WESTCAPS

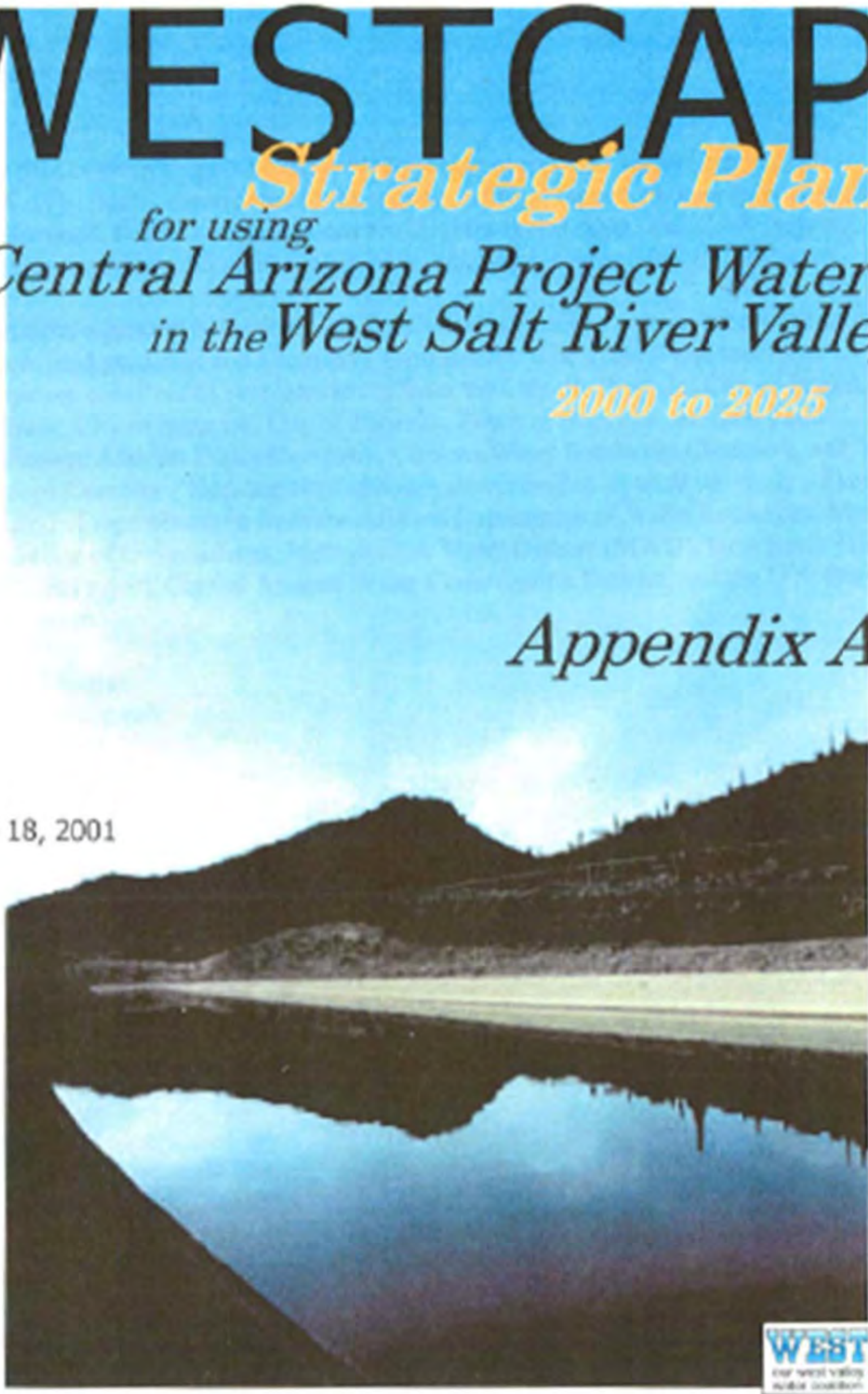
Strategic Plan

for using
Central Arizona Project Water
in the West Salt River Valley

2000 to 2025

Appendix A

May 18, 2001



WESTCAPS
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ABBREVIATIONS AND ACRONYMS

ACC	Arizona Corporation Commission
ADEQ	Arizona Department of Environmental Quality
ADWR	Arizona Department of Water Resources
AMA	Active management Area
ac-ft	acre-foot
ac-ft/day	acre-foot per day
ac-ft/yr	acre-foot per year, acre-feet per year
AWBA	Arizona Water Banking Authority
AWS	assured water supply
BOR	U.S. Bureau of Reclamation
CAGRD	Central Arizona Groundwater Replenishment District
CAP	Central Arizona Project
CAWCD	Central Arizona Water Conservation District
CC&N	Certificate of Convenience and Need
cfs	cubic feet per second
CRIT	Colorado River Indian Tribes
D/DBPR	Disinfectant/Disinfection By-Products Rule
DOI	Department of the Interior
EPA	Environmental Protection Agency
ESRV	East Salt River Valley
ESWTR	Enhanced Surface Water Treatment Rule
ft^3/day	cubic feet per day
GRIC	Gila River Indian Community
LAU	lower alluvial unit
MAU	middle alluvial unit
M&I	municipal and industrial
MCL	maximum contaminant level
MF	microfiltration
mg/L	milligrams per liter
mg	million gallons
MGD	million gallons per day
MWD	Maricopa County Municipal Water Conservation District No. 1
NB-WTP	North Beardsley Water Treatment Plant

NCI	Navigant Consulting, Incorporated
O&M	operation and maintenance
OM&R	operation, maintenance and replacement
ppb	parts per billion
ppm	parts per million
psi	pounds per square inch
RID	Roosevelt Irrigation District
RO	reverse osmosis
RUS	Rural Utilities Service
SB-WTP	South Beardsley Water Treatment Plant
SROG	Arizona Municipal Water Users Association Sub-Regional Operating Group
SRP	Salt River Project
SRV	Salt River Valley
TDS	total dissolved solids
THM	trihalomethanes
TOC	total organic carbon
UAU	upper alluvial unit
UF	ultrafiltration
USBR	U.S. Bureau of Reclamation
USDA	U.S. Department of Agriculture
USDOI	U.S. Department of the Interior
WESTCAPS	coalition of West Valley Central Arizona Project Subcontractors
WMC	West Maricopa Combine
WTP	water treatment plant
WSRV	West Salt River Valley
WPA	water planning area

CHAPTER I

Background

Also referred to as the West Salt River Valley (WSRV), the WESTCAPS member area is located west of Phoenix, and includes the City of Phoenix.

WESTCAPS members are comprised of Arizona State Land Department, Arizona Water Company, Citizens Water Resources, City of Glendale, City of Goodyear, City of Peoria, City of Phoenix, City of Surprise, Town of Buckeye and West Maricopa Combine.

WESTCAPS advisors include the Arizona Corporation Commission, Arizona Department of Water Resources, Arizona Municipal Water User's Association, Bureau of Reclamation, Central Arizona Project, Maricopa County Flood Control District, Maricopa Water District, Agua Fria-New River NRCD, Salt River Project, United State Geologic Survey, WESTMARC, Arizona Department of Environmental Quality, Maricopa Association of Government and Arizona Water Banking Authority.

The WESTCAPS members goal is to determine a feasible solution to meet the WSRV future water supply and water delivery needs. WESTCAPS' mission is to develop workable alternatives for its members, and to provide their customers with a cost-effective, sustainable, reliable, and high-quality water supply through partnerships, and cooperative efforts in regional water resource planning and management emphasizing Central Arizona Project utilization.

When the future demand for water supplies was projected, it was assumed sufficient water supply allocation(s) could be obtained to meet all of the demand projections (Unlimited Supply). [See figure A-1, "A West Salt River Valley Groundwater Totals from Scenario 23, Base Case (AF/YR).”]

On June 30, 2000, the WESTCAPS’ members approved a strategy designating the construction of five water treatment plants to help meet the WSRV future water supply needs. Three of the facilities are already constructed and one is currently being designed. North Beardsley Regional water treatment plant (WTP) and South Beardsley Regional (WTP) are studied further in this report. This report also presents configurations to refine the two sites, North and South Beardsley WTPs, and associated infrastructure strategy. [See figure A-2, "A WESTCAPS Strategy, As Adopted by the General Committee on 6/30/00, (Assuming adequate surface water supply to meet projected demand)".]

The report also studies the possibility that the existing allocated surface water supply may be all the water that is available in the area (Limited Supply). The report also details how the water supply allocations and facilities to be constructed to deliver water will be studied. Also presented is the year the water supply capacity is exceeded. [See Figure A-3, "A WESTCAPS Strategy, As Adopted by the General Committee on 6/30/00, (Limited by amount of anticipated supplies)".]

Further refining the WESTCAPS Strategy (6/30/00) is the WESTCAPS Strategy (9/15/00) (See Figure A-4). Refining the original strategy repositioned the North and South WTPs to locations that increase the:

- Access to available water supplies
- Regional area served with surface water
- Reliability
- Flexibility
- Available gravity pressure head to the delivery system

The three existing facilities and interconnections remain the same. The WESTCAPS Strategy (9/15/00) will be used as the layout to compare study configurations in this appendix, Appendix A.

Figures A-1 8x11 color figures (arcview).

Figures A-2   8x11 color figures (arcview).

Figures A-3, 8x11 color figures (arcview).

Figures A-4, 8x11 color figures (arcview).

CHAPTER II

Introduction

In order to evaluate if an alternative treatment and direct delivery of Central Arizona Project (CAP) water is feasible, certain assumptions were made as to how much CAP water would be available for use. As of this publication, the exact quantity of CAP water available for use by each of the water providers has not been determined.

The amounts of limited and unlimited water supplies that will be available for use by the new water treatment plants will be water supplies not currently assigned to other regional facilities. For purposes of this study, it can be assumed for the Direct Delivery Alternative that the CAP water available from each of the water providers includes two base supply options.

- 153,344 Acre-feet per year – This amount assumes that an unlimited water supply will be acquired to meet the necessary project demand.

- 65,681 Acre-feet per year – This amount assumes that the water supply available for use is limited by possible allocated water rights.

Two WTP configurations were evaluated for each of the water supply configurations. The first of these involves construction of a single treatment plant located along the CAP system. The second involves construction of two water treatment plants. One would be located along the CAP system, and the other located along the Beardsley Canal, which belongs to the Maricopa Water District (MWD).

Modular capacity growth of the WTP configurations is evaluated for increasing plant production capacity in stages of construction to meet the estimated demand by the years 2005, 2015 and 2025.

A summary illustrating the options, configurations, stages and corresponding capacities is shown in Table A-1, Summary of Configurations and Stages.

Table A-1 - SUMMARY OF CONFIGURATIONS AND STAGES

CONFIGURATION	CAPACITY (acre-feet per year)
Unlimited Supply Capacity	
One Treatment Plant (along CAP Canal System)	
Phase 1 - Build Plant to estimated demand at 2005	53,700
Phase 2 - Expand Plant to estimated demand at 2015	98,677
Phase 3 - Expand Plant to ultimate demand at 2025	153,344
Two Treatment Plants (CAP Canal System & Beardsley Canal)	
Phase 1 - (North) Plant to estimated demand at 2005	0
(South) Plant to estimated demand at 2005	51,329
Phase 2 - (North) Plant to estimated demand at 2015	45,138
South Plant to estimated demand at 2015	53,539
Phase 3 - Expand Both Plants to ultimate demand at 2025	153,344
North Plant to estimated demand at 2025	64,485
South Plant to estimated demand at 2025	88,859
Limited Supply Capacity Configuration	
One Treatment Plant (along CAP Canal System)	65,681
Two Treatment Plants (CAP Canal System & Beardsley Canal)	65,681
North Plant to estimated supply available at 2010	38,910
South Plant to estimated supply available at 2010	26,771

CHAPTER III

Common Elements for the Supply Configurations

The operations and some design elements for the various configurations for both the Beardsley and CAP canals will be similar. Water delivery, through a canal side gravity turnout, must be constructed and integrated into the canal side slope. From the turnout, water will then pass through a metering vault prior to delivery to an optional raw water reservoir, and then is fed to the WTP. The treated water is then stored in a clearwell reservoir from which water is delivered to the main distribution pipeline.

The water providers' point of delivery from the WTP, and capacities, are determined by the water providers demand and location of the area centroid (center of the Water Planning Area (WPA)) for potable water deliveries into each individual system. Reservoirs tanks are designed and located to provide storage and surge protection. The distribution system was designed to take advantage of existing infrastructure and opportunities for operational cooperation among the water providers.

A more detailed description of specific common elements that make-up the various configurations follows.

Power

Power will be brought to the site from one of several existing high voltage lines. When the total amount of power needed for the project is determined, additional electric sources may be required. Presently, the CAP transmission system provides the electric power for the nearby CAP facilities. Sufficient electric capacity to operate the project on these lines may exist, but CAWCD would need to approve the use of the lines. Appropriate wheeling and transmission agreements would need to be negotiated, as well. In the event that CAP transmission power could not be purchased, Arizona Public Service and Maricopa Water District facilities could provide additional supply. However, these facilities would require a new electric substation. The current cost for power is approximately 40 to 90 mills. This study assumes 60 mills for all calculations. The water providers are responsible for securing electric power for the WTP.

Operations and Water Costs

Water costs from the CAWCD were calculated using the most recent municipal and industrial (M&I) rate schedule (CAWCD, July 2000). (Attached at end of Appendix A, CAP memorandum dated July 7, 2000) The CAWCD's price for CAP water is comprised of a capital component and a delivery component that covers maintenance and energy costs. The total cost is commonly referred to as the "postage stamp" rate. For the cities in the Phoenix metropolitan area, the postage stamp rate applies to water delivered to the CAP canal on the canalside. Canalside is defined at the limits of the CAP canal right of way.

The Beardsley Canal water costs are not calculated and further studies will still need to be conducted on the cost of using the canal. Also, the additional costs for transporting CAP water through the Beardsley Canal will need to be addressed. Currently, the cost to wheel water in the Beardsley Canal is unknown. Further study and consideration for the cost of using the Beardsley will need to be discussed with the MWD.

This report does not address the cost of leasing water allocations from another allottee. Leasing Indian water supplies and transporting them through the CAP will need to be discussed with the CAWCD and the CAP allocation lessor.

Operational Considerations

All the schedules for the operation of pumping plants, turnouts, and check structures are set by CAWCD using a computer model called the Aqueduct Control Software. The operating strategy used to analyze the aqueduct and determine the operation of each structure is called the constant volume method. The goal is to keep the water in each pool at a constant water surface level by regulating flow through the gated check structures. That will allow operators to make quick changes in the aqueduct operation. This provides flexibility in the operation because flow changes can be made almost immediately if the aqueduct remains at a constant storage capacity. (Attached at the end of Appendix A, CAP Memorandum, dated May 29, 1998, Operating Procedures.)

The water delivery orders are updated daily and are entered into the computer using the Aqueduct Control Software. The Aqueduct Control Software creates a 1-hour turnout and check structure schedule. The operators review and approve the schedules before the control system implements them.

The CAP water users turnout delivery schedules are used as information entered into the Aqueduct Control Software, along with the state of the aqueduct. This information is placed in the system's database after being transmitted to the master station by remote terminal units (RTU) located at the various structure sites along the aqueduct. The program then determines the necessary operations at each structure so that water orders are met. Major changes that vary from normal operations (off-normal or emergency conditions) are detected by the control system, allowing operators time to respond and make corrections. The Aqueduct Control Software automatically corrects minor changes in the system.

When off-normal or emergency conditions are cleared, the computer scheduling programs are rerun using the current aqueduct conditions. The revised schedules are reviewed, approved, and implemented by operators. Off-normal and emergency conditions are closely coordinated with Salt River Project power schedulers, the water users, and Colorado River operations when necessary.

Maricopa Water District Section

The canal's available water capacity in the Beardsley Canal corresponds directly to the current irrigation water deliveries. System improvements currently underway will change available capacities and improve water delivery for WTP operations. These improvement costs were not included in the cost comparisons at the end of Chapter III or Appendix A. A hydraulic analysis for additional capacity and cost is provided in a separate report by MWD.

Turnout Types

Turnouts are canal structures that control water flow to customers. CAP turnouts consist of gravity structures, pumps, or a combination of both. Table A-2 shows the types of turnouts that are used by various municipal CAP water subcontractors.

Turnout design is influenced by topography at the proposed turnout location and the maximum, normal, and control water surface elevations of the canal;[1] the elevation of the point of water delivery; and economic considerations. Pump turnouts are required if the elevation of the normal water surface at the diversion point is lower than the elevation of the point of delivery. In general, capital, and operation and maintenance (O&M) costs associated with gravity turnouts are less than for pump turnouts.

[1] The maximum water surface elevation is the highest elevation of the water in the canal at which the canal should be operated. The difference between this elevation and the top of the canal lining represents the operating freeboard. Freeboard is the designated capacity in the canal to protect the conveyance system from overtopping due to sedimentation in the canal, flows from storm runoff, increased water depth resulting from a rougher friction coefficient than used for design, and wave action or surges which accompany sudden changes in flow. Normal water surface elevation represents the normal operating level in the CAP Canal. The control water surface elevation represents the lowest elevation that the turnout can operate and still deliver the required capacity.

Table A-2. - Types of turnouts used by CAP water subcontractors		
Municipality	Location	Type of turnout
Glendale	59th Avenue and the CAP Canal	Gravity
Mesa	Between McKellips and Brown Roads and the CAP Canal	Gravity
Chaparral City Water Company	Near Shea Boulevard and the CAP Canal	Pump
Phoenix	Cave Creek and Deer Valley Roads and the CAP Canal	Gravity and pump
Scottsdale	Pima Road and the CAP Canal	Gravity and pump

Selection and design of turnouts takes into account hydraulic losses associated with the type of turnout being considered. The standard maximum hydraulic loss used in the design of most CAP turnouts is 2 feet and is used to determine the delivery water surface downstream of the turnout.

The proposed turnouts for this study are located along Reach 9 of the Hayden-Rhodes CAP Aqueduct and the MWD Beardsley Canal. Topographic maps show that, at these locations, land surface elevations decrease south of the CAP Canal and east of the Beardsley Canal. Therefore, turnouts diverting water to the south or east of the canals will not require pumping to service the WTPs.

Gravity Turnout

Figures A-5 and A-6 show the general plan and section of a typical 145 and 325-cfs canalside gravity structures. The structure includes a trashrack, turnout inlet structure, pipe(s), and acoustic meter structure.

