Yoder Report of June 1961 (Reference 1, p. 9). Especially encouraging was Well 7 which had been drilled in 1953, and had a specific capacity of 30.1 gallons per minute per foot of drawdown. Well 8, which had been drilled in 1958, had a specific capacity of 20.5 gallons per minute per foot of drawdown. Well 6, which had been drilled in 1949, was shallower than Well 7 or Well 8, had a specific capacity of only 7.5 gpm/ft. Based upon the promising specific capacities of deeper Wells 7 and 8, it was indicated in the May 1974 report that as many as four additional wells could be drilled in the general area east of the City. Another favorable factor in ground water development east of the City was the May 1964 report of the U. S. Bureau of Reclamation stating that many wells in the High Prairie area had been abandoned as farmers switched from ground water to canal water. As a result of this change, the Bureau estimated that a ground water safe yield of about 9000 acre-feet would become available. It was not made clear in my 1974 report that the High Prairie area is at least six miles to the east of Elmira. Until very recently, I was not aware that the figure of 9000 acre-feet was being interpreted as the safe yield of the small area of the Elmira Road well field. As will be discussed later in this report, a determination of the safe yield of a deep ground water system such as that tapped by the Elmira Road well field could not have been predicted in 1974. Such a determination could only have been made after the additional wells had been drilled, after they had been pumped heavily, and after the effects of that pumping on year round static water levels had been studied.

Development of the Elmira Road Well Field

The first step in the development of the Elmira Road well field was to drill additional wells to establish the horizontal extent of the deep productive aquifers which had been found in Wells 7 and 8 (old numbers). The first two new wells were drilled in 1974-75 (old Wells 10 and 9, present Wells 7 and 8) to the east of the known productive area. It was desired to locate the test holes for the new wells close to the Elmira Road pipeline so as to minimize tie-in costs. From the test hole information and electric logs, there was evidence that the productive gravels became thinner to the east of Well 7 (present Well 4). Specific capacities of Wells 10 and 9 (present Wells 7 and 8) were disappointing -- in the range of only 11-12 gpm/ft. This would mean that the anticipated pumping rates could be achieved only at the expense of considerably greater drawdowns. Well 9 (present Well 8) was initially tested at 1600 gallons per minute with a drawdown of 145 feet. For the month of June 1983, it was pumped at an average rate of 1412 gpm. For the month of June 1976 , Well 10 (present Well 7) was pumped at an average rate of 1550 gpm.

Old Well 11 (present Well 2) was drilled in 1979 at the northwest corner of Nut Tree Road and Elmira Road. Again, a location close to the Elmira Road pipeline was desired. The driller's log and electric log showed that the base of the gravels was at a shallower depth than at old Well 7 and that the aquifer was somewhat thinner than at old Well 7. Well 11 was tested initially at 1400 gpm with a drawdown of 128 feet. This specific capacity was also disappointing -- only 11.3 gpm/ft.

For Well 12 (present Well 5), a site was acquired by the City south of Elmira Road, but close enough to the Elmira Road pipeline to minimize tie-in costs. The test hole was drilled in August 1980, and the driller's log and electric log both showed thinner gravels than at Well 7. Well 12 was initially tested at 1500 gpm with a drawdown of 91 feet. Although the specific capacity, at 16.5 gpm/ft, was better than the other three new wells, it was still only about half the specific capacity of Well 7. During March 1982, Well 12 was pumped at an average rate of 1547 gpm.

The development of the Elmira Road well field revealed that the area of productive aquifers initially found in the two early deep wells, especially Well 7, did not come up to the early promise. Specific capacities to the east, south, and west of Well 7 turned out to be substantially less than at Well 7. The main effect of this is that high well production can be achieved in this compact well field only with large drawdowns and considerable well interference. The area north of Well 7 (present Well 4) has not yet been tested.

Ground Water Geology East of Vacaville

The main aquifer east of Vacaville consists of gravels near the base of the Tehama formation. The general geology of the Vacaville area is shown on Plate 1 of United States Geological Survey Water Supply Paper 1464. The wells at the City's Water Treatment Plant (6N/1W-22F1, F2, F3, and F4) are shown plotted on the outcrop area of the Tehama formation (QTt). Information on these old P.G.& E. wells is given on page 390 of WSP 1464. The dip symbols near these wells show that the Tehama formation is inclined 15 to 25 degrees from the horizontal so that the formation becomes deeper to the east. These relationships are shown on cross-section F-F on Plate 9 of WSP 1464. This cross-section represents a vertical slice through the earth somewhat south of Elmira Road; its location is shown on Plate 1. The geologic relationships along Elmira Road are similar, as shown by the attached sketch (Figure 1). The base of the Tehama formation is at the ground surface just west of the Water Treatment Plant. At Nut Tree Road (Well 2) the main gravel zone is between depths of 340 and 710 feet. Farther east, at Well 4, the main Tehama gravels are found between depths of 560 and 865 feet. At Leisure Town Road, the base of the Tehama gravels may be as deep as 1200 feet. The individual gravel layers are lenticular, which means they may be thick, thin, or absent at any particular geographic location.

Nature of the Aquifer

The basal gravel zone of the Tehama formation is a pressure or confined aquifer. It is overlain by a shallow (unconfined) aquifer with a true water table (Fig. 2). The pumping of the deep aquifer at the Elmira Road well field has no effect on the shallow water table. When the deep aquifer is pumped, a cone of depression is created in the pressure (also called the piezometric or potentiometric) surface. This cone will grow and deepen until the flow in the confined aquifer equals the amount of water being pumped from the well field. If the pumping rate is decreased, the cone will show recovery by becoming shallower and of smaller geographic extent. The pumping cone generated by the Elmira Road well field can not grow indefinitely to the west because the aquifers sweep up to the surface of the ground and the tilted layers beneath the Tehama gravels form an impermeable barrier which can not feed water into the cone (Fig. 3). The flow of ground water into the cone is primarily from the east. It is characteristic of such confined aquifer well fields to respond only to changes in pumping rather than to wet and dry years. Recharge related to rainfall can be difficult to demonstrate -- even over a period of decades. The classical example of this type of aquifer is the Dakota sandstone in South Dakota. A recent (1983) report by the U. S. Geological Survey (Water Supply Paper 2237, Ref. 6) sheds some light on the sources of water to such deep confined aquifers. One of the conclusions of that report was that most of the water pumped from the confined aquifers comes from flow across the confining layers. When wells perforated in the confined aquifer are pumped, and the aquifer is depressurized within the cone of depression, there is a flow into the aquifer through the relatively tight beds above and below the aquifer. Such water joins the water which is flowing within the aquifer toward the center of pumping (Fig. 4). When the water flowing toward the well field equals the amount of water being pumped, the cone becomes stabilized. The rate of flow from the confining beds into the aquifer is extremely slow, and there is no perceptible lowering of the overlying water table. Despite this slow flow rate, the area of the cone is so large that the cumulative flow from the confined beds is important.

When wells in a confined aquifer are as closely spaced as in the Elmira Road well field, it is to be expected that there will be substantial well interference, which is illustrated in Fig. 5. Substantial drawdown is necessary to cause the water in the aquifer to flow toward the well field, and it is an inescapable fact that the greater the amount of water which is pumped, the greater the drawdown must be to create a gradient steep enough to cause the necessary flow into the well field. However, there is a limit on how deep a pumping level should be. This is discussed on page 76 of U.S.G.S. Water Supply Paper 2220, Ref. 5), where the "lowest practical pumping level" is related to the uppermost screen. In Well 4 (old Well 7) the pumping level is below the shallowest perforated zone. Well 4 has a hole in the casing and is now beyond its useful life. When a replacement well is drilled for Well 4, the shallower zones should not be perforated.

Safe Yield

The term "safe yield" has been a source of much confusion and misunderstanding (Ref. 7). For the Elmira Road well field no "Reservoir Method" can be used. The only applicable method of determining the maximum rate of withdrawal for this compact well field is the "Transmissibility Method" which treats the Tehama gravels as a conduit rather than as a reservoir (Ref. 7, p. 5-1). The safe yield can be considered as the annual rate of withdrawal from the well field which will result in a year-to-year stabilized cone of depression (or pumping hole). The data needed for such an analysis are:

1. Accurate production records for each well.

2. Systematic water level measurements.

The historical well production records for the Elmira Road well field are excellent. Unfortunately, the static water level measurements are sketchy, and in some instances, of doubtful accuracy. The available static water level measurements as compiled by Michael Abramson are given in Table 1. Because of the lack of systematic water level measurements, a reliable safe yield determination can not be made at this time. Only the roughest indications can be gleaned from the existing information. Assuming the water level measurements are correct, between February 22, 1982 and April 26, 1984, the static water level in Well 8 dropped only 2.8 feet. During this 26-month period, the well field was pumped at a rate of 7800 acre-feet per year. It will take systematic static water level measurements for all wells at no less than monthly intervals over at least two consecutive winters to get even a rough figure for the annual pumping rate at which the pumping hole will stabilize. Then the production and static water level data should be reviewed each year. An annual pattern of water level fluctuations will probably result from periods of heavy pumping and reduced pumping. In systems like this , there may be "hysteresis" effect due to the difference between the way the aquifer responds after an increase in pumping as compared with the response when pumping is reduced.

Additional Wells in the Elmira Road Well Field

As discussed above, the Elmira Road well field, as it is now developed, is a compact group of wells with substantial well interference and excessive drawdown. The pumping hole is deeper than it would be if the wells were spread over a larger area. A well drilled in the area less than a half mile north of Well 4, an area which is as yet untested, may tap a productive portion of the Tehama gravels. Pumping of such a well may be expected to cause a further deepening of the pumping hole if pumping is increased rather than kept the same and distributed over a larger area.

Well 1 at the Water Treatment Plant is scheduled to be equipped with a new pump. Well 1 is now 50 years old and has collapsed between a depth of 484 feet and its original depth of 600 feet; it can not be relied on indefinitely. The aquifers beneath the Water Treatment Plant (which may include minor sands and gravels below the Tehama formation) have shown no long term decline of static water level. A replacement well could be drilled at this site and the pumping rate should be about the same as Well 1 (200 gpm).

Well 4 was drilled in 1953 and is now 32 years old. The casing is ruptured at a depth of 475 feet and is so pitted and encrusted that the contractor has advised against cleaning or repairing of the casing. A replacement well could be drilled at the same well site and the specific capacity should be high. To avoid having the pumping levels below the top perforations, the shallower zones now perforated in Well 4 (190-240 and possibly 310-340) should not be perforated in the replacement well. This reduction in perforated thickness could result in some reduction of specific capacity in the replacement well as compared with the initial specific capacity of Well 4, which was about 30 gpm/ft. The problems with Well 4 have been well and pump problems rather than aquifer problems. Thus, a replacement well as little as 50 feet away from Well 4 would tap aquifer materials which have been unaffected by the historic pumping of Well 4. Pumping both Well 4 and a replacement well on the same site would result in severe well interference.

This is not to say that such a plan is impossible. If such a course were chosen, the replacement well should be as far away from Well 4 as possible, bearing in mind that drawdown (and interference) are related to the square of the distance. Thus, if the two wells were 200 feet apart, they would have only one fourth as much interference they would have if they were only 100 feet apart. To avoid having the pumping levels below the perforations, with consequent problems of cascading water and air entrainment, the shallower zones now perforated in Well 4 (190-240, and perhaps 310-340) should not be perforated in the replacement well.

The next oldest well is Well 6 (old Well 8) which was drilled in 1958. It is now 27 years old and about at its normal life expectancy. Well 6 was rehabilitated recently by mechanical scratching of the perforations, and by bailing. Removal of encrustations from the perforations was effective but not complete, so re-establishing the initial specific capacity will probably not be possible. However, a replacement well as little as 50 feet away should be expected to have as high a specific capacity (20.5 gpm/ft) as Well 6 had at the time of drilling in 1958.

The four newer wells have all been drilled since 1974. Heavy pumping has contributed to sand problems, which in turn have caused pump wear. As necessary, any of the four wells could be replaced by wells on the same well site, with the expectation that the specific capacities would be as high as the original wells had when first drilled.

Additional Well Fields

There are little available data on the presence of or productivity of the basal Tehama gravels to the east of the Elmira Road well field. About 6 miles east of Elmira, electric log information from an oil well (Amerada Comber 1) shows coarse-grained layers near the base of the Tehama formation between depths of 1600 and 2300 feet. As shown on Plate 1 of U.S.G.S. Water Supply Paper 1464, this well is in the northwest quarter of Section 19, T. 6 N., R. 2 E. The electric log is shown on Section F-F , Plate 9. Although individual gravel layers may thicken, thin, or disappear, the zone of gravels near the base of the Tehama formation has a strong likelihood of being present beneath the Easterly Sewage Treatment Plant, as suggested on Figure 1. The existing water well at the Easterly Plant went only to a depth of 692 feet, and would not have reached the basal gravels of the Tehama formation. If it were desired to locate a deep water production well in this vicinity, a test hole should be drilled to a depth of at least 1500 feet, and an electric log run. If the electric log indicators are favorable, a production well could be constructed nearby. A single pumping well in this area would not experience the interference effects of a well in the Elmira Road well field.

