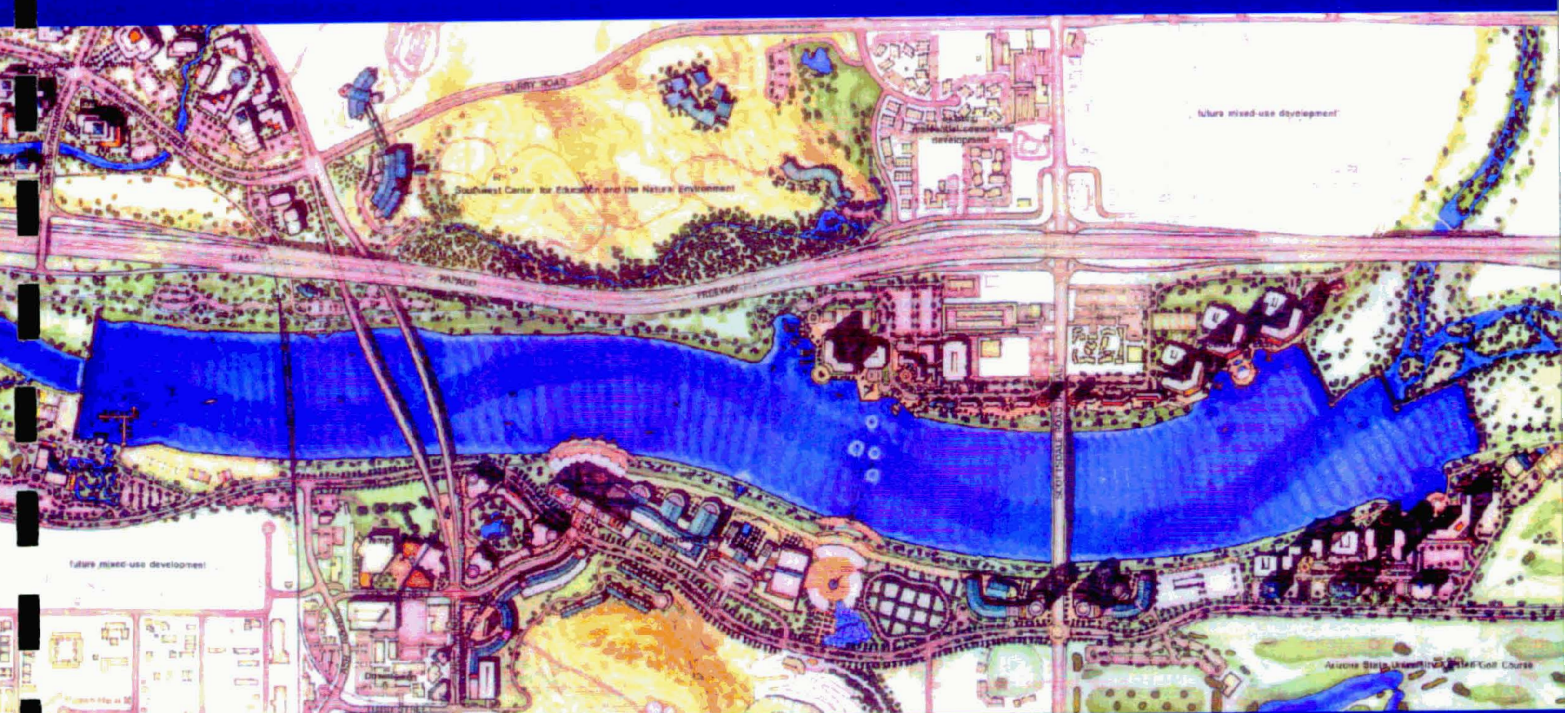
Rio Salado Engineering Report



Tempe

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City of Tempe

Rio Salado

Engineering Report

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Executive Summary

This Engineering Report represents the findings of a one-year study of the engineering feasibility of creating a Town Lake as part of the Rio Salado project. The report is intended to provide City decisionmakers with information regarding Town Lake and alternatives for lake water supply in a format that complements information being generated by the City regarding overall water resource management planning within the City of Tempe. As such, this report makes no recommendations regarding water supply. However, four alternative conceptual plans based on the primary sources of water supply are presented in Section 8. This report includes the following information:

- Alternative methods of lake construction. These methods are primarily alternative approaches for controlling seepage losses to minimize impacts on the existing hydrogeology of the project area (Section 4).
- Alternative projects for protecting the lake from low quality stormwater runoff (Section 4).
- Alternative approaches for supplying the lake with water (Sections 5 and 6).
- Techniques for managing the lake's water quality (Section 7).

Eight technical memorandums were prepared in earlier phases of this investigation of engineering feasibility that included the following topics:

- Alternative types of dam structures.
- Alternative lake locations and lake sizes.
- Other technical information regarding Salt River hydraulics, lake water quality, and permitting.

During this investigation, the Rio Salado Technical Committee, City staff, and City Council provided direction to the project team. Key direction included:

- Beneficial uses. Town Lake will be used primarily as a boating lake of sufficient water quality to be permitted for partial body contact. Consideration may be given to fishing on the basis of catch-and-release or for food consumption, but overall fishing is not currently a high priority beneficial use. Swimming in Town Lake will not be allowed due to safety considerations and the high cost of consistently maintaining a highly transparent water quality.

- Lake location. The lake will be created by two dams: the "downstream" dam located approximately 1,500 feet west of Mill Avenue, and an "upstream" dam located at the confluence of Indian Bend Wash and the Salt River. Selection of the dam locations establishes the lake surface area, volume, depth, and hydrogeological setting that can be used to determine lake construction features and engineering requirements.

- Dam types. The dams will be air-inflatable rubber fabric types, keyed to a concrete foundation.

Two major decisions have yet to be made:

- What is the source of water supply to sustain Town Lake?

- What is the most cost-effective and environmentally "safe" method of controlling seepage losses from the lake? Seepage control must recognize known landfills and groundwater contamination that could be adversely impacted by raising the groundwater elevations in the vicinity of the lake.

Water Supply Options

There are five separate water supply options. Each of these options has variations for a subset of 10 supply options. Among these 10 options, some have further sub-alternatives to consider. The primary sources are reclaimed water from either the existing Kyrene Water Reclamation Facility (WRF) or the proposed North WRF, or Salt River Project (SRP).

The options cover a range in capital cost from under $500,000 to over $20,000,000[1]. Operations and maintenance costs for these options range from nothing to over $2,000,000 per year. The supply costs do not include the basic costs of wastewater treatment (sunk costs of Kyrene WRF or future costs of the proposed North WRF).

The options range from direct reuse of existing reclaimed water produced at the Kyrene WRF (or the proposed North WRF) to extensive additional treatment schemes combined with aquifer storage and recovery systems. The

1 Estimates of construction cost include construction and allowances for contingency, administration, and engineering. See Section 10 for limitations.

additional levels of treatment provided for the reclaimed water directly affect the water quality of the lake. This report categorizes lake water quality in terms of transparency. Transparency, or clarity, can be measured scientifically with a Secchi disk. A Secchi disk is a white and red round disk that can be placed at varying depths in the lake. If the disk is visible at a 2-foot depth, the interpretation is that the transparency is 2 feet. Each of the alternative water supplies has been characterized with the resultant probable average lake transparency. Transparency will be lowest for a lake directly supplied with reclaimed water. Transparency will be high (clear) for a lake supplied with recovered water (reclaimed water stored in the aquifer and then pumped into the lake). In general, the higher the desired transparency, the higher the costs associated with supply.

Stormwater Management

The nutrients and other constituent pollutants usually found in urban runoff pose a significant threat to the water quality of Town Lake. A range of alternative stormwater management schemes was evaluated. The range of cost is $3,300,000 to $11,420,000. The lower cost is estimated for a lake supplied by either the existing Kyrene WRF or future North WRF. The higher cost is related to a lake supplied by SRP. The alternatives are based on intercepting and diverting a portion of the runoff that would otherwise enter the lake. Additional system components include constructing an upstream dam to retain nuisance runoff and detain large flows, and a plan to continually dewater the Price Road Tunnel to reduce the impacts of stale discharges from the tunnel. If the selected supply option entails an urban SRP reservoir, and SRP requires that all stormwater is diverted around the lake, a more conservative design incorporating a higher capacity bypass would be needed.

Seepage Losses

One of the major costs associated with creating Town Lake could be related to the means and methods for controlling water loss from the sides and bottom of the lake. Alternative technologies that are presented in this report include the conventional approach of lining the lake. Also included is an innovative approach using the underlying rock surface as the lake "bottom" in conjunction with slurry walls to form the sides of the lake. Even more innovative is an approach whereby the water is allowed to seep out the sides (and bottom where the hardrock is deep) only to be recovered with wells and pumped back into the lake.

The liner and slurry wall techniques have a much higher construction cost (from $14 to $20 million, depending on depth) but low maintenance cost. (Assuming the liner is placed below the scour depth, there would be no replacement cost.) The pumped seepage recovery system has a relatively low capital cost ($4,500,000), but a high annual cost ($200,000) associated with energy costs for pumping, maintenance of equipment, and the more intensive groundwater monitoring that would be required.

Lake Quality Management

Maintaining the quality of the lake water over time will require a well-planned, proactive program. Management options include aeration and circulation of the lake; withdrawal of water from the deeper, more stagnant areas of the lake; physical and chemical treatment; and possibly the use of fish to control weeds. The costs for management are extremely variable and will depend on the final source water and transparency required. The annual maintenance cost, on an extreme basis, could range from $200,000 to over $500,000 per year. This wide range in annual cost is related to the range in lower to higher lake water qualities established by the alternative sources of supply.

Alternative Project Concepts

Elements of the project are described in Sections 4 and 6. These elements include the dams, methods of seepage control, creation of a shoreline for the lake, pipelines to transport water to the lake, wells for recovering reclaimed wastewater stored in an underground aquifer, canal turnouts, and possible treatment process additions at either the existing Kyrene WRF or the future North WRF. These project elements can be selected to create numerous concepts for the final "total project" alternative.

Section 8 presents four concept plans as examples of how the various elements of the project could be selected to provide insight into the estimated capital and annual costs associated with a "complete" lake and supporting infrastructure.

These four concept plans are based on the primary differences between the sources of water supply as follows:

> ***Concept 1.*** This concept uses the existing Kyrene WRF plus additional treatment focused on reducing the phosphorus content of the reclaimed water. The additionally treated reclaimed water is piped to the lake.
>
> ***Concept 2.*** This concept uses the existing Kyrene WRF whereby the reclaimed water is stored in an aquifer via surface recharge techniques on city-owned land south of Elliot Road near Kyrene Road (the Hardy Farm site). The reclaimed water is recovered using wells north of Broadway Road near Mill Avenue, then piped to the lake.
>
> ***Concept 3.*** This concept is based on supply from the future North WRF located south of the Rio Salado Parkway near Priest Drive.
>
> ***Concept 4.*** This concept uses SRP water supplied from the SRP Tempe Canal. In this concept the lake would function as an SRP transport system, allowing movement of SRP water from the Tempe Canal to the Grand Canal, and also function as an equalizing reservoir in the SRP system.

These alternative "complete" lake projects are illustrated in Figures 8-1 through 8-4.

Section 1
Introduction

Background

Prior to the 1940s, the Salt River was a perennial stream providing water to the Valley of the Sun for irrigation and recreation. Following the developments of the Salt River Project, the river became a dry riverbed for most of the year, flowing only in response to large rainfall events. Over the years, sand and gravel extraction from the riverbed and floodplains, and the creation of several landfills dramatically altered the environment and habitat of the Salt River, creating an eyesore where a riparian oasis once existed.