Designed to prevent 1.5 to 2-inch material from entering the diversion, the trashrack is sloped to a slope ratio of 1.5:1 to correspond to the canal lining slope. Under normal operating conditions, the water current flowing in the CAP Canal is adequate to keep trash from going through or being trapped on the trashrack. However, if debris and aquatic weeds that accumulate become a problem, the trashrack will require periodic cleaning. Some trashracks incorporate an automated trashrake system to help remove accumulated material, while others require debris to be removed manually.

The turnout inlet structure consists of concrete bay(s) with an operation deck. The canal invert and the amount of debris bed load determine how deep the turnout structure inlet is located below the water level. The size of the turnout structure deck is designed to accommodate access to the trashracks, mounting gate controls, and if installed, automatic trashrake equipment.

The gate controls are to operate a gate on the back wall of the inlet structure to control the flow to the pipe inlet leading to the metering vault. From the metering vault, the pipeline flow is directed to a users valve structure that will allow water to be delivered to the WTP.

Figure A-5 drawings Turnout (8-1/2 x 11 black and white)

Figure A- 6 drawings (8-1/2 x 11 black and white)

Design Sizing

The turnout size used for the majority of CAP municipal water providers is based on a formula using 11- percent of the yearly allocation (converted to an average daily volume) with an additional 50-percent peaking factor constructed into the turnout. The following is an example of an allocation for 10,000 acre-feet per year.

> The monthly allocation is 1,100 acre-feet per month and is determined by taking 11-percent of 10,000 acre-feet per year. The average daily allocation is then determined to be 36.7 acre-feet per day. This is calculated by dividing the monthly allocation (of 1,100 acre-feet per month) by 30 days per month. The average daily peak capacity is 55 acre-feet per day (28 cfs) and is calculated by increasing the daily acre-foot allocation of 36.7 by 50-percent. This value is used for design purposes and is equivalent to almost twice the normal yearly allocation. Turnout sizing for each option studied are listed below in Table A-3, and assumes the turnout size to be twice the actual yearly allocation. The turnout size, in CFS, is shown under the column labeled "2 X CFS".

Table A-3 TURNOUT SIZING

Description	Acre-Feet/Year	2 x CFS
Unlimited Supply Capacity Option		
Study area, well demand, unlimited supply, year 2025	153,344	423.6
North CAP WTP, study area, Unlimited Supply (2 WTP study)	64,485	178.1
South MWD WTP, study area, Unlimited supply (2 WTP study)	88,859	245.5
Limited Supply Capacity Option		
Current CAP Allocations available, Limited supply	65,681	181.4
North CAP WTP, study area, Current CAP Allocations, Limited Supply (2 WTP study)	38,910	107.5
South MWD WTP, study area, Current CAP Allocations, Limited Supply (2 WTP study)	26,771	74.0

Canal Operation and Maintenance (O&M) Practices

CAP Canal

CAP outages are unlikely for the upstream sections of the proposed turnout location. During previous preventative maintenance outages, inverted siphons and canal sections have been replaced and repaired. Because the canal is designed to continuously handle scheduled water deliveries, annual operational outages are not conducted, nor are they anticipated. Continuous operation is anticipated, but is not provided contractually. Contingency plans for outages are discussed in the next paragraphs.

The O&M practice of ***estimated delivery outage duration*** is selected by determining the number of days a particular CAP feature will be inoperable while undergoing maintenance. This is usually based on maintenance schedules and practices of the operating agency; in this instance it would be the CAWCD. Future evaluations of the CAP O&M practices may reflect more accurate operating conditions and more probable outage durations.

The ***schedule of the delivery outage*** O&M practice being activated depends on the CAWCD's maintenance schedules and practices. The CAWCD indicated it intends to schedule its maintenance aqueduct activities during the fall and winter months.

The ***monthly CAP water demand*** O&M practice is based on a monthly usage variation chart showing residential with light industrial and commercial average yearly water use for the years 1995 to 1998.[2] (see figure A-7, Monthly Usage Variation).

The evaporation losses, averaging about 72 inches a year, are based on the publication *Arizona Climate*, University of Arizona Press, data, verified by the 1988 through 1995 Arizona Meteorological Network.

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[2] This chart was included in a March 12, 1999, letter from MDWID.

Insert Figure A-7

CAP Turnout Scheduling

Final water orders are requested one day in advance by CAP water customers with emergency changes allowed if needed. Using these orders, the Aqueduct Control Software generates updated turnout schedules that are approved or modified by the operator. For example, modifications to the turnout schedules would be needed if a power failure occurs along the aqueduct and the pumping plants cannot deliver the water flows needed to meet the water user's demands. Approved schedules are transmitted to the RTU at the turnout sites, assuring they are met. The RTU sends flowmeter readings to the control operating center where the operator monitors them. Turnout gates may be adjusted remotely or locally if an unscheduled change is needed.

MWD - Beardsley Canal Turnout

Information of the turnout types, method of design sizing, water delivery schedule and operation practices have not been completed for the addition of municipal water services to the Beardsley Canal. Discussed below are the outage criteria that may affect a WTP system.

The O&M practice of ***estimated delivery outage duration*** is selected by determining the number of days a particular canal feature will be inoperable while undergoing maintenance. This is usually based on maintenance schedules and practices of the operating agency. In this instance, it is the MWD. Future evaluations of the O&M practices may reflect more accurate operating conditions and more probable outage durations.

The ***schedule of the delivery outage*** O&M practice being activated depends on the MWD's maintenance schedules and practices. The MWD indicated it intends to schedule its aqueduct maintenance activities during the fall and winter months.

The ***monthly projected MWD domestic water demand*** O&M practice is also based on figure A-7 for a monthly usage variation chart showing residential with light industrial and commercial average yearly water use for the years 1995 to 1998[3] (see figure A-7). Additional data regarding the irrigation deliveries are also enclosed (Beardsley canal report).

Evaporation losses, averaging about 72 inches per year, are based on the publication *Arizona Climate,* University of Arizona Press, data verified by the 1988 through 1995 Arizona Meteorological Network. It is important to note that volume loss from evaporation and infiltration is not included in the water and wheeling costs of the Beardsley.

[3] This chart was included in a March 12, 1999, letter from MDWID, will also be used for the MWD Beardsley canal WTP.

Water Treatment Facilities

Water Treatment Facilities Operational Overview

The accounting of allocation for the CAP water delivery begins at the canal side turnout constructed and integrated into the canal side slope. The combined water flows will pass through a metering vault structure and then a valving structure allowing it to be delivered to a raw water reservoir, or diverted directly to the water treatment plant. If water pretreatment, reserve capacity, or sedimentation is required, the optional raw water reservoir could be constructed and integrated into the system.

The canal and reservoir water surface elevations delivered to the water treatment plants are designated to correspond to the normal water elevations of the canal and reservoir minus the operating bandwidth of a design delivery low water surface elevation. For the canal, water surface delivery elevation will be normal water surface minus 5 feet. For the raw water reservoir, this is minus 10 feet, with an option to incorporate high volume - low head pumps. This will require that the reservoir be operated at or near full capacity for the majority of the year. Delivery of water below the top 10 feet of the reservoir will require pumping to the WTP. After treatment through the WTP treatment trains, water will then be delivered to Clearwell Forebay Reservoir.

The clearwell forebay reservoir, or forebay, is designed to store treated water, provide water for water deliveries that use gravity to move the water, and water to be used by the intake side of a possible future pumping plant. The gravity pipelines and pumping plant will deliver water to the distribution pipeline and finally to the individual turnouts in the WPA service areas. A 1-day reserve capacity water treatment plant, wells, and potable water supply for reservoirs is required as part of the reliability of the distribution system. The distribution pipeline to the water providers' turnout includes reservoirs, tanks, relift pumping plants, piped turnouts, valves, pumps, pressure reducers, and various sizes of pipe. These features, combined with existing wells, boosters, and reservoirs, will provide fire flows and peak daily and hourly demands.

Water Treatment Plant

Customer service preferences, existing concentrations of constituents in the source water, public health, water quality, and cost effectiveness determine the type of water treatment to use. The source water will require treatment to accommodate seasonal and operational changes in concentrations of the constituents' in the water. A major factor for selecting and designing a water treatment plant will be how flexible it is to accommodate these changing water conditions.

Additional considerations for types of treatment to be used include requirements that will be able to deal with *Giardia*, *Cryptosporidia*,, nitrates and arsenic. Arsenic levels are more of a concern for groundwater and not surface water supplies.

The water treatment plants evaluated will use conventional treatment methods with two different treatment trains[4]. One is direct filtration, and the other is conventional filtration. The direct filtration treatment train provides for receiving untreated water at the filter beds, with no pretreatment. The raw water needs to be of a very good quality with low turbidity for this method to work The conventional filtration treatment train requires some sort of disinfections and particle flocculation treatment prior to delivering the water to the filter beds. Conventional filtration is preferred since CAP water quality varies from each sampling location site and the time of year the sample was taken.

The overall treatment process preferred in this study includes using untreated (raw) CAP water, a gravity turnout, the option to construct a raw water reservoir, low head pump, screens, aeration, ozone, chemical pretreatment (disinfection and coagulants), rapid and flash mixers, flocculation, sediment beds, filters, post-disinfection, corrosion control, and a finished (potable) water reservoir. In addition, the conventional filtration treatment train will include options to bypass certain processes during those times that CAP water quality is good allowing the plant to be operated very nearly like a direct filtration plant. See figure A-8 and 9 for the plan and hydraulic profiles of a WTP. Descriptions of the major features of a conventional water treatment system are discussed in the following paragraphs.

[4] Treatment train is a term used to briefly list in order the primary physical features and processes of various types of water treatment plants.

Figure A-8, Plan, WTP

Figure A-9, Hydraulic Profile, WTP

Screens and Aeration
Prior to treatment water flows by gravity and/or is pumped by auxiliary pumps[5] from the raw water reservoir to the screens. The screens keep large debris, leaves, sticks, fish, and clumped algae from continuing on to the flash mixer process. An aeration facility will be made a part of the screen structure and used as a bypassed option in the treatment process. Aeration is recommended where taste and odor of the water is objectionable, high concentrations of carbon dioxide exist, or concentrations of iron and manganese are higher than 0.3 mg/L. Iron concentrations in CAP water have been known to exceed 0.3 mg/L periodically.

Ozone and Coagulant Additives
After the raw water passed through the screens and aeration process, the water flows through an ozone treatment process. See figure A-10 for schematic sketch. Ozone is added to the water, which kills most organisms. Ozone can also assist in controlling taste and odor related substances, reduces tri-halo methane (THM) byproducts, and prevents the growth of algae and slimes in downstream treatment processes. In addition, a chemical coagulant will be added to the water between this structure and the flash mixing basins. Coagulants help fine particles agglomerate, or clump together, to form large particles. Coagulants also help in adjusting pH levels.

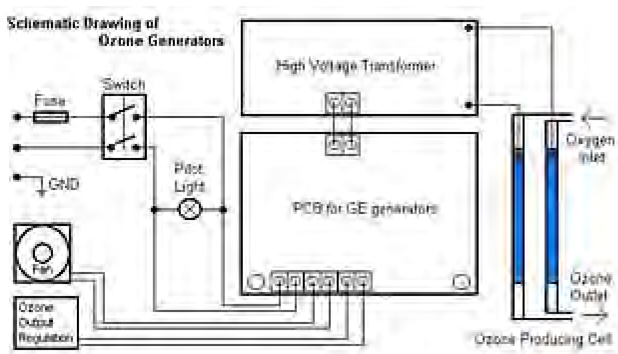


Figure A-10, Schematic Drawing of Ozone Generator

Flash Mixing Basins
After the ozone and coagulant additives process, water is enters flash mixing basins. In this step, the chemicals added to the water during the previous treatment train steps are thoroughly mixed with mixing paddles. If needed, additional chemicals can be added at this juncture.

Flocculation and Sedimentation Basins
The particles in the water entering flocculation and sedimentation basins from the flash mixing

[5] The auxiliary pumps will only be necessary during times when levels in the raw water reservoir are low.

basins are now allowed to settle by reducing the velocity of the flow. Flocculated particles, or floc, consisting of all precipitated particles formed as a result of adding the coagulant, are collected at the bottom of the basins. The floc is then siphoned out of the basins, and eventually disposed of in landfills or used on crops or tree farms. The remaining water then flows to the filter. It should be noted that the effectiveness alum as a flocculate is a factor of water temperature. This will have a bearing on the effectiveness of the flocculation process and the backwash cycle of the filters.

Filters
Water enters the filters after being treated in the flocculation and sedimentation basins. Also, during times when the quality of CAP water permits, flocculation and sedimentation processes can be bypassed, allowing the water to flow from the ozone structure directly to the filters. The filters will remove the remaining suspended particles. For this evaluation, it was assumed that a multimedia filtration system consisting of four layers of filtering media was used. This filtering media consists of charcoal, sand, garnet, and gravel layers. The flow rate through the filter is designed for 2 to 3 gallons per minute per square foot. The filter beds require back washing. The backwash water is processed in clarifier basins and recycled. An additional air system is installed to save water and purge the filter beds more effectively during backwash.

Post Disinfection and Chemical Treatment
After filtration, the water is chlorinated to kill remaining disease-causing organisms and provide a chlorine residual for the distribution system. For this evaluation, it is assumed chloramines, is used as a disinfectant. If required, other chemicals can be added to assist with corrosion control. (Note, Chloramines may be replaced with Chlorine for the preferred water properties. See references to Tucson WTP and changes made to residual disinfection chemicals in the clearwell storage reservoirs.)

Clearwater Forebay Reservoir
After the treatment process the finished water enters and is stored in a clearwater forebay reservoir. The reservoir allows additional chlorine contact time and residual mixing in order to complete the treatment disinfection of the stored water. After the water is disinfected, it is available for use by consumers. A total system reservoir design capacity is anticipated to be at least 30 percent of the maximum daily use. The forebay reservoir is also designed to allow equalization during daily fluctuations of the pump and gravity demands that exceed the combined available production of the WTP and groundwater well pumping. The reservoir provides the distribution system with a reserve capacity to meet fire and domestic supply demands when they are at the peak.

Standard Design Criteria

Design criteria determine the size of the water treatment facility and distribution pipelines. Average and peak water use demands and fire flow capacity represent major factors used to determine what the size of the facilities to treat, transport, and store water. For this study, the Annual Average Water Requirement is computed for an area of mostly residential and light commercial and light industrial populations. [See Table A-4, "Water Demand, Annual Average Water Requirements (gpcd)".] This capacity is adjusted down for water savings programs and adjusted upwards for intensive water use facilities. The average is multiplied by the population to determine the average demand per capita per day, or by year.

TABLE A-4
Water Demand - Annual Average Water Requirements (gpcd)

Water Use	GPCD
Residential	75 to 130
Commercial/industrial	70 to 100
Public	10 to 20
Loss and waste	10 to 20
Total	165 to 220 gpcd

(Excluding fire fighting)
(Mostly residential and light commercial/industrial)
(gpcd - gallon per capita per day)

The seasonal and daily peak demand multipliers are used to determine the size of the water treatment, distribution, and storage capacity of a water system. Standard demand multipliers are shown below in Table A-5. For this study, a peaking factor for the treatment facilities and pipelines will be 1.5 times the average demand. Any other peaking requirements above this amount will be provided by groundwater pumping or reservoir storage.

From Table A-5, it is possible to multiply the factors by each other in order to arrive at the theoretical peak demand. For example, in July, it is possible that a daily demand of 1.3 x 1.8 = 2.34 (or greater) above the average is the theoretical peak demand. This value reflects the capacity to be provided by the water treatment plant, well water production, and reservoir water supply. During the lower demand period, (at night) the system should be operated so that the storage capacity recovers. For example, the daily 8-hour peak demand drawdown of the reservoirs would require the off-peak water production and supply of demand for 16 hours to refill the reservoirs and recover.

In addition, growth factors should be used to define the yearly growth of the population, and the demand. The growth factor to be used in the study follows the "Demand Data From Scenario 23 (Revised 2/23/00)", [See Table A-6]). The figure showing the related Water Provider Areas (WPA) graphically is Figure A-1 at the beginning of Appendix A. The table represents the demand projections of continued well water use for the years 2000 to 2025 in 5-year increments of an increasing rate per year. The projection of demand and population growth resembles a steep, linear rate per year.

TABLE A-5
Demand Multipliers for Peak Periods

<table>
<tr><td colspan="2"><u>Consumption time/period</u></td><td><u>Multiplier</u></td></tr>
<tr><td colspan="2">Winter</td><td>0.80</td></tr>
<tr><td colspan="2">Summer</td><td>1.30</td></tr>
<tr><td colspan="3"></td></tr>
<tr><td></td><td>Maximum daily</td><td>1.5-1.8</td></tr>
<tr><td></td><td>Maximum hourly</td><td>2.0-3.0</td></tr>
<tr><td></td><td>Early morning
(before showers)</td><td>0.25-0.40</td></tr>
<tr><td></td><td>Noon</td><td>1.5-2.0</td></tr>
</table>

Required fire flows represent the additional water storage and delivery capacity that must be maintained without interfering with normal deliveries. Fire flows are assumed for this study to be 1,500 gpm for 8 hours. The maximum design consideration is 10 hours, which can be a factor to be considered for establishing the size of a reservoir, minimum pipe sizing and the minimum water distribution system pressures.

The water treatment and distribution system layouts tend to conform to certain design location criteria. This study uses raw water storage and gravity water delivery systems as much as is practical. The order of preference for design and layouts for the WTP system for this study are the following.

1. Gravity delivery using a raw water source that is higher in elevation than the subsequent treatment and delivery infrastructure. This allows for the hydraulic head to be easily maintained.

Table A-6, "Demand Data From Scenario 23 (Revised 2/23/00)", 8 1/2 by 11 landscape

2. Pumping of either raw or potable water from a lower elevation to a higher water storage elevation. Requires pumping water to reservoir storage so that an average demand capacity of 24 hours exists.

3. Pumping without major water storage. This requires the use of peak pumping capacity and power and large backup pump sizes. Of concern when operating a large system is control of the cost of power, operation and maintenance, reliability and efficiency.

A finished water reservoir has a minimum water storage capacity in order to account for emergency water demands. In order to provide flexibility and maximize efficiency of the water treatment plant production, and minimize pumping, additional water storage capacity is included in the reservoir. The reservoir storage is typically 15 to 30-percent of maximum daily use. The total reservoir capacity for this system will be 30-percent. For the total study, 30 % storage equals the following.