The nearest known high specific capacity wells in shallow zones (less than 500 feet deep) are almost 6 miles northeast of Elmira in Section 1, T. 6 N., R. 1 E. Another area of shallow high specific capacity wells lies about 8 miles east of Elmira in Section 16, T. 6 N., R. 2 E. Developing shallow well fields in either of these areas would, of course, require substantial pipeline costs.

Two deep wells were drilled west of Leisure Town Road, just west of the American Home Foods Sewage Treatment Plant. The Chevron West Well was drilled to a depth of 1627 feet in 1965, and was perforated between depths of 200 and 1600 feet. It was tested on April 20, 1977 and produced 810 gpm with a drawdown of 65 feet, for a specific capacity of 12.5 gpm/ft. The Chevron East Well was also drilled in 1965 and to a depth of 1920 feet. Pump tests in 1977 and 1983 indicate specific capacities of 6.5 and 4.7 gpm/ft, respectively. Several deep wells in Section 1, T. 6 N., R. 1 W. have shown very poor results. With regard to this area, the following comments are offered:

1. There is no large area of high specific capacities. The average is probably closer to 5 gpm/ft than to the 12.5 gpm/ft reported for the Chevron West Well.

2. Additional wells in this area would produce severe interference effects on the existing Chevron wells.

3. The two Chevron wells are each 20 years old, which in the event of purchase by the City, raises the question of how much useful life remains.

4. There is the question of the net gain if the City were to assume the demands of American Home Foods in return for pumping from this rather mediocre area.

Conclusions and Recommendations

1. The deep confined aquifers in the Elmira Road well field are gravel layers and lenses near the base of the Tehama formation. Water pumped from the well field consists of water stored in the aquifers and leakage from the confining layers. These waters join and flow toward the centers of pumping. Such aquifers respond to increases of pumping and to decreases of pumping, but not to recharge related to wet and dry periods. Compared to most municipal well fields, this compact well field has relatively low specific capacities with a large amount of mutual well interference.

2. It is imperative that static water levels in all wells in the Elmira Road well field be measured at least monthly. Before measuring the static level, the well must have been idle for about 10 hours, or overnight. At the time the static water level is measured, the length of time that the pump has been off should be noted.

3. A determination of the safe yield of the Elmira Road well field must await the accumulation of an adequate data base of monthly static water levels and monthly production from each well. The minimum time required would be two successive winters. Because of an expected difference between the manner in which the aquifer would respond to an increase in the overall rate of pumping as compared with the manner it would respond to a decrease in pumping, it may take several years of studying pumpage in relation to changes of static water levels before a refined safe yield determination may be possible.

4. In order to detect changes in well efficiency and pump efficiency, pumping levels should be measured weekly. Pump efficiency tests on each well should be made once a year, or more frequently if indicated by the pumping level changes.

5. The possible extension of the basal Tehama gravels to the east of the Elmira Road well field can be determined by a test hole drilled at the Easterly Sewage Treatment Plant. This hole should be drilled to a depth of at least 1500 feet, and electric logged.

6. Shallow aquifers (less than 500 feet deep) with high specific capacity wells are known in one area 6 miles northeast of Elmira and in another area 8 miles east of Elmira.

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GEOLOGIC CROSS-SECTION ALONG ELMIRA ROAD

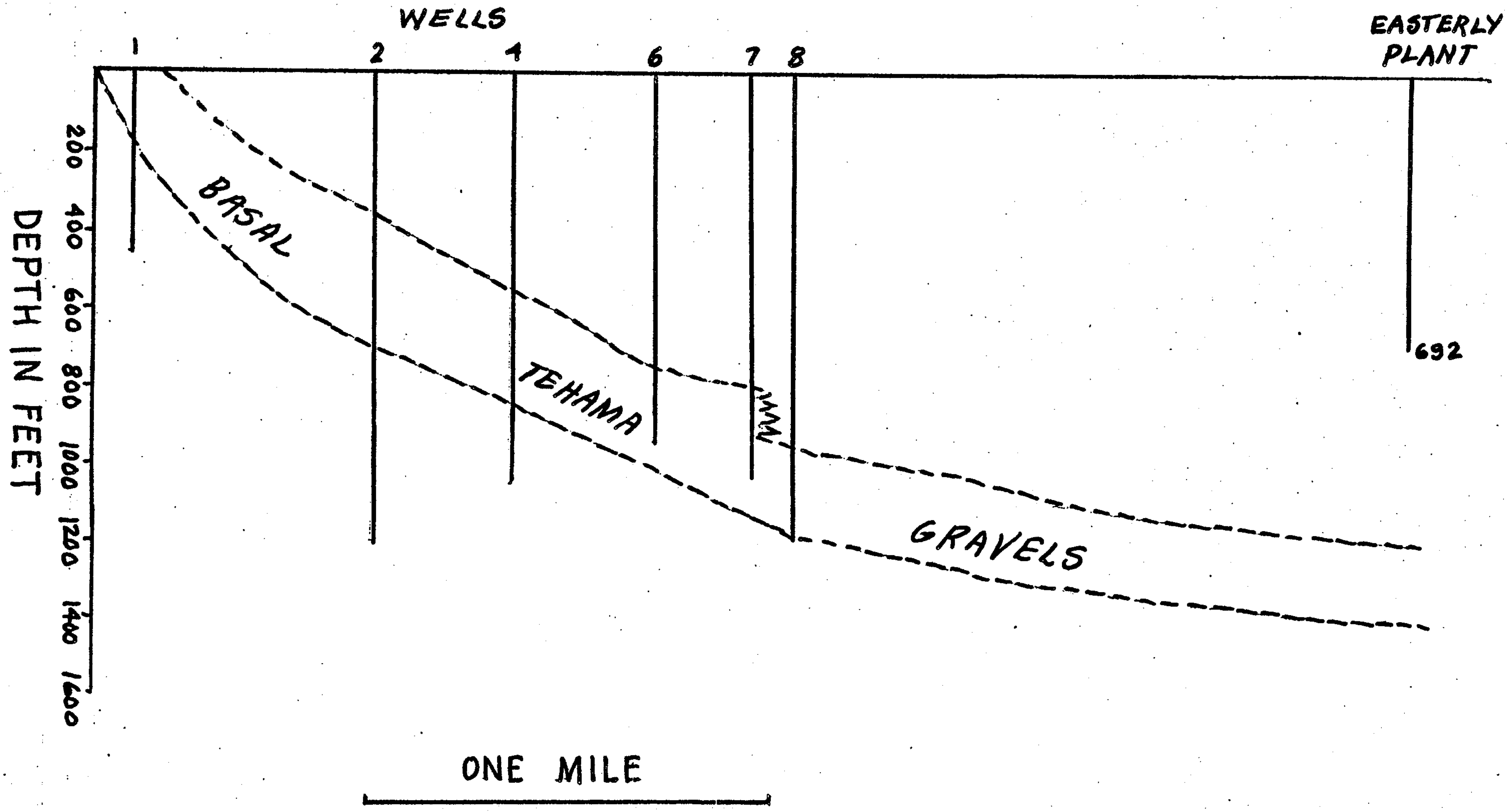

FIGURE 1

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4/85

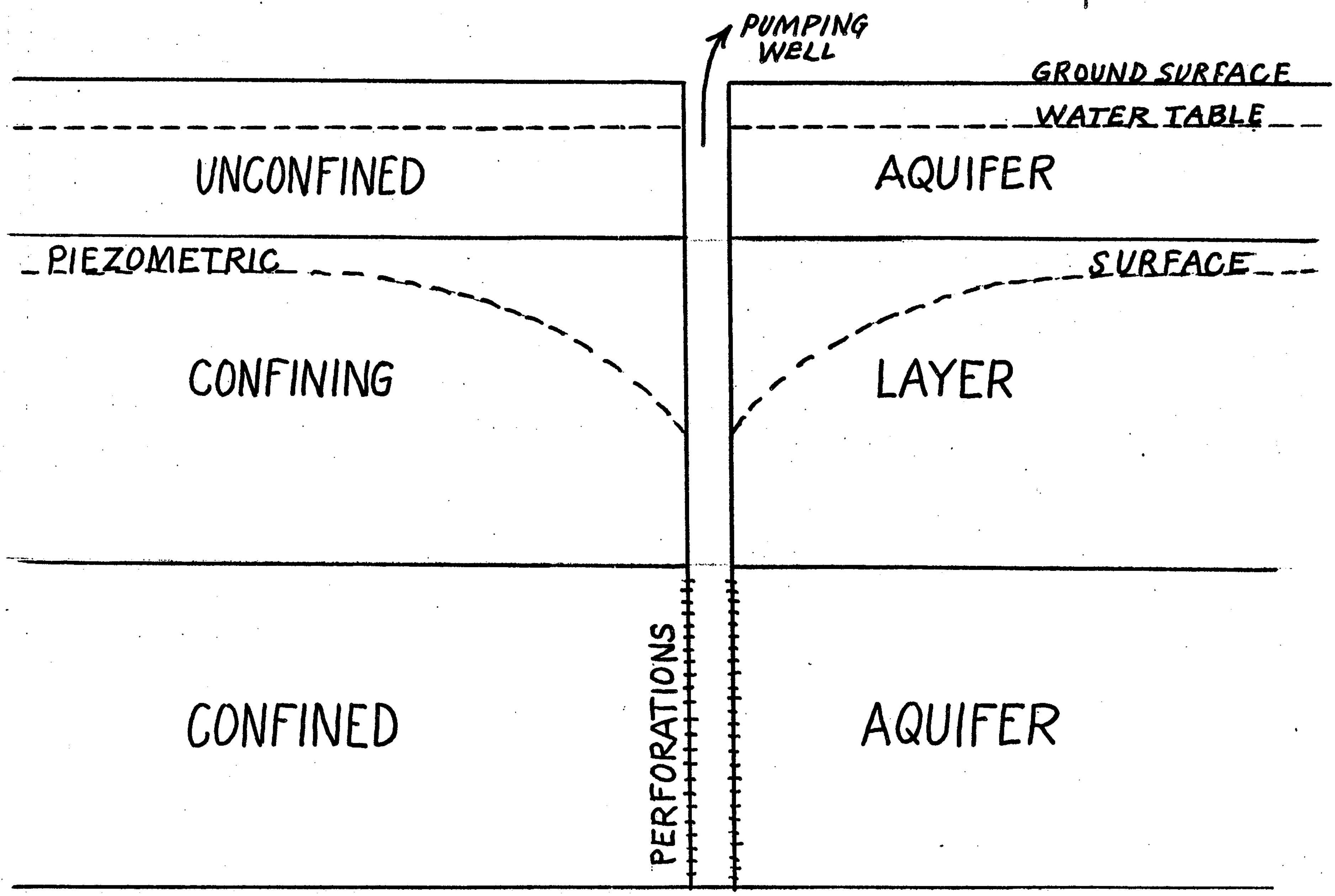
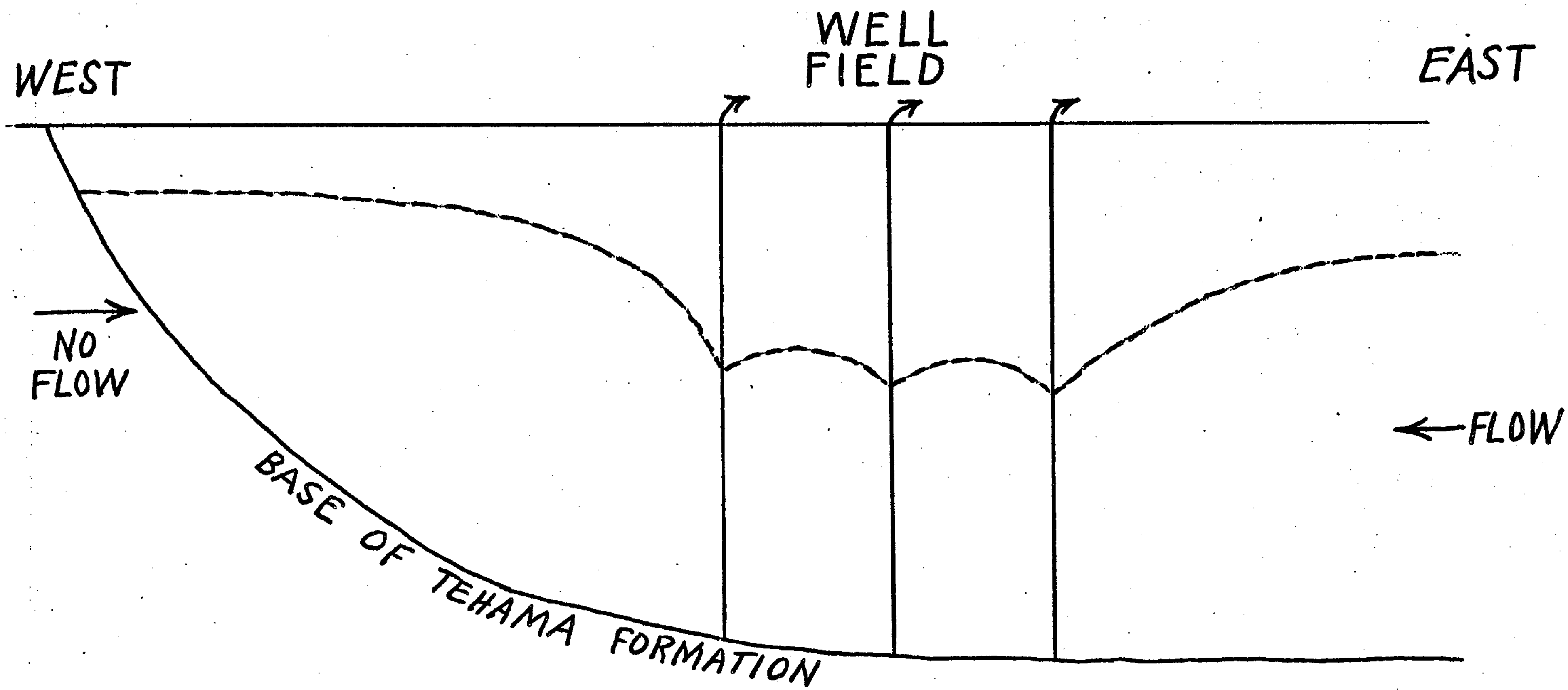