In 1966, students in the ASU College of Architecture conceived an ambitious plan to restore the Salt River through creation of a series of lakes and streams. The project covered over 38 miles from Granite Reef Dam to the Gila River. The City of Tempe eventually assumed a leadership role in promoting the "Rio Salado" project, focusing on the portion of the river within the City boundaries.

Today the vision of Rio Salado encompasses an area from McClintock Drive to the Hohokam Expressway and includes a variety of commercial, recreational, and residential developments. The focal point of the project is a 200-acre recreational lake which will extend from about 1,500 feet west of Milll Avenue, east to the Indian Bend Wash. "Town Lake" will provide a gathering place for the Valley just as Hayden's Ferry once did near what is now Old Town Tempe.

In undertaking this bold renaissance of the Salt River, the City of Tempe faces the challenges of ensuring a reliable supply of water; creating a major water feature in the riverbed without compromising its flood control capabilities; and avoiding any adverse impacts on area Superfund sites and landfills. Further, the project requires that a high quality, aesthetically pleasing lake be developed and maintained using source water of possibly limited quality, in an adverse environment for such water bodies.

Objectives

The Rio Salado Engineering Feasibility project is one step in the continuing phases of implementation of the Rio Salado project. The objective of this report is to conclude the engineering feasibility of the major physical facilities needed for the lake. The project must satisfy a confusing gamut of regulatory requirements and permitting issues.

This Engineering Report summarizes the findings and conclusions of the engineering work completed thus far. This report defines the water demands of the lake (Section 3) and outlines the design of the project's physical facilities (Section 4). It also details the opportunities and constraints for the selection of a source water supply for Town Lake (Sections 5 and 6), and the needs for long-term management of the lake water quality (Section 7).

Implementation of the project requires further predesign and final design investigations as well as agency coordination. As part of this study, preliminary discussions were conducted with the regulatory agencies and a general strategy for negotiating the regulatory maze was developed. Section 8 describes an implementation strategy for the next stages of the Rio Salado design and permitting activities.

Project Documentation

The engineering work performed to meet the project objectives began in February 1991. A series of Technical Memorandums (TMs) was produced documenting the findings of the various work elements. The TMs delivered to the City of Tempe were:

- TM 1 Data Inventory
- TM 2 Permitting Constraints
- TM 3 Water Balances
- TM 4 Salt River Hydraulics
- TM 5 Stormwater Management
- TM 6 Aquifer Storage and Recovery
- TM 7 Surface Water Development
- TM 8 Town Lake Feasibility Study

TMs 1 through 7 were produced only in draft form and were intended to provide preliminary findings to the Rio Salado Technical Committee. A compendium of TMs 1 through 7 was reprinted as a separate project deliverable. As new and additional information was developed during the course of the work, these TMs were not updated, and therefore may not represent the latest or most accurate information. TM 8 was produced in final form for general use by the City.

Two workshops were held with Tempe staff. At the workshops, the consultant staff presented findings to, and received direction from, the Rio Salado Technical Committee. Direction was also received as part of the concluding meeting focussing on the draft of this report. The results of these workshops and project coordination meetings guided the work efforts as the project progressed.

Acknowledgments

This study was prepared by CH2M HILL with guidance and assistance from the City of Tempe Mayor and Council, and the City's Engineering Department staff. In addition, the City's Planning and Redevelopment and Water and Wastewater Department staff provided invaluable assistance. The Rio Salado Technical Committee also provided input and direction throughout the project. Assistance was also provided by Salt River Project, Arizona Department of Transportation, Arizona State University, the Bridgestone Engineered Products Company, and Aquatic Dynamics, Inc.

Limitations

The findings and recommendations presented herein are based on information either provided to, or developed by the Consultant for the purposes stated above. The information represents the best available information at the time of the work effort. Ongoing work by the City of Tempe, changes in institutional and regulatory policy, changes in cost or availability of materials, and other factors may impact the accuracy of the information provided. Some of the conclusions of this study are based on limited information and assumptions coordinated with Tempe staff. These assumptions have been identified as such where possible.

Currently the City is pursuing ongoing investigations into the feasibility of surface recharge near the Kyrene WRF, and is developing a comprehensive City-wide water/wastewater master plan. These studies were not complete as of this writing. These results are, therefore, not reflected in this report.

Section 2
Project Design Objectives

The design of Town Lake must consider a wide range of objectives and design criteria. In many cases these objectives are conflicting or competing. For instance, water quality is enhanced with a deeper lake; however, costs are reduced with a lower dam height. The evaluations performed for this study included consideration of many of these criteria and objectives, however the scope of the evaluations was limited to the engineering considerations. Land use, economic impacts, financing, and other related issues have been generally excluded from this study. These issues have, in some instances, been incorporated into the evaluation process through input and direction by City of Tempe staff. This section describes some of the more significant objectives used in evaluating project alternatives.

Beneficial Uses

Perhaps the most obvious lake design objective is to maximize the beneficial uses that the lake will support. The most desirable lake design supports the widest range of uses. The potential uses are (listed from most difficult to attain to least difficult):

- Swimming (full body contact)
- Sailboarding (partial body contact)
- Fishing for human consumption
- Boating (incidental contact)
- Catch and release fishing
- Passive recreation (no contact)

The level of use that can be attained depends on several factors. Federal, state, and county agencies such as the Arizona Department of Environmental Quality (ADEQ), and the Maricopa County Health Department (MCHD) each have requirements that impact lake uses. Aesthetic characteristics of the lake will also determine the range of uses. The ability to create and maintain a lake of sufficient water quality to support these uses depends on the quality of the source water and the degree to which the lake water quality is managed. Specific design recommendations will depend on the selected level of use. This report is based on City direction that swimming will not be permitted in the main body of water comprising Town Lake. All other beneficial uses are of continuing interest.

Flood Control

The Salt River is the primary conveyance facility for flood water from the Salt River and Verde River watersheds through the Phoenix valley. The design flood for the river in the project area is the 100-year event, about 215,000 cfs. Recently completed and ongoing channelization projects in the area are intended to ensure that the design flood is safely conveyed through the valley. The Arizona Department of Water Resources (ADWR) and the Flood Control District of Maricopa County (FCDMC) will require that the Rio Salado project not jeopardize the capacity of the river to contain flood water, even in the event of a dam failure.

The Salt River is a complex and dynamic system. Physical changes to one reach of the system invariably affect the rest of the system. The ability of the river to transport sediment is one of the characteristics that must be carefully considered when modifications to the river are proposed. The final project must minimize sediment transport-related impacts to the Salt River system. Specific design criteria for flood control include:

- The capacity of the channel and bridge structures to pass the design event must not be compromised.
- The water surface elevation during a 100-year flood event must not be increased by more than 1 foot.
- The flood wave that would result from a spontaneous failure of the dam must be contained in the channel.
- The single-event general scour downstream of the lake must not significantly increase.
- The equilibrium slopes of the channel must be maintained.

Environmental Impacts

The development of Rio Salado must consider impacts on the local groundwater aquifers. Specifically, the potential impacts to the North and South Indian Bend Wash Superfund sites, and several area landfills must be considered. ADWR and U.S. Environmental Protection Agency (EPA) concerns and regulatory constraints must be incorporated into the design of the project components. Environmental constraints and issues include the Section 404 permit requirements that resulted from the City's Salt River channelization project, National Pollutant Discharge Elimination System (NPDES) requirements, and other Clean Water Act provisions.

As a primary design objective, the Rio Salado project must attempt to achieve "zero impact" on existing groundwater contamination. This objective is reflected in the recommendation for design criteria that will isolate the lake from the local groundwater system by limiting or controlling seepage from the lake.

To further define specific design recommendations and criteria, specific geotechnical, geophysical, and hydrogeological investigations were performed for this project. These studies provided greater understanding of subsurface conditions and groundwater flow characteristics. Details of these studies were provided in TM8 and are summarized later in this report. These studies, in addition to recommended predesign investigations and ongoing monitoring programs discussed in later sections, are intended to help meet the objective of "zero impact."

Section 3
Water Demands

The base water demands for Town Lake are lake evaporation and seepage (infiltration through the bottom and sides of the lake). Additional water demands include irrigation water for landscaping the Rio Salado project developments and creation of artificial wetlands.

Evaporation

Evaporation rates in the Phoenix area are among the highest rates found in the United States. Data on monthly pan evaporation rates for the Phoenix area from 1960 to 1991 reveal strong seasonal fluctuations, with the lowest rates occurring in mid-winter and the highest rates occurring in late spring and early summer. Recent data (1989 through 1991 records) suggest that, due to "heat island" effects, the evaporation rate in Phoenix has been increasing since the mid 1970s.

Evaporation rates are influenced by solar radiation, relative humidity, temperature, wind, atmospheric pressure, and other factors. In general, the smaller and more shallow a body of water, the higher the evaporation rate. Hence, a correlation factor is often used to relate pan and lake evaporation. For this evaluation, lake evaporation is estimated as 70 percent of pan evaporation rates.

Average pan evaporation rates vary by about 15 percent from year to year. A safety factor of 30 percent has been used to account for this variability on a maximum month basis.

Figure 3-1 illustrates monthly evaporation demand estimated for a 200-acre lake. The annual average evaporation demand for the selected lake would be approximately 1.1 million gallons per day (mgd). The monthly rates would vary from 0.4 mgd in December to 1.7 mgd in June (2.2 mgd with "safety factor").

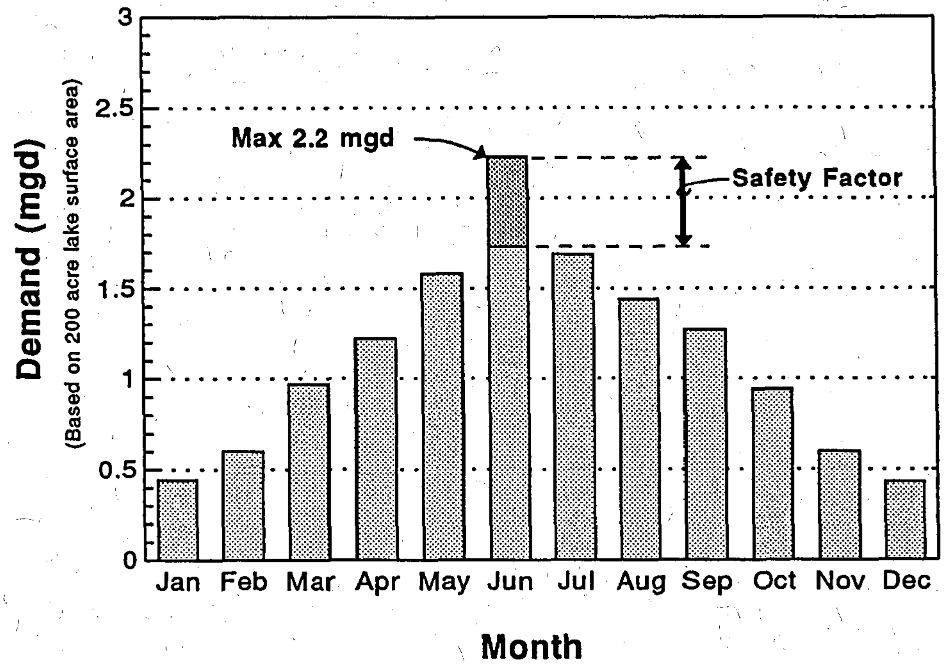


Figure 3-1
Monthly Evaporation Demand

Seepage Losses

A detailed discussion of the potential loss of water through the lake bottom and sides, seepage losses, was presented in the Feasibility Study, TM8. Field programs, including drilling and monitoring of groundwater levels, allowed calculations of potential seepage losses. Based on the work described in TM8, Town Lake configurations without seepage control may be expected to lose an average of approximately 0.2 feet/day per square foot of lake area (during steady state conditions). The actual seepage range varies depending on the depth of the water in the lake and the location. The rates are lower between about Priest Drive and Mill Avenue and are higher east of Mill Avenue to McClintock Drive. As described in Appendix A of TM8, these estimates are approximate (-50% to +100%) and do not account for reduction over time due to siltation and subsequent clogging. With available seepage control technologies, seepage may be reduced to approximately 0.01 feet/day/per square foot. Three seepage control methods were investigated for this report. Slurry trench cutoff walls, liners, and well recovery systems are described in Section 4.