- 75-percent of average annual demand
- 56-percent of July and August Average Daily Demand
- 25-percent of Hourly Peak Demand

Water pressure and elevation of reservoirs and facilities above the service areas are related to economics, types of uses, size of distribution system and length. The type of use, operational flexibility, operational control available, and efficiency of the water distribution system determine water pressure design requirements of a system. Residential use is defined as homes; fire flows for fire fighting; commercial, such as public businesses and stores; industrial, which may have special needs; and turf watering. Each of these has volume and pressure flow requirements.

For residential/domestic uses, the water pressure is typically pressure controlled to between 25 and 40 psi. Fire hydrants demand a minimum pressure of 60 psi, which allows for up to 30 percent friction loss in the fire hoses. Commercial and industrial applications typically require pressures of 75 psi and higher.

The volume of flow to high demand areas with minimum and undersized pipe sizing can be compensated for with increased pressures in conjunction with the use of operational controls. Typically, pressure reducing valves and control center monitoring can use pressure to offset minimal pipe sizing to maintain water delivery.

Due to the large economic considerations of pipe diameter, pressure and frictional losses related to the velocity of flow, a criterion to standardize the designs is used for all applications of this study. Standard design criteria for elevation and distribution system are the following.

- Pipe sizes to maintain velocities of approximately 5 feet per second.

- Pipe delivery pressures of 70 psi will be maintained to compensate for pressure losses and allow for flexibility of operation.

Further analysis of individual distribution systems will be required to determine economical pipe sizing to fully utilize available pressure.

Water Treatment Plant Location Assumptions

Current and future residential population projections will ultimately determine where water treatment plants will be located and constructed, with the size of the first plant being determined by current demand. By the year 2025, future water treatment plant(s) will have the planning flexibility to be located closer to areas of greater population density. See Figures A-11, Residential Units in Place 2000, and A-12, Residential Units Added 2000 to 2025 for graphic population intensity changes.

Presently, the Citizen Utility area north of Grand Avenue and the Goodyear service area around I-10 highway encompass the two major population areas. The Citizens area south of Grand Avenue and the Goodyear service area around I-10 dominate future service areas.

In addition to the current and future population densities in the area, the topography of the land will also be considered in determining where water treatment plants will be located. This information will aid in placing the plants in locations where gravity service pressures can be used instead of pumping the water at an added expense. Consideration is geven to transporting raw water in an open channel which is far less expensive and hydraulically more efficient than transporting by pipeline. The above statement is true as long as the hydraulic head conserved by transporting in a pipeline is not required for current or future delivery pressures. Additional consideration is given in balancing where the water treatment plant is to be located in relation to the raw water supply source flowing in the open channel, and how great the distance will be to pipe the treated water.

Future water treatment technologies will also have a bearing on location of facilities. An example is the current use of micro-, ultra-, nano- and reverse osmosis filtering to improve water treatment and quality. The high water pressure required for many of these filtration types can use gravity systems to provide the pressure, rather than pumping. Consideration for conservation of

Figure A-11, Residential Units in Place 2000

Figure A-12, Residential Units Added 2000 to 2025 for graphic population intensity changes.

pressure head is noted for this study, but is not discussed further except in the concept of future flexibility.[6]

Source Water Quality

The source water is untreated, or raw CAP water which generally contains impurities, such as suspended solids, total dissolved solids (TDS), iron, manganese, coliform bacteria, and toxic chemicals. An understanding of the characteristics of source water, such as turbidity, alkalinity, pH, and color are required in order to provide proper treatment. CAP water does not exceed MCLs established under the Safe Water Drinking Act. Of note are that deliveries from Lake Pleasant through Waddell Canal is a blend of Colorado and Agua Fria River waters, with the Colorado River supplying approximately 90 percent of the water. The water quality for the proposed plants will differ from the water delivered to Glendale's Pyramid Peak WTP and to the Tucson area in the following way.

The Colorado River has a TDS level of approximately 660 milligrams per liter (mg/L) versus approximately 430 mg/L for the Agua Fria River. Due to in-channel evaporation the resulting TDS levels are 743 mg/L for CAP water prior to storage in Lake Pleasant and 697 mg/L for delivery of the blend below Lake Pleasant.

CAP water also contains certain natural organic compounds, which in combination with the disinfectant chlorine, react to form trihalomethanes (THMs). High concentrations of THMs have been shown to cause cancer in laboratory animals. The filtration and disinfection process will be designed to remove as much of the organic and disease-causing organisms prior to disinfecting with chlorine and using the best available technology and management practices.

Table A-7 lists the average water compositions developed from 1994 through 1997 for CAP water at four sampling sites. ("Reverse Osmosis Treatment of CAP Water for the City of Tucson," draft, November 1998).

6 Facilities such as the Olivenhain Municipal Water District in Encinitas, California has constructed a 25 mgd ultra filtration plant instead of convention treatment. Costs are comparable ($30 million). Modules can increase future capacity by 1mgd increments. The plant uses energy efficiency to earn credit for power from hydraulic falling head. Land size is 75% smaller. Finished water quality is better, and less chemicals. (ref. August 2000 Civil Engineering magazine.)

Table A-7. Average water compositions October 1993 to December 1997 and design water compositions

Parameter	USGS analysis No.	Unit	Colo. River below Parker Dam 09427520	CAP Canal at MP 7.98 near Parker 09426700	Colo. River average of two stations	Estimated Colo. River composition in 2015 (w/743-mg/L TDS)[1]	CAP Canal at MP 162.3 at 7th St. 09427100	CAP Canal at MP 252 near Coolidge 09427300	CAP Canal average of two stations	Estimated CAP Canal composition for 743-mg/L TDS Colo. River[1]
pH	400		8.1	8.3	8.2	8.2	8.4	8.7	8.5	8.5
Ca	915	mg/L	76.0	75.1	75.6	??	70.0	69.9	70.0	79.2
Mg	00925	mg/L	29.0	28.7	28.8	??	27.8	28.1	28.0	31.6
Na	00930	mg/L	99.0	98.4	98.7	??	92.5	93.5	93.0	105.2
K	00935	mg/L	4.7	4.6	4.7	??	5.0	4.9	4.9	5.6
HCO_3	00453	mg/L	158.8	153.5	156.2	??	156.9	151.2	154.0	174.3
SO4	00945	mg/L	269.0	272.5	270.7	??	245.7	249.1	247.4	280.0
Cl	00940	mg/L	91.7	90.6	91.2	??	83.7	84.0	83.8	94.9
F	00950	mg/L	0.4	0.3	0.3	??	0.4	0.4	0.4	0.4
SiO_2	00955	mg/L	7.7	8.0	7.9	??	9.1	8.3	8.7	9.8
As	01000	µg/L	2.3	2.3	2.3	??	3.3	3.3	3.3	3.7
Ba	01005	µg/L	125.9	128.4	127.2	??	115.9	123.4	119.6	135.4
B	01020	µg/L	137.9	140.4	139.2	??	142.0	141.6	141.8	160.4
Fe, total recoverable	1045	µg/L	94.7	52.8	73.8	??	52.0	465.2	258.6	292.6
Fe, dissolved	01046	µg/L	< 3.0	< 3.0	< 3.0	< 3.0	8.3	18.6	13.4	15.2
Mn[2]	01055	µg/L	28.0	19.2	23.6	??	41.6	88.9	65.2	73.8
Sr[3]		µg/L	1,092.0	1,079.7	1,085.9	1,228.8	1,194.9	1,198.0	1,196.5	1,353.9
TDS, NF/RO sum[4]		mg/L	736.6	732.0	734.3	ERR	691.2	689.7	690.5	782.7
TDS, 180 C	70300	mg/L	693.0	682.2	687.6	??	649.0	646.7	647.9	733.1
TDS, sum[5]	70301	mg/L	658.4	654.8	656.6	??	614.1	617.5	615.8	696.8

[1] The design compositions are obtained by multiplying the average compositions to the left by the ratio of the projected mean TDS, sum, in 2015 below Parker Dam with new salinity controls (743 mg/L) and the above 1994-97 average Colorado River TDS, sum, of 656.6 mg/L. The ratio is: 1.132.

[2] Values listed for Fe and Mn are unrepresentatively high because the averages do not include below-detectable observations.

[3] For Colorado River Water, Sr is estimated from Sr/(Ca+Mg) ratios at the Water Quality Improvement Center, April - June 1998. For CAP Canal water, Sr is estimated from Sr/(Ca+Mg) ratios at Tucson Water February - June 1998.

[4] Membrane manufacturers frequently refer to the sum of constituents as TDS. This TDS does not subtract any alkalinity and reports silicon species as SiO_2. For waters in this study, it is related to TDS, sum, by: TDS, NF/RO sum = TDS, sum + 0.508*HCO_3 - 0.27*SiO_2 + concentrations of solutes other than those in foot- note 5.

[5] This is the estimated average of "TDS, sum (70301)" in U.S. Geological Survey Water-Data Reports. It is calculated to correspond to TDS by evaporation at 180 C by: TDS, sum = 0.6*alkalinity + Na + K + Ca + Mg + Cl + SO_4 + SiO_3 + NO_3 + F.

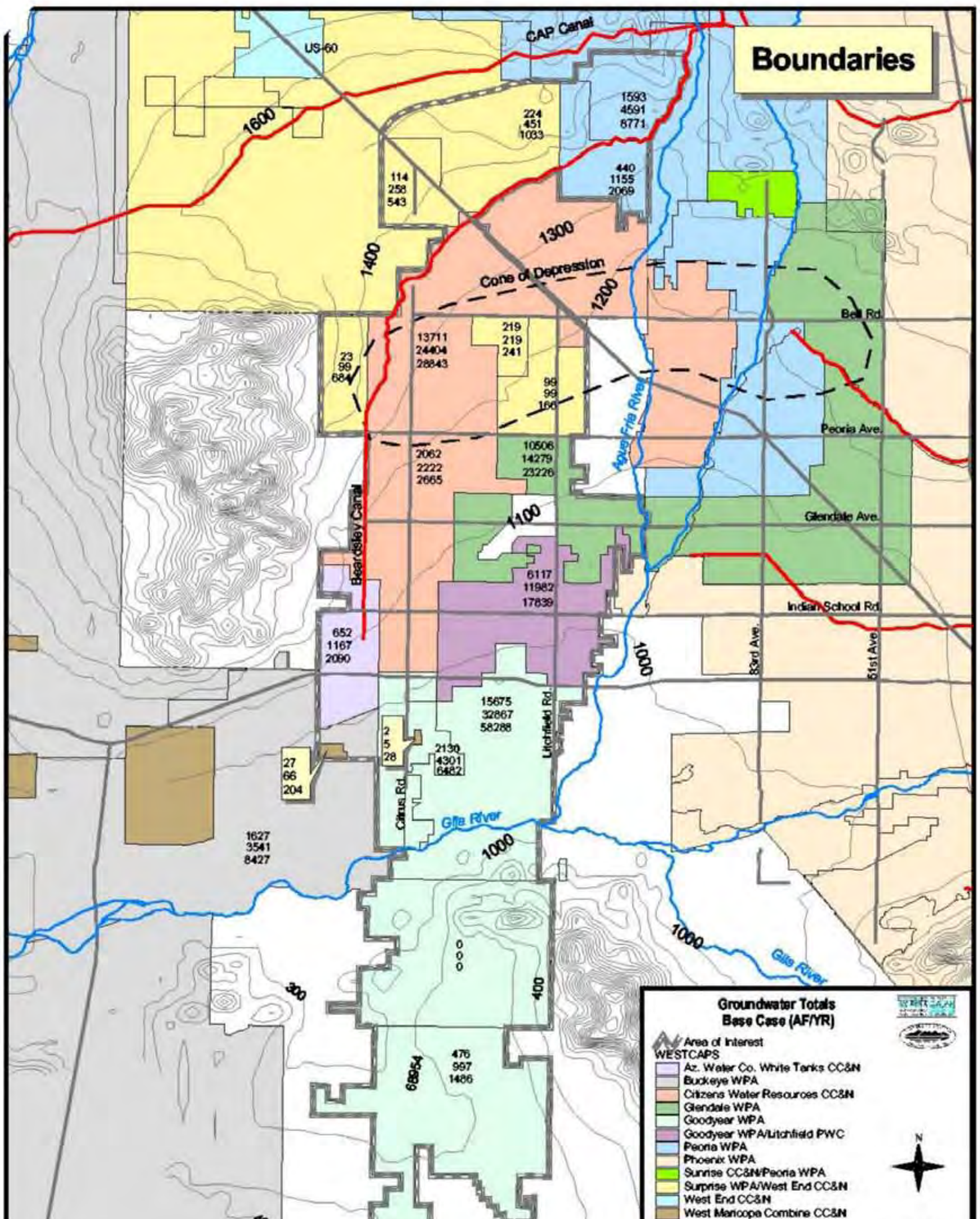
Pressure Zones in the Study Area

Typically, water systems are separated into geographic areas with similar land surface elevations, similar pressure gradients, or certain pressure requirements. These geographic areas are called zones and are separated areas of minimum and maximum pressure areas per elevation. Turnouts are subsequently located to correspond to the pressure zones within each individual system. For this study, the 100-foot elevation interval contours are used as a common and general pressure zone standard. See table A-8 for a listing of pressure zones and pertinent physical parameters. These pressure zones represent the delivery pressure achieved by pressure reducing valves or booster pumps from the trunk line or lateral. See figure A-13 for the 100-foot topographic lines for the study area.

Table A-8. Regional Zone Boundaries and Highwater
Regional common elevations

		Boundaries Service Elevations	
Zone	Highwater Elevation	(86.5) 85 psi[1] Minimum elevation	(43.26) 40 psi[1] Maximum elevation
A	1000	800	900
B	1100	900	1000
C	1200	1000	1100
D	1300	1100	1200
E	1400	1200	1300
F	1500	1300	1400
G	1600	1400	1500
H	1700	1500	1600
I	1800	1600	1700
J	1900	1700	1800

Note: [1] Pounds per square inch.



Boundaries
CAP Canal
US-60
1600
1400
1300
1200
1100
1000
400
300
Cone of Depression
Bell Rd.
Peoria Ave.
Glendale Ave.
Indian School Rd.
83rd Ave.
51st Ave.
Beardsley Canal
Agua Fria River
Litchfield Rd.
Citrus Rd.
Gila River
1593
4591
8771
224
451
1033
440
1155
2069
114
258
543
219
219
241
13711
24404
28843
23
99
684
99
99
166
10506
14279
23226
2062
2222
2665
6117
11982
17839
652
1167
2090
15675
32867
58288
2
5
28
2130
4301
6482
27
66
204
1627
3541
8427
0
0
0
476
997
1486
68954
Groundwater Totals
Base Case (AF/YR)
Area of Interest
WESTCAPS
Az. Water Co. White Tanks CC&N
Buckeye WPA
Citizens Water Resources CC&N
Glendale WPA
Goodyear WPA
Goodyear WPA/Litchfield PWC
Peoria WPA
Phoenix WPA
Sunrise CC&N/Peoria WPA
Surprise WPA/West End CC&N
West End CC&N
West Maricopa Combine CC&N
N
1 0 1 2 Miles

Overview of the Geology of the West Salt River Valley

Introduction

Westcaps infrastructure strategies (6/30/00 and 9/15/00) were two appraisal level plans which focused on treating and distributing renewable CAP water supplies to various municipalities and water providers in the West Salt River Valley. The communities in the west valley have planned to rely less on groundwater pumping, and thus require the infrastructure to take delivery of surface water. The sought after planned reduction pumping will help to mitigate the current and historic groundwater declines which have plagued the WSRV for decades. This pumping has led to land subsidence and the degradation of water quality as the water levels have dropped.

These appraisal reports include two regional water treatment plants, storage reservoirs, and pumping capability to convey the treated and potable water through a trunk pipeline and to a number of customer turnouts (Figure A-4). The North Beardsley Regional Water Treatment Plant is located at the CAP Canal north of US-60. The South Beardsley Regional Water Treatment Plant would take water from the Beardsley Canal just above Peoria Road along Perryville Road. The north and south trunk pipeline segments follow a north-south route along Sarival Road to the southern terminus about 10 miles south of the Gila River. Twelve turnouts (five on the west side of the trunk pipeline) extend from the trunkline from one to about five miles. This two plant layout forms a "corridor" bounded by the Hieroglyphic Mountains and CAP Canal on the north, the Agua Fria River and 107th Avenue on the east, the Sierra Estrella Range/Buckeye Hills and Gila River on the south, and the White Tank Mountains, Beardsley Canal, and Perryville Road on the west. The corridor study area is shown on Figure A-14, GEO-1.

This appendix presents an overview of the regional geologic framework and physiography of the West Salt River Valley (WSRV) sub-basin. It is a look at the foundation geologic conditions and potential geologic hazards, such as (flooding, subsidence, etc.) relevant to the planning and design of the plants and pipelines in the west valley corridor. The corridor discussion begins with the depth to rock and surficial soil types and projected groundwater levels in the water treatment plant and pipeline route areas.

This information was compiled from available reports from a number of agencies – mainly the Bureau of Reclamation, US Geological Survey, Arizona Geological Survey, and Arizona Department of Water Resources, cited and referenced herein.

Figure A-14, GEO-1, West Salt River Valley "Corridor" Study Area.