PUMPING WELL
GROUND SURFACE
WATER TABLE
UNCONFINED AQUIFER
PIEZOMETRIC SURFACE
CONFINING LAYER
PERFORATIONS
CONFINED AQUIFER

Fig. 2

JFM
4/85



FLOW INTO PUMPING HOLE

FIGURE 3

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4/85

SOURCES OF WATER TO PUMPING HOLE

(SEE U.S.G.S. WATER SUPPLY PAPER 2237)

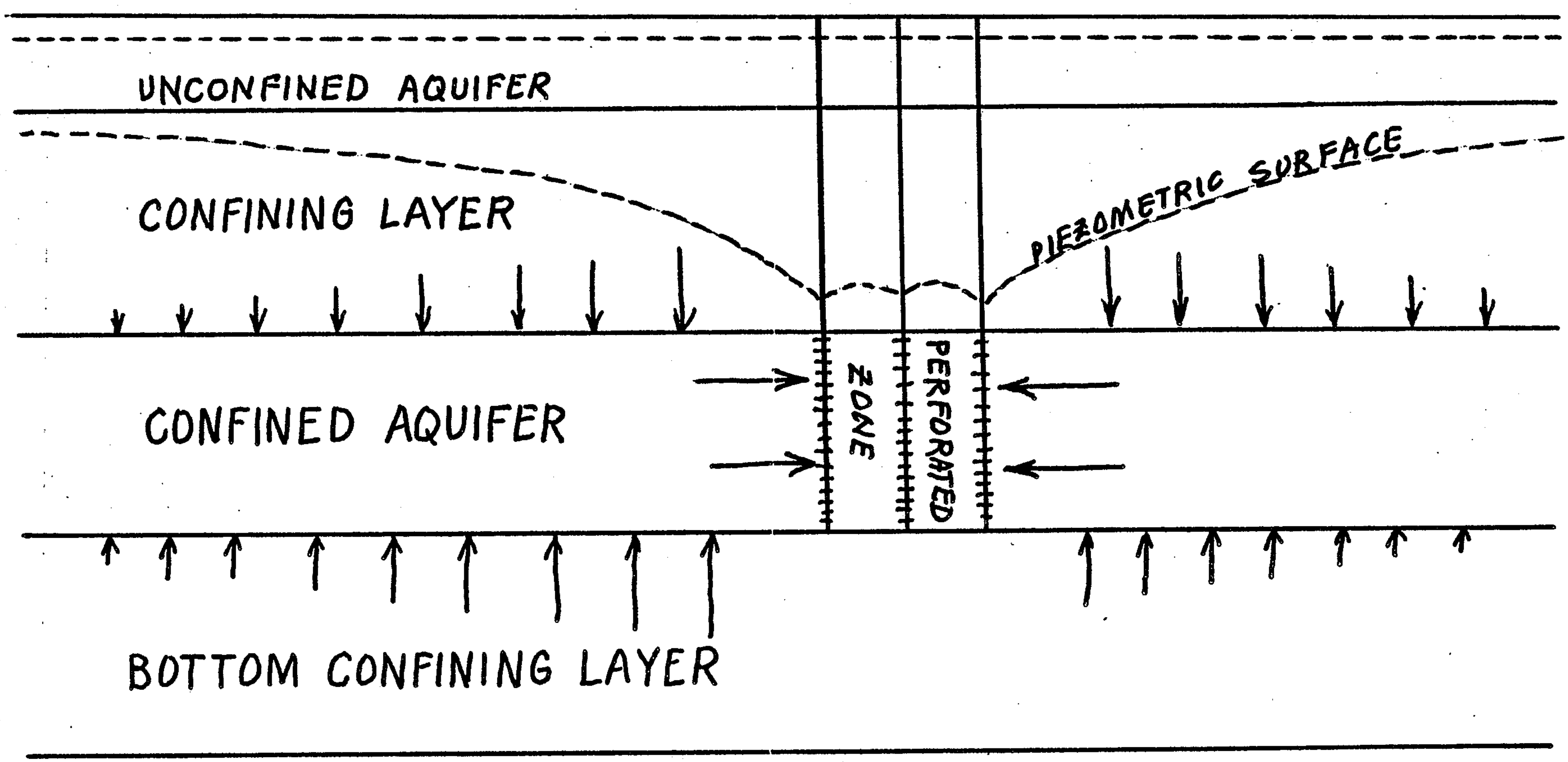

FIGURE 4

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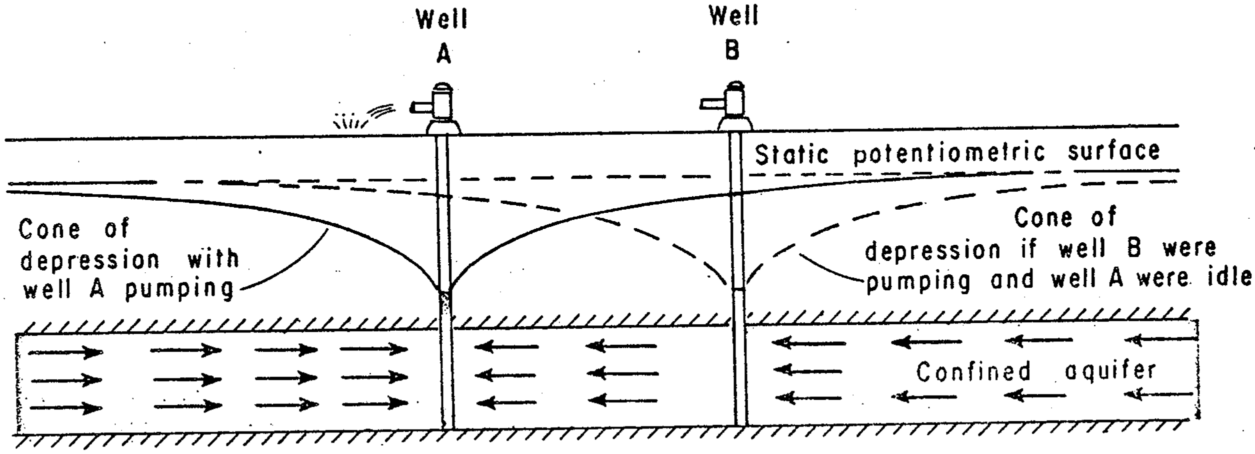
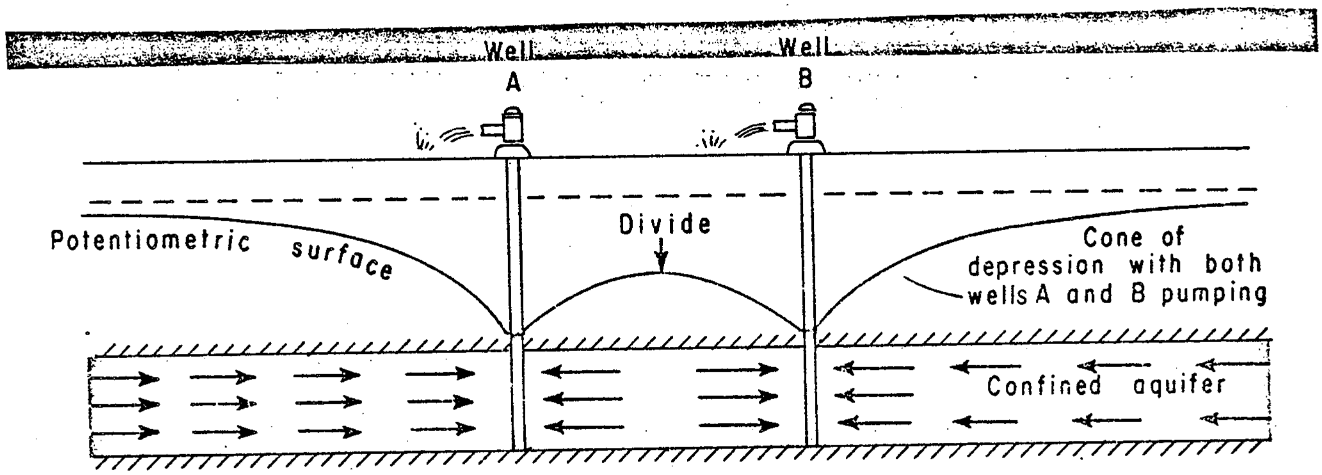
4/85

WELL INTERFERENCE

FIGURE 5
JFM
4/85



(1)



(2)

Pumping a well causes a drawdown in the ground-water level in the surrounding area. The drawdown in water level forms a conical-shaped depression in the water table or potentiometric surface, which is referred to as a *cone of depression*. (See "Cone of Depression.") Similarly, a well through which water is injected into an aquifer (that is, a recharge or injection well) causes a buildup in ground-water level in the form of a conical-shaped mound.

The drawdown (s) in an aquifer caused by pumping at any point in the aquifer is directly proportional to the pumping rate (Q) and the length of time (t) that pumping has been in progress and is inversely proportional to the transmissivity (T), the storage coefficient (S), and the square of the distance (r^2) between the pumping well and the point. In other words,

$$s \approx \frac{Q,t}{T,S,r^2} \qquad (1)$$

Where pumping wells are spaced relatively close together, pumping of one will cause a drawdown in the others. Drawdowns are additive, so that the total drawdown in a pumping well is equal to its own drawdown plus the drawdowns caused at its location by other pumping wells (1) (2). The drawdowns in pumping wells caused by withdrawals from other pumping wells are referred to as *well interference*. As sketch 2 shows, a divide forms in the potentiometric surface (or the water table, in the case of an unconfined aquifer) between pumping wells.

At any point in an aquifer affected by both a discharging well and a recharging well, the change in water level is equal to the difference between the drawdown and the buildup. If the rates of discharge and recharge are the same and if the wells are operated on the same schedule, the drawdown and the buildup will cancel midway between the wells, and the water level at that point will remain unchanged from the static level (3). (See "Aquifer Boundaries.")

City of Vacaville - Static Water Levels

Date	(OLD # 3) Well #1	(OLD # 11) Well #2	(OLD # 7) Well #4	(OLD # 12) Well #5	(OLD # 8) Well #6	(OLD # 10) Well #7	(OLD # 9) Well #8
29-Jan-73	152.00 3		130.00			130.00	
30-Jun-73					131.00 ?		
02-Oct-73	114.00						
04-Mar-75							96.50
07-Aug-75							144.00
09-Mar-76							144.00
30-Jun-78			160.00 ?				
31-Mar-80		170.00					
17-Oct-80				205.75			
22-Feb-82							203.00
02-Mar-82							204.00
27-Apr-82							208.00
28-Apr-82							213.00
04-May-82							214.00
05-May-83	100.00						
08-Sep-83					250.00 240		
26-Sep-83			237.00				
07-Dec-83			241.00 ?				
30-Jan-84					228.00		
27-Feb-84					234.00		
05-Mar-84			224.00				
09-Apr-84			212.00				
26-Apr-84	122.66	215.00	212.00 15	206.82	206.83	210.35	205.80
10-May-84				201.00			
02-Aug-84	117.00						
15-Aug-84				242.90			
01-Oct-84	115.00						

TABLE I

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4/85

APPENDIX B

URBAN WATER MANAGEMENT PLAN FOR THE CITY OF VACAVILLE

Following is the main text from the Urban Water Management Plan for the City of Vacaville prepared by Nolte and Associates in 1986. Water demand projections and the supply/demand analysis have been superceded by the 1989 Draft Water System Master Plan, but the basis concepts of the management plan remain valid.