In summary, without seepage control or under conditions of collecting seepage with wells, annual seepage may range in the order-of-magnitude of 16,000 ac-ft (14.1 mgd) for a lake surface area of 200 acres. With effective seepage controls, annual net seepage losses for a 200-acre lake may range in the order-of-magnitude of under 0.2 (theoretically zero with pumped recovery methods) to 400 ac.ft. (0.4 mgd) based on liner construction techniques. Actual seepage will vary with clogging and natural variability within the geologic and man-placed materials.

Landscape Irrigation

The demand for landscape irrigation water depends on the size of the area to be irrigated, vegetation type, and method of irrigation. Final landscaping plans have not yet been developed, so no exact estimates have been made. However, landscape irrigation is an important water demand, and therefore merits consideration.

Preliminary estimates of areas that will be irrigated by the City were prepared by City planning staff and are shown in Figure 3-2. The landscaped areas are categorized as turf and non-turf areas. This estimate indicated that 12 acres of turf and 48 acres of non-turf areas would be irrigated. Turf irrigation requires relatively large quantities of water compared to other types of landscape materials. The annual irrigation requirement for bermuda lawn overseeded with winter rye grass is approximately 6 ac-ft per acre. A bermuda grass lawn that is not overseeded in winter requires approximately 4.5 ac-ft per acre. For this assessment, 6 ac-ft/ac was used.

Water consumption for plants commonly used in arid to semi-arid areas range from 10 to 20 inches per year for low water use varieties. Middle-use plants range from 20 to 35 inches per year and higher-use plants use 35 to 50 inches per year (University of Arizona, 1977). For this estimate, non-turf landscaping was assumed to consume 24 inches per year.

Based on the City's estimate of future landscaped areas, the irrigation demand will range from 0.04 mgd in December to 0.33 in June.

An additional landscape-related demand to be considered is the proposed wetlands area downstream of the lake as described in the *Wildlife Habitat Master Plan* (HNTB, 1990). This report does not quantify water needs, however based on about 3.5 acres of planting areas, a rough approximation of the average monthly demand ranges from 0.02 mgd in December and January to 0.14 mgd in June and July.

Total Water Demand

The total base demand for source water is the sum of evaporation and seepage demands described above. Figure 3-3 illustrates the relative magnitude of these monthly source water demands for a 200-acre lake (based on a constructed liner system). The average demand is about 1.7 mgd; the peak month demand is about 2.6 mgd (3.1 mgd with an evaporation safety factor).

These estimates of water demand are approximate and will vary with weather conditions, types and amounts of landscaping, sedimentation and scouring of

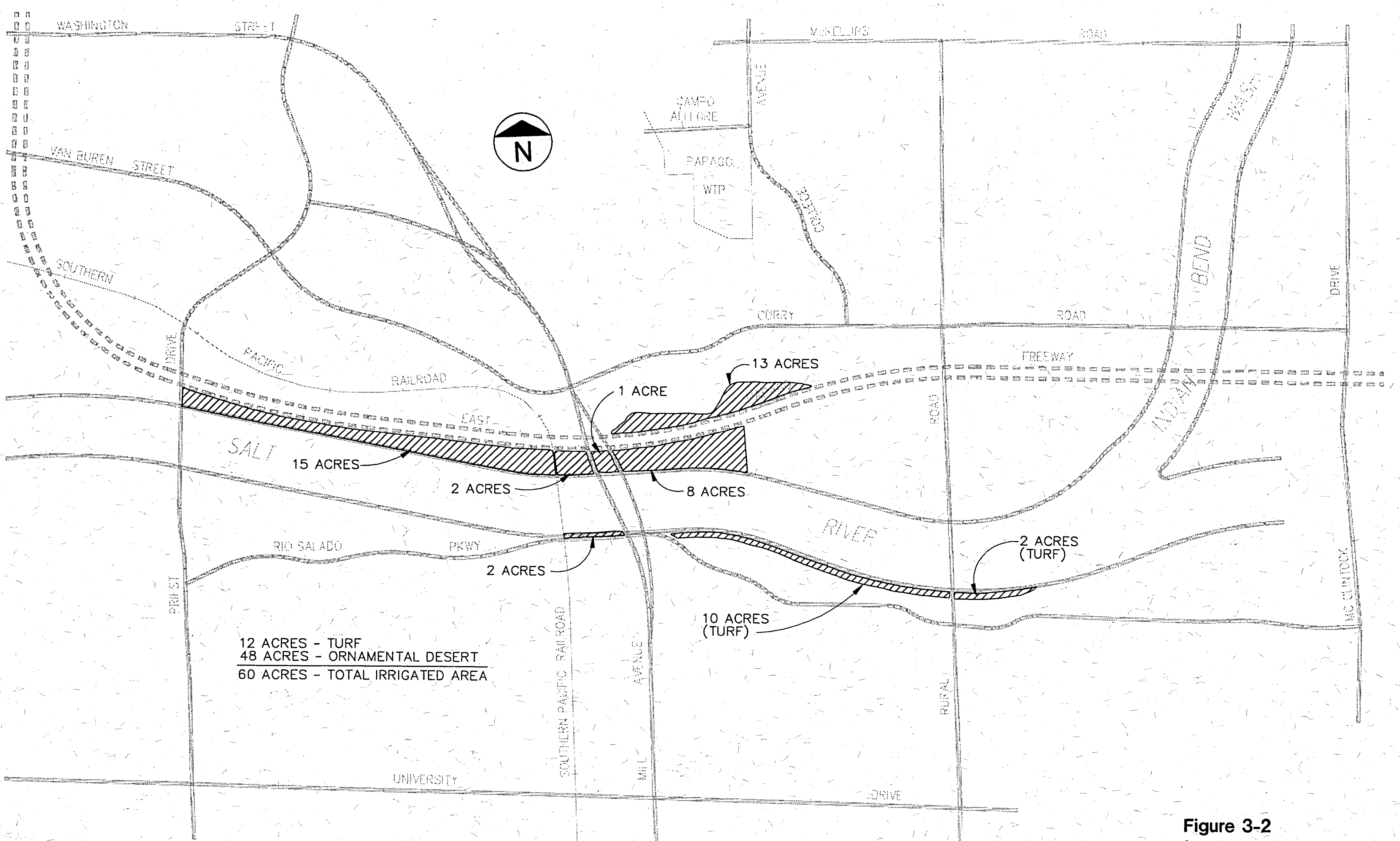


Figure 3-2
Areas to be Irrigated

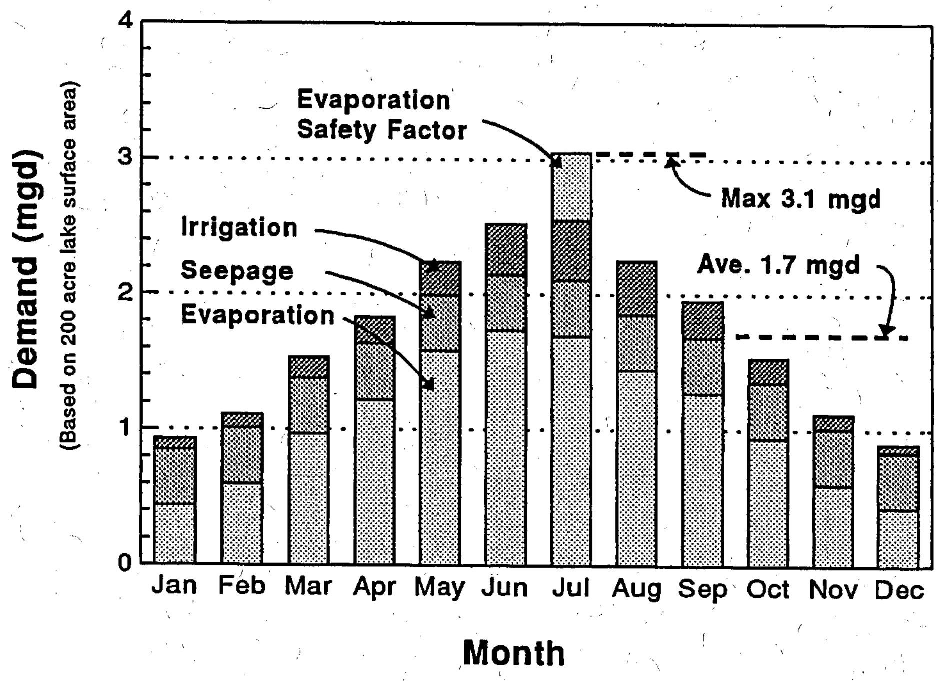


Figure 3-3
Monthly Water Requirements

the lake bottom, seepage control, and lake management practices. For instance, fountains, sprayers, and other aesthetic water features may increase rates of evaporation.

The application of aquifer storage and recovery (ASR) facilities to this project would provide storage capability for responding to seasonal and operational variations of the evaporation and irrigation demand. This would allow the primary water source to be sized for the average demand, rather than the maximum monthly demand.

Continuing monitoring of the lake's water balance after construction is recommended, and will likely be required as part of an Aquifer Protection Permit (APP). Monitoring of the evaporation rate will require measurement of temperature, humidity, windspeed and direction, and solar radiation. Both standard evaporation pans and floating-type pans should be incorporated into the monitoring program. The floating pans will be more accurate than standard pans, and can be used for a short time to calibrate the standard pan rate for long-term monitoring.

Section 4
Physical Facilities

The physical components of Town Lake include the dams or impoundment structures—the main downstream dam and the upstream dam which will serve to establish the extent of the lake; seepage control and lining systems; stormwater bypass and management systems; and, channel bank modifications required to create shoreline, access, boating facilities and other user amenities. In addition, facilities to manage the lake water quality will be required.

This section summarizes the physical facilities.

Impoundment Structures

The most important structural component of the lake is the dam. During the feasibility phase of the project, various dam types were evaluated in detail using specific project criteria, including hydraulic and sediment transport-related flood control impacts, life cycle costs, aesthetics, reliability, safety, and operational flexibility (TM4).

The potential dam and gate alternatives that were considered for the Rio Salado project include three basic configurations:

- Movable gates
- Fixed weirs
- Fuse plugs

The alternatives considered for each basic configuration included tainter gates; bascule or bottom-hinged leaf gates; inflatable dams, both water- and air-filled; ogee crest weirs; labyrinth weirs; and fuse plug configurations with sections set at different blowout elevations and with mechanical gates for passing lower, more frequent flows.

Several alternatives were eliminated after preliminary evaluation. Tainter gates did not meet flood control criteria. Water-filled inflatable dams were eliminated from further consideration for safety and operational constraints.

Labyrinth weirs were eliminated because of hydraulic and sediment transport constraints.