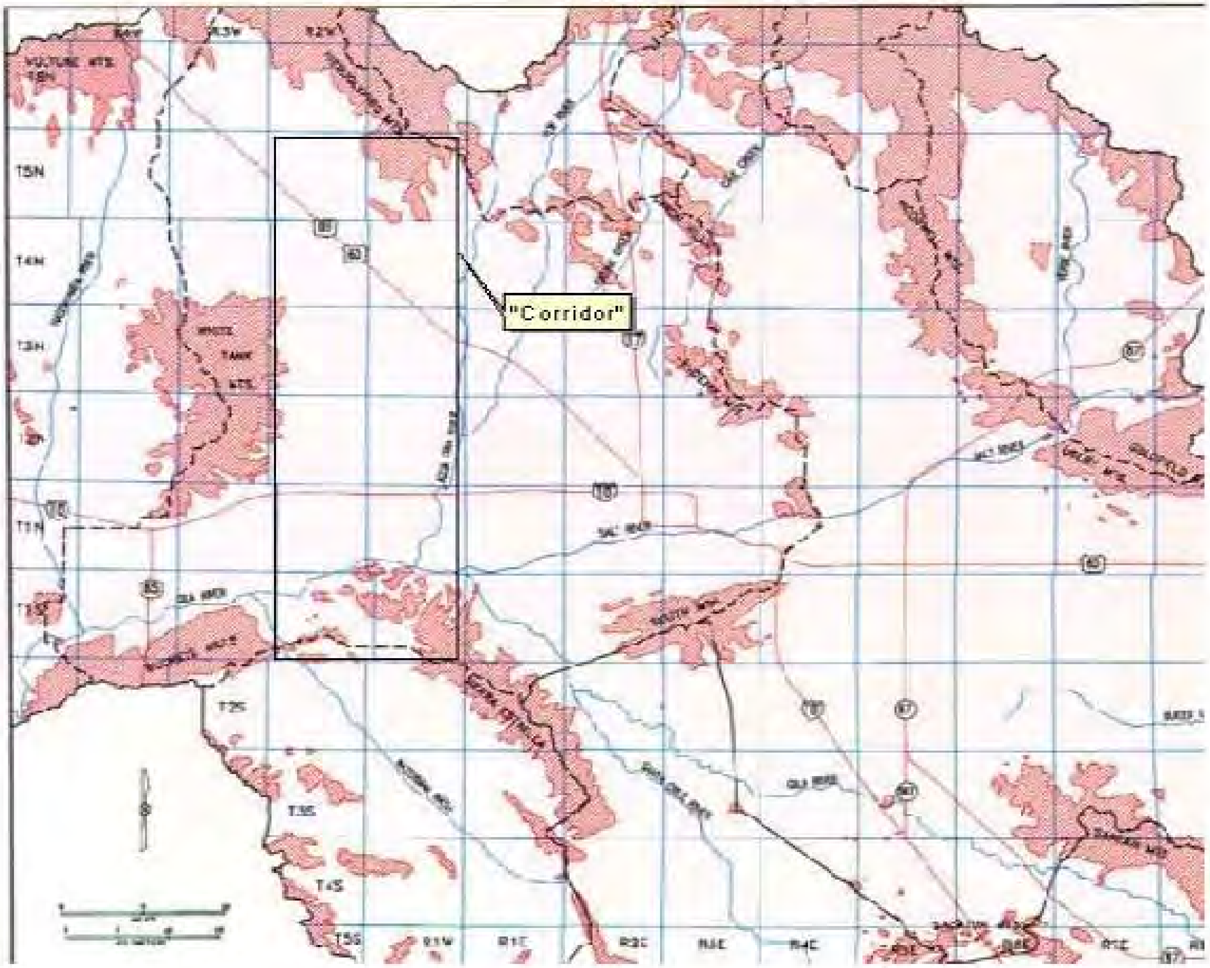


Figure A-14, (GEO-1). West Salt River Valley "Corridor" Study Area
(Modified from ADWR Fig.1, Modeling Report No. 6)

Regional Framework

The WSRV is one of seven sub-basins in the Phoenix Active Management Area and covers 1,300 square miles, or about 850,000 acres, in the Arizona basin and range physiographic province.
The Hieroglyphic Mountains, the Union Hills, and Phoenix Mountains on the north and east, and by South Mountain, the Sierra Estrellas, and Buckeye Hills define the sub-basin on the south. The White Tank Mountains and Hassayampa River define the west limits of the basin. The west valley surface topography ranges from about 800 feet in the Gila River floodplain to 2,000 feet above sea level (amsl) on alluvial fan piedmonts in the northwest portion of the basin towards Morristown. Most of the west valley ranges between 1000 to 1300 feet amsl with the alluvial valley surfaces rising at shallow topographic gradients northwards.

The WSRV is drained by broad, normally dry and shallow river drainages, the largest being the Salt and Agua Fria Rivers which are tributaries to the west flowing Gila River. The Gila River is perennial downstream of the 91st Avenue Waste Water Treatment Plant because of effluent discharges and groundwater flow convergence. Groundwater underflow enters the WSRV sub-basin from the ESRV sub-basin around South Mountain, and from the Hassayampa sub-basin in the northwest corner. Groundwater exits the sub-basin near Arlington.

The WSRV sub-basin is a northwest-trending structural basin up to 2 miles in depth. It formed over approximately the last 15 million years from high angle block faulting of the surrounding crystalline basement complex. Over this time period, the basin filled with sediments shed from the surrounding highlands. These sediments were carried by ancestral streams, and in some places sediments formed by evaporation in shallow lakes to form the present basin-fill geomorphology.

Previous investigators have subdivided these water-bearing basin sediments into three hydrostratigraphic units based on their lithology and hydrologic properties. These three units are termed the Upper, Middle, and Lower Alluvial Unit Aquifers. These Tertiary to Quaternary aged alluvial basin fill deposits consist of interbedded, unconsolidated to caliche cemented gravels, cobbles, sand, silt, and clay of alluvial fan, playa, and fluvial origin. In the WSRV, the three units collectively range from 3,000 to over 10,000 feet thick in the central portion of the basin. Coarser alluvial fan deposits and the finer basin-fill deposits often interfinger along the mountain fronts.

Included within the lower alluvial unit in the broad central portion of the WSRV is a salt dome structure referred to as the Luke Salt Dome or lake body. This body was formed as water evaporated from stratigraphically younger successions of playa lakes leaving behind fine-grained evaporite deposits. This salt body is below the water table depth that most lower unit wells pump from, although there are some wells that extend below the top level of the dome off to the sides. This unit is included with the impermeable bedrock for modeling purposes in the SRV groundwater model. In ADWR's Modeling Report No. 6, the salt dome is shown in elevation, as high as sea level elevation or about 1,300 feet below ground level.

The upper alluvial unit (UAU) is dewatered in much of the valley and its water-bearing northern extent occurs roughly south of Bell Road and west of Scottsdale Road. It ranges from zero to 400 feet thick. The MAU is up to 1600 feet thick while the LAU is several thousand feet thick, to possibly as thick as 2 miles adjacent to the salt dome. The units tend to feather out and thin against the rising piedmont slopes and on pediments of the mountain cores. The MAU is locally dewatered east of the White Tanks, south of Luke AFB, and in Deer Valley (Brown and Pool, 1989). The basal LAU was deposited on a reddish fanglomerate/clastic unit referred by investigators as the red unit. These sediments accumulated contemporaneously with and after the late stages of the basin and range faulting as the basin subsided. These lower units are in fault contact with the precambrian basement rock at the valley margins (Anderson and others, 1990).

In most places these alluvial units are bounded by generally impermeable bedrock. The bedrock underlying and surrounding the sub-basin aquifers is composed chiefly of precambrian to mid-tertiary crystalline metamorphics and granitic intrusives with some younger volcanic flows and intercalated volcanics within the lower unit. Some late tertiary exposures and remnants of the reddish fanglomerate and other well-indurated clastic sedimentary rocks prominently outcrop around the valley. Examples are the Tempe Buttes and the head of Camelback Mountain.

Summary of Geology in the West Valley Corridor

This section summarizes the geology within a north-south rectangular corridor in the west, one-half of the WSRV sub-basin. This rectangle covers the geographic area T5N to T1S, R1W to R2W, G&SR Meridian, and encompasses the proposed infrastructure layout. The corridor is bounded by the Hieroglyphic Mountains and CAP Canal on the north, the Agua Fria River and 107th Avenue on the east, and the Sierra Estrella Range/Buckeye Hills and Gila River on the south. It is bounded on the west by the White Tank Mountains, Beardsley Canal, and Perryville Road. These mountains are composed of Precambrian crystalline rocks (schist, gneiss, granites, and metavolcanics), and in some places tertiary volcanics. The "station-to-station" geology will be referenced where necessary using major roads.

The North Beardsley Regional Water Treatment Plant is located at the CAP Canal five miles north of US-60 on Sarival Road. The South Beardsley Regional Water Treatment Plant is located at about Perryville and Peoria Roads at the base of the White Tanks near the Beardsley Canal. A pipeline would trend north-south along Sarival Road to the southern terminus about 10 miles south of the Gila River.

Depth to bedrock may be shallow and it is exposed in places roughly one to two miles southeast of the North Water Treatment Plant (some isolated volcanic bedrock outcrops exist near Dynamite and Reems Roads). Shallow to exposed bedrock occurs at the Sierra Estrella Park along Baseline and Sarival Roads near the Gila River. A large triangular shaped graben (downdropped block forming a structural depression) forms the west valley floor with the steeply dipping basin-bounding faults roughly defined at the surface by US-60 on the north, Citrus Road on the west, and Southern Avenue on the south (US Geological Survey, Water Resources Investigations (WRI) Report 88-4202, Fig.3, Sheet 1 of 5). The bedrock floor of this graben is interpreted to be more than 1,500 feet deep in the area of Luke Air Force Base. This depth is also about 1500 feet between the Agua Fria River to the east and about Citrus Road where the South Water Treatment Plant is located on the west. The surface hingeline expression of the west-side basin-bounding fault appears to occur beneath the South Water Treatment Plant area.

R.T. Moore and R.J. Varga (1976) show the surficial alluvial materials exposed at the surface in the west valley (Map I-845-J, not included). This map shows that the ground surface from the North Water Treatment Plant southwards to where US-60 crosses the Beardsley Canal is composed of heterogeneous mixtures of fine sands, gravel, and some boulder-size, alluvial fan and terrace deposits. Moore and Varga describe this deposit as generally well consolidated with the larger fragments in a variably calcium carbonate cemented (caliche and clay) matrix of sand and fines. It occurs as discontinuous lenses or pockets and thin sheets. Zones of strong caliche can exist in fan deposits along the White Tank foothills and associated with Agua Fria terraces, and could present some minor excavation problems.

From the US-60/Beardsley Canal crossing southwards along the pipeline route, to the Gila River floodplain, the alluvium is finer-grained floodplain deposits. They describe the alluvium as loose to moderately consolidated, mostly poorly graded sands, silts, and clayey sand with gravel and cobbles. The Agua Fria and Gila River stream channels typically contain coarser sand, more gravel, and larger cobbles. Brown and Pool (1989) mention that upper alluvial deposits typically contain at least 80-percent sand and gravel along the Salt and Gila River channels, and from Agua Fria River deposition in an area northeast of Luke Air Force Base.

In drillhole (B-3-2) 15dbb, located about a mile east of the Beardsley Canal, approximately at Citrus and Thunderbird Roads (just northeast of the proposed South Beardsley Water Treatment Plant), the combined upper and middle alluvial unit thickness is about 500 feet. Surface elevation is about 1,300 feet. The percent sand and gravel on Section A (Brown and Pool, 1989) is shown as about 30 to 60 percent between 800 and 1000-feet amsl. The basin-bounding fault (top of rock) on this section is about 1,600 feet below ground surface.

In this area, the water table elevation is presently at about 800 feet. A groundwater simulation (the WESTCAPS Strategy of September 15, 2000) shows that by year 2010 the water table has recovered to about 830 feet, and by year 2025, to about 880 feet amsl. Groundwater at depths exceeding 400 feet will not be an issue during excavation in this area.

In another drillhole, (B-2-2) 10ccc, also located about one mile east of the Beardsley Canal (along Citrus Road) near Glendale Avenue, the upper alluvium is approximately 250 feet thick, the middle unit (MAU) about 150 feet thick, and the LAU about 1,000 feet thick. Ground surface elevation here is about 1,150 feet amsl. This places the projected top of rock and fault at about 1,400 feet below the ground surface (250 feet below sea level defined by NGVD, 1929). Percent sand and gravel in the lower one-half of the UAU ranges from about 20 to 80-percent (Section D of the Brown and Pool report). Here, the depth to the regional water table is projected to be between 300 to 400 feet.

Drillhole (B-1-2) 9ada2, at approximately Citrus Road and I-10 shows the UAU is roughly 200 feet thick and the depth to rock possibly 1,800 feet from gravity geophysics. Ground surface elevation at Sarival Road and I-10 is about 1,000 feet amsl, and depth to water is simulated to be between 145 to 170 feet between years 2000 and 2025, and 60 to 70 feet at Lower Buckeye Road. The groundwater is sufficiently shallow within approximately a half-mile of the Gila River to possibly affect construction. Groundwater seepage into into any potential pipeline excavation should not pose problems a short distance south of the Gila River to the terminus of the pipeline.

In Reclamation's 1976 report, *Geology and Groundwater Resources Report, Maricopa and Pinal Counties, Arizona (Volume 1)*, for the Phoenix/Buckeye sub-area, includes a general geologic map showing two surficial valley fill units in the WSRV is included. Quaternary Channel and Floodplain deposits (unconsolidated sand, gravel, and fines with local caliche) occur along the Agua Fria and Gila Rivers. Early Quaternary to late tertiary deposits make up most of the corridor between the White Tanks and the Agua Fria and Gila Rivers. These deposits are described as variably consolidated and with caliche common in the alluvium overlying shallow bedrock pediments.

Volume 2 (Reclamation, 1976) includes three geologic cross-sections trending through the corridor. Geologic Section L-L trends north-south through the corridor area and through the Luke Air Force Base to south of the Gila River. Drillhole (B-4-1) 18 bda near the Beardsley Canal/US-60 shows the top 70 feet is composed of gravel and clay overlying 450 feet of silt and clay. Further south near the AFB, drillhole (B-3-1) 32 dda shows the upper 100 feet is sand followed by about 300 feet of silt and clay with gravel. Drillhole (B-1-1) 28 bca, located about a mile north of the Gila River, shows about 50 feet of sand and gravel overlying gravel, sand, and clay. The sand and gravel thickens to several hundred feet as the Gila River is approached.

The left end of Section M begins near the Beardsley Canal and trends eastward across the corridor through the AFB. In most of the drillholes, the upper 50 to several hundred feet consists of sand, or sand with gravel.

Geologic Hazards

Of all the geologic hazards identified in Arizona (flooding, landslides, earthquakes, expansive and collapsible soil conditions, radon, and others), perhaps the most problematic in central Arizona basins (and especially the WSRV) is land subsidence, and in some places, earth fissuring. As much as 18 feet of land has subsided in the Luke Air Force Base area because of groundwater withdrawal. P'ewe' (1989) shows that the area bounded by the CAP Canal, Gila River, the White Tanks, and the Agua Fria (the corridor) is characterized by subsidence greater than 0.5 feet between the period 1905-1985.

Differential land subsidence due to groundwater withdrawal resulted in over 15 feet of ground lowering in an area near Luke Air Force Base (Fellows, 1993). This has caused damage to nearby infrastructure. Fellows goes on to say that in September 1992 when 4 inches of rain fell in one day, the resulting flood (the effects worsened by the land subsidence), exceeded the capacity of the Dysart Drain causing property flooding. At the time the Dysart Drain no longer relieved floods, but exacerbated them since land subsidence caused the drain to slope incorrectly.

On a Bouguer Anomoly map in the 1976 Reclamation report (volume 1), an earth fissure (1964) is shown trending northeast-southwest 1.5 miles across sections 25 and 36 of T3N, R2W (between Glendale and Peoria Roads about 3 miles northwest of the Luke Air Force Base and southeast of the proposed South Beardsley Water Treatment Plant). Another fissure is shown on this map extending north-south across Section 2 of T2N, R1W about one-half mile west of the Agua Fria River. Another shorter fissure is mapped occurring between Litchfield Park and Luke Air Force Base. These fissures are also shown on Water Resources Investigations Report OFR-78-83, sheet 2 of 2 by Laney et al. These fissures seem to ring around and encompass the Luke Air Force Base. It has been postulated by some investigators that these features were formed in response to a combination of water level declines and resulting differential compaction of the alluvium associated with the margins of the Luke salt body at depth.

Overbank flooding is a concern for the Agua Fria and New River drainages as they are relatively broad and shallow, and the known occurrence of near surface caliche and /or clayey zones on alluvial fans and terraces would tend to prevent infiltration. Runoff from the highlands to the north and mountain front runoff from the White Tanks (alluvial fan flooding) would sheetwash south and southeast towards the southern, lower and flatter portion of the corridor (below about Glendale Avenue). The number of damaging flood events in Buckeye and Goodyear between 1962 and 1983 was less than five as shown on P'ewe' (1989) Figure 9.

Although the US Army Corps of Engineers flood control dams should lessen the concern regarding flooding in the west valley, floodplain and alluvial-plain hazard maps of the area may show further study is warranted. An article by the AGS (Pearthree, P.A.,1991) on alluvial-fan flood hazards in Arizona refers to several open-file reports (see references) documenting the surficial geomorphology and flood potential adjacent to the White Tanks. This source should be consulted for feasibility designs.

CHAPTER IV

The Unlimited Supply Configuration

One Treatment Plant (along CAP Canal System)

Water Treatment Plant Site Description

Reach 8 of the CAP Hayden Rhodes Aqueduct begins with the Hassayampa Pumping Plant, which is located 17.9 miles upstream of the planned Water Treatment Plant turnout located in Reach 9 (see figure A-15, Location Map). Two check structures are located between the pumping plant and the proposed turnout location and a third check structure is 2 miles below the proposed turnout location. Also downstream of the proposed turnout is another check structure.

Water is delivered through the CAP system from the proposed turnout located on Reach 9 of the Hayden-Rhodes Aqueduct which is located in the general area of Sarival Road, the CAP canal and north of the State Highway 89 (Grand Avenue) intersection (see figure A-15, Location map of Reach 9). Water delivered from the CAP Canal will be delivered to the water treatment plant. It is anticipated that no reservoir to store and pretreat raw water will be part of the initial design stage. After treatment, finished water is stored in a covered clearwell forebay reservoir. A gravity flow trunk line delivers the water to the south and east. A pumping plant is planed to lift the water for delivery via a distribution pipeline to higher elevations, or pressure zones, close to the plant with insufficient service pressure.

For purposes of this study, the proposed treatment plant site will include the water treatment plant, clearwell forebay reservoir, the pumping plants, and an associated electrical substation. This section of Reach 9 has a capacity under normal flow conditions of 3,000 cfs (2,172,000 AFY). Two check structures located between the Hassayampa Pumping Plant and the turnout will control the water surface elevations of the canal pools upstream of the turnout, and the check immediately downstream will control the pool in which the turnout is located. Operationally, locating the delivery point close to the downstream check structure of a pool provides the deepest water depths and the highest available volume of the check pool water.