RECOMMENDED URBAN WATER MANAGEMENT PLAN

FOR

THE CITY OF VACAVILLE

In Response to

California Assembly Bill No. 797

PREPARED BY

GEORGE S. NOLTE AND ASSOCIATES

January 1986

R0001-B

TABLE OF CONTENTS

Section	Page
I. INTRODUCTION	I-1
II. WATER USE	II-1
A. HISTORICAL RECORDS	II-1
B. CURRENT USES	II-1
C. FUTURE PROJECTIONS	II-5
D. SUPPLY DEFICIENCY ANALYSIS	II-5
III. CONSERVATION MEASURES	III-1
A. WATER CONSERVATION AGENCIES	III-1
B. EDUCATION AND PUBLIC INFORMATION	III-4
C. WATER MANAGEMENT PROGRAMS	III-6
D. REGULATIONS	III-8
E. WATER EMERGENCY PLAN	III-9
IV. BENEFITS OF CONSERVATION MEASURES	IV-1
A. VOLUME PRODUCTION	IV-1
B. VALUE OF SAVED WATER	IV-5
V. COSTS OF CONSERVATION MEASURES	V-1
VI. BENEFIT COST COMPARISON	VI-1
VII. IMPLEMENTATION SCHEDULE	VII-1
VIII. ADDITIONAL CONSIDERATIONS	VIII-1
A. WASTEWATER RECLAMATION	VIII-1
B. WATER EXCHANGE OR TRANSFER	VIII-1
C. WATER PRESSURE MANAGEMENT	VIII-2
D. IMPACTS OF THE CONSERVATION PLAN	VIII-2
REFERENCES	R-1
APPENDICES	
A. ASSEMBLY BILL NO. 797	A-1
B. RECOMMENDED WATER MANAGEMENT PLAN FOR SOLANO COUNTY FLOOD CONTROL AND WATER CONSERVATION DISTRICT	A-2
C. ESTIMATED COST OF CONSERVATION MEASURES	A-3

I. INTRODUCTION

In accordance with State of California Assembly Bill No. 797, the Urban Water Management Plan for the City of Vacaville is presented in this report. The bill requires all water suppliers in the State of California with more than 3,000 customers or a demand exceeding 3,000 acre-feet annually to prepare and adopt such a plan. A copy of the bill is provided in Appendix A.

In 1982, State Water Project water service contractors, under the Governor's Executive Order B 68-80, were required to prepare water management plans, herein referred to as B 68-60 plans. The City of Vacaville was included in the plan prepared for Solano County Flood Control and Water Conservation District. Other cities included in this regional plan included Vallejo, Benicia, Fairfield, and Suisun City. The plan meets most of the requirements in Assembly Bill No. 797 but is deficient in some respects. In a letter dated December 13, 1984 from Suzanne Butterfield of the Department of Water Resources, Office of Water Conservation, to Eugene Knapp of the Solano County Flood Control and Water Conservation District, the deficincies of the B 68-60 plan in meeting the requirements of Assembly Bill No. 797 are outlined. The deficiencies are summarized below.

1. Estimates of projected water use should be updated.
2. Current conservation practices should be updated.
3. Specific impacts to the service area as a result of conservation measures should be given further consideration.
4. A supply deficiency analysis should be made for the specific service area.
5. Wastewater reclamation potential should be addressed for the specific service area.
6. Wastewater exchange/transfer potential should be made for the specific service area.
7. Water pressure problems must be addressed.
8. The incremental cost of expanded water supplies should be estimated.

Despite these deficiencies, the conservation measures recommended in B 68-60 plan are still applicable. Therefore, much of the conservation plan proposed herein is based on the B 68-60 plan.

The scope of work in the following report is intended to meet the requirements of Assembly Bill No. 797 as a minimum. Additionally, the water conservation requirements set forth in Amendment 7 of the contract between the Department of Water Resources and the Solano County Flood Control and Water Maintenance District will be met. In particular, this amendment calls for specific goals in the volume of water conserved as a result of the Assembly Bill No. 797 conservation plan.

The first task in the following report is presentation of historical and projected water use for the City of Vacaville and comparison with available supplies. Next, conservation measures from the B 68-60 plan are briefly summarized and modified in some cases to suit the specific needs of Vacaville. Following, benefits from the conservation plan in terms of the volume of water saved and the associated dollar value of the savings are estimated. The savings are then compared with the costs of implementing the conservation programs. An implementation schedule for the recommended measures is then presented in tabular form. Finally, additional consideration of wastewater reclamation, water exchange or transfers, and water pressure are discussed.

A list of abbreviations used throughout this report is provided below:

Abbreviation	Definition
AF	acre feet
AF/yr	acre feet per year
gpd	gallons per day
gpcd	gallons per capita per day
sq ft	square feet
mgd	million galons per day
MG	million gallons

II. WATER USE

The discussion of water use is divided into Historical Records, Current Uses, Future Projections, and Supply Deficiency Analysis.

A. HISTORICAL RECORDS

Values of water production and population from 1970 through 1984 are presented in Table II-1. Water production values are based on actual city records. Population data originated from a number of sources including census data in 1970 and 1980, official counts in 1976 and 1985, and city estimates for January 1 of the remaining years. The population data was adjusted for Table II-1 to reflect approximate mid-year levels. From the water production and population records, per capita demand is calculated. There is an apparent trend in increasing per capita demand between 1970 and 1984. Exceptions to this trend include the drought in 1976-77 and the wet period of 1982-83. One possible explanation for the increase in per capita demand is the influx of industrial development into Vacaville over the same period.

B. CURRENT USES

The division of water use between residential, commercial, industrial and other demands is shown in Table II-2. The information is based on the city records of metered water use in 1984. Clearly, residential demands at 68.4 percent of the total are the dominant use in Vacaville. The residential portion of 1984 demands is further subdivided as shown in Table II-3. The division of residential use is based on three assumptions used by the Department of Water Resources for preparation of B 68-80 plans (DWR, 1983, Appendix B):

1. Interior residential water demands are equal to 80 gpcd for single family and multiple family dwellings combined.

2. Multiple family dwelling exterior use is equal to six-tenths (0.6) of single family dwellings.

3. The partition of interior residential use is 42 percent toilet, 30 percent bath/shower, 14 percent laundry, 6 percent dishwashing, 4 percent cooking, 2 percent kitchen faucet, and 2 percent bath faucet.

TABLE II-1

HISTORICAL WATER PRODUCTION
AND POPULATION IN VACAVILLE

Year	Water Production[1] (gpd)	Population[2]	Per Capita Water Production (gpcd)
1970	3,690,000	21,980	168
1971	4,052,090	23,430	173
1972	4,197,000	24,950	168
1973	4,389,000	26,450	166
1974	4,674,000	27,430	170
1975	5,090,000	28,930	176
1976	6,126,000	31,120	197
1977	5,595,000	35,100	159
1978	6,480,000	37,680	171
1979	7,110,000	40,880	174
1980	7,882,000	43,370	182
1981	8,326,000	44,790	186
1982	8,340,000	46,780	178
1983	8,723,000	47,800	182
1984	9,997,000	49,070	203

[1] City records.
[2] Estimates based on city records.

TABLE II-2

1984 METERED WATER USE IN VACAVILLE

Category of Use	Water Consumption (gpd)	Water Consumption % of Total
RESIDENTIAL		
Single Family	4,969,200	54.28
Multiple Family/Mobile Home	1,292,400	14.12
Subtotal	6,261,600	68.40
COMMERCIAL		
General Commercial	387,500	4.23
Hotels/Motels	27,300	0.30
Restaurants	167,900	1.83
Construction Water	11,500	0.13
Subtotal	594,200	6.49
INDUSTRIAL		
Basic Foods	498,700	5.45
CMF	694,800	7.59
Class I/Class II	481,300	5.26
Subtotal	1,674,800	18.30
GOVERNMENT		
City Water	413,400	4.52
Schools	132,500	1.45
Parks	37,400	0.41
Subtotal	583,300	6.38
OTHER		
Fire Protection	1,800	0.02
Other	37,700	0.41
Subtotal	39,500	0.43
TOTAL	9,153,400	100.00

SOURCE: City of Vacaville records.

TABLE II-3

ESTIMATED DIVISION OF RESIDENTIAL
WATER DEMANDS FOR VACAVILLE IN 1984

Category of Use	Water Consumption (gpd)	% of Total
EXTERIOR		
Single Family	1,940,100	21.19
Multiple Family	396,000	4.33
Subtotal	2,336,100	25.52
INTERIOR (Single Family and Multiple Family Combined)		
Toilet	1,648,700	18.01
Bath/Shower	1,177,700	12.86
Bath Faucet	78,500	0.86
Laundry	549,600	6.00
Dishwashing	235,500	2.57
Kitchen Faucet	78,500	0.86
Cooking	157,000	1.72
Subtotal	3,925,500	42.88
TOTAL	6,261,600	68.40

SOURCE: Nolte 1985; see paragraph II.B for assumptions.

Additionally, it is assumed that the number of single family water meters installed in Vacaville in 1984 is equivalent to the number of single family dwellings. Furthermore, single family dwellings are assumed to have 3.10 cap/du in 1984 while multiple family dwellings only have 2.88 cap/du.

The percentage of total water use in 1984 for each category is assumed to be typical of future water use patterns. The percentages are used to determine future demands in each category which are subsequently used to calculate reductions in demand as a result of conservation measueres.

C. FUTURE PROJECTIONS

Projections of water demand are calculated from expected residential, commercial and industrial development in Vacaville. Recommendations from the City Staff regarding the number of new single family and multiple family dwelling units and the square footage of industrial and commercial developments are given in Table II-4. Demands for the respective types of development are based on information and research performed for the upcoming Water Master Plan for the city prepared by George S. Nolte and Associates. For the city as a whole, it is assumed that the number of people per dwelling unit decreases from 3.0 in 1986 to 2.6 in 2005 and remains at 2.6 thereafter. It is also assumed that annual average demands of single and multiple family homes are 420 gpd and 320 gpd, respectively. Industrial and commercial demands are estimated as 86 gpd and 129 gpd per 1,000 square feet, respectively. Projected population and water demands for Vacaville are given in Table II-5.

D. SUPPLY DEFICIENCY ANALYSIS

A graph of projected annual demand and known supplies is shown in Figure II-1 for Vacaville from 1986 to 2010. The three sources of water include ground water, Solano Irrigation District, and North Bay Aqueduct allotments. The SID allotment is shown as a constant 5600 AF/yr. However, this constant value of 5,600 AF/yr ignores the constant buildup of water allotments from SID as a result of the agreement between the City and SID which stipulates that 2.5 AF/yr will become available for each acre of land developed in the north industrial area up to a maximum of 2,350 acres. This quantity is ignored because of uncertainty about the buildup rate of this land. Thus, the water supply estimates will be conservative. The NBA allotment increases from 100 AF/yr in 1986 to 6,100 AF/yr by 1996. The quantity of ground water available is a function of the recharge potential of the deep aquifer used for domestic supply. Currently, the recharge potential is not accurately defined, so a broad range of potential annual use from 3,000 to 9,000 AF/yr is shown in Figure II-1. At 3,000 AF/yr the aquifer will probably be recharged on an annual basis, but at 9,000 AF/yr the recharge potential may be exceeded. Thus, the higher levels of ground water withdrawal should not be maintained for more than a few consecutive years.

In Figure II-1 two demand lines are plotted. The first shows the projected demands in Table II-5 which represents the "no-conservation case." The second demand curve accounts for the projected water savings as a result of the proposed conservation measures discussed throughout the remainder of this report. Apparently, without conservation, Vacaville has ample water on an annual basis until about 2008 when ground water withdrawal becomes excessive. On the other hand, with conservation measures, there should be ample water supply until nearly the year 2015. However, as noted earlier, these projects do not account for water from development of the north industrial area. Such entitlements of water could accommodate an additional 10 to 15 years of growth if they were all acquired.

On a short-term basis, such as a peak day, Vacaville is not expected to have deficiencies in water supply. The major factors preventing short-term deficincies are listed below.

1. Ground water can be used at high rates during summer months and low rates during the winter to dampen fluctuations in the amount of surface water required.

2. Utilization of two surface water supplies assures continued availability of some surface water at all times.

3. New treatment and raw water storage facilities (NBA) are in the planning stages.

4. Increased distribution system storage will be recommended in the upcoming master plan for Vacaville water facilities.

TABLE II-4

PROJECTED DEVELOPMENT IN VACAVILLE[1]

Year	Residential: Single Family Units	Residential: Multiple Family Units	Industrial (sq ft)	Commercial (sq ft)
1985	300	200	350,000	50,000
1986	400	150	450,000	60,000
1987	450	150	550,000	80,000
1988	450	200	650,000	140,000
1989	500	200	700,000	60,000
1990	600	250	750,000	80,000
1991	600	250	750,000	80,000
1992	600	250	750,000	80,000
1993	600	250	750,000	80,000
1994	600	250	750,000	80,000
1995	600	250	750,000	80,000
1996	600	250	750,000	80,000
1997	600	250	750,000	80,000
1998	600	250	750,000	80,000
1999	600	250	750,000	80,000
2000	600	250	750,000	80,000
2001	600	250	750,000	80,000
2002	600	250	750,000	80,000
2003	600	250	750,000	80,000
2004	600	250	750,000	80,000
2005	600	250	750,000	80,000
2006	600	250	750,000	80,000
2007	600	250	750,000	80,000
2008	600	250	750,000	80,000
2009	600	250	750,000	80,000
2010	600	250	750,000	80,000

SOURCE: City of Vacaville staff estimates.

1 The revised residential development figures are 290 single family (SF) and 1,500 multiple family (MF) in 1985, and 500 SF and 200 MF in 1986 through 1988. These revised figures would be important in a short-term water supply analysis but have minimal impact on the findings in this long-range water conservation plan.