Based on the findings of the detailed evaluations, three alternatives were presented to the Rio Salado advisory committee. These alternatives were:

- Air-inflatable rubber dam
- Combination bascule (leaf gate) and multiple fuse plug
- Side channel weir

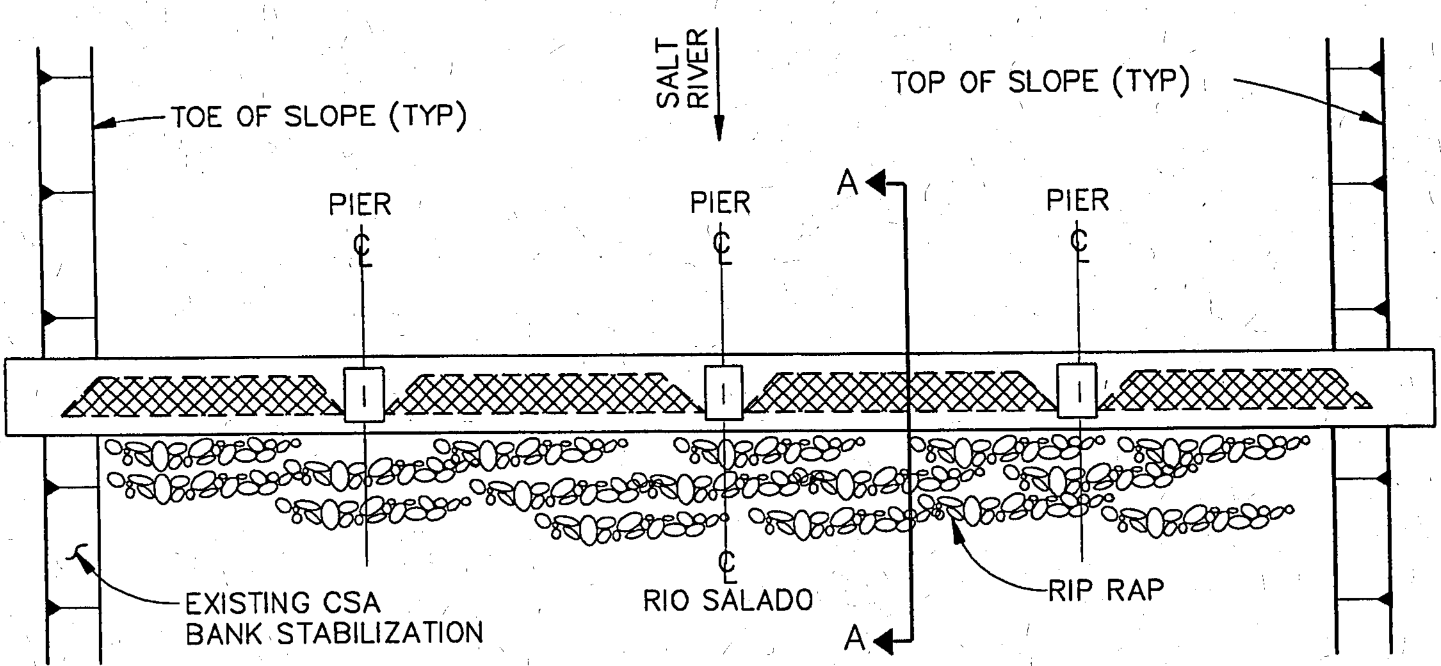
Following consideration by the committee, a 16-foot-high air-inflatable rubber dam was selected. The dam was evaluated assuming it consisted of four 210-foot-long dam segments, with three intermediate piers. Each pier would be 18 feet high, with a 5-foot top width and 1:1 side slopes. In addition to the main impoundment dam, similar dams were recommended at the upstream end of the lake to act as stormwater retention structures and limit the lake area and depth. The inflatable dam configurations are shown schematically in Figure 4-1. Each dam segment should be independently operable to allow flexibility for low flow and sediment passage, and to be able to exercise each segment for maintenance checks. One of the manufacturers of inflatable dams claims that the 16-foot dam could be overtopped by about 6 feet without inducing instability.

Some of the key advantages and disadvantages of the air-inflatable dam relative to fixed weir and fuse plug options are outlined below.

Table 4-1
Air-Inflatable Dam

Advantages	Disadvantages
• Dam should perform well during anticipated flood events • Dam backwater effects are minimal • Construction and design is not complex because: - Long spans allow fewer piers - Suitable foundation conditions exist • Operations and maintenance is less complicated because: - Dam deflates without electrical power - Rubber material withstands sand erosion in high velocity flooding - Less sediment trapped by dam	• Long material delivery time • Untested design parameters, including: - Dam height exceeds tallest previous installation - Design life of rubber bag is unproven - Some potential for vandalism • Manufacturers/suppliers are limited

As currently proposed the project includes a downstream 16-foot-high inflatable rubber dam (three sections) constructed near the existing soil cement grade control structure between Priest Drive and Mill Avenue and three inflatable dams upstream connected by concrete floodwalls at the confluence of Indian Bend Wash and the Salt River (see Figure 4-2).



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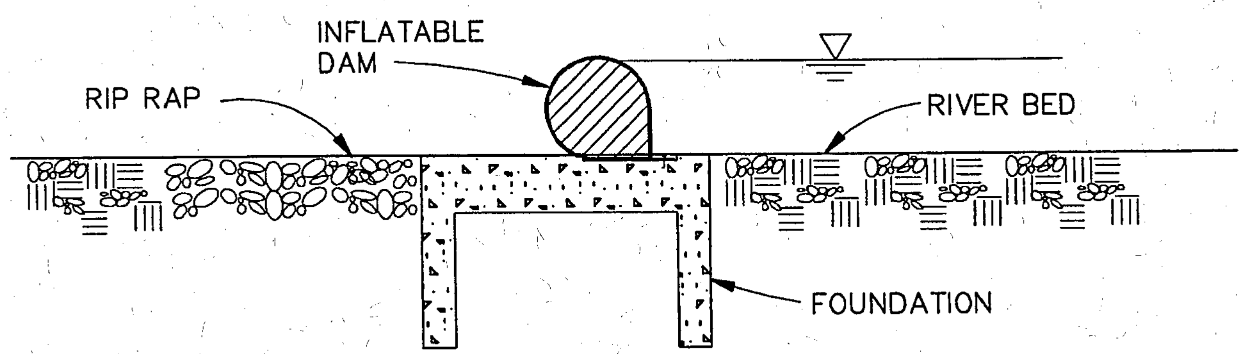

SECTION A-A
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Figure 4-1
Inflatable Dam Detail

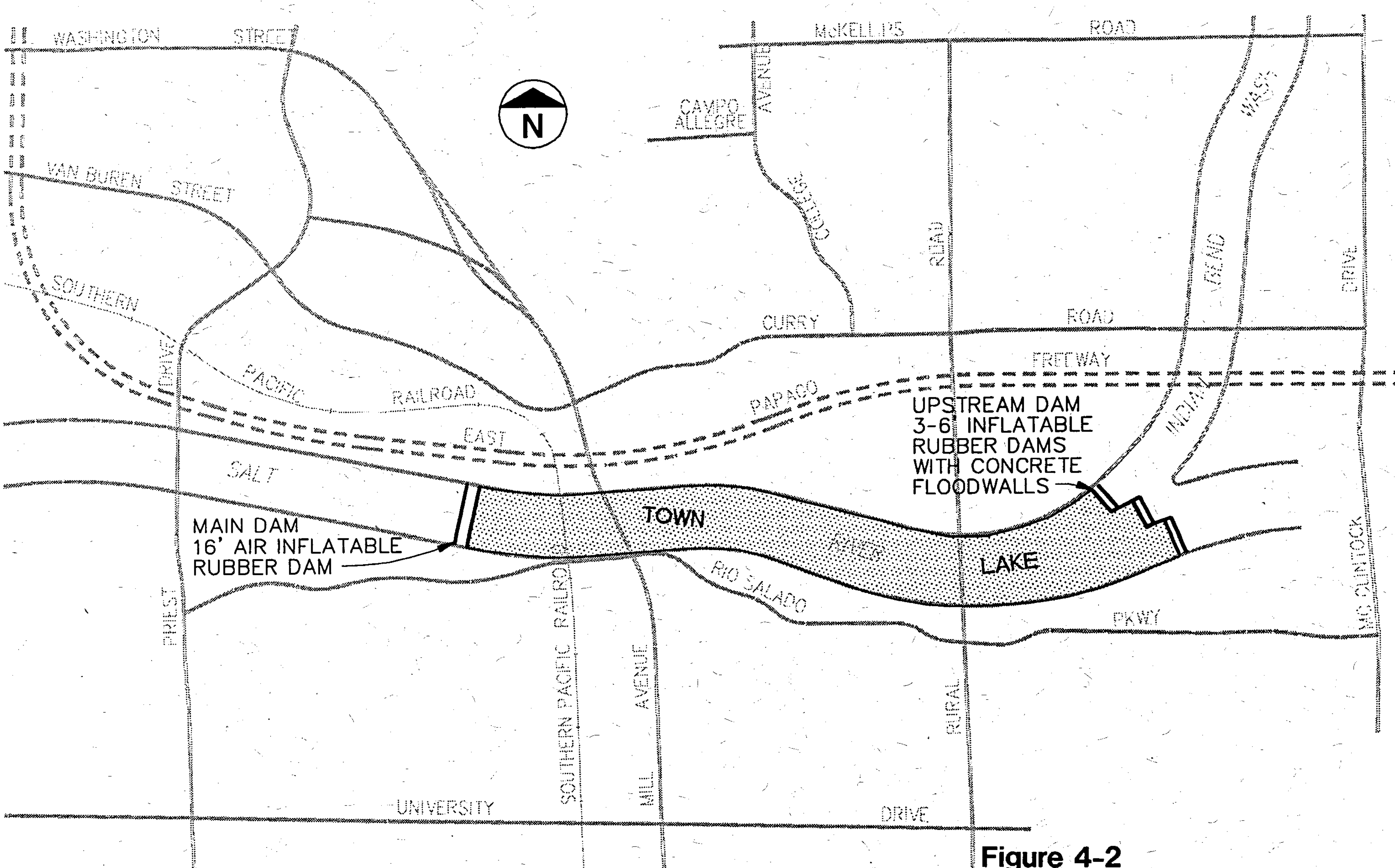


Figure 4-2
Limits of Town Lake
Grade Control Structure #4
to Indian Bend Wash

Impoundment Dam Instrumentation and Control

To preserve the flood control function of the Salt River channelization, the impoundment dams must offer flexible and reliable means of deflation and provide the minimum possible obstruction to flood flows. Both manual and automatic inflation/deflation controls may be installed. A typical installation may include several independent automated safety systems such as:

- A Programmable Logic Controller (PLC) operated valve system to maintain the water and pressure levels for preselected operating conditions. This system may be configured to interface with SRP and FCDMC ALERT or SCADA telemetry systems.

- A water elevation actuated valve controlled by a device such as a float/counterweight or pressure transducer to deflate the dam in the case of high upstream water surface.

- Rupture disks to safeguard against over-inflation.

Depending on the degree of control, redundancy, and operational criteria desired, each of the rubber dam sections or bags can be independently plumbed and controlled. The conceptual design recommendation of the Bridgestone Engineered Products Company, a vendor of inflatable dams, included at a minimum, a single 900 CFM blower with 6-inch piping for the downstream dam. They indicated that resultant inflation and deflation times of 86 and 40 minutes respectively could be expected.

For safety and redundancy, at least two blowers should be installed at each dam location and each bag should be independently plumbed and controlled. The selection and final design of the systems should be coordinated with SRP, ADWR, and FCDMC.