Water treatment plant design delivery elevation is anticipated to range between normal water surface elevation to minus 5 feet. Elevation 1,535.87 is defined as the normal water surface at Sarival Road (ground elevation 1,543: slope 10 feet/1,320 feet to the south). Topography of the area naturally allows the gravity flow through the treatment trains of the water treatment plant to flow from the north to the south, or southwest. This would minimize the required earthwork for the facility. Locating the WTP close to the source water maximizes the use of available canal reservoir capacity and topographic elevation. See figures A-16, Granite Reef Aqueduct, Reach 10 for Typical Aqueduct Sections and Figure A-17 and 18 for Plan and Profiles of the CAP Canal alignment.

Because of the location of the WTP in relation to Lake Pleasant, normal operations of the CAP do not access Lake Pleasant water. Reverse flows of the CAP to the water treatment plant turnout is possible during severe outages using the static water levels from the Waddell Turnout and pump lifts using temporary pumps and the check structures. For the purposes of this study, the water treatment plant site will include the water treatment plant (including any advanced treatment facilities), clearwell forebay reservoir, and an associated electrical substation. The site lies south of the CAP Canal and in the vicinity of Sarival Road, along a southerly extension of Sarival Road, between the CAP Canal and the Beardsley Canal alignments (see figure A-18, Standard One Plant Layout).

The 200 foot elevation of the CAP canal at the proposed turnout near Sarival Road is beneficial for water delivery under gravity pressure head. Areas south of the CAP location were initially considered for the WTP site, and may still be considered as an optional location. The Sarival Road alignment was selected for the existing road right of way and the lack of urban development. The rapid urban development occurring near the canal, especially with regard to the road alignments, will hamper the development of the alignment. Assuming projected build-out of the infrastructure, urban development would occur prior to treatment plant development which would rapidly decrease the options for rights-of-way acquisitions, increased easement and installation costs, and limit the location options for facilities. No other facility constraints or operational constraints are evident at this time. See Figures 20 (a,b), "Photos of the CAP Canal and Sarival Road Alignments".

Figure A-15, Location Map. Reach 9.

. figures A-16, Granite Reef Aqueduct, Reach 10 for Typical Aqueduct Sections

Figure A-17 , Plan and Profiles of the CAP Canal alignment.

Figure A-18 for Plan and Profiles of the CAP Canal alignment.

Figure A-19 Site Plan , WTP

Figure A-19 is a computer-generated layout of a 30 to 60 MGD water treatment plant, conventional filtration system, with ozone, raw water reservoir and 20 mg clearwell reservoir. Note, a raw water reservoir is not included as part of the CAP WTP layouts.

(compgenwtp.doc)

Figures 20 (a,b), Photos of the CAP Canal and Saraval Road Alignments.

Figures 21 (a,b), Photos of the CAP Canal and other road alignments to the east of Saraval Road Alignments.

Figure A-22, Standard One Plant layout, North WTP

Water Treatment Plant Sizing

The single water treatment plant for the unlimited supply configuration is sized for a yearly average capacity of 153,344 acre-feet per year. Since it is envisioned that the ultimate capacity will not be needed for at least two decades, the capacity will be constructed in stages (modules) that are phased in over time. To phase in the capacity, certain portions of the plant will be constructed for ultimate capacity in order to allow expansion and addition of equipment over time. This will involve additional initial cost to construction. The important feature of the initial plant construction consists of two parallel treatment train modules by the year 2005 with a capacity of 48 MGD and a 20 MG clearwell reservoir. Future expansion in year 2015 includes an additional 40 MGD expansion and a second 20 MG clearwell reservoir. Final expansion by the year 2025 is for an additional 49 MGD for a total capacity of 137 MGD.

Hydraulic Profile

A hydraulic profile is calculated for the conventional water treatment system. A maximum hydraulic head loss profile of 19 feet is allowed between the entrance to the screens and the finished water reservoir. Figure A-9, "Grade and Hydraulic Profiles", illustrates the profiles of the original ground surface for the proposed site, the hydraulic profile of the water treatment plant, and the excavation profile for the centerline of the water treatment plant. The hydraulic profile will be the same for all of the plant sizes and will be used for determining the position of the plant to the raw water source or if low head, high volume pumping will be used to feed water to the WTP.

Pumping Plant

To move treated (potable) water to the first 2 turnouts identified as locations for introduction of water into local distribution systems, several pumping plants will be required. See Figure A-22, Standard One Plant layout. It was important to identify not only the location of these turnouts, but also the pressure zone within which each turnout is located. Overall, it was determined that water would be boosted or lifted from elevation 1,500 (at clearwell forebay reservoir) to higher elevations.

The total dynamic head (TDH) in the distribution system is primarily made up of the static difference in elevation plus the dynamic pressure developed as water moves through pipelines. A primary pumping plant for the delivery of treated water will be required for service to the east and north. The pumping plant will mainly pump water from clearwater forebay reservoir to service parts of Peoria.

In some areas, inline booster pumps provide additional pressure to laterals or turnouts where pipeline pressure is less than required for service delivery, such as in the service areas of Surprise.

Design Criteria

From the clearwell reservoir, the majority of the water is designed to be delivered by gravity. Along the main trunk line to the south, the first two miles incorporate pipe laterals with booster pumps to meet pressure zone deliveries. All deliveries close to the water treatment plants to the west, north and east require pumping capability. Determining the boosters and lift plants capability requires understanding the complex interaction among power, pipe and material, and pumping plant costs. Although the exact design of relift plants is beyond the scope of this study, several configurations are developed in order to understand relative costs.

Pumping Plant Layout

Water from the clearwater forebay reservoir is fed through a header and suction-side manifold, with individual feed lines to each pump. The discharge of the individual pump manifolds is consolidated into a single discharge header. Flow through the header is measured with ultrasonic instrumentation housed in a metering structure. An air chamber for handling surges within the pipe is needed due to a high pressure head a long discharge pipeline.

A supervisory control system integrated with the water treatment plant and distribution system will provide monitoring and system control. The supervisory control will provide operational status and system control for any shutdowns during emergency conditions and eliminate the need for pumping bypass or return flows. The supervisory control system monitors water treatment plant production, clearwell forebay reservoir water elevation, down gradient reservoirs, and pump operations at the pumping plants. Floats or pressure transducers in the reservoirs and air chambers will be wired to send signals to turn the pumps on or off. The operating water surface elevation of the reservoir is intended to remain fairly constant under normal conditions.

The minimum pressure gradient along the length of the discharge is determined to ensure that water column separation does not occur. This condition could occur if the pumping plant experiences a total power failure. This minimum gradient is used as a basis for the design of the pump discharge line to ensure against failure due to collapse. Control facilities are installed to avoid the condition of water column separation if the computed minimum pressure gradient at any point in the discharge line falls below the vapor pressure of water. Typically, this is accomplished using an air chamber or surge tank.

Gravity Delivery System

The southern service area will rely on a gravity fed system to adequately provide water at sufficient pressure. Valve controlled outlet pipes control the gravity pipelines from the clearwell reservoir. The minimum pressure gradient along the length of the gravity discharge is determined to ensure that water column separation does not occur. This condition could occur if the control valve or pressure reducing valves malfunction, pipe velocity exceeds available capacity, total power failure or system over demand lowers the pipe pressure. This minimum gradient is used as a basis for the design of the trunk line to ensure against dropping below minimum gravity delivery pressures, vapor pressure, column separation and failure due to collapse. Control facilities are installed to avoid the condition of water column separation if the computed minimum pressure gradient at any point in the discharge line falls below the vapor pressure of water. Typically, this is accomplished using air and vacuum valves, air chamber or surge tank.

Conveyance and Distribution System

Reclamation developed feasibility level designs and cost estimates for various distribution system alignments and configurations for this study. The alignments and configurations are developed through extensive consultation with water providers. The criteria used in determining the most cost-effective routing are listed below.

- Delivery areas
- Future delivery capacities
- Locations of reservoirs
- Rights-of-way
- Environmental impacts
- Power costs
- Engineering criteria
- Geology
- Archeological impacts

The design of the distribution system alignments, appurtenances, and configurations underwent several iterations before culminating in the selections presented.

Alignment

An appraisal-level study was conducted for delivering water to the study area. Possible areas of trunk pipelines were determined by researching available easements, predicting area development and anticipating congestion of existing utilities and features. A standard distribution system layout was selected for comparing alternatives. Twelve major delivery points, or turnouts, are located along the distribution system with distances to deliveries ranging from the clearwell reservoir (0 miles) to 35 miles.

The alignment is described as follows. The CAP Water Treatment Plant clearwell reservoir elevation is 1,500 and is located at the CAP canal and approximately Sarival Road. From the clearwell, the pipeline continues to the south, paralleling the Sarival Road alignment for seven and one half miles to Bell Road, situated at elevation 1,300. As the pipeline traverses south the gravity fed line is reduced in size incrementally as deliveries are made to major laterals and service areas. The pipeline splits approximately at Peoria Avenue and feeds a distribution line that provides water to the west, and provides water at an increase in elevation in order to store the water in a floating reservoir at elevation 1,400.

The main trunk line continues south, and other laterals branch off to the east and west from the main trunk line service to the centroid of the service areas identified.

The size of pipes and alignment were determined by hydraulics pipeline analysis, and land topography. Peaking, normal capacity, length, friction losses, and service delivery pressures in year 2025 were considered in order to project the size of pipes needed for the project. Elevations of reservoirs were to maximize the use of gravity for filling and delivery. The locations of reservoirs were selected in order to maximize the hydraulic efficiency for pipe sizes and delivery to major turnouts. See the tables at the end of the Appendix A, Exhibits, Hydraulics, On WTP, Unlimited Supply.

To define the input data for the distribution system analysis, the delivery area is divided into sections by ownership and natural boundaries. For each of the areas, the centroid of the area is assumed to be the delivery point for a distribution system. The capacity of the delivery of each area is defined and the delivery required is identified for comparison between different alternatives and scenarios.

The sizing of the main trunk lines is based on conveying one and a half times the average daily demand at a velocity of no more than five feet per second. The maximum daily demand is calculated at a velocity not to exceed seven feet per second. These criteria will keep the head losses consistent for the delivery comparison of booster pumps and gravity systems. The floating reservoirs are used to balance the daily water production, demand and provide system reserve capacity, discussed below. The delivery system is designed to provide standard water system pressures of 70 psi to the connection points of the centroids or municipal systems. Note that a separate study at the end of this section shows that there is an advantage to using pipe sizes of a larger diameter where possible. The comparison relates the variables of the volume of capacity compared to diameter and the relative friction head loss (see Exhibit number 9 at the end of the Appendix A).

The location of the trunk line is generally aligned from north to south. The exact alignment it will follow is not to be determined. A bandwidth of road locations is summarized and published in the main study and the actual position will be determined by the system pressure criteria, access of available right of way and easements, service delivery areas, geology, incorporation of more than one water treatment plant, and other interconnected facilities. In general, the trunk line is anticipated to run west of Reems Road due to current house and infrastructure development on Reems Road and to the east.

The lateral pipes off the main trunk line will carry water system zone pressures ranging between 40 to 90 psi (average 70 psi) for delivery to the municipal systems. To control the pressure, in line reducing valves and other standard appurtenances are installed. Capacity of the laterals is calculated for the maximum daily demand at a velocity not to exceed seven feet per second. Twelve-inch diameter pipe is the smallest size considered for this study.

Booster Plants

Booster plants are required to deliver water to some of the turnouts located along the distribution line.

Booster pumps are used to lift water to meet service pressures and overcome system friction losses and topography. The capital cost and energy cost of a system increases substantially with each required pump installation and/or higher pressure demand.

Booster pump facilities are installed for areas with less than 140 feet of head available (approximately 70 psi). It is important to note that normal delivery pressures are between 40 and 85 psi. The pressure above the minimum 40 psi required in this study is to account for head losses of system and reserve pressure head for peak demand periods and future delivery capacities.

Extension of water service to the south of the Gila River will be by booster pumping from the main trunk line. Analysis shows that groundwater pumping south of the Gila River is more economical because of the relatively small amounts of water for the length of pipeline that is planned. The main design criteria favors the cost of local groundwater pumping or booster pumping a lesser distance in a smaller pipe to that of a larger size pipe from the north WTP.

Turnout areas requiring minimal pumping are shown in the detailed hydraulics for the one treatment plant system. The summary is shown below in Table A-9.

Table A-9,
Peak Capacity and Total Dynamic Head for the One WTP System

Turnout Number	WPA Description	Peaked Capacity (CFS)	Total Dynamic Head Pumped (Feet)
1	Peoria #5	18.2	**176**
2	Surprise #3	2.1	**145**
3	Peoria #6	4.3	41
4	Surprise #8	1.1	40
12	Goodyear #4	3.1	**87**

Note: For this CAP water treatment plant delivery system the delivery head for turnouts 1, 2, and 12 are the only deliveries that ***must*** have pumping to make any deliveries to those areas. Turnout areas 3 and 4 are considered occasional pumping to meet full (peak) demand capacity. Details and cost of pump stations and pumping can be found in the Hydraulics and Cost Summary tables at the end of this section

Operations

The design flow rate is 136.9 MGD (153,355 AFY). The maximum flow rate of 205 MGD (230,000 AFY, 317 cfs is one and a half times the average annual allocation) was the value calculated for the hydraulic flow in the main trunk line only. The maximum flow rate assumes ground water will be blended with CAP water supply to meet the demand with some contingency. A detailed hydraulic analysis of combined groundwater and direct delivery system is beyond the scope of this study. The flow allows for distribution pipeline peaking, distribution variances, and future operational considerations. The data provided by water providers were used for estimating the size of each turnout and subsequent flow regime.

The infrastructure components for each area delivery include a tee, an upstream shut off valve, a meter, a pressure reducing valve (if needed), and a downstream shut off valve.

System Storage

For this study, the clearwell and trunk line distribution system reservoirs can provide water for a minimum of 18 hours to the delivery area during a short term water supply interruption and/or power outage. The infrastructure could provide up to a 1-day reserve supply by combining the reservoir capacities and groundwater wells. The storage reservoirs should be constructed at higher elevations so gravity deliveries may be used in case of pump or power failures. The storage provided will supply an average daily demand for emergency use by the water providers. The average daily demand capacity incorporated into this system is about 100 million gallons.

Operationally, all reservoirs in the system are filled during off peak periods. The location of the system reservoirs are in areas adjacent to the slopes of the White Tank Mountains, outside the area of the White Tank Regional Park boundaries. See Figure A-23, White Tank Mountain Regional Park Boundary Line.

Water Providers Turnouts

The turnout locations were chosen to maximize the delivery of the domestic water supply into the water providers systems and to be integrated into the future plans of each provider. The sizing of each turnout was determined by consulting with each water provider using the water provider area well water projections. Turnouts are located at the centroid of the water planning area to correspond to pressure zones for each individual system. See table A-5 for a listing of existing pressure zones and pertinent physical parameters. Each water provider ultimately decides the sizing and location for turnouts. See figure A-22 for turnout location.

The water providers' turnouts are sized for approximately twice the anticipated CAP allocation. This allows for water deliveries to include groundwater or future additional CAP allocations that may be acquired. Each turnout includes a tee from the mainline with a gate valve and a blind flange. If required, a pressure reducer will be included. Any boosters past the gate valve are the responsibility of the water provider. The system is designed so that some deliveries are made to the White Tanks reservoirs, and some are made directly to the systems. The cost per turnout is approximately in the range of $10,000 to $30,000 each. See table A-10 for a listing of turnouts.

Table A-10,
Turnout Descriptions - Single WTP - Unlimited Supply

	2025 DEMAND			
TURN OUT #	LOCATION (WPA) DESCRIPTION	TURNOUT CAPACITY		GROUND ELEV.
		MGD	cfs	feet
1	Peoria #5		18.2	1500
2	Surprise #3		2.1	1500
3	Peoria #6		4.3	1360
4	Surprise #8		1.1	1410
5	Citizens Agua Fria		59.8	1250
6	Glendale Out of Service		48.1	1100
7	Citizens Agua Fria #2		5.5	1150
8	LPSCO		37.0	1030
9	Az. Water Co. White Tanks		13.4	1060
10	Goodyear Outside		4.3	980
11	Goodyear #2		120.8	970
12	Goodyear #4		3.1	1100

Figure A-23, White Tank Regional Park, Boundary for Floating Reservoirs

11x17

Cost Summary

For comparison purposes, costs associated with construction of facilities and acquiring rights-of-way are summarized below. These costs do not include other factors that may increase the total, such as mitigation for endangered species, recreational facilities, architectural aesthetics, or cultural resource mitigation. Construction costs are an average, and many related factors such as quantity of pavement replacement, extent of utility relocation, drainage crossings, traffic control, and neighborhood disruptions, will affect the total. The costs are representative of what can be expected for this project. More refined costs will be developed when and if the project moves to a design phase.

Possible Federal participation in the design and construction of various portions or features of a direct delivery alternative, see Table A-11 below, will affect the amortization rate. Potential Federal participation is also summarized below.

Table A-11
POSSIBLE FEDERAL PARTICIPATION

Direct Delivery Alternative feature	Possible Federal participation
Reservoir	Yes
WTP	No
Booster plant	Yes
Distribution pipeline	Yes

Operations and Maintenance Costs

Operations and maintenance costs are based on delivering 153,344 acre-feet of treated water per year. Routine maintenance and contingency funding for the repairs is included in the annual operating costs. Summary of operation and maintenance (O&M) costs for this system are shown in the following tables with the details of factors contributing to O&M costs. Costs are detailed on a cost per acre-foot and cost per 1,000 gallons basis. (See "One WTP System, Unlimited Supply, Water Treatment Plant Total Annual Cost ($) and Pipeline For Single WTP, Pipeline, One Big North Plant, Distribution Pipeline System Total Annual Cost ($)").

Water costs are based on the CAWCD’s "Final 2000 Rate Schedule" for M&I use. The rate varies from $102 per acre-foot for the year 2000, increasing to $129 per acre-foot in the year 2004. The CAWCD's price for water is comprised of a capital component and a delivery component that covers maintenance and energy costs. The total cost is commonly referred to as the "postage stamp" rate. Since the cost of CAP water will vary over time, a value of $150 per acre-foot has been selected to calculate representative costs that will be used to compare alternatives.

The issue of acquiring additional water supplies from other allocation owners is currently projected at $1,500 per acre-foot not including O&M charges. This cost can be inserted into future calculations. As of this publication, a 16,000 acre-foot allocation of Beardsley Canal water is available. The cost of this water and its use through the Beardsley Canal is unknown at this time.