TABLE II-5

ESTIMATED WATER DEMAND AND POPULATION
IN VACAVILLE

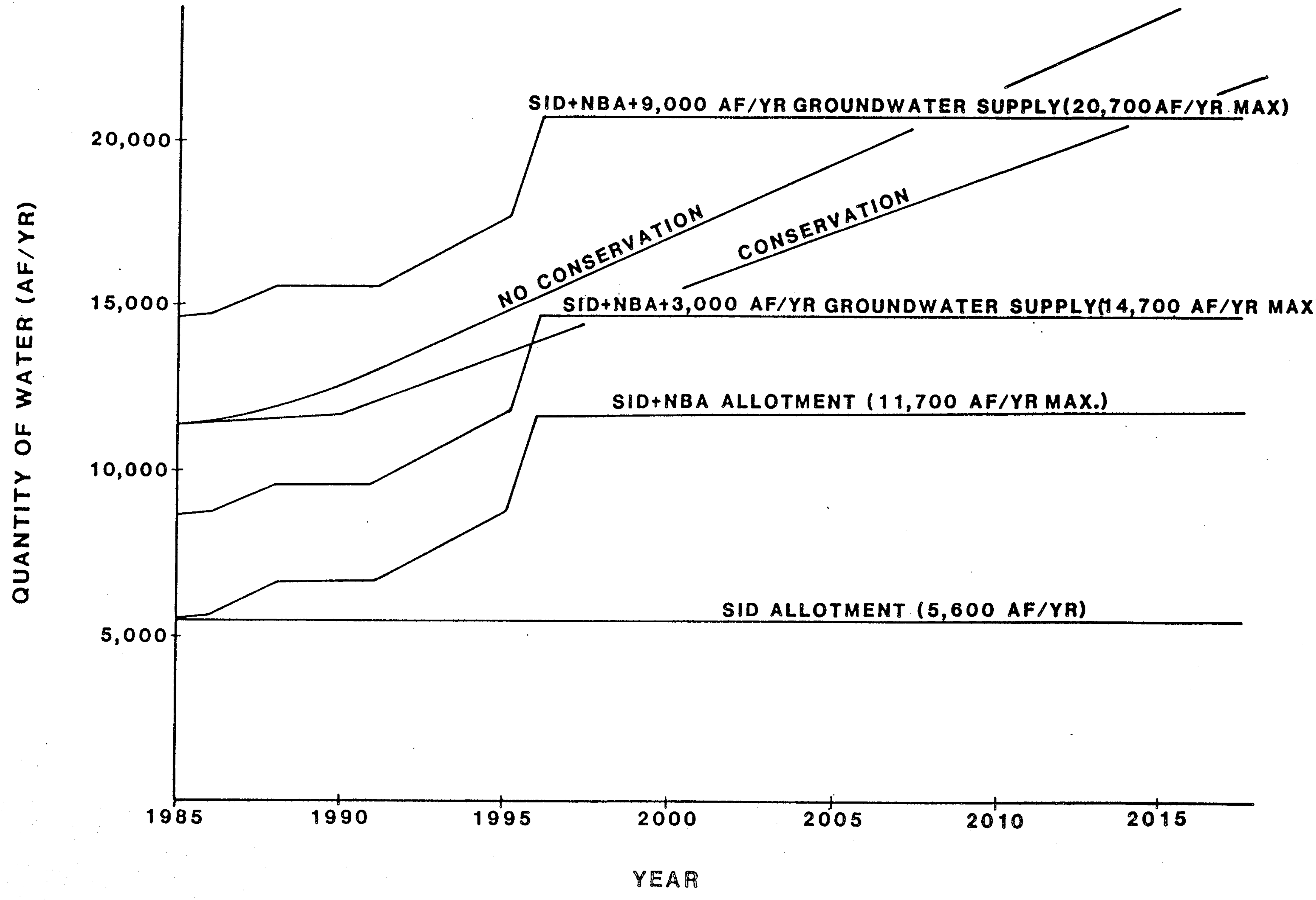
Year	Average Day Water Demand (gpd)	Population
1985	10,226,000	50,800
1986	10,489,000	52,450
1987	10,083,000 [1]	54,238
1988	10,410,000	56,156
1989	10,752,000	58,207
1990	11,159,000	60,672
1991	11,556,000	63,120
1992	11,973,000	65,551
1993	12,380,000	67,965
1994	12,787,000	70,362
1995	13,194,000	72,742
1996	13,601,000	75,105
1997	14,008,000	77,451
1998	14,415,000	79,780
1999	14,882,000	82,092
2000	15,229,000	84,387
2001	15,636,000	86,665
2002	16,043,000	88,926
2003	16,450,000	91,170
2004	16,857,000	93,397
2005	17,264,000	95,607
2006	17,671,000	97,817
2007	18,078,000	100,027
2008	18,485,000	102,237
2009	18,892,000	104,447
2010	19,299,000	106,657

SOURCE: Nolte, 1985.

[1] California Medical Facility demands taken off city system in 1987.

FIGURE 2-1

WATER DEMAND AND AVAILABLE
WATER SUPPLY FOR VACAVILLE



III. CONSERVATION MEASURES

The list of water conservation measures which are considered in this report are shown in Table III-1. The current status of each measure is noted in the table. A brief description of each of the measures is provided in the following pages. The description of the measures summarizes the contents of the B 68-60 plan and modifies the plan where appropriate for specific conditions in Vacaville. A copy of the B 68-60 plan is provided in Appendix B for a more complete discussion of the measures.

A. WATER CONSERVATION AGENCIES

An organization or committee in charge is an important part of the conservation plan. The main functions of the organization should be: (1) to promote water conservation awareness, (2) to develop specific conservation programs, (3) to oversee implementation of the programs, and (4) to collect information and evaluate performance of the programs. It is recommended that this organization should function on both a regional and local basis.

Many conservation measures are most effective and economical on a large scale; so the conservation plan proposed in this report should be an integral component of a larger regional plan. The cities included in the regional plan are assumed to be Benicia, Fairfield, Suisun City, Vacaville and Vallejo, and the regional organizational body is referred to as the "lead agency" throughout the remainder of this report. Formation of the lead agency is assumed to be the responsibility of the Solano County Flood Control and Water Conservation District in conjunction with the respective city governments. It is recommended that the lead agency include representatives of residential, commercial, industrial and governmental interests. Other groups such as public service or professional organizations would also provide valuable representation.

In addition to the lead agency, a local organization referred to as the "support agency" should be closely tied to the city government of Vacaville and in particular the Community Development Department. It is anticipated that the support agency will have similar functions to the lead agency except it will work on a local level and emphasis will be on implementation of conservation measures. It is assumed that similar support agencies will be established in each of the other major cities in Solano County.

TABLE III-1

WATER CONSERVATION MEASURES

Measure	Status [1]
A. WATER CONSERVATION AGENCIES	R
B. EDUCATION AND PUBLIC INFORMATION	
1. Conservation Literature	
a. General Brochure	I
b. Landscape Brochure with Plant List	I
c. Brochures for Specific Users	R
2. Previous Year's Use on Water Bills	R
3. Promotional Measures	
a. Publicity Campaigns	I
b. Public Speaking Presentations	I
c. Demonstration Low Water Using Landscapes	R
d. Promotional Campaign with Nurseries	R
4. Work with Large Water Users	R
5. In-School Education	R
6. Information on Federal and State Laws and Programs	R
C. WATER MANAGEMENT PROGRAMS	
1. Water Loss Reduction Techniques	
a. System-Wide Water Audit	I
b. Leak Detection Program	I
c. Meter Calibration and Replacement Program	I
d. Corrosion Control Program	R
e. Valve Exercising Program	I
2. Metering Existing Customers	C
3. Device Distribution	C
4. Meter Loan Program	R
5. Pricing	R

(Continued)

TABLE III-1

WATER CONSERVATION MEASURES
(Continued)

Measure	Status [1]
D. REGULATIONS	
1. Environmental Impact Reports/Statements	R
2. Water Waste Reduction Programs	R
3. Water Conservation Ordinances	
a. Requirements of Low Water Users	R
b. Self-Closing Faucets	I
c. Low Water Using Landscapes	R
E. WATER EMERGENCY PLAN	R

[1] Status Key: R = Implementation recommended.
I = Increased effort recommended.
C = Currently in full use.

Both the lead and support agencies are integral parts of the conservation measures described through the remainder of this report. Some of the costs and responsibilities of the plan are assigned to each agency.

B. EDUCATION AND PUBLIC INFORMATION

Education and public information practices are intended to alter water use patterns to reduce consumption. Possible ways to inform users which are suggested herein are with brochures, promotional campaigns, and education in schools.

1. Conservation Literature

Conservation literature serves to inform the public of ways in which to save water and acts as a reminder to be conscientious of water use. As in the B 68-60 plan, three types of literature should be considered for distribution including a general water conservation brochure, a landscape water conservation brochure with a plant list, and literature with information pertaining to specific users. Further discussion of the literature is provided in Appendix B.

The conservation literature should be prepared and printed by the lead agency to take advantage of any economies of scale, but distribution should be primarily the responsibility of the support agency. The focus of distribution efforts should be through the mail, and a frequency of two to three times a year is recommended. Additional ways of making the literature available include distribution at public gatherings, speaking engagements, libraries, schools and in public offices.

2. Previous Year's Use on Water Bills

Noting the previous year's use on the water bills increases user awareness of water consumption and motivates the user to improve conservation efforts. In Vacaville, records of water use can be stored in the city's computer for two years (12 billing periods). This capacity will facilitate implementation of this recommended conservation measure.

3. Promotional Measures

Promotional measures recommended for consideration in Vacaville include publicity campaigns, public speaking presentations, demonstrations of low water using landscapes, and promotional campaigns with nurseries.

Publicity campaign measures include public service announcements or press releases concerning use of water. Typically, these measures should be promoted on a regional basis by the lead agency to take advantage of media that will reach all of the individual cities. However, the support agency should also contribute with posters, bumper stickers, or ads in local papers.

Public speaking presentations are recommended on both the local and regional levels. The support agency should have a representative available to give speeches to community groups, professional associations, local government agencies, businesses and schools. Slide shows, workshops or speeches should also be promoted by the lead agency.

It is also recommended that consideration be given to establishing demonstration low water using landscapes to help eliminate the misconception that these landscapes are dry and unsightly. Selection of the specific sites should be the responsibility of both the lead and support agencies. The locations of the demonstrations should be widely publicized to draw attention to them, and specific information of important water saving aspects of the landscapes should be available so other landowners will be able to implement similar measures themselves.

Complimentary with the promotion of demonstration lanscapes is involvement of nurseries in conservation programs. By encouraging nurseries to regularly supply and identify low water using plants and to supply literature on low water using landscapes, individual landowners will be able to effectively plan their landscapes. Responsiblity of working with the nurseries should be with both the lead and support agencies.

4. <u>Work with Large Water Users</u>

The support agency should give special attention to large water users in their service area by promoting voluntary water audits or metering programs. The promotion could be conducted by literature, contact by phone or advertisements. The support agency should be available for assistance in establishing such programs but should not assume the lead role in the programs because the individual users will best know their own water needs.

5. <u>In-School Education</u>

In-school education of water use and water conservation is highly recommended. In-school education will help develop good habits in the youth which they will carry into adulthood. Also, by bringing home and adopting conservation practices, the students will encourage their families to conserve water. The lead agency and the support agency should work with local school districts to promote these educational programs. Teacher-training workshops and acquisition of supplies such as literature or devices should be performed by the lead agency.

6. Information on Federal and State Laws and Programs

The lead agency should assume the responsibility of keeping track of water conservation laws and programs and passing it on to the support agencies and local governments.

C. WATER MANAGEMENT PROGRAMS

There are a variety of programs that can be conducted by the water utility to reduce water losses and water consumption. Water loss reduction programs include water audits, leak detection, meter calibration, corrosion control and valve exercising. Reduced water consumption can be promoted by device distribtion and the rate structure for water prices.

1. Water Loss Reduction Techniques

The first step in reducing water losses from the system is to determine the quantity and source of water losses. In 1984, metering records for Vacaville show an average annual consumption of 9.153 mgd with water production of 9.997 mgd. The difference is equal to 9.2 percent of the metered uses. This discrepancy is considered significant, so a water audit should be considered.

In the B 68-80 plan, a leak detection program is recommended for the agency's side of the water distribution system if unaccounted losses exceed 10 percent of the demand. In Vacaville, the losses are slightly less than 10 percent, but it is assumed that a leak detection program would probably be beneficial. The actual need for a leak detection program will be more apparent following results of the water audit. Possible components of a leak detection program include locating leaks by mechanical/accoustical devices and investigating customers with abnormally high water usage. Currently, Vacaville checks for leaks on an as-needed basis and checks for unusually high water bills. However, increasing the level of effort should be considered.

A scheduled meter calibration and replacement program is another important means to detect and reduce unaccounted water losses. It is recommended that Vacaville consider developing such a program.

Although Vacaville currently replaces old pipe with non-corrosive new pipe, further investigation into the need for a corrosion control plan is recommeneded. Components of the program may include inspection and cathodic protection.

Previously, Vacaville had a valve exercising program. This practice should be restarted, and a schedule should be developed. The exercising program should also accompany a replacement program for non-working valves.

2. Metering Existing Customers

Currently, all water taken through permanent services in Vacaville is metered. Continued metering of existing and future customers is highly recommended. The possibility of metering construction water in the future should also be considered.