Impoundment Dam Predesign Activities

Prior to finalizing the design of the impoundments, several pre-design investigations are recommended to confirm or modify the criteria and assumptions used for this concept design. These activities include both geotechnical and hydraulic investigations.

Geotechnical Investigations

The main dam location is near a cement stabilized alluvium (CSA) grade control structure. This structure may be incorporated into the dam foundation. Geotechnical evaluations are recommended to assess the foundation requirements for each dam and the characteristics of the grade control structure. In addition, the capability of the newly constructed bank protection to withstand rapid drawdown conditions should be confirmed. Specific activities include:

- Review Arizona Department of Transportation (ADOT) data from construction of the soil cement grade control structure including as-built plans, geotechnical report, construction records, and photos taken during construction.

- Drill core holes through the grade control structure to obtain information on the structure, the rock below the structure, and the interface between the structure and the rock.

- Drill soil borings and rock cores along the proposed upstream and downstream dams. Borings or rock cores should be extended a minimum of 10 feet into competent bearing material.

- Perform laboratory testing of samples collected during soil borings and rock coring. Testing will be performed for geotechnical parameters required for preliminary design of the dam foundation and abutments.

- Perform a preliminary analysis of the dam's foundation system and abutments. The preliminary analysis will include evaluation of the proposed foundation systems for bearing capacity, settlement, lateral loading resistance, lateral stability, uplift pressures, and seepage.

Hydraulic Investigations

The primary purpose of the channelization of the Salt River is to provide flood control. Prior to final design, a detailed hydraulic analysis of the Salt River from I-17 to Price Road should be prepared that reflects the final dam and bank configuration. A sediment routing/scour analysis of the same reach should be included. In addition a dynamic model simulating the rapid deflation of the dams may be required by ADWR Division of Dam Safety.

Impoundment Dam Costs

The contingencies for the dams are based on estimates provided by the Bridgestone Engineered Products Company. Detailed cost information provided in TM4 and TM8 was updated to incorporate the additional cost of the upstream dam configuration proposed by Tempe staff. The results are summarized in Table 4-2 below.

Table 4-2 Impoundment Cost Opinion	
16-foot inflatable rubber dam (downstream)	$6,880,000
6-foot inflatable rubber dam (upstream)	4,260,000
Foundation and upstream floodwall	720,000
Total	$11,780,000

Seepage Control

To reduce seepage from the lake, three general methods were considered:

- Lining the lake, thus reducing seepage through the bottom and sides of the lake.
- Constructing cutoff walls along the lake boundary, thus reducing the seepage through the aquifer beneath the lake.
- Collecting the seepage with wells, and returning the pumped water to the lake.

Linings

There are many alternatives for lining materials, including those constructed in place such as compacted clay, soil cement, or asphalt, and those manufactured offsite such as PVC and geosynthetic clay. The differences between these types are the cost, ease of installation, hydraulic consistency of the finished liner, and resistance to scour. All of these factors will need to be evaluated during final design.

A description of several lining options and design considerations follows. Typical permeabilities are shown in units of centimeters per second (cm/s) for comparison. Resultant losses for various lake configurations were estimated and are presented in a later section.

Compacted Clay Lining

Compacted clay lining should consist of approximately 1 to 2 feet of compacted clay imported to the site. The clay would be placed in thin lifts and compacted with several passes of equipment to achieve a consistent low permeability lining. The clay lining should be protected from scour and will require continuous watering during construction and when the lake is empty to prevent cracking and desiccation of the clay. A volume of in-situ channel material equal to the volume of clay would be removed to maintain the channel profile.

Geosynthetic Clay Lining

A geosynthetic clay lining is a layer of bentonite clay between two geotextile membranes. The material is manufactured offsite and shipped in rolls. The material is installed by unrolling it on the prepared surface. Seams require overlapping. Permeabilities on the order of 10^{-8} cm/s could be attained. A disadvantage of the geosynthetic clay lining is because it is thin, it could more easily be damaged by scour than the compacted clay.

PVC Lining

The PVC lining is similar to the geosynthetic clay lining in that it would arrive onsite in rolls, be unrolled on a prepared surface, and overlapped. The difference is that the overlaps of PVC lining must be cemented together. The PVC lining would require a geotextile over the top to protect the lining from scour similar to the compacted or synthetic clay. The cost of a PVC lining is similar to the geosynthetic clay lining.

Soil Cement and Polymer Asphalt

Other constructed-in-place lining alternatives are soil cement and polymer asphalt. The soil cement is a mixture of the river sands and gravels, cement, and water with less cement and water than typical concrete. The material would be placed and compacted similar to the compacted clay liner. Permeabilities of less than 10^{-5} cm/s can be expected with additives to prevent cracking. The polymer asphalt is constructed using paving equipment similar to that used for construction of an asphalt roadway. Polymers are added to reduce the permeabilities of the lining. Permeabilities of less than 10^{-9} cm/s have been reported with this type lining. Both the soil cement and polymer asphalt linings would have greater scour resistance than the clay or PVC linings and would require less scour protection.

A typical section for liner placed beneath the channel bed is shown in Figure 4-3.

Channel Scour

As noted above, several of the lining options are susceptible to damage from scour of the channel bed during flood events. The cost of the liner will be dependent on the depth to which it is buried and the selection of an appropriate burial depth will be determined by the acceptable degree of risk of damage or failure to the liner. To help define the relationship between the burial depth and installation costs of lake lining systems and the risk of damage or failure of the lining system, the probable depth of scour during a range of flood events was estimated.

As part of the Salt River hydraulic design for the Rio Salado area, CRSS Commercial Group, Inc., prepared a sediment transport and scour analysis of

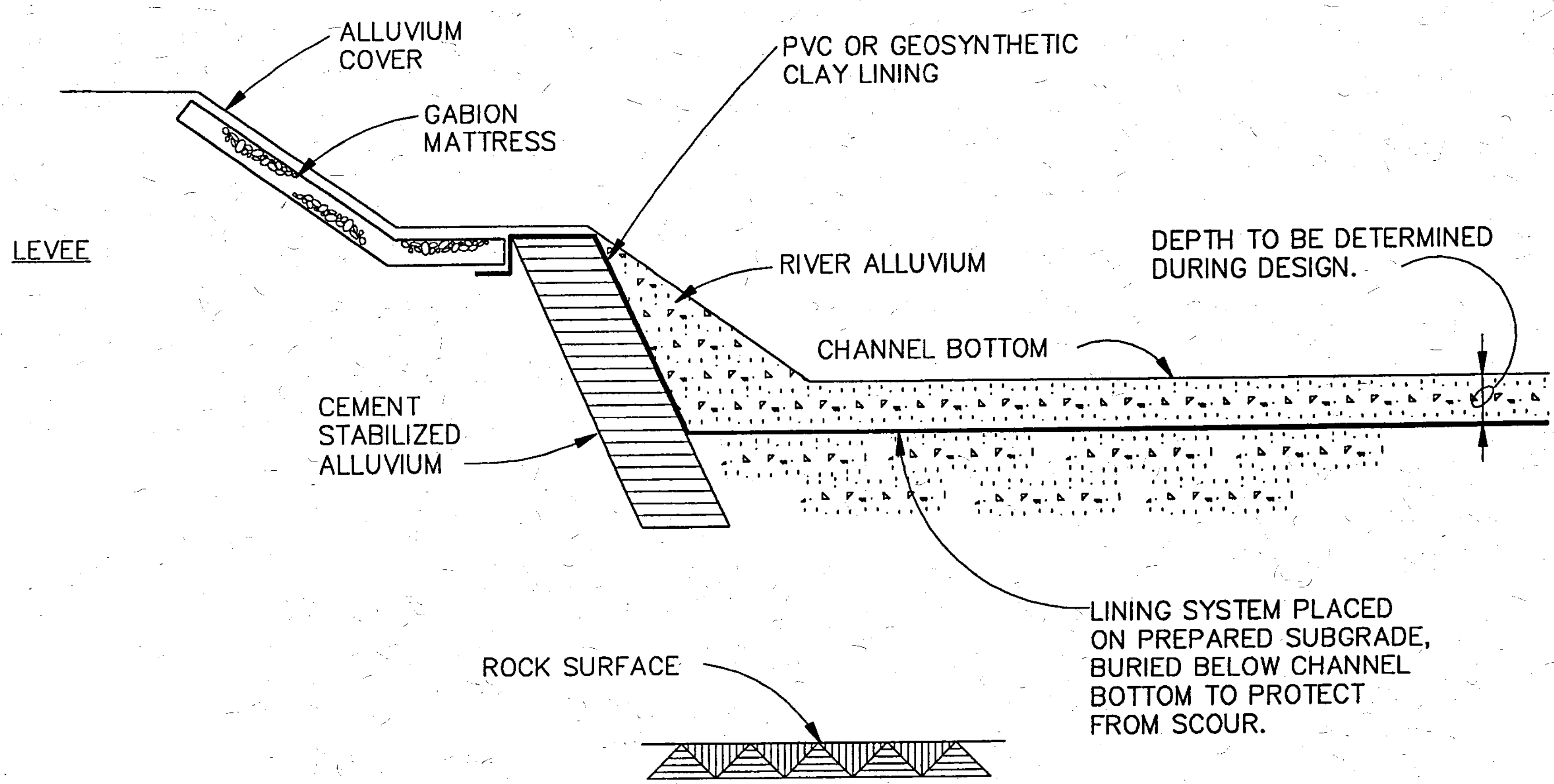


Figure 4-3
Conceptual Lake Liner Detail

the reach (CRSS, 1990). The CRSS report was intended to provide recommended depth of toe down for the CSA bank protection along this reach of the Salt River. The results of the CRSS study were used and extrapolated to apply to a range of flood events from the 5-year discharge of 40,000 cfs to the 100-year discharge of 215,000 cfs. This analysis, summarized below, was not intended to be a design-level evaluation and the final burial depth selected for any liner system should be re-evaluated during project design. The design parameters for the bank protection required that it withstand scour during the 500-year flood event. Because of the magnitude of damage that would occur should failure of the bank protection occur, a very conservative analysis and a high safety factor were used. The consequences of damage or failure to the liner are much less severe and therefore the design criteria used in the CRSS scour analysis may not be appropriate for the design burial depth for a lake lining system.

The CRSS study concluded that the existence of larger gravels and cobbles in the channel bed materials would limit the depth of scour during a flood event through a process called armoring. In this process, the finer material is scoured away and the larger sediment sizes remaining i.e., larger gravel and cobbles, form a layer which resists further channel degradation.

For the design of the bank protection, CRSS multiplied the boundary sheer calculated for each reach by a safety factor of 1.5. This factor generally increases the predicted maximum depth of scour by a factor of 2 or more. Therefore, the resulting scour depths are conservative.

CRSS evaluated the scour depth for a flow of 250,000 cfs. For this application, the basic hydraulic parameters and sediment characteristics and the basic method of analysis reported by CRSS were applied to lower discharge rates to develop a relationship between discharge and scour depth. The computations were performed both with and without the shear stress safety factor.

The results of this evaluation are shown in Table 4-3 below. The range of predicted scour depths, using the CRSS safety factor, varied from 2 feet for the 5-year flood to 9 feet for the 100-year event. These scour depths represent an average across the channel width. The actual scour depths will vary both laterally and longitudinally throughout the lake area.