In order to provide a fair estimate of energy costs to operate the system, the estimate includes required pumps and energy cost and also auxiliary booster pumps and energy costs. Required pumps are the system components that are to perform normal daily operations of the system. The other pumps and facilities required during emergency and peak water demand periods are the auxiliary systems, which are also included in the costs, but are to maintain the system operations within the operating parameters. An example of auxiliary facilities pumping costs is to meet the minimum design pressure requirements of laterals under all operating conditions. The alternative assumes the pumping cost is borne by project sponsors using a rate of 60 mills per kWh, which represents the rate for interruptible power for large industrial users.

Right-of-Way Cost

North Water Treatment Plant

The land area needed is anticipated for the facilities, is primarily state or county land, with some private land at an estimated cost of $10,000 per acre. A Reclamation land cost survey was not performed. The estimate may or may not reflect actual prices or future increases in land value for this area. Consideration should be given to acquisition of additional area to allow for future water treatment plant expansion, recreational opportunities, environmental mitigation of the water treatment plant, and alignments and reservoir locations.

Capital Costs

Capital cost for the one water treatment plant layout and distribution pipeline system is calculated in year 2000 dollars. All major facilities required for service delivery to the year 2025 are included in capital costs. A summary of the major features to be constructed and costs are shown in the Exhibits, at the end of the Appendix, "Cost Estimates" with description, cost, and year to be constructed. The tables list the cost for the water treatment plant facilities and on a separate list is the distribution system with the associated major facilities. Each list also includes percentages for contingencies, engineering and administration cost. Included without contingencies is the associated land acreage and cost.

Total capital costs for a "Single North Water Treatment Plant" design layout is $430,641,000 with a cost per 1,000 gallons of $1.31.

The Unlimited Supply Configuration
Two Treatment Plants (along CAP Canal System and Beardsley Canal)

Water Treatment Plant Site Descriptions

CAP Canal WTP (North WTP)

For the "Two WTP Configuration", a North Water Treatment Plant on the CAP canal and a South WTP on the Beardsley Canal is configured in the layout. WTP sizings are smaller and the required land acreage is shown at the end of this appendix on the "Cost Estimate" sheets. Total regional WTP capacity of 153,344 is divided between a North WTP on the CAP and the South WTP on the Beardsley Canal. CAP Canal and WTP descriptions are the same as discussed in the previous section.

Beardsley Canal WTP (South WTP)

The alignment of the Beardsley Canal is shown on the previous Figures A-1 through A-4. A canal layout in plan view is shown on figure A-24. An elevation profile of the canal is shown on figure A-25, Profile of Beardsley Canal Invert. The current water source for the Beardsley canal is the MCMWCD#1 (MWD) turnout on Reach 9 of the CAP. Historically, the Beardsley canal had, and still can, receive deliveries from Lake Pleasant, five miles north of the CAP turnout location. The MWD Turnout on the CAP is Milepost 146.7, just before the entrance to the Agua Fria Siphon. This location provides only Colorado River water. Normal operations of the CAP do not access Lake Pleasant water, which is a combination of Agua Fria River and CAP Colorado River. Reverse flows of the CAP to the MWD turnout is possible during severe outages using the static water levels from the Waddell Turnout. An alternative is to also use the historic Beardsley Canal headworks from the lower lake at Lake Pleasant. For the purposes of this study, the water source is CAP water delivered through the Beardsley Canal. The WTP site will include the WTP (including any advanced treatment facilities), clearwell forebay reservoir, and an associated electrical substation.

Water will be delivered through the Beardsley Canal to a new WTP turnout located in the area between Bell to Cactus roads (see figure A-24, Beardsley Canal Capacity Study.). This section of canal has a capacity under normal flow conditions of 290 cubic feet per second (cfs). From the CAP inlet of the Beardsley Canal to Cactus road, there are seven hydraulic control structures. Operationally, locating the delivery point close to the downstream check structure of a pool

Figure A-24, Beardsley Canal plan view.

Figure A-25, Beardsley Canal Profile View provides the deepest water depths and the highest available volume of the check pool water. The canal section slope between the areas of Bell to Cactus Road is of a gentle slope at approximately elevation 1330. See figure A-25 for elevation Profile of the Beardsley Canal invert. The canal in this area borders on the western boundary of the delivery area along the White Tank Mountain range to maintain the elevation for gravity delivery above the agricultural fields. Ground topography slopes downhill from a westerly to an easterly direction. A canal water supply to a WTP by gravity is possible in this area. Note that the WTP could be placed farther south of Cactus Road, but the elevation and the capacity of the canal drop quickly. The estimated costs for modifications to increase capacity by 50,000 acre-feet per year are shown on Figure A-24 for the area below Cactus Road. The additional elevation loss below Cactus represents flexibility and economics lost to utilize gravity head pressures.

MWD currently holds a 139-foot wide right of way for the Beardsley Canal from the Camp Dyer Diversion at Lake Pleasant to approximately Cactus Road. The remaining right of way south of Cactus Road is 75 feet wide. The width of right of way available south of the Cactus Road area may become an acquisition and improvements cost factor for canal capacity and WTP location.

Currently, the Beardsley Canal is sized and operated to meet the demands for agricultural irrigation uses. The additional available capacity of the Beardsley Canal has been calculated and presented in the Beardsley Canal Capacity Study by Navigant. The improvements to the system to provide summer peaking demands for a WTP and agriculture can be determined from the following summarized information. Photo Figures A- 26 through A-30 give an area overview and views of canal sections of relative size along the MWD alignment. Additional aerial and ground photographs of the canal are attached as exhibits at the end of Appendix A.

Beardsley Canal Capacities

The capacities of the Reaches are shown in the following table, Table A-8, Beardsley Canal Capacity Study. Costs of capacity increases to the system are shown on Table A-13, Beardsley Canal Capacity Study, Capital Costs of Improvements. Cost for these improvements were not included in the cost comparisons at the end of chapter III or Appendix A.

Figure 26, (A,b), Pictures

Figures A-27 (a,b), Picutres

Figure A-28 (a,b), Pictures

Figure A-29 (a,b), Pictures

Figure A-30, one picture of Beardsley alignment.

TABLE A-12
Beardsley Canal Capacity Study,
Summary of Estimated Maximum Reach Capacities

Reach No.	Reach Description	1993 MWD High Flow (cfs) (1)	1995 July Flow Demand (cfs)	July 1995 Available MWD Capacity for WTP in (cfs)	Estimated Maximum Capacity (cfs) (2)	Estimated Maximum Capacity (mgd)
I	Lake Pleasant to CAP Inlet	230	202	198	400	255
II	CAP Inlet to Grand Avenue	230	197	103	300	195
III	Grand Avenue to Bell Road	230	187	113	300	195
IV	Bell Road to Greenway Road	200	168	122	290	185
	Greenway Road to Waddell Road	179	168	122	290	185
	Waddell Road to Cactus Road	144	168	122	290	185
V	Cactus Road to Peoria Avenue	119	86	4	90	55
	Peoria Avenue to Olive Avenue	99	86	4	90	55
	Olive Avenue to Northern Avenue	82	86	4	90	55
VI	Northern Avenue to Glendale Avenue	67	47	43	90	55
	Glendale Avenue to Camelback Road	39	47	33	80	50
VII	Camelback Road to Indian School Road	25	13	61	74	45

1. Flows measured by MWD in 1993 as maximum capacities constrained by existing demands. MWD has since made improvements to the Beardsley Canal. In some instances, MWD exceeded canal freeboard constraints used for computing the estimated maximum capacity.
2. Based on six inches of freeboard.
Data summarized from Beardsley Canal Capacity Study Phase I and II by Navigant

TABLE A-13
Beardsley Canal Capacity Study
Capital Costs of Improvements to Achieve Minimum Delivery Requirements

Canal Reach	50,000 af (69 cfs)	100,000 af (138 cfs)	150,000 af (207 cfs)	200,000 af (276 cfs)	300,000 af (414 cfs)
I - Lake Pleasant to CAP Inlet	$0	$0	$934,000	$1,140,000	$12,721,000
II - CAP Inlet to Grand Ave.	$0	$13,000	$30,000	$2,785,000	$27,168,000
III - Grand Ave. to Cactus Rd.	$0	$326,000	$871,000	$1,187,000	$12,769,000
IV - Cactus Rd. to Camelback Rd.	$4,805,000	$5,314,000	$7,495,000	$7,854,000	$9,782,000
V - Camelback to Thomas Rd.	$1,697,000	$1,814,000	$2,330,000	$2,442,000	$3,084,000
Total Cost	$6,502,000	$7,467,000	$11,660,000	$15,408,000	$65,524,000

Data summarized from Beardsley Canal Capacity Study Phase I and II by Navigant

Water Treatment Plant Sizing

North WTP
The north water treatment plant for the two plant, unlimited supply option will be sized for a yearly average capacity of 57.6 MGD (64,485 acre-feet per year). With the concentration of existing water service areas in the south, the north plant will not be required until 2010. Since ultimate build out will not be needed for at least fifteen years, the infrastructure is constructed in stages that are phased in over time. In order to be able to phase in the capacity, certain portions of the plant will be constructed for ultimate capacity to accommodate expansion and addition of equipment over time. Conceptually the initial treatment plant will consist of two parallel treatment trains with the option of additional treatment trains. Ultimately, the land needed for this WTP layout is approximately 45 acres.

The notable features of the initial WTP are two parallel treatment train modules (by the year 2015) operating at a capacity of 41 MGD with a 10 MG clearwell reservoir. The planned expansion by the year 2025 will increase the capacity by an additional 17 MGD and add a second 10 MG clearwell reservoir.

South WTP

The south WTP of the two plant system will be constructed to provide service for the highest concentration of current water demands. With time, other areas will be added. See figure A-31, "South WTP Constructed First, 2005 - Demand." The initial capacity will be 45.8 MGD (51,329 acre-feet per year). The greater part of the delivery is by gravity, with an interim pump delivery system installed for delivery areas to the north. As the demands increase, the North WTP will be constructed in stages to absorb the additional capacity required and replace the interim pump deliveries with gravity delivery. See figure A-32, "Standard Two Plant Layout". Eventually the two WTPs will be networked together, which will increase the flexibility and reliability of delivering surface waters.

The 2025 capacity for the south WTP will be constructed in stages over time to ultimately deliver 79.3 MGD (88,859 acre-feet per year). Land area for the WTP layout is estimated at 59 acres.

Raw Water Reservoir - South WTP Only

A raw water reservoir is an optional recommendation for the south WTP. Acquisition of the additional land area and provisions for design in the WTP hydraulic profile are needed. This reservoir provides detention time and storage to the supply source. The capacity provides for pretreatment of the water from the canal and reserve supply for operation. Operation considerations for inclusion of this facility include the following.

- Planned maintenance outages
- Bypassing canal storm inflows
- Stabilizing differing water quality
- Storage of pumped well water
- Pretreatment of the water
- Sedimentation

The primary function is comparable to the Phoenix Union Hills WTP reservoir. Capacity and dimensions of that reservoir are18 acres of land area, a depth of 25 feet and a 145 MG raw water impoundment.

Under the terms of all CAP water service subcontracts, no user is guaranteed a certain water quality. Although it is uncertain as of this publication, the same may be true for the wheeling of CAP water through the Beardsley canal. Water delivery interruptions could occur during emergency and maintenance outages of the aqueduct upstream of the proposed turnout. Storm runoff and short-term outages are anticipated as part of start-up operations. Continued capital improvements of the Beardsley canal are being implemented at this time, which would contribute to the total reliability of delivery and water quality. The improvements would offset the design of reliability features for the WTP.

After treatment at the WTP, finished water is stored in a covered forebay reservoir. A gravity flow trunk line delivers the water to the south and east. A booster and a pumping plant is incorporated to lift the water and deliver via a distribution pipeline to higher elevations, and/or floating reservoir and into pressure zones close to the plant.

The yearly average capacity of the south WTP in year 2025 is 79.3 MGD. The south plant carries all deliveries until the North WTP is functional in 2015. This will require some interim pumping to service areas until those areas are converted to gravity deliveries. The notable features of the initial plant will consist of two parallel treatment train modules in the year 2005 with a capacity of 52 MGD and a 10 MGD clearwell reservoir. Future expansion of the plant will occur by 2025 to include an additional 31 MGD of treatment capacity and a second 10 MG clearwell reservoir. Note that the balance of the system reserve capacity is retained in floating reservoirs as part of the distribution system.

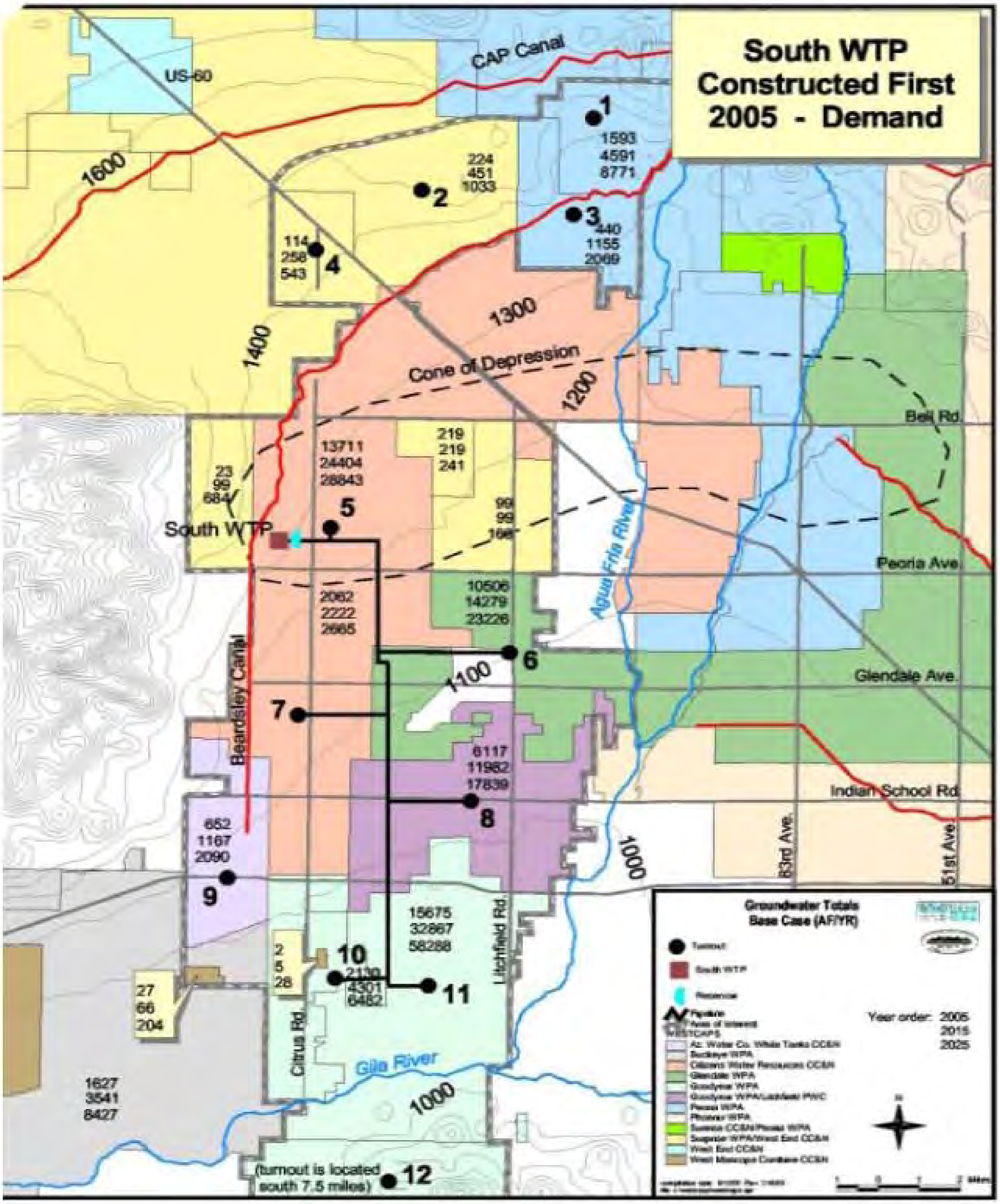
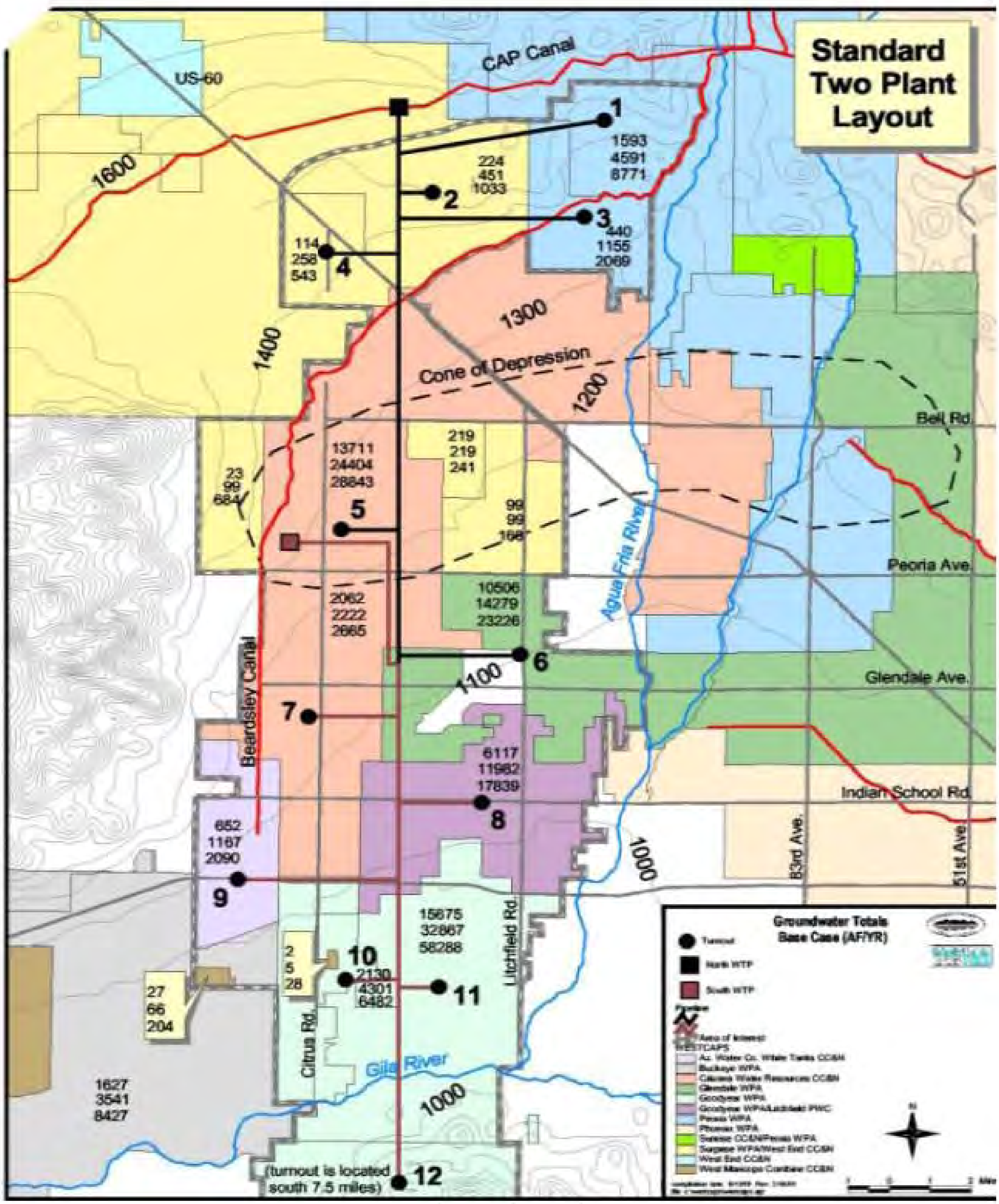