3. Device Distribution

Distribution of water saving kits is recommended as part of the conservation plan. The kits should include a displacement bag for the toilet, two sets of shower flow restrictors and two dye tablets. The actual method of distribution used should be selected by the support agency; but for purposes here, a mass mailing program is assumed. It is recommended that the recipients should be limited to pre-1980 residences as homes built after that time likely have low water use facilities already in operation. The major distribution efforts should be repeated approximately once every five years. Kits are currently available in Vacaville upon request, and this practice should be continued in the future for the period between mailings.

The lead agency is assumed to be responsible for advertising and public relations of the device distribution program on a regional level. The lead agency is also assumed to be responsible for ordering the kits for all of the cities in Solano County to reduce the unit cost, but it is assumed that the support agency pays for the kits and distribution.

4. Meter Loan Program

In order to help evaluate the water use patterns in large public, industrial and commercial facilities, it is recommended that a meter loan program be considered in Vacaville. In such a program, the support agency could acquire meters of various sizes, notify large water users of their availability, and assist in setting up metering plans.

5. Pricing

Vacaville is currently reviewing water pricing structure in-house and with a private consultant. Possible considerations include but are not limited to: (1) adoption of a seasonal pricing structure, (2) elimination of minimum charges and service charges, (3) adoption of a low water rate for low levels of water use, (4) different rates for various classes of water use, and (5) surcharges for water consumption in higher level pressure zones.

D. REGULATIONS

Education and public information programs encourage conservation through voluntary efforts, and lack of participation can be a significant problem. Therefore, regulations requiring mandatory participation in water conservation efforts should be considered. Inclusion of water conservation measures in Environmental Impact Reports, adoption of a water waste reduction program, and establishment of water conservation ordinances are potential types of regulations.

1. Environmental Impact Reports/Statements

Local government agencies responsible for reviewing EIR's should require water conservation practices to be evaluated as mitigation measures whenever applicable. This practice would have a minor cost to the city but could potentially be very effective, particularly in new subdivision construction.

2. Water Waste Reduction Program

The main task of initiating a water waste reduction program is to establish ordinances for eliminating wasted water. The focus of attention should be on exterior use and in particular unnecessary application of water that runs off of the intended application site. The ordinances should include definitions for wasted water and should establish an enforcement mechanism.

The responsibility for forming the program should be with the support agency, although the lead agency may assist with guidance and promotion. Implementation of the program should be the responsibility of the support agency.

3. Water Conservation Ordinances

The support agency should consider adopting new water conservation ordinances in Vacaville. Three types of new ordinances are suggested for evaluation. First, all large water users should be required to submit a conservation plan to the city when applying for a hookup to the city water system. The plan should identify quantities and uses of water and proposed conservation measures. Second, self-closing faucets and other conservation measures should be required in heavy-use public restrooms. Third, all new public facilities, commercial, industrial or multi-family developments should be required to incorporate low water using features into their landscapes such as drip irrigation systems, low water use vegetation or alternate sources of water. The support agency should work with the Fire Department, Community Development Department, and experienced landscape personnel to review plans for new construction and evaluate the landscape designs. For further discussion concerning these ordinances see Appendix B.

E. WATER EMERGENCY PLAN

Although it is not intended to conserve water on a regular basis, an emergency plan should be established for periods of low water availability. The plan should be set up by the support agency in consultation with the lead agency. It should include a "staged" series of measures to be used under worsening water shortage conditions. A brief outline of one possible series of staged measures is provided as follows. However, a thorough investigation will be needed to select the final plan.

Non-Public Measures:

1. Stop or reduce irrigation of city parks.
2. Request that the California Medical Facility conserve water.
3. Request that major industrial users in the area conserve water.

Public Measures:

4. Use media announcements to encourage voluntary conservation of water.

Public Measures Under Proposed Water Waste Ordinances:

5. Limit lawn watering and irrigation of residential, commercial and industrial areas to early morning hours.
6. Prohibit all lawn watering, irrigation or other luxury activities such as car washing.

IV. BENEFITS OF CONSERVATION MEASURES

In The Recommended Water Management Plan for Solano County Flood Control and Water Conservation District (B 68-60 plan), the estimated volume of water saved from the total conservation plan is given for the years 1990, 2000 and 2010, but a breakdown of the volume of water saved for different components of the plan is not provided. The objectives of the following section are to increase the level of detail of the water saving estimates and to re-estimate savings based on the projected demands in Table II-1. From the estimates of the volume of saved water, a dollar value is estimated to the water savings.

A. VOLUME REDUCTION

The amount of water saved as a result of conservation programs is based on a DWR publication titled "Appendix B, Methodology for Estimating Urban Water Savings" (California DWR, 1973, Appendix B). This is the same publication used for estimates in the B 68-60 plan. A list of the criteria used to evaluate water savings is presented in Table IV-1. Estimated reduction in water consumption is given in Table IV-2. The savings are based on the demand and population estimates contained in Table II-5 and the categories of use shown in Tables II-2 and II-3.

It should be noted that the estimated savings are predicted for the conservation measures discussed in Section III, and additional savings as a result of existing laws or trends are not considered. It is assumed that the existing laws and trends will result in reduced water consumption regardless of whether a conservation plan is implemented or not; so credit for these savings cannot be attributed to the plan. A list of existing laws and trends as noted in the B 68-60 plan is given in Table IV-3.

The total volume predicted to be saved in Vacaville as a result of the conservation plan increases from approximately 0.7 mgd in 1990 to over 2.3 mgd in 2010. Per capita savings in this same period are predicted to be 11.9 gpcd by 1990 and as high as 22.3 gpcd by 2010. The amount of water saved each year is considerably less than the quantities predicted in the B 68-80 plan. The discrepancy is principally attributed to the higher population projections and water demands in the B 68-60 plan.

TABLE IV-1
CRITERIA FOR ESTIMATING WATER REDUCTION

Category of Savings	Criteria
Residential Interior - Single Family - Multiple Family	5% reduction in interior use after 1990 for single family dwellings (metered); 2.5% in multi-family dwellings (unmetered). Excludes reduction from retrofit showers and toilets.
Commercial	5% reduction by 1990 and 7.5% by 2000 for all commercial use.
Governmental	10% reduction by 1990 and 15% reduction by 2000 for all governmental use.
Industrial	2.5% reduction by 1990 and 5% reduction by 2000 of all industrial use.
Existing Landscapes	20% reduction in exterior use for every residence in compliance. 25% compliance by 1990 and 50% compliance by 2000. Applicable to residencies constructed by 1985.
New Single Family Landscapes	Same as existing homes plus: 40% additional reduction for low water use landscapes for each residence in compliance. 10% compliance in 1990, 25% in 2000, and 50% in 2010.
New Multiple Family Landscapes	Same as existing homes plus: 30% reduction by 1990 and 40% by 2000 for all residences. 100% compliance as a result of low landscape water use ordinance.
Retrofit Existing Toilets	14% reduction in water use with water bags and 25% retention of these devices. Only considered in residences existing in 1980 before the low-flush toilet law.
Retrofit Existing Showers	50% reduction in water use with low-flow shower heads and 13% retention of these devices. Only considered in residencies existing in 1980 before the low-flow shower law.
Water Loss Reduction Program	50% unauthorized water loss from leaks, 32% of leaks cost effective to repair. 50% of these repaired by 1990 and 50% by 2000.

SOURCE: California DWR, Appendix B, 1982.

TABLE IV-2

ESTIMATED WATER REDUCTION FOR VACAVILLE (gpd)

Category of Savings	Year 1990	1995	2000	2005	2010
Residential Interior					
- Single Family	184,600	218,300	252,000	285,600	319,300
- Multiple Family	27,300	32,300	37,300	42,300	47,200
Commercial	36,300	42,900	74,200	84,200	94,100
Governmental	71,200	84,200	145,700	165,200	184,700
Industrial	29,900	35,300	81,600	92,400	103,300
Existing Landscape	130,500	130,500	261,000	261,000	261,000
New Single Family Landscapes	36,600	92,100	287,000	389,900	739,100
New Multiple Family Landscapes	29,100	75,300	146,600	199,100	251,700
Retrofit Existing Toilets	49,700	49,700	49,700	49,700	49,700
Retrofit Existing Showers	44,600	44,600	44,600	44,600	44,600
Water Loss Reduction Program	82,130	133,500	224,200	254,100	284,100
Total Reduction (gpd)	721,930	938,700	1,603,900	1,868,100	2,378,800
Total Reduction (AF/year)	809	1,052	1,797	2,093	2,665
Total Reduction (gpcd)	11.9	12.9	19.0	19.5	22.3

TABLE IV-3

EXISTING LAWS AND TRENDS FOR WATER CONSERVATION

Item	Description
Toilet Law	Law requires 3.5 gal toilets instead of 5 gal in all construction after 01/01/78.
Shower Law	Law requires maximum flow of 2.75 gpm instead of 6.00 gpm in all construction after 01/01/80.
Faucet Law	Law requires maximum flow of 2.75 gpm in kitchen faucets instead of 3.5 gpm in all construction after 01/01/80
Shower and Faucet Replacement	As old fixtures wear out, they will be replaced with low-use devices.
Clotheswashers	New clotheswashers use about 15% less water than old ones.
Dishwashers	New dishwashers use about 25% less water than old ones.
Industrial Waste Disposal Laws	Increasing regulations and costs of wastewater disposal encourage industry to use less water.

SOURCE: California DWR, Appendix B, 1982.

Ammendment 7 of the contract for North Bay Aqueduct (NBA) water between the Department of Water Resources and the Solano County Flood Control and Water Conservation District calls for specific goals for the volume of water saved in the conservation plan under Assembly Bill No. 797. The goals specified are 750 AF/year by 1990; 1,400 AF/year by 2000; and 2,070 AF/year by 2010. Clearly, the savings predicted in Table IV-2 meet these goals.

B. VALUE OF SAVED WATER

The value of saved water will be realized by two principal merchanisms. First, any reduction in consumption will correspond with a reduction in annual operation and maintenance costs. Second, reduction in consumption will effectively delay the time when new facilities need to be constructed. Because it is very difficult to determine exactly what facilities will be needed and how long their construction can be delayed, a dollar value is not estimated for this type of savings. Instead, the value of saved water is wholly attributed to reduction in O&M costs, and it is recognized that this estimate will be a conservative estimate of actual conditions.

Any reduction in water demand as a result of conservation will most likely be reflected in reduced ground water consumption for two reasons. First, NBA and SID allotments are contracted supplies, and any water not used is considered a loss of resources to the city. Second, the deep aquifer used for water supply has been heavily used in the past five years and will probably be heavily used in the next three years. Therefore, it is important that ground water use be reduced in the long term to prevent overdraft of the aquifer.

The dollar value of any savings in ground water is attributed entirely to pumping energy, chlorine and flouride costs. Because all well pumps will likely be used at some minimum production level each year (regardless of the amount of water saved by conservation), labor and materials for routine maintenance of the wells is considered to be nearly a fixed cost and is neglected in the following estimates. Typically, the wells must supply about 575 feet of head. If the pumps and motors operate at 75 percent and 90 percent efficiency, respectively, and the cost of energy is $0.09/KWH, then the pumping cost is $242/MG (million gallons). Flouride and chlorine costs are approximately $2/MG and $1/MG, respectively; so the total variable cost to supply well water is about $245/MG.

The total annual dollar savings as a result of the proposed conservation plan is shown in Table IV-4, and the total value in terms of 1985 dollars of all savings over a 25-year period is presented in Table IV-5. If the rate of escalation in construction costs is equal to the time value of money, then 1985 dollars will equal the present value. For this report, 1985 dollars is considered a reasonable estimate of present value and is used for all costs.

TABLE IV-4
ESTIMATED ANNUAL DOLLAR SAVINGS TO VACAVILLE
FROM THE CONSERVATION PLAN [1]

Category of Savings	Year 1990	1995	2000	2005	2010
Residential Interior					
- Single Family	$ 16,500	$ 19,500	$ 22,500	$ 25,600	$ 28,500
- Multiple Family	2,400	2,900	3,300	3,800	4,200
Commercial	3,200	3,800	6,600	7,500	8,400
Governmental	6,400	7,500	13,000	14,800	16,500
Industrial	2,700	3,200	7,300	8,300	9,300
Existing Landscape	11,600	11,600	23,300	23,300	23,300
New Single Family Landscapes	3,300	8,300	25,600	34,900	66,100
New Multiple Family Landscapes	2,600	6,700	13,100	17,800	22,500
Retrofit Existing Toilets	4,500	4,500	4,500	4,500	4,500
Retrofit Existing Showers	4,000	4,000	4,000	4,000	4,000
Water Loss Reduction Program	7,300	11,900	20,000	22,700	25,400
Total Estimated Savings	$ 64,500	$ 83,900	$143,200	$167,200	$212,700

[1] 1985 dollars.

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TABLE IV-5

ESTIMATED TOTAL SAVINGS TO VACAVILLE
FROM THE CONSERVATION PLAN FROM 1986 TO 2010

Category of Savings	1985 Dollars
Residential Interior	
- Single Family	$ 504,200
- Multiple Family	74,600
Commercial	130,800
Governmental	256,800
Industrial	134,000
Existing Landscapes	420,100
New Single Family Landscapes	546,200
New Multiple Family Landscapes	246,600
Retrofit Existing Toilets	111,100
Retrofit Existing Showers	99,700
Water Loss Reduction Program	378,100
Total	$2,920,200

V. COSTS OF CONSERVATION MEASURES

The cost to implement the conservation measures in the plan is based on Appendix A of the State Water Project Management Plan, Summary Report (California DWR, Appendix A, 1973). This reference is the same used in B 68-80 plan, but the costs have herein been scaled up to reflect their 1985 value. Costs include both materials and labor. Labor costs are incurred by both the support agency and the lead agency. All support agency staff time is paid by Vacaville. However, only 25 percent of the lead agency expenses are assigned to Vacaville. Twenty-five percent was selected based on projected 1990 populations in the B 68-80 plan. Vacaville has about 25 percent of the population in the area served by the lead agency (Vallejo, Benicia, Fairfield, Suisuin City, Vacaville).