Table 4-3
Liner Cover Depths for Varying Flood Return Periods

Flood Return Interval	Discharge (cfs)	Cover Depth With Safety Factor	Cover Depth Without Safety Factor
5-year	40,000	2 feet	1.5 feet
10-year	93,000	3 feet	2 feet
50-year	160,000	6 feet	3.5 feet
100-year	215,000	9 feet	5 feet

The cost of installation under each of these assumptions is shown in Table 4-4. Based on this cost comparison, the conclusion is that a burial depth. representing a design storm of between 10 and 50 years is appropriate. The estimates shown in Table 4-4 are based on a slurry wall control method west of Mill Avenue. The variable depths apply to the lake segment east of Mill Avenue.

Table 4-4 Liner Costs for Varying Depths of Cover	
Depth of Cover	**Capital ($ Million)**
2 Feet	14.0
3 Feet	15.6
6 Feet	20.2
9 Feet	25.0

It should also be noted that there is some uncertainty as to the accuracy of the adopted peak discharge rates for the higher frequency storms, i.e., 5- through 25- or 50-year storms. Some previous investigators have expressed the opinion that the accepted U.S. Army Corps of Engineers hydrology overestimates the discharges during these more frequent storm events and therefore, selecting a level of protection for the 10-year event may in fact provide protection for a much higher rate of flow.

Cutoff Walls

A cutoff wall involves the construction of a low-permeability, below-grade wall along the north and south sides of the lake. The most effective cutoff wall is a full cutoff of the aquifer beneath the site which would extend from the bottom of the CSA to rock or some other low permeability contact. Partial cutoff walls, extending only part-way to rock would also reduce the seepage rate by reducing the available area for the water to flow. Past experiments (USBR, 1977) have shown that a partial cutoff wall, extending 50 percent of the depth to rock may reduce seepage 25 percent; a cutoff wall extending 80 percent of the depth to rock may reduce seepage by 50 percent.

Under some lake configurations, the high water level will be above the CSA. To reduce the seepage rate through the gabions at the sides of the lake, the gabions above the CSA should be grouted. During pre-design, the effects of rapid drawdown (lowering the water level) in the lake on the stability of the levees should be analyzed.

There are various methods of constructing cutoff walls including cutoff trenches, sheet piling, mixed-in-place concrete pile curtains, slurry walls, and grouting of alluvium. Based on available subsurface information, the slurry wall method appears to be the most appropriate and cost-effective method for the Rio Salado site.

The slurry wall method is illustrated in Figure 4-4. This technique uses a water-bentonite mixture to support the sides of a trench. The trench is excavated by a backhoe with the excavated material placed beside the trench. Backfill material, typically a well-graded sand and gravel is mixed with bentonite and placed in the trench. The backfill displaces the slurry and forms a low permeability barrier. Permeabilities of less than 10^{-6} cm/s are typical for slurry walls. A practical depth limit is 60 feet below the ground surface for slurry walls constructed with a backhoe. Greater depth walls can be constructed by using specialized equipment.

The cost of the slurry wall depends on the amount of slurry needed to fill unforeseen large voids, caving of trench wall, the occurrence of large boulders, and the general nature of the material available for backfill. At the Rio Salado site some slurry loss and caving of the trench wall should be expected. A large percentage of the excavated material is probably suitable for backfill after the larger cobbles and boulders are removed.

Well Recovery

Well recovery is a third option for controlling seepage losses from Town Lake. A preliminary recovery design consists of 10 wells situated around the eastern perimeter of the lake. Each well captures a portion of the seepage flow, and discharges it back to the lake (Figure 4-5). One variation of this scheme is to collect the seepage in a network of pipes and deliver the combined flow to Tempe's Papago Water Treatment Plant. From the treatment plant, the water would be distributed for potable use.

Two types of wells are used for the recovery system. "Ranney" wells (Figure 4-6) can be installed west of Rural Road, where the depth of the wells are constrained by shallow bedrock. Ranney wells are constructed with horizontal casings placed radially from a central pumping facility. Conventional vertical turbine wells (Figure 4-7) are planned east of Rural Road where depth to hardrock permits deeper well construction.

Preliminary estimates of well yields indicate that four Ranney wells pumping at 1,000 gpm, and six vertical wells pumping at 1,500 gpm, may be sufficient to capture the infiltration losses. Additional hydraulic testing is required during preliminary design to refine the estimates of required well yield.

Figure 4-7 is a schematic diagram of a typical well installation, discharging directly to the lake. A conceptual cost estimate for the 10-well recovery system is $2.61 million plus approximately $200,000 per year to operate.

Additionally, a pipe network ranging in size from 10 inches to 36 inches in diameter may be used to collect the well flow and convey it to Tempe's Papago WTP north of the river. Figure 4-8 shows this concept. The