South WTP
Constructed First
2005 - Demand
CAP Canal
US-60
1600
1400
1300
1200
1100
1000
Cone of Depression
1
1593
4591
8771
2
224
451
1033
3
440
1155
2069
4
114
258
543
5
13711
24404
28843
219
219
241
23
99
684
99
99
168
South WTP
2062
2222
2665
10506
14279
23226
6
7
6117
11982
17839
8
652
1167
2090
9
15675
32867
58288
10
2130
4301
6482
2
5
28
11
27
66
204
1627
3541
8427
(turnout is located south 7.5 miles)
12
Beardsley Canal
Agua Fria River
Gila River
Citrus Rd.
Litchfield Rd.
83rd Ave.
51st Ave.
Bell Rd.
Peoria Ave.
Glendale Ave.
Indian School Rd.
Groundwater Totals
Base Case (AF/YR)
Year order: 2005
2015
2025

FIGURE A-31


Standard
Two Plant
Layout
CAP Canal
US-60
1600
1400
1300
1200
1100
1000
Cone of Depression
Bell Rd.
Peoria Ave.
Glendale Ave.
Indian School Rd.
83rd Ave.
51st Ave.
Litchfield Rd.
Citrus Rd.
Beardsley Canal
Agua Fria River
Gila River
1
1593
4591
8771
2
224
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2069
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13711
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15675
32867
58288
10
2
5
28
2130
4301
6482
11
27
66
204
1627
3541
8427
(turnout is located
south 7.5 miles)
12
Groundwater Totals
Base Case (AF/YR)
Turnout
North WTP
South WTP

FIGURE A-32

Hydraulic Profile

The hydraulic profile for both plants are identical, except for the planned raw water reservoir for the south plant. A maximum hydraulic head loss of 19 feet is allowed between the entrance to the screens and the finished water reservoir. Figure A-9 shows typical profiles of the original ground surface for both sites (elevations are for the north WTP, subtract 200 feet for the approximate south WTP elevations), the hydraulic profile of the WTP, and the excavation profile for the centerline of the WTP. The use of low head, high volume pumping can be used instead of a gravity hydraulic profile to position the raw water reservoir and WTP in relation to the canal water surface.

Pumping Plant

North WTP (CAP)

The pumping plants planned for the northern area, as part of the two plant system, is identical to the treatment plant designed for the "One North Plant system". The pumping requirement also remains the same.

South WTP (Beardsley)

The south WTP will require three additional pump stations to meet service pressures for delivery to Citizens Agua Fria #2, Arizona Water Company White Tanks, and Goodyear #4. The Citizens Agua Fria #2 pumps are temporary and water service would be changed to gravity service when the North WTP comes on line and the pipelines are networked. Water service to Arizona Water Company White Tanks and Goodyear #4 will continue to be pumped service. The economics of pumping relatively small amounts of water through a length of pipe (pipe costs) costs less to deliver from the south WTP than delivery from the North WTP by gravity.

Design Criteria

With two WTP's providing the regional supply, the size of the trunkline pipes are decreased and the distance potable water is transported is minimized. However, the higher elevation areas in the southern WTP service area may require additional booster pumping. Keeping design criteria constant, the two plant system saves 6% of capital costs through less transportation and slightly more pumping over the single plant system (see cost estimate sheets). The criterion for design only considers the capital costs of pumps, power and pipe sizing for comparison of different configurations. The future costs of energy and capital replacement are not clearly defined for this study.

Pumping Plant Layout

Each of the pumping plant layouts are described in the prior section for the single "North WTP Configuration.

Gravity System

The gravity delivery system criteria are similar to the single "North WTP Configuration" criteria.

Conveyance and Distribution System

The conveyance and distribution system is the same as the single "North WTP Configuration" with the following exception.

Alignment

An appraisal-level study was conducted for delivering water to the study area. Twelve delivery points (laterals and turnouts) are planned along the distribution system. These twelve points vary in distances, from zero to 26 miles, from the WTP.

The south WTP and distribution system is planned first for construction. See Figure A-31, "South WTP Constructed First". The pipeline alignment originates at the Beardsley canal WTP clearwell, elevation 1300, with a trunk line to the east and to the south. Distribution lines from this plant service the WPA's of Glendale-Out of Service, Citizens Agua Fria and Goodyear. The main pipeline alignment follows a standard layout that is used in all the configuration studies. The balance of the laterals and trunk lines are constructed as the demand is required. Initially, the Citizens Agua Fria area would be high lift pumped, but after the North WTP is constructed, the pumped deliveries can be converted to gravity and the high lift pumps used for system emergencies. The south system will incorporate a floating reservoir at an elevation to provide reserve capacity beyond what is to be provided as part of the clearwell reservoirs. From the clearwell, the trunk line continues to the south. Other laterals to the east and west from the trunk line service the centroid of the WPA's identified. South of the Gila River, booster pumping of the pipe lateral system is anticipated due to the low delivery capacity required and the cost of pumping compared to increased pipe sizing.

The northern portion of the main trunkline will be constructed after year 2010. The pipeline construction begins at the CAP WTP clearwell reservoir at elevation 1500. From the clearwell, the pipeline continues south, paralleling the Sarival Road alignment for 7.5 miles to Bell Road, situated at elevation 1300. See Figure A-32. As water deliveries are made off of the main gravity fed trunkline the size of the pipe is reduced incrementally. The alignment is planned to connect to the existing laterals for Citizens Agua Fria and Glendale Out of Service, then connects to the existing southern distribution system from the Beardsley WTP.

After the north and south WTP and pipelines are installed, the operation of the combined systems will differ from the individual systems. Initially, three pipelines will transport water from the south WTP clearwell.

One pipeline connects a clearwell reservoir pump system to a floating reservoir at elevation 1400 and delivers that water to the west of the WTP up the slope of the White Tank Mountain Range. That reservoir will primarily be filled under normal operating conditions by gravity service from the north WTP.

The second distribution line will interconnect with the Citizens Agua Fria area by booster pumps that are serviced in the future by the North WTP.

The third pipeline will deliver water by gravity to the main trunkline oriented north-south to feed the WPA's to the east and south of the South WTP.

Selection Considerations for Pipes

There are other items considered when establishing design criteria for comparison of configurations. The staged construction and capacity of pipe installations are a factor when implementing infrastructure over the long term. Several design criteria become important when considering variable operating conditions.

A variable cost for the installation of the pipeline system can be anticipated as the land is developed concurrently with pipe installations. See table of pipe costs and additional costs for congested areas and developed areas in the exhibits attached to the end of Appendix A.

A velocity of 5 feet per second was used to compare all pipes in this study. This simplifies the selection of various pipe sizes and pipe materials in an appraisal level study. Note that larger pipes have less friction loss. One large pipe has less friction loss than two smaller pipes of equal capacity. By understanding this engineering principle, consideration is given to sizes greater than approximately 36" pipe diameter because of the operating benefit over time compared with the cost of the pipe itself. An even greater consideration is given to this principle when factoring in population growth over time.

The above considerations do not affect the pipes for this study as much as the pipe sizing will become a consideration of the "limited supply" presented later in this appendix (see the table and graph in the exhibit).

Booster Plants

Same criteria as are discussed in the one North WTP configuration applies. Booster plants are required to deliver water from the trunkline to some of the turnouts located along the distribution line. For the two plant system, boosters will be required for the Arizona Water Company and White Tanks area. The Goodyear #4 area will require an intermediate high head booster pump as part of the lateral.

Turnout areas requiring intermittent or full time pumping to meet pressure delivery are shown in the detailed hydraulics of the two-treatment plant system. The summary is detailed in Table A-14. The intermittent pumping is shown in normal type and the full time pumping is shown in bold type.

Table A-14
Peak Capacity and Total Dynamic Head for the Two WTP System

Turnout Number	WPA Description	Peaked Capacity (CFS)	Total Dynamic Head Pumped (Feet)
1	Peoria #5	18.2	**176**
2	Surprise #3	2.1	**145**
3	Peoria #6	4.3	41
4	Surprise #8	1.1	40
5	Citizens Agua Fria	59.8*	90*
7	Citizens Agua Fria #2	5.5	63
9	Arizona Water Co. W.T.	4.3	8
12	Goodyear #4	3.1	**87**

Note: For this water delivery system, the delivery head for turnouts 1, 2 and 12 are the only deliveries that must have pumping to make deliveries to those areas. Turnout areas 3, 4, 7 and 9 are considered occasional pumping to meet full (peak) demand capacity.
* Turnout #5, Citizens Agua Fria will be converted from south WTP pumped delivery to North WTP gravity delivery in year 2015.

Operations

The design flow rate of the North WTP and the South WTP are 57.6 and 79.3 MGD, respectively (North 64,485, south 88,859 AFY. Total annual capacity of 153,344 acre-feet per year.) The maximum flow rate is 86.4 MGD for the north and 118.9 MGD for the south, (134 cfs, 184 cfs, design capacity of 1.5 times annual allocation) was used for the hydraulic computations in the main pipeline section only. The system components for each delivery would include a tee, an upstream shut off valve, a meter, a pressure reducing valve (if needed), and a downstream shut off valve. All other components are as previously discussed in the One North WTP.

System Storage

Criterion for storage is similar to what was previously discussed in the One North WTP. The total storage capacity of the system is divided among multiple planned sites. The storage provided will supply an average daily demand for emergency use by the water providers. The average daily delivery capacity designed into this system is about 100 million gallons. A clearwell reservoir is designed for each WTP site. The location of the reservoirs are separate from the water treatment plants and are designed in areas adjacent to the slopes of the White Tank Mountains, outside the area of the White Tank Regional Park boundaries.

Water Providers' Turnouts

These turnouts are the same size as planned for the One North WTP, but the sequence of construction and whether pumping is required in the early development years is dependent on the available head. See figure A-32 for turnout location.

The water providers' turnouts are sized for twice the anticipated CAP delivery, and are shown in Table A-15 below.

Table A-15
Turnout Descriptions - Two WTP's - Unlimited Supply

	2025 DEMAND			
TURN OUT #	LOCATION DESCRIPTION	TURNOUT CAPACITY		GROUND ELEV.
		MGD	cfs	feet
	Service Thru North WTP			
1	Peoria #5		18.2	1500
2	Surprise #3		2.1	1500
3	Peoria #6		4.3	1360
4	Surprise #8		1.1	1410

	Service Initially Thru the South WTP and Transfers to the North WTP			
5	Citizens Agua Fria		59.8	1250
6	Glendale Out of Service		48.1	1100
	Service Thru South WTP			
7	Citizens Agua Fria #2		5.5	1150
8	LPSCO		37.0	1030
9	Az. Water Co. White Tanks		13.4	1060
10	Goodyear Outside		4.3	980
11	Goodyear #2		120.8	970
12	Goodyear #4		3.1	1100

Cost Summary

The cost summary follows the same criteria as discussed in the one North WTP. The differences in the configuration for the two WTP layout (costs and design) criteria are discussed below.

Operations and Maintenance Costs

Operations and maintenance costs are based on delivering from the north and south WTP’s, along the main trunkline distribution system, a total of 64,485 and 88,859 acre-feet per year of treated water respectively. General maintenance and contingency funding for repairs is included in the annual operating costs.

Additional costs for operating the two WTP's is included in the cost comparison.

Additional costs of transporting CAP water through the Beardsley Canal, MWD system, have not been included in the cost estimates.

An additional 16,000-acre foot allocation of MWD water rights from Lake Pleasant can be used in this study. The cost of this water and its use through the Beardsley canal or the CAP canal is unknown at this time.

Right-of-Way Cost

The land area anticipated to be needed for the North WTP facility is the same as discussed in the one north WTP section. The land area anticipated to be needed for the facilities is primarily State, county, and some private land. An estimate of $10,000 per acre is used for the land in this area. A Reclamation land cost survey was not performed.

The cost of the land needed for the south WTP on the Beardsley canal will most likely be privately owned, and land owned by MWD adjacent to the Beardsley Canal. An estimate of $10,000 per acre is applied based on a cost survey of the area and may or may not reflect actual increases in land value anticipated for this area. A Reclamation land cost survey was not performed and the estimate may or may not reflect actual price increases anticipated for this area.

Capital Costs

The total capital costs for the two water treatment plant design layout is $429,713,000 with a cost per 1,000 gallons of $1.32. Note that the cost of the optional raw water reservoir for the south WTP is not included and would add $0.035 per 1,000 gallons to the $1.32 cost per 1,000 gallons.

CHAPTER V

The Limited Supply Configuration
One Treatment Plant (along CAP canal system)

Water Treatment Plant Site Description

CAP Canal WTP (north WTP)
This configuration is almost identical to the Single Plant Configuration, as previously shown in the "Unlimited Supply" section. The exception is that less land area is required and the need for future land expansions is not considered. This severely limits the future development of this WTP site since it is anticipated that surrounding housing and infrastructure developments will occur in the future.

Water Treatment Plant Sizing

The single water treatment plant, for the limited supply option, will be sized for a yearly average capacity of 65,681 acre-feet per year. The constructed plant will consist of 3 parallel treatment trains with no future expansions. Plant buildout and maximum delivery capacity is reached prior to the year 2010. Therefore, the design consideration is for full production by 2010. It should be noted that Peoria, Surprise and Citizens Agua Fria own the majority of the CAP allocations in the region. The balance of the service areas, mostly the southern area, does not have sufficient surface water allocations that would provide for growth demands to the year 2010. The WTP is sized for the total CAP water allocations available and the assignment of ownership has not been made. See the turnout capacity distributions for the Limited Supply, years 2005, 2010, 2015 and 2025.

Hydraulic Profile

The maximum hydraulic head loss allowed is 19 feet, which is standard when compared to other configurations for constructed WTP facilities.

Pumping Plant

The criteria for pumping plant design and layout are nearly identical as is illustrated in the previous section for the One North WTP. The notable differences are the size of the pumps and the hydraulic heads developed for proper delivery. One difference to note is the Goodyear #4 area does not require hydraulic pumping. A gravity system provides water more economically with a pipe of a specific size assuming a specific delivery capacity versus long term pumping.

Gravity System

The gravity delivery system criteria are also the same as in the single north WTP except with the addition of the Goodyear #4 can now be delivered by gravity instead of booster pumping. Design allows for the conversion to gravity by the conserved head, smaller diameter pipes, and higher head class that can economically be used in this single plant, smaller capacity system.

Conveyance and Distribution System

Similar criteria used from the Unlimited, One North WTP layouts is used to determine pipeline sizing and configuration for the Limited supply configuration. The one noted difference is that the pipelines and appurtenances will have no future available capacity. The transportation cost of water per volume unit mile is expected will be higher than previous configurations.

Alignment

Similar design criteria used for the Unlimited Supply, is applied for the Limited Supply layout.

Selection criteria for pipes are discussed in the exhibits at the end of the appendix. The guidelines provided in the exhibit on sizing and capacity also apply to the Two WTP and the smaller design configurations, which follows this section. The pipe sizing/capacity details are not included in this appendix report, but are presented only for information.

Booster Plants

Booster plants will deliver the water to some of the turnouts located along the distribution line, similar to the single North WTP layout, but with smaller pumping equipment and variable water pressure and flow characteristics.

Water provided to areas south of the Gila River is gravity fed for this configuration. Through engineering analysis, a gravity system is possible because of the small capacity of the trunk line, thus allowing higher available pressure with little increase in pipe costs.

In order to deliver water is some turnout areas, pumping is needed. The details of this pumping is shown in the following table, Table A-16, Peak Capacity and Total Dynamic Head for the One WTP System. Details of the hydraulics of the one treatment plant system are shown in the exhibit at the end of Appendix A.

Table A-16
Peak Capacity and Total Dynamic Head for the One WTP System

Turnout Number	WPA Description	Peaked Capacity (CFS)	Total Dynamic Head Pumped (Feet)
1	Peoria #5	12.3	**169**
2	Surprise #3	4.9	**141**
3	Peoria #6	2.9	71
4	Surprise #8	2.6	75
12	Goodyear #4	0.8	No pumping Required

Note: For this CAP water treatment plant delivery system, the delivery head for turnouts 1 and 2 are the only deliveries that ***must*** have pumping to make any deliveries to those areas. Turnout areas 3 and 4 are considered occasional pumping to meet full (peak) demand capacity. Turnout 12 is less capacity in this configuration, and is gravity delivery. Details and cost of pump stations and pumping can be found in the Hydraulics and Cost Summary tables at the end of this section

Operations

The operation of the Unlimited One North WTP previously discussed is similar to the One North WTP with Limited Supply. However, the Limited One North WTP will operate at a flow rate of 58.6 MGD (65,680 acre-feet per year) and a maximum flow rate of 88 MGD. The distribution system can peak at 95,521 AFY (136 cfs, 1.5x design capacity). This distribution capacity was applied toward the design of the main pipeline section.