Assumptions used to estimate costs and the cost calculations are presented in tabular form in Appendix C. The 1985 value of all costs from 1986 to 2010 are presented in Table V-1. In Table V-2 total costs for the first five years of the plan are presented.

In cases where Vacaville has already begun implementation (excluding metering) of a conservation measure, costs are estimated as if the program had not been started. Although this procedure will tend to overestimate additional expenditures to Vacaville, it allows the costs to be comparable with the benefits to evaluate the economics of the conservation plan. However, there are a number of cost items that are neglected in the previous estimates or that may change significantly when it comes time to implement the measures as listed below.

1. Costs for formation of a corrosion control plan are estimated, but costs for implementation of the program are neglected. These costs may vary significantly and will depend on the extent of corrosion control measure needed.

2. Costs for establishing a meter calibration and replacement schedule are estimated, but the costs of calibrating and replacing new meters are neglected. Costs to Vacaville may vary significantly with this program depending on the need of the program.

3. Although costs are estimated for a leak detection program, replacement costs for old facilities are not included.

4. Costs for the valve exercising program are estimated, but repacement costs of old valves are not included.

TABLE V-1

1985 VALUE OF COSTS TO IMPLEMENT
WATER CONSERVATION MEASURES FROM 1986 TO 2010 [1]

Measure	1985 Dollars
A. WATER CONSERVATION AGENCIES	$ 23,700
B. EDUCATION AND PUBLIC INFORMATION	
1. Conservation Literature	300,200
2. Previous Year's Use on Water Bills	4,800
3. Promotional Measures	103,700
4. Work with Large Water Users (Including Meter Loan Program)	130,100
5. In-School Education	72,300
6. Information on Federal and State Laws and Programs	11,500
C. WATER MANAGEMENT PROGRAMS	
1. Water Loss Reduction Techniques	
a. System-Wide Water Audit	85,600
b. Leak Detection Program	149,000
c. Meter Calibration and Replacement Program	7,400
d. Corrosion Control Program	5,000
e. Valve Exercising Program	336,000
2. Meter Existing Customers	-0-
3. Device Distribution	400,700
4. Meter Loan Program	[See B.4]
5. Pricing	10,000
D. REGULATIONS	
1. Environmental Impact Reports/Statements	-0-
2. Water Waste Reduction Programs	51,700
3. Water Conservation Ordinances	169,500
E. WATER EMERGENCY PLAN	4,300
TOTAL PRESENT WORTH	$1,865,800

[1] Costs already incurred by Vacaville on measures already started are not accounted for.

TABLE V-2

ESTIMATED CONSERVATION PROGRAM COSTS
TO VACAVILLE FOR THE FIRST FIVE YEARS

Year	Total Annual Cost [1]
1986	$ 66,800
1987	92,400
1988	63,300
1989	61,900
1990	59,200

[1] 1985 dollars.

VI. BENEFIT COST COMPARISON

The two main reasons for implementing a water conservation program are to preserve water supply in conjunction with the "conservation ethic" for environmental reasons, and to reduce operational costs to the water utility. With regard to the "conservation ethic," it self-evident that any measure aimed at reducing unnecessary use is good. However, from an economic point of view, not all measures are beneficial. Based on the costs and benefits of the proposed conservation plan estimated in Sections IV and V, the economic feasibility is discussed in the following papragraphs.

Ideally, it would be desirable to take each component of the conservation plan and compare the costs to the benefits and implement only those items which reduce long-term expenses to the city. However, this comparison is not possible from the information provided because there is not a one-to-one correlation between categories of savings and the specific conservation measures. For example, reduction in residential interior water use will be the result of a variety of conservation measures, including conservation literature, promotional measures, in-school eduction, and price structures. Similarly, a promotional campaign with nurseries may influence landscape water use in the governmental, industrial, commercial or residential sectors. Despite these comparison problems, some conclusions may be drawn from a benefit cost analysis as listed below.

1. The total 1985 value of all benefits ($2,920,200 - Table IV-5) over a 25-year period exceeds the costs to implement the program ($1,865,800 - Table V-1), so overall it appears that Vacaville would benefit economically from the conservation program with the assumptions used in this report. However, as noted previously some costs are neglected in the estimates. Also, the estimates can only be considered as rough approximations of actual costs, and they may deviate significantly in actual practice.

2. In the first few years of the conservation plan, costs will exceed benefits for two reasons. First, many of the measures have high initial expenditures; and second, many of the measures take a number of years to fully implement or are only effective on new construction.

VII. IMPLEMENTATION SCHEDULE

An implementation schedule for the conservation plan is shown in Table VII-1. The schedule should be viewed as tentative because the final decisions regarding when and if the measures are implemented will be at the discression of the support agency. In Table VII-1 the letter 'A' symbolizes an "active" effort on implementation. Where an initial investigation is required before implementation of a measure the letter 'S' is used for "study" or "initial organization." In the case of demonstration low water using landscapes, the sysmbol 'P' is used for "passive" effort because, once established, the demonstration landscapes do not need little further work.

TABLE VII-1
IMPLEMENTATION SCHEDULE [1]

Conservation Measure	1986 - 1990					1991 - 1995				
	86	87	88	89	90	91	92	93	94	95
A. WATER CONSERVATION AGENCIES	S	A	A	A	A	A	A	A	A	A
B. EDUCATION AND PUBLIC INFORMATION										
1. Conservation Literature	S	A	A	A	A	A	A	A	A	A
2. Previous Year's Use on Water Bills	S	A	A	A	A	A	A	A	A	A
3. Promotional Measures										
a. Publicity campaign	A	A	A	A	A	A	A	A	A	A
b. Public speaking presentations	A	A	A	A	A	A	A	A	A	A
c. Demonstration low water using landscapes	S	P	P	P	P	P	P	P	P	P
d. Promotional campaign with nurseries	S	A	A	A	A	A	A	A	A	A
4. Work with Large Water Users	A	A	A	A	A	A	A	A	A	A
5. In-School Education	S	A	A	A	A	A	A	A	A	A
6. Information on Federal/State Laws and Programs	A	A	A	A	A	A	A	A	A	A
C. WATER MANAGEMENT PROGRAMS										
1. Water Loss Reduction Techniques										
a. Water audit	A	A	A	A	A	A	A	A	A	A
b. Leak detection program	-	A	A	A	A	A	A	A	A	A
c. Meter calibration and replacement program	-	S	A	A	A	A	A	A	A	A
d. Corrosion control program	-	S	A	A	A	A	A	A	A	A
e. Valve exercising program	A	A	A	A	A	A	A	A	A	A
2. Metering Existing Customers	A	A	A	A	A	A	A	A	A	A
3. Device Distribution	A	A	A	A	A	A	A	A	A	A
4. Meter Loan Program	-	A	A	A	A	A	A	A	A	A
5. Pricing	S	A	A	A	A	A	A	A	A	A
D. REGULATIONS										
1. Environmental Impact Reports	A	A	A	A	A	A	A	A	A	A
2. Water Waste Reduction Program	-	S	A	A	A	A	A	A	A	A
3. Water Conservation Ordinances	-	S	A	A	A	A	A	A	A	A
E. WATER EMERGENCY PLAN	S	-	-	-	-	-	-	-	-	-

[1] Key: A = Active Effort; P = Passive Effort; S = Study or Initial Organization

VIII. ADDITIONAL CONSIDERATIONS

Wastewater reclamation, water exchange or transfer and pressure management are important topics to consider in the management plan. Each of these topics is briefly discussed in the following pages. Afterwords, impacts of the conservation plan other than the costs and benefits to the water utility are presented.

A. WASTEWATER RECLAMATION

Currently, wastewater from the Easterly Treatment Plant enters Alamo Creek which leads to Cache Slough. Although the wastewater is used for irrigition by the Solano and Marine Prairie Irrigation Districts, none is used in a manner which would offset uses of available domestic water; and the wastewater does not significantly recharge the deep aquifer used by Vacaville.

If the wastewater were used for watering city parks or meeting industrial and other demands, reductions in the demand for domestic water supply could be realized. Such uses should be encouraged in the future. However, the high cost of infrastructure facilities and the ability to find suitable uses for the wastewater will limit the potential for wastewater reclamation.

B. WATER EXCHANGE OR TRANSFER

A water exchange or transfer of treated wastewater for fresh water could potentially benefit Vacaville by increasing available domestic supply and putting the reclaimed wastewater to good use by irrigating crops and/or replenishing ground water. One arrangement of this type currently exists between SID and Fairfield in which treated wastewater is used for irrigation in exchange for fresh water.

However, the potential for making a similar exchange in Vacaville is limited by two factors. First, additional infrastructure facilities would be needed to transport the exchanged water, and such facilities could be very expensive. Second, for the exchange to be beneficial to the recipient of the wastewater, productive land suitable for irrigation with treated wastewater must be available.

C. WATER PRESSURE MANAGEMENT

Water pressure is an important factor which influences the amount of water lost through leaks in the system and the rate of consumption through fixtures such as faucets or shower heads (the greater the pressure, the faster the water passes through leaks or fixtures). Currently, pressures throughout the City of Vacaville range from over 100 psi, which is considered very high, to near 30 psi, which is considerd low. Higher pressures are generally found in the vicinity of Elmira Road near the well field and treatment plant. Lower pressures are typically found in the northwest end of town.

Much of the wide range in pressures is attributed to elevation differences between the two areas. However, headloss in the distribution system is also a contributing factor. For example, consider a single pipe with no vertical slope which has a well on one end and a water user on the other. If the pipe is large, headlosses will be small; so the pressure at the well and user ends of the pipe will be nearly equal. But if the pipe is small and headloss is high, pressures at the well must be greater in order to have the same pressure at the user end.

The city staff is well aware of the pressure problems, and long-term solutions to rectifying the situation are under consideration. Primarily, larger distribution pipes, which will reduce headloss and pressures, and new reservoirs will be planned in accordance with the forthcoming Master Plan.

D. IMPACTS OF THE MANAGEMENT PLAN

In addition to the economic impacts to the water utility, economic impacts to the consumers, environmental, social, health and technological impacts are potential concerns of the water conservation plan. In Table VIII-1, the relative impacts for each of these areas are identified for each component of the conservation plan. Impacts are identified with a "+" for positive impact, "o" for no impact, and "-" for negative impact. In some cases the conservation measure may have both positive and negative impacts depending on how implementation occurs or depending on the point of view taken.

Generally, economic impacts to the consumers as a result of the conservation plan are positive. Some measures may reduce water consumption and thus lower total water bills. Furthermore, the conservation plan may have economic benefits to the city and consumers by effectively delaying the time of construction for new facilities. However, in some instances, additional costs to the consumer may be realized. For example, water conservation ordinances regarding lawn watering may cause residents to have to modify sprinkler systems or landscapes.

Environmental impacts of the conservation measures are generally mixed although not extreme. For example, changes in landscapes as a result of water conservation and landscape ordinances may be attractive to some people and unattractive to others. Wastewater reclamation and water exchange practices are also potentially controversial environmental issues.

As with environmental impacts, social and health impacts from wastewater reclamation and water exchange may be considered positive or negative. However, there are also some social and health impacts that are clearly positive. Education and public information, device distribution and water waste reduction programs should add to the social interactions of local government and the community. The water audit, leak detection program, corrosion control program and other measures which improve upkeep of the water system will reduce health risks in the community by minimizing the potential for contamination of the water supply.

Technological impacts are those which concern the use or implementation of new and/or innovative mechanical devices for the water system or for the consumer's side of the service connection. No adverse technological impacts are expected from the conservation plan, and in many cases the conservation plan will stimulate the use of new technology in the community.

TABLE VIII-1
POTENTIAL IMPACTS OF CONSERVATION MEASURES

Conservation Measure	Impact				
	Economic[1]	Environmental	Social	Health	Technological
Education and Public Information	o	o	+	o	o
System-Wide Water Audit	o	o	o	+	+
Leak Detection Program	o	o	o	+	+
Meter Calibration and Replacement Program	+/-	o	o	o	o
Corrosion Control Program	o	o	o	+	+
Valve Exercising Program	o	o	o	o	o
Device Distribution	+	o	+	o	o
Meter Loan Program	+/-	o	+	o	+
Pricing	+/-	o	o	o	o
Environmental Impact Reports/ Statements	+/-	+	+	+	+
Water Waste Reduction Program	+/-	+	+	o	+
Water Conservation Ordinances	+/-	+/-	+/-	o	+
Water Emergency Plan	o	o	+	o	o

(Continued)

TABLE VIII-1
POTENTIAL IMPACTS OF CONSERVATION MEASURES (CONTINUED)

Conservation Measure	Impact				
	Economic[1]	Environmental	Social	Health	Technological
Wastewater Reclamation	o	+/-	+/-	-	+
Water Exchange/Transfer	o	+/-	+/-	+/-	+
Pressure Management	+	+	o	o	+

Key: + = Positive impact
o = Negligible impact
- = Negative impact

1 Economic considerations are for consumers and not the water utility.

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APPENDIX C

NOTES ON CAPITAL COST ESTIMATES

Appendix C provides supplemental information on the assumptions and basis for cost estimates for the recommended capital improvements. For improvements which are discussed in the body of the report, reference is made to the report section where the improvement is discussed. The information in Appendix C is intended to supplement and not reiterate the report's discussion.

Costs quoted herein are comprehensive budget level costs and include construction cost plus the other expenses identified in Section 9 of the master plan. All costs are 1990 dollars.

A. WATER PRODUCTION IMPROVEMENTS

1. **Water Plant Clearwell Reroof**

   The cost is based on actual bid price.

2. **NBR Plant Construction, Section 5,E**

   The current comprehensive cost for the 40 mgd NBR plant is about $40,000,000. Vacaville's share is 36% of the total, which is about $14,400,00.

3. **Well Field Study**

   The cost will be determined upon selection of a proposal to perform this work.

4. **NBR Plant CIP**

   The cost estimate was provided by the City.

5. **Well Rehabilitation**

   The cost will be determined in conjunction with the well field study to be conducted in 1990.

6. **Disinfection and Operations Study**

   The cost is based on the consultant's fee estimate plus an allowance for water quality testing done in conjunction with the study.

7. **Replacement Well, Section 3, C**

   The cost estimate was provided by the City.

8. DE Plant Improvements, Section 5, B

Booster Pump Station Upgrade

The project includes addition of 4 new vertical pumps with electrical equipment, construction of a block building to house the pump controls, demolition of existing equipment, installation of a surge tank, and relocation and replacement of the flow meter.

Water Plant Building Expansion

The project includes 2,500 ft^2 of building floor space, HVAC, plumbing, electrical improvements, sitework and a minor amount of locker facilities, laboratory facilities, and furniture.

Chemical Addition Facilities

The estimated cost includes the following components: 1) a chemical addition building to house chlorine, ammonia and fluoride addition facilities, 2) an addition to the existing chlorine building to be used to store chemicals for the wells, 3) chlorination equipment, 4) fluoridation equipment, and 5) ammonia feed equipment. It is assumed that existing chlorination and fluoridation equipment will be reused to the extent possible.

Waste DE Drying Beds

The estimated cost is based on four side by side drying beds, each 20 ft wide and 50 ft long. The beds would be set below grade with ramped access on one end. The cost includes structural and mechanical improvements and sitework.

Filter to Waste Piping

The estimated cost is based on installing automated valves which are tied into the turbidity meters at the DE plant plus the associated mechanical and structural work.

DE Plant Controls

The cost is based on completely replacing the existing timers and electromechanical relays with programmable logic controllers (PLCs).

Body Feed System

The cost estimate includes new positive displacement pumps.

Air Supply System

The cost includes a new 2,000 gal. air receiver with appurtenances and miscellaneous piping plus an enclosure around the existing compressors.

Main Switchboard Improvements

The cost estimate includes a new MSB for the filtration building with an automatic transfer switch to allow the generator to run the pump station while keeping the filtration building on normal supplies.

Clearwell Baffeling

The cost is a budgeting number and is not based on the DE Plant Preliminary Design Report. The City's 5 year CIP budget originally had this cost budgeted for chlorine monitoring equipment, but this equipment is now covered in the chemical addition facilities cost estimate.

9. **Well Field Chloramination, Section 3, C**

Costs for well field chloramination could vary substantially dependent upon the method employed. Addition of ammonia feed facilities at each well would cost about $60,000 per well which is less than the budgeted amount. Piping all the wells together for chloramination at a single location could exceed $3,000,000. Selection of a method will be done in conjunction with the 1990 Disinfection and Operations Study.

10. **Well Field Development, Section 3, C**

Well costs are based on new facilities at new sites, including ammonia addition facilities at each well. Costs also include approximately 3,000 LF of 12 inch pipe to connect the well to the distribution system.

11. **NBR Plant Expansion, Section 5,E**

The estimated construction cost for 6.7 mgd at the NBR plant is $3,200,00. This includes design, construction, administration, etc. Costs could vary significantly and will need refinement during preliminary design.

12. **Replacement Well, Section 3,C**

The cost is based on completely new facilities at a new site including ammonia addition facilities. Costs could be reduced through reuse of an existing site and existing equipment.

13. **NBR Plant Expansion, Section 5,E**

The estimated construction cost for 10 mgd at the NBR plant is $4,500,000. This includes design, construction, administration, etc. Costs could vary significantly and will need refinement during preliminary design.

14. **Replacement Well, Section 3,C**

The cost is based on completely new facilities at a new site including ammonia addition facilities. Costs could be reduced through reuse of an existing site and existing equipment.

B. WATER TRANSMISSION AND DISTRIBUTION IMPROVEMENTS

1. **Downtown Water Main Improvements, Section 7,G**

The cost is based on actual construction prices.

2. **Nut Tree Road Water Main, Section 7,E**

A portion of the pipeline on Nut Tree Road from Elmira Road to I-80 was constructed ahead of schedule in association with road improvements. The cost is based on actual construction prices. See improvement number 22.

3. **CMF-North Water Meter Station**

The cost was provided by the City.

4. **Elmira Road Pipeline, Section 7,E**

This improvement has been designed and bid. The cost is based on the bid price plus an allowance for additional costs.

5. **Browns Valley Road Pipeline, Section 7,E**

This improvement has been designed and bid. The cost is based on the bid price plus an allowance for additional costs.

6. **Browns Valley Reservoir, Section 6,A**

This project is currently under construction. The cost is based on the actual bid price plus an allowance for additional costs.

7. **Water Main Capacity Program**

Specific improvements have not been identified but funds have been set aside for replacement of old and undersized pipelines and to upsize facilities installed privately with new development. Budget costs for this item have been established by the City.

8. **SCADA Facilities, Section 5,C**

Costs for the SCADA system are based on those listed in the Water System SCADA Master Plan (Nolte, 1990).

The first phase improvements include implementing the minimum master station equipment plus equipment at well No. 9 and equipment with the proposed Vine Street water system. All remaining costs have been distributed equally over a five year period. Remaining costs include equipment additions to the master station plus remote facilities for all wells and Zone 1 reservoirs. Costs are not included for upgrading the Hidden Valley and Wykoff systems as these costs are included in the improvements needed to implement proposed Zone 2. Costs are also not included for SCADA equipment for the DE plant as these are included in the DE plant improvement projects.

The costs in the SCADA system master plan do not include instrumentation and mechanical equipment improvements which will be needed at existing facilities to fully utilize the SCADA system. Therefore, without itemizing exactly what equipment may be needed, we have allocated some costs (included in the totals) for this work in each of the years listed in Table 9-4. Specifically, $30,000 has been included in 1990 and $15,000 has been included in each of the subsequent years.

9. **Reservoir Rehabilitation, Section 6,B**

The project includes modification of the inlet outlet piping on both Butcher reservoirs and on the Buck reservoir. The cost of this work is estimated to be $280,000 assuming only minor structural modification of the valve pits, and reuse of existing valves. Additional funds have been budgeted for corrosion and miscellaneous site improvements.

10. **Tranquility Pump Station Improvements, Section 8,F**

The cost was provided by the City.

11. **ACAD Implementation**

The cost is yet to be determined.

12. **Wykoff Zone 2 Improvements, Section 8,E**

The estimated costs are based on developing the Zone 2 water system and improving the Wykoff water system. Costs to expand the system for the West Valleys are not included. The cost estimates are very preliminary at this time because the locations of the reservoir and pumping facilities have not been specified.

New Pumping Station (first increment)

It is assumed that the pumping station is constructed in the general vicinity of Buck Avenue and Alamo Drive. Initially, the building is assumed to be constructed with ample space to handle both Zone 2 and Wykoff pumps, but only mechanical facilities for the Wykoff system are installed. The cost estimate includes a standby generator for the Wykoff pumps and a 500 ft, 24 in. suction pipe for the pump station.

Wykoff System Pipelines

The cost estimate covers 4,500 ft of 8 in. pipe. Approximately 2000 ft would be placed on Wykoff Drive between Tranquility Land and Buck Avenue. The remaining pipe would be used to connect the pumping station with the Wykoff distribution system.

New Zone 2 Reservoir

The cost estimate covers a concrete reservoir and appurtenances plus sitework and a moderate length access road. The location of the reservoir has not yet been specified, and the actual site characteristics can impact the cost substantially.

Pumping Station, Second Increment

The cost covers addition of pumps, and other mechanical and electrical improvements to serve Zone 2. It is assumed that structural and site improvements would be covered in the first phase of construction of the pump station in 1990.

Alamo Drive and Foothill Drive Pipeline

The cost estimate covers 6,000 ft of 24 in. pipe connecting the pumping station and reservoir. It is assumed that the reservoir is located near Foothill Drive, east of Pleasants Valley Road, so pipeline length and cost could increase substantially if the reservoir is located elsewhere.

Pipeline to Form Zone 2

The cost estimate includes 960 ft of 12 in. pipe from North Alamo Drive to Fruitvale Avenue across Alamo Creek, 1,700 ft of 8 in. pipe on Fruitvale Avenue between North Orchard Avenue and Stinson Avenue, and $40,000 for miscellaneous improvements such as cutting and capping pipelines.

Pleasants Valley/Vaca Valley Pipeline

The cost estimate includes 13,000 ft of 12 in. pipeline on Pleasants Valley Road and Vaca Valley Road completing a loop into the North Orchard Avenue Area.

13. **Noonan Reservoir**

The cost is yet to be determined.

14. **New Zone 1 Reservoir, Section 6, B**

The cost is based on a 5 MG concrete water tank with somewhat more difficult construction and a longer pipeline connecting the tank to the system than with the Browns Valley Reservoir.

15. **Wykoff/Zone 2 Improvements, Phase 2, Section 8,E**

See Item B-12.

16. **North Browns Valley Road Pipeline, Section 7,E**

The project includes 8,140 ft of 18 in. pipeline along north Browns Valley Road and Vaca Valley Parkway. Average construction complexity is anticipated.

17. **Peabody Road Pipeline, Section 7, E**

The project cost is based on 3,500 LF of 36" pipe on Peabody Road from California Drive south to the end of the existing 24" pipe. Average construction complexity is anticipated.

18. **Wykoff/Zone 2 Improvements, Phase 3, Section 8,E**

See Item B-12.

19. **North Leisure Town Road Pipeline, Section 7,E**

The project includes 6,420 ft of 18 in. pipeline along north Leisure Town Road. Average construction complexity is anticipated. An allowance for costs to cross I-80 has been included.

20. **Monte Vista Avenue Pipeline, Section 7,E**

The project includes 2,030 ft of 12 in. pipeline along Monte Vista Avenue. Moderate to difficult construction complexity is anticipated.

21. **New Zone 1 Reservoir, Section 6, B**

The cost is based on a 5 MG concrete water tank with somewhat more difficult construction and a longer pipeline connecting the tank to the system than with the Browns Valley Reservoir.

22. **Nut Tree / I-505 Pipeline, Section 7,E**

The project includes 8,410 ft (less 2,700 ft already installed) of 18 in. pipeline from Elmira Road to I-505 and 9,840 ft of 18 in. pipeline continuing to Vaca Valley Parkway. Moderate to difficult construction complexity is anticipated. An allowance for costs to cross I-80 has been included.

23. **NBR Pipeline, Section 7,E**

The project cost is based on 9,500 LF of 30" pipe on Peabody Road from the NBR plant to the City. Average construction complexity is anticipated.

24. **New Zone 1 Reservoir, Section 6,B**

The cost is based on an above ground 5 MG steel reservoir with somewhat more difficult construction conditions and a longer pipeline connecting the reservoir to the distribution system than included with the Browns Valley Reservoir. The cost will not cover construction of a concrete tank.

C. **LOCAL PIPELINE IMPROVEMENTS**

1. **Gonsalves-Lockie Pipelines, Section 7,F**

Costs for the 12 in. pipelines in this project are assumed to be paid by the developer.

2. **Akerely Drive Pipeline, Section 7,F**

The project includes 4,500 ft of 12 in. pipeline along Akerely Drive. Relatively easy construction complexity is anticipated under the assumption that the pipeline is placed in conjunction with road improvements.

3. **Leisure Town, Midway, Eubanks Pipeline, Section 7,F**

The project includes 15,500 ft of 12 in. pipeline on Leisure Town Road and Midway Road. The portion on Eubanks Drive is assumed to be constructed by developers in the area. Relatively easy construction complexity is anticipated under the assumption that the pipeline is placed in conjunction with road improvements. An allowance is included for crossing I-505.

4. **Midway Road Pipeline, Section 7,F**

The project includes 5,300 ft of 12 in. pipeline along Midway Road. Relatively easy construction complexity is anticipated under the assumption that the pipeline is placed in conjunction with road improvements.

5. **Allison Drive Pipeline, Section 7,F**

The project includes 12,500 ft of 12 in. pipeline along Allison Drive. Relatively easy construction complexity is anticipated under the assumption that the pipeline is placed in conjunction with road improvements.