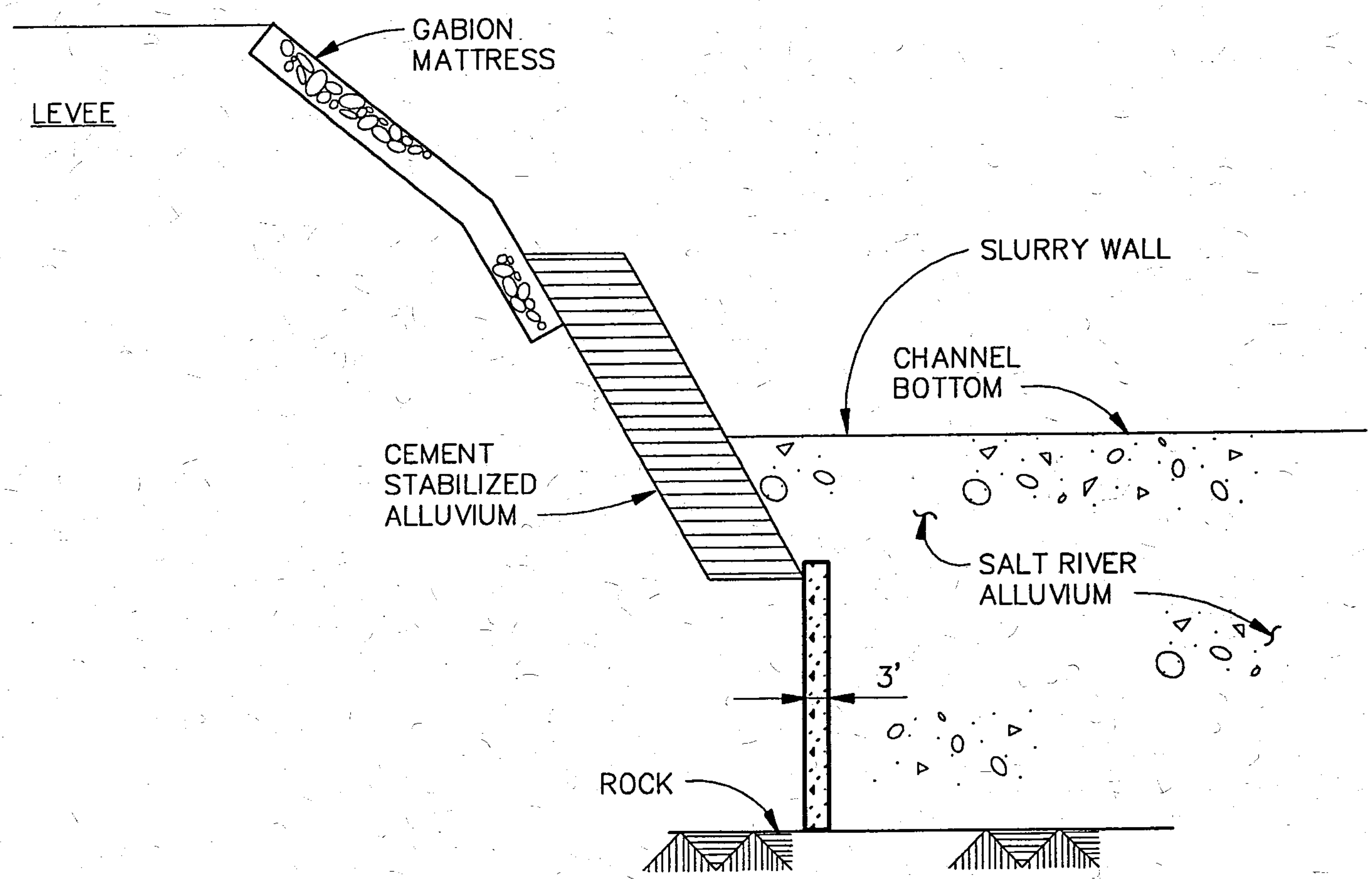


Figure 4-4
Slurry Cut-Off Wall

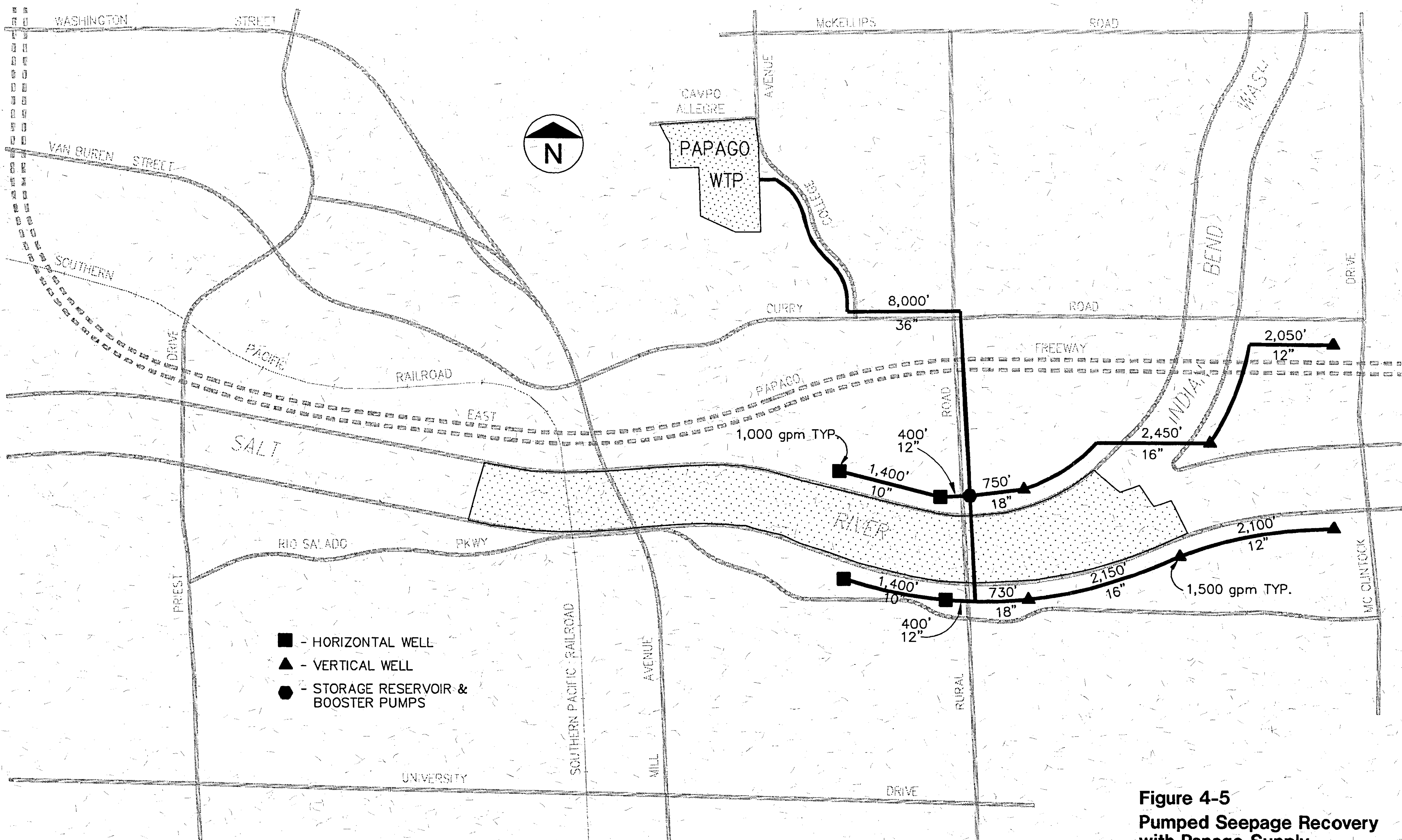


Figure 4-5
Pumped Seepage Recovery with Papago Supply

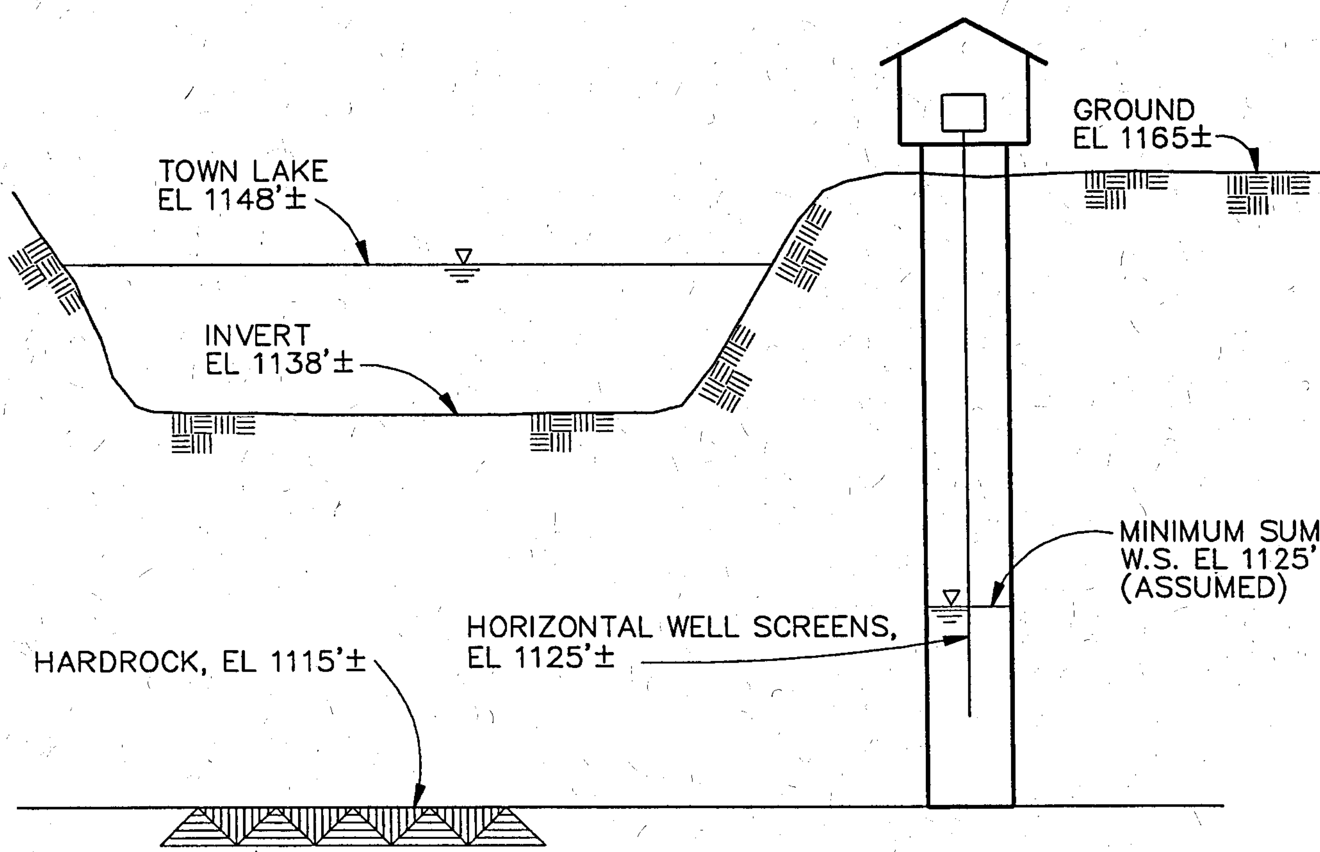


Figure 4-6
Horizontal Well

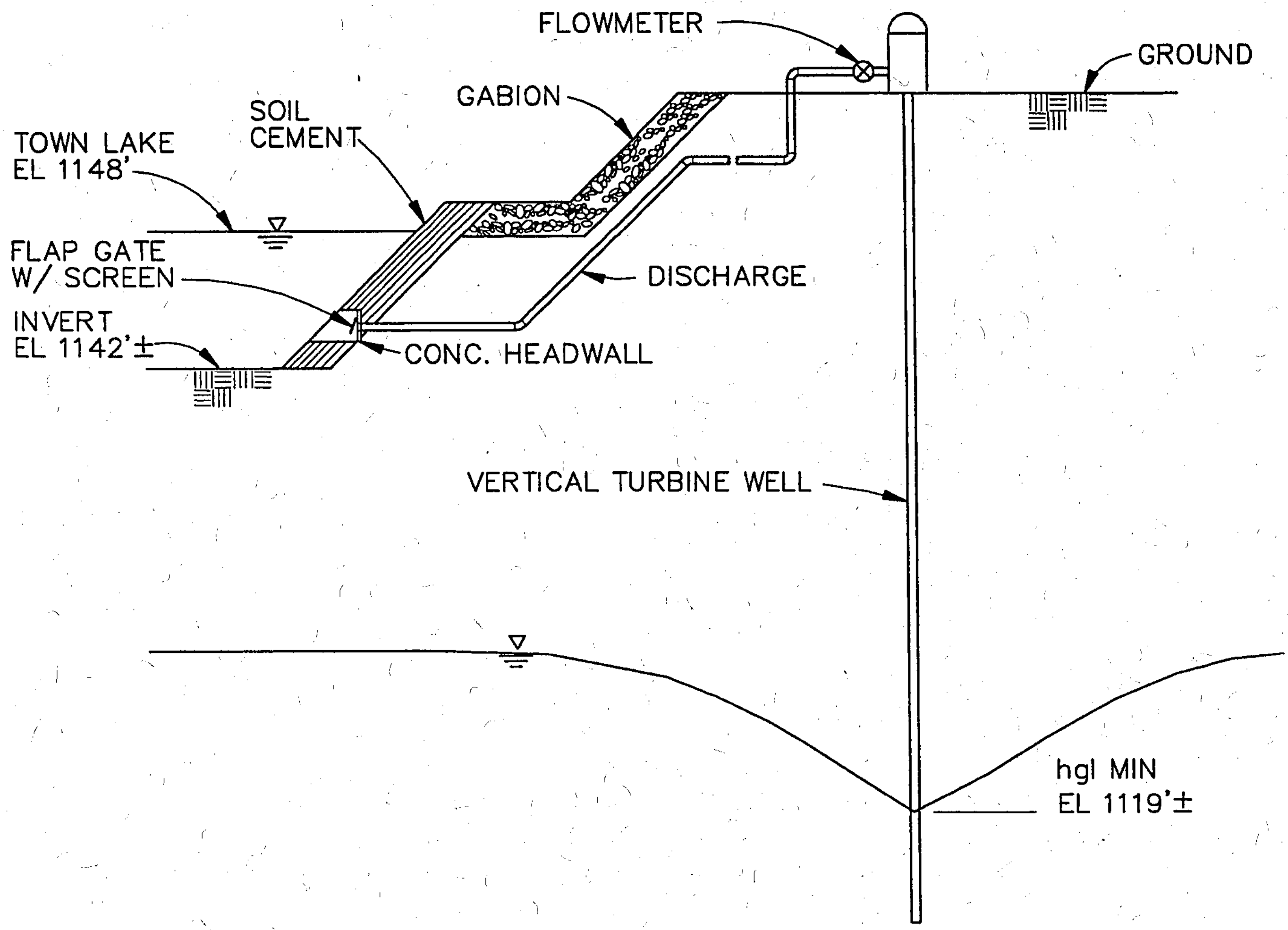


Figure 4-7
Vertical Well

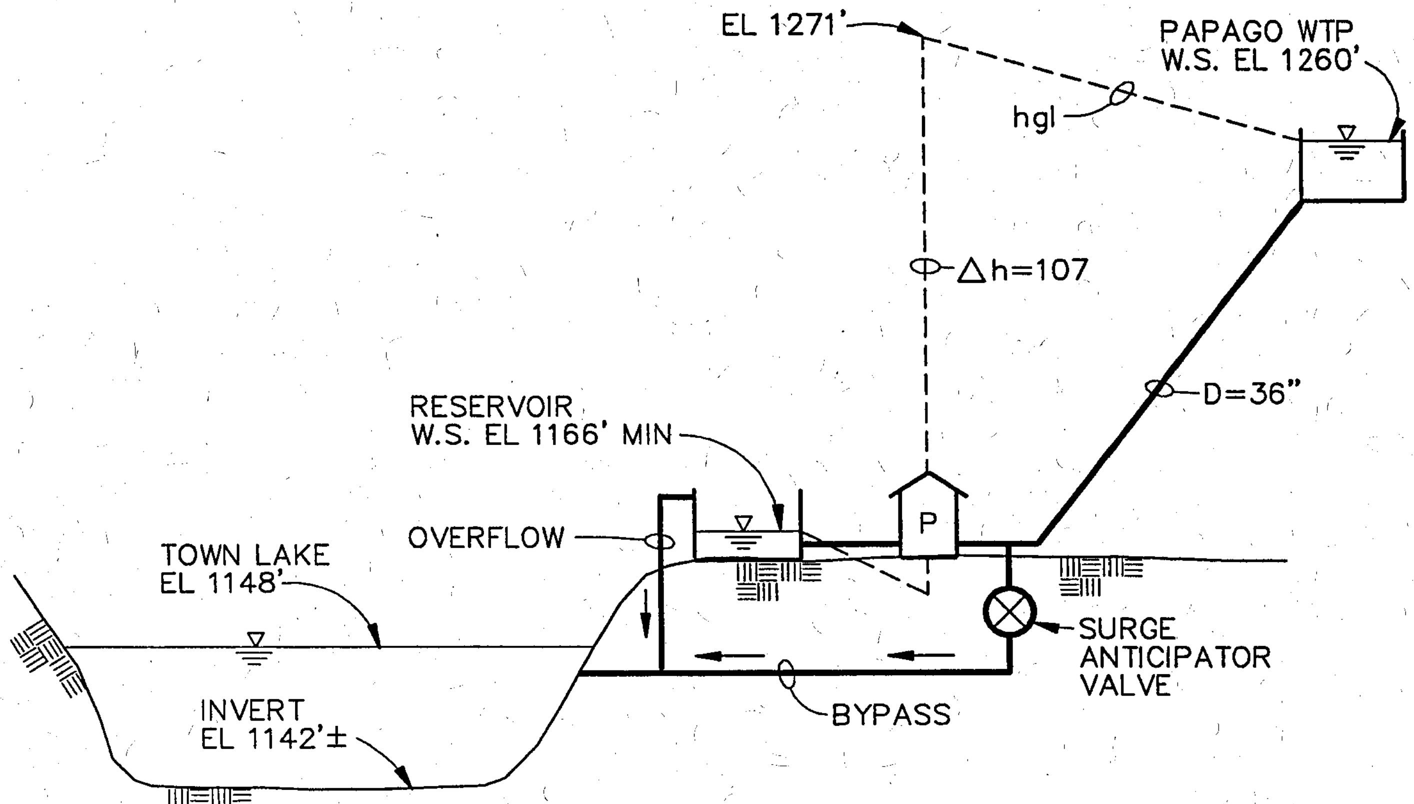


Figure 4-8
Papago WTP Supply Option

estimated cost of the collection and conveyance system is $5.48 million plus $170,000 per year to operate. While this well recovery alternative requires significantly less capital cost, the O&M costs are substantial. In addition, pumped recovery of seepage does not clearly meet the design objectives of isolating the lake from the local aquifer. APP permitting would be more demanding and ongoing monitoring requirements would be significant by greater than required for a cutoff/liner system.

Seepage Control Predesign Activities

Hydrogeologic Investigations

The choices between the seepage control options depend strongly on the distribution of hydraulic conductivity of materials beneath the lake. The current interpretation is that granite and sandstone materials of low hydraulic conductivity are present at depths of less than 40 feet in the area between Priest Drive and Mill Avenue. These conditions appear to make this area favorable for cut-off walls, if needed, and unfavorable for wells.

Of particular importance is the extent and hydraulic conductivity of breccia materials at depths of 35 to 100 feet in the area between Mill Avenue and McClintock Drive (TM8). These materials may not extend east of Rural Road but could extend to the half-way point between Rural Road and McClintock Drive where Indian Bend Wash enters the Salt River. If these breccia materials have high hydraulic conductivity, they would make cut-off walls ineffective, but may allow for effective vertical wells. Thus, the parameters needed to evaluate the effectiveness of cut-off walls (extent and hydraulic conductivity of the breccia materials) are the same as those needed to evaluate the effectiveness of vertical wells.

Testing of hydraulic conductivity will be needed for predesign as well as for permitting (unless the bed liner option is chosen) for the area east of Mill Avenue. The extent of the breccia materials can be investigated with dual-wall drilling as was done in Phase II with the addition of continuous coring in selected boreholes. Testing of hydraulic conductivity can be reliably accomplished with pumping tests. The following predesign investigations are necessary to further evaluate the pumped system of seepage control:

- Drill boreholes and install 2-inch blank and steel casing at 17 sites. These sites will primarily be in the riverbed between Mill Avenue and McClintock Drive.

- Install and pump five test wells in the breccia materials. During installation, continuous wireline core will be collected from three of the test well boreholes to depths of 200 feet. Coring will allow identification of the breccia materials as opposed to alluvial sand and clay fill of younger geologic materials. This evaluation was not conclusive based on drill cuttings alone from the Phase II drilling. Five 100-foot deep test wells should be installed with 8-inch steel blank and slotted casing. Each of the wells should be located adjacent to existing 2-inch piezometers and pumped at rates between 50 and 100 gallons per minute for either 12 or 24 hours. Water levels will be measured in selected wells during the pumping period for each well and also during an equal amount of time of recovery for each well. Four of the tests should include 12 hours pumping and 12 hours recovery, and 1 test should include 24 hours pumping and 24 hours recovery. Water pumped during the tests will be conveyed away from the pumping sites.

The data from this work will be used to:

- Resolve uncertainty about the character of the breccia materials as opposed to younger alluvial clay and sand materials.
- Refine the maps of extent of the breccia materials.
- Estimate hydraulic conductivity of the breccia materials.

Based on the above interpretations, groundwater model simulations will be conducted to refine estimates of lake seepage losses under conditions of cut-off walls or vertical wells and Ranney collector wells. The cost-effectiveness of each seepage control option can then be determined and compared to the cost of bed lining which has been assumed to be effective at seepage control.

The data collected in this work and the interpretations derived from the groundwater model simulations will provide the basis for predesign. Data from the two previous phases of work would also be incorporated.

Geotechnical Investigations

In addition to the hydrogeologic investigations described above, several specific geotechnical activities are recommended. These activities would provide detailed information on the characteristics of the river bed and banks. The results of these investigations will provide the data needed for final design of the seepage control structures. Specific activities are described for both cut-off walls and lake liners.

Slurry Cut-off Walls

- Review ADOT data from construction of river channel bank protection including as-built plans, geotechnical report, construction records, and photos taken during construction.
- Evaluate stability and seepage characteristics of the cement stabilized alluvium and the gabion mattress bank protection under different lake conditions.

- Excavate pits near the slurry wall alignment. The purpose of the test pits is to determine the construction excavation requirements for the slurry wall and obtain material samples for testing.

- Perform laboratory testing of recovered soil samples required for design of the slurry wall including grain size analysis, clay content, moisture content, and atterberg limits.

- Drill 5 to 10 cores through the cement stabilized alluvium bank protection to perform permeability test and evaluate seepage and strength characteristics.

- Explore area commercial, private, and City-owned borrow sources for fine-grained materials to include in the slurry mix.

- Perform slurry mix design tests using the material proposed for slurry wall construction.

Lake Lining

- Review ADOT data from construction of river channel bank protection including as-built plans, geotechnical report, construction records, and photos taken during construction.

- Explore area commercial, private and city-owned borrow sources for suitable low permeability material for lining and lining cushion sand.

- Perform a detailed evaluation of proposed lining systems alternatives. The evaluation will include degree of seepage control provided, material availability, cost, constructability, expected design life, and appearance. Preliminary construction details will be developed for the recommended alternative.

Seepage Control Costs

Detailed cost information for liners and cutoff walls was presented in TM8. Table 4-5 below summarizes that information and includes the well options presented earlier in this section.

Table 4-5
Seepage Control Costs

	Capitol Cost	Annual Cost
Slurry Wall/Liner Combination	$14,300,000	
Well Collection—Lake Return	2,610,000	200,000
Well Collection—Papago Delivery	7,130,000	370,000

Stormwater Management

Stormwater represents both a potential resource and a potential threat to the Rio Salado project. Stormwater is a source of additional water to the lake, but pollutant loads carried in runoff discharges may result in adverse lake water quality impacts. The implementation of stormwater management practices can enhance the resource value of runoff discharges while minimizing potential impacts to lake water quality.

As expected for a desert environment, the average storm volume, intensity, and annual number of storms in Phoenix are low compared to other parts of the nation. The average storm produces 0.42 inches of rain over 8.1 hours. In addition, the time between storm events is long, averaging 579 hours, or just over 24 days. Rainfall occurs 1.4 percent of all hours in Phoenix, based on the average storm duration and time between storms.

The major sources of urban stormwater that affect the Rio Salado site include:

- Indian Bend Wash
- Price Road Drain
- Tempe/Scottsdale
- Mesa
- Salt River Pima-Maricopa Indian Community

The two largest watersheds are Indian Bend Wash and the Price Road Drain. Indian Bend Wash drains a major portion of Scottsdale north of the Rio Salado site, while the Price Road Drain conveys stormwater from much of Mesa, and Chandler, south of the Salt River. In addition, 14 existing stormdrain outfalls have been located that discharge into the Salt River in the reach proposed for Town Lake.

Stormwater Quality

During the initial phase of the Rio Salado project, local data on stormwater quality from urban areas were obtained from the City of Tempe, ADOT, FCDMC, and the City of Mesa. The data are from grab samples collected during wet and dry weather conditions and from the Price Road Tunnel. In all, data from 111 samples were provided from 23 sites. Data from FCDMC included 6 wet-weather samples from 5 sites and 16 dry-weather samples from 6 sites. One dry-weather sample was provided by the City of Mesa. Subsequent to that TM additional samples of water were provided by Tempe staff for outfalls in the project area.

These data indicate that local wet-weather samples contain higher levels of total suspended solids, biological oxygen demand, organic nitrogen, nitrate, ortho-phosphorous, and copper, compared to median urban data reported by the EPA. The local wet-weather samples contain lower concentrations of lead and zinc, and nearly equal concentrations of total phosphorous compared to median urban data.

The pollutant concentrations from the dry-weather samples were typically lower than the wet-weather and tunnel samples. The data appear to indicate that dry-weather flows contribute a much smaller pollutant load compared to wet-weather flows. The data also suggest that detention storage in the Price Road Tunnel may provide some pollutant removal, especially for heavy metals. Copper, lead, and zinc concentrations from tunnel samples were below detection limits in dry-weather samples. One grab sample taken from the Price Road Tunnel, however, showed extremely high fecal coliform concentration, exceeding 90,000 CFU/100 ml. This sample may indicate high variability of stormwater quality from individual sources.

Stormwater Management Options

A range of stormwater management options were evaluated in TM5. The options included wet detention ponds, dry retention ponds, and bypass and diversion devices. Five alternatives were recommended for further consideration, as described below.

Alternative 1—No Action. Under this scenario, existing stormwater outfalls will continue to discharge directly to the Salt River Channel.

Alternative 2—Pump from the Price Road Drain. The Price Road Tunnel is an 18-foot-diameter inverted siphon located adjacent to Price Road. Discharges from this tunnel may pose a significant threat to the lake water quality during high flows. Two pumps with a combined capacity of 10 cfs have been installed in a permanent concrete structure near 5th Street and Price Road in Tempe. The pumps are currently used by ADOT to periodically drain the tunnel for maintenance and inspection. The pumps discharge stormwater into an existing 72-inch Tempe storm drain beneath the Price Frontage Road. The storm drain outfalls at the Salt River channel.

The affect of dewatering the Price Road Tunnel is to reduce the average annual quantity of stormwater to the Rio Salado site by approximately 8 percent, and the pollutant load from the source by about 45 percent. Alternative 2 would have an inconsequential effect on the average annual flow-weighted concentration of pollutants from all sources, but would reduce the single event loading from large events.

Alternative 3—Alternative 2 Plus an Upstream Dam and Retention Pond. The construction of an upstream dam and retention basin at the east end of the Rio Salado site would be beneficial for stormwater management. The dam could be used to impound or divert stormwater flows in the Salt River channel, including runoff originating from Indian Bend Wash, Price Road, Mesa, and the Salt River Pima-Maricopa Indian Community (SRPMIC).

Much of the pollutant load in urban stormwater is bound to sediment particles. Providing even small volumes of detention storage may, through settling, reduce pollutant loads downstream. A 5-foot dam would impound approximately 290 acre-feet of stormwater based on the width and slope of proposed channel improvements. The basin would remove roughly 80 percent of the pollutant loads from upstream sources, and reduce the average stormwater input to Rio Salado by approximately 28 percent.

Alternative 4—Alternative 3 Plus a South Bank Bypass. Alternative 4 includes the improvements discussed in previous alternatives, plus a bypass system to intercept and divert a portion of the water in Tempe's existing storm drains along the south side of the Salt River. A bypass system with a design capacity of twice the "average" storm would remove about 80 percent of the annual pollutant load. On that basis, a system for bypassing Tempe's existing outfalls requires a capacity of about 200 cubic feet per second (cfs).

Alternative 5—High Capacity Bypass. Alternative 5 is similar to Alternative 4. In this alternative, bypass pipe is constructed adjacent to the south hard bank to intercept and divert stormwater discharges around the lake. Alternative 5, however, includes a diversion/inlet structure behind the upstream dam to divert and bypass stormwater flows from the Salt River. Alternative 5 provides the greatest flexibility and control over stormwater input to the lake, at the highest cost.

Stormwater Management Plan (for Non-SRP Lakes)

Based on the selected location of Town Lake and information obtained subsequent to the completion of TM5, refinements to the original south bank bypass alternative (Alternative 4) were investigated and are presented in this section. The refinements include modifications to the south bypass concept, improvements to Dorsey