System Storage

The system storage is designed similarly to the Unlimited Supply configuration, except that the Limited Supply Configuration requires less storage, which is in relation to the design delivery of the system. The system storage for the "Unlimited" storage is 100 million gallons. The system storage for the Limited configuration is 45 million gallons.

Water Providers' Turnouts

The turnout locations and descriptions are the same as the Unlimited Supply, One North WTP, except the turnouts vary in size due to a limited water supply using this approach. Deliveries for each area are from current CAP allocations for each WPA.

Table A-17
Turnout and Capacity Data, One WTP

	SUPPLY AVAILABLE ONE WTP			
TURN OUT #	LOCATION DESCRIPTION	TURNOUT CAPACITY		GROUND ELEV.
		MGD	cfs	feet
1	Peoria #5		12.3	1500
2	Surprise #3		4.9	1500
3	Peoria #6		2.9	1360
4	Surprise #8		2.6	1410
5	Citizens Agua Fria		51.6	1250
6	Glendale Out of Service		6.3	1100
7	Citizens Agua Fria #2		4.8	1150
8	LPSCO		10.0	1030
9	Az. Water Co. White Tanks		3.8	1060
10	Goodyear Outside		3.6	980
11	Goodyear #2		32.5	970
12	Goodyear #4		0.8	1100

Cost Summary

This summary is similar to the previously discussed section entitled One Water Treatment plant.

Operations and Maintenance Costs

For operations and maintenance, the criteria of the cost summary for the Limited, One North WTP is similar to the One North WTP, Unlimited Supply configuration. However, operations and maintenance costs are based on delivering 65,680 acre-feet of treated water per year. Routine maintenance and contingency funding for the repairs is included in the annual operating costs.

The layout envisioned for the WTP does not provide for future expansion or improvements of the treatment process or distribution conveyance system. For example, the WTP and pipelines would have to be taken out of service to make any improvements to the treatment train or increase capacity due to constraints imposed by the original design criteria.

Right-of-Way Cost

Ownership of lands and the costs associated with land ownership are discussed in detail in the previous Unlimited Supply, One North WTP configuration. Because of a Limited Supply configuration for a regional solution, less land area is required for the Limited North WTP. Expansions were not factored into the future for the Limited Supply configuration, nor for pipeline right of ways. Additionally, land requirements are less intensive for tanks, booster stations and reservoirs.

Capital Costs

The total capital cost for the One Limited Supply WTP, for local area delivery, is estimated at $219,706,000. Treatment and delivery costs are $1.47 per 1,000 gallons.

The Limited Supply Configuration
Two Treatment Plants (along CAP Canal System and Beardsley Canal)

Water Treatment Plant Site Descriptions

The Two WTP configuration provides treated water for the WSRV from two separate locations. Both WTP's are undersized for water deliveries to their areas after the year 2010. Both the WTP's are smaller than the One WTP configuration.

CAP
The CAP location is situated in the area of the CAP canal and Sarival Road. The North WTP site description is similar to the Unlimited Supply Configuration of the Two WTP's, as previously discussed. The area for the facility is smaller, and the treatment trains are fewer than the single North WTP configuration. Provisions for future plant expansions are not provided.

Beardsley Canal

The Beardsley Canal WTP location is approximately in the area of the Beardsley Canal and Cactus Road. Provisions for future plant expansions are not provided. The area for the facility is smaller, and the treatment trains are fewer than the Unlimited Supply for Two WTP configuration.

The capacities of the Beardsley Canal, for the reaches described, are shown in Table A-7. The costs for providing additional capacities to the system are shown in Table A-8. The cost of increasing the canal capacity to support a smaller WTP along the Beardsley Canal is less costly for this configuration. Note that the costs of these improvements are not included in the cost comparisons at the end of chapter III of the report, or in this Appendix A cost summary.

Water Treatment Plant Sizing

North WTP
The North WTP for the Two WTP, Limited Supply configuration is sized for a yearly capacity of 34.7 MGD (38,909 acre-feet per year). The concentration of existing service areas are to the east and south. New service areas to the north and west would require pumping to service those areas. The capacity of this plant will be utilized quickly after it is constructed since the current CAP allocations are mainly held by Peoria, Surprise and Citizens. The treatment plant would be constructed by the year 2005 and consist of two 17.5 MGD parallel modules for water treatment. The land area required for the plant is approximately 29 acres.

South WTP

The south WTP of the Two WTP configuration is constructed to service the areas to the east and south of its planned location as gravity delivery. The anticipated less demand areas of the west and the north will have water delivered to them through booster pumping. The CAP allocation owners of this plant currently do not have CAP allocations sufficient to provide for expected demand. Using current projections the supply capacity of 23.9 MGD (26,771 acre-feet per year) is expected will be exceeded by the year 2010. The plant is expected to be constructed by the year 2005 and consist of two 12 MGD parallel modules for water treatment. The land area required for the plant is approximately 21 acres.

It should be noted that the design for both the north and south WTP's for this configuration and sizing are of a capacity to allow for the consideration of designs using ultra filtration modular facilities. Also, the land area required for an ultra filtration plant is less than the land area required for a conventional filtration plant. However, when factoring in construction costs and land costs, both systems are similar on a total cost basis.

Raw Water Reservoir - South WTP

The raw water reservoir is the same as described as in the previous Two WTP configuration, Unlimited Supply section. The reduced land area required for the reservoir is estimated at six surface acres in size. The depth would remain at 25 feet, as previously discussed. The volume capacity of the raw water would be proportionally reduced to 50 MG to match the WTP capacity.

Hydraulic Profile

A maximum hydraulic head loss profile of 19 feet is allowed between the entrance to the screens and the finished water reservoir. Figure A-3 shows the profiles of the original ground surface for the proposed site, the hydraulic profile of the WTP, and the excavation profile for the centerline of the WTP.

Pumping Plant

The pumping plant requirement for the North plant as part of the two plant system is also a requirement in the Unlimited Two WTP configuration. The pumping capacities and pipeline sizing will be reduced for this Limited Two WTP configuration compared to the single North WTP or the Unlimited Two WTP configuration. The south WTP pumping requirement remains unchanged, but it is worth noting that the pumping head required for the Goodyear #4 delivery system increases. The small pipe size required to transport the smaller delivery capacity increases the pumping head required to make the delivery.

The Design Criteria and the Pumping Plant Layout are identical, except for the smaller pumping units.

Gravity System

The gravity delivery system sizing and pressure criteria are also the same as presented in the Unlimited Supply, Two WTP configuration.

Conveyance and Distribution System

The same criteria used to size the Two WTP layout, Unlimited Supply, is used to determine pipeline sizing and appurtenances for this Limited Supply configuration. The difference is that the pipelines and features are installed with no future available capacity. The cost for transportation of water per volume unit mile for the Two WTP layout, Limited Supply, will be the highest of the configurations studied.

Alignment

The pipe alignment is identical as discussed in sections above, except that the following economic observation should be noted. The connection of the 7 miles of 48 inch diameter connecting pipeline between the north and south distribution system (length of trunk pipeline between turnouts #4 and #5) is not cost effective. The decreased capacity of this configuration and the distance between major service delivery areas does not justify the interconnection pipeline to be installed to provide for network reliability or flexibility for delivery. Providing two WTP's increases the reliability to the region. The cost of two WTP's therefore does not justify the cost of interconnecting pipelines for this limited supply. The two plant system for it's configured capacities are feasible as a limited area water service (sub-regional), but are not effective as combined regional plants as sized. The use of existing well water to provide flexibility and supply becomes a greater factor for operations of the system if the interconnecting pipeline is not installed.

The system will be analyzed as follows to be consistent with the prior configurations. Applying the original design criteria, the layout of the northern alignment will service turnouts 1 though 6. The alignment originates from the CAP WTP clearwell reservoir elevation 1500. From the clearwell, the pipeline continues south, paralleling the Sarival Road alignment for 7.5 miles to Bell Road, situated at elevation 1300. As the main trunkline delivers water southward by gravity, deliveries are made to major laterals and service areas. As these deliveries are made, the size of the pipe is reduced incrementally. The alignment will connect to existing laterals belonging to Citizens Agua Fria and Glendale Out of Service, then connects to the existing Southern distribution system from the Beardsley WTP.

The two WTP systems are situated to make deliveries with the maximum available pressure, accomplished by using gravity. Booster pumping is required for the rest of deliveries. A storage reservoir is installed to store the North and South WTP reserves at elevations 1400.

Booster Plants

Booster plants will be required to deliver the water from the distribution to the turnouts located along the distribution line.

The same criteria discussed in the Two WTP configuration, Unlimited Supply, applies to the Limited Supply option, except there will be no conversion of pump to gravity delivery since both WTP's will be constructed at the same time. Booster plants will be required to deliver water from the distribution to the turnouts located along the distribution line. For the two plant system, boosters will be required for the Arizona Water Company and White Tanks area. The Goodyear #4 area will require a high pressure booster pump inline of their lateral.

Turnout areas requiring pumping at intermittent times are shown in the detailed hydraulics of the two-treatment plant system. The summary is shown below in Table A-18.

Table A-18
Peak Capacity and Total Dynamic Head for the Two WTP System

Turnout Number	WPA Description	Peaked Capacity (CFS)	Total Dynamic Head Pumped (Feet)
1	Peoria #5	12.3	**178**
2	Surprise #3	4.9	**149**
3	Peoria #6	2.9	46
4	Surprise #8	2.6	47
7	Citizens Agua Fria #2	4.8	63
9	Arizona Water Co. W.T.	3.8	1
12	Goodyear #4	0.8	**291**

Note: For this water delivery system, the delivery head for turnouts 1, 2 and 12 are the only deliveries that must have pumping to make deliveries to those areas. Turnout areas 3, 4, 7 and 9 are considered occasional pumping to meet full (peak) demand capacity.

Operations

The design flow rate of the North WTP and the South WTP is 34.7 MGD and 23.9 MGD, respectively (the north delivers 58,363 AFY, and the south delivers 40,156 AFY). The maximum flow rate for the North plant is 52.1 MGD and the South plant is 35.8 MGD (80.6 cfs, 55.5 cfs, peaked at 1.5 times the annual average allocation or a maximum overload rate of the plants is 50%). The design concept was applied toward the hydraulic computations for the main pipeline section only. The system components for each lateral delivery include a tee, an upstream shut off valve, a meter, a pressure reducing valve (if needed), and a downstream shut off valve.

System Storage

The system storage is designed similarly to the Two WTP, Unlimited Supply configuration, except that the Limited Supply Configuration requires less storage, which is in relation to the design delivery of the system. The system storage for the "Unlimited" storage is 104 million gallons. The system storage for the Limited configuration is 34.2 million gallons.

Water Providers' Turnouts

The turnouts and sizing for the Two WTP layout, Limited Supply, is identical to the turnouts for the single North WTP configuration, but the sequence and whether pumping is required in the early or later development years is dependent on the available system delivery pressure. See figure A-1 for turnout location. The water providers' turnouts are sized for twice the anticipated CAP delivery. In Table A-19 below the turnout capacity is provided for the Two WTP layout.

Table A-19
Turnout Descriptions - Two WTP's - Limited Supply

	SUPPLY AVAILABLE TWO WTPs			
TURN OUT #	LOCATION DESCRIPTION	TURNOUT CAPACITY		GROUND ELEV.
		MGD	cfs	Feet
	Service Thru North WTP			
1	Peoria #5		12.3	1500
2	Surprise #3		4.9	1500
3	Peoria #6		2.9	1360
4	Surprise #8		2.6	1410
5	Citizens Agua Fria		51.6	1250
6	Glendale Out of Service		6.3	1100
	Service Thru South WTP			
7	Citizens Agua Fria #2		4.8	1150
8	LPSCO		10.0	1030
9	Az. Water Co. White Tanks		3.8	1060
10	Goodyear Outside		3.6	990
11	Goodyear #2		32.5	970
12	Goodyear #4		0.8	1100

Cost Summary

The following is a summary of the costs for the Two WTP configuration.

Operations and Maintenance Costs

Operations and maintenance costs are based on North and South WTP water deliveries, 38,909 and 26,771 acre-feet of treated water per year, respectively. Routine maintenance and contingency funding for the repairs is included in the annual operating costs.

The cost of CAP water is expected to continue to vary over time. A value of $150 per acre-foot has been selected to calculate representative costs that will be used to compare alternatives.

The additional cost of transporting (wheeling) CAP water through the Beardsley Canal has not been determined as part of this report and is not included in the cost analysis or summary.

Additionally, the issue of acquiring incremental water supplies from other allocation owners is a cost that can be provided as part of future calculations.

There is also a 16,000 acre foot allocation of Beardsley water that can be applied towards water purchases. The cost of this water and its use through the Beardsley canal is unknown at this time.

The alternative assumes that the pumping cost is borne by project sponsors using a rate of 60 mills per kWh, which represents the rate for interruptible power for large industrial users.

Right-of-Way Cost

Land ownerships and the costs associated with land development are similar to what has previously been discussed. The land area required for the south WTP facilities is primarily MWD owned. The land needed for the distribution system is primarily county and private land. Less land area is required for the Two WTP configuration, since expansions have not been planned for the land or facilities. The planning for the smaller two plant system does not depreciably reduce the size of the pipeline right of ways. Land areas for facilities, such as tanks and reservoirs, will also not depreciably be reduced in size.

An estimate of $10,000 per acre is estimated as the land value, which is discussed in more detail in previous sections. Some thought should be given to the acquisition of the surrounding area to allow for future WTP expansion, recreational opportunities, possible flood control benefits, and environmental mitigation. However, regardless of land that may be purchased in the future, it is worth noting that the configuration, as presented, does not allow for future plant expansion.

Capital Costs

The total capital costs for the Two WTP, Limited Supply layout for the area of delivery is $220,580,000 with a cost per 1,000 gallons of $1.52. It should be noted that the cost of the optional raw water reservoir for the south WTP is not included and would add $0.043 per 1,000 gallons.

CHAPTER VI

Summarization of Appendix A Report

Preferred Configuration

The preferred option is the Two WTP, Unlimited Supply configuration. For purposes of this report the configuration is planned though the year 2025. However, the benefits of the layout and design extend beyond the year 2025. By making provisions for future plant expansions to meet projected growth, the benefits from the costs of construction are greater than the implementation of the other configurations. The other configurations include the two "Limited Supply" layouts that are facilities initially constructed with the maximum delivery capacity but with little expansion capability in the future. The other configuration is the Single North WTP, "Unlimited Supply", which would be constrained by the single water production location and the high initial cost of construction because of the large pipe size and length of the main trunk line.

In general, the configuration, which provides the greatest benefit, is that which specifies the larger sizes for the WTP's. Two large plants best represent the reliability, flexibility, reserved capacity, and full utilization of each portion of the expanded facilities. Though the cost is greater than the one large north plant configuration, the design and operational saving of having two plants offsets the initial costs.

Although the cost of the pipe distribution system is proportional to the distance the water is transported and the size of pipe used, a larger size pipes requires less pumping cost for similar water delivery due to friction costs with a smaller size pipe. However, one of the benefits of this layout is that the pump operational costs are minimal because gravity is used to provide service pressure. The larger the WTP and the larger the pipes, the greater the economic benefits for effort of construction, operation and maintenance. This is shown in the cost per 1,000 gallons summary. Additional advantages of larger pipes are reliability, future flexibility, reserved capacity, and the extent of regional water service coverage.

What this report provides is the technical background for how a configuration was selected. But reliability is a factor that can only be implemented by policy.

And system flexibility can only be recognized as a benefit by the owners of the system. Flexibility can provide the ability to utilize facilities beyond the design service life and design capacity, and reserve or conserve assets such as automation, system pressure, alternative water sources, and reserved water storage. The two large plant system, when connected, provides these capabilities by constructing seven miles of trunk line that connect the two facilities.

In final, this configuration is designed so that utilizing gravity pressure minimizes operation costs. The wonder of the location of the CAP and Beardsley canal is that the majority of the service areas lie downhill of these surface water sources. One of the benefits of this configuration is minimizing the dependence on power consumption. Projected power supply is unknown, and this report cannot predict the ability to contract a firm fixed power supply.

Table A-20

WATER TREATMENT PLANTS AND DISTRIBUTION PIPELINE SYSTEM TOTAL ANNUAL COST ($)

SUMMARY Description	Annualized capital	O&M	Total annual Cost	Cost per acre-foot	Cost per 1,000 gallons
Unlimited Supply Capacity, One Treatment Plant and pipes **	$36,044,618	$29,194,503	$65,239,121	$425.44	$1.306
Unlimited Supply Capacity, Two Treatment Plants and pipes *	$35,966,967	$30,026,879	$65,993,846	$430.36	$1.321
Limited Supply Capacity, One Treatment Plant and pipes **	$18,389,362	$13,172,556	$31,561,918	$480.54	$1.475
Limited Supply Capacity, Two Treatment Plants and pipes **	$18,462,514	$13,925,528	$32,388,041	$493.12	$1.513

* Shown in following tables is the summary cost breakdown of the "Unlimited Supply Capacity, Two WTP" configuration.
* *See exhibits for summary cost breakdown of these configurations.

Table A-21

WATER TREATEMENT PLANTS Description	Annualized capital	O&M	Total annual Cost	Cost per acre-foot	Cost per 1,000 gallons
Unlimited Supply Capacity, North Treatment Plant	$8,960,055	$2,601,118	$11,561,173	$179.28	$0.550
Unlimited Supply Capacity, South Treatment Plant	$12,414,804	$3,336,266	$15,751,070	$177.26	$0.544

Table A-22

PIPELINES - TWO WTP SYSTEM Description	Annualized capital	O&M	Total annual Cost	Cost per acre-foot	Cost per 1,000 gallons
Unlimited Supply Capacity	$14,592,108	$1,087,895	$15,680,003	$102.25	$0.314

Table A-23

COST OF CAP WATER Description	Annualized capital	O&M	Total annual Cost	Cost per acre-foot	Cost per 1,000 gallons
Unlimited Supply Capacity			$23,001,600	$150	$0.460

Table A-24

OPTIONAL (not included in above totals)

South WTP only	RAW WATER RESERVOIR Total Annual Cost ($)				
Description	Annualized capital	O&M	Total annual Cost	Cost per acre-foot	cost per 1,000 gals
Unlimited Supply Capacity South WTP	$992,515	$29,645	$1,022,160	$11.50	$0.035
Limited Supply Capacity South WTP	$365,602	$10,920	$376,522	$14.06	$0.043

CHAPTER VII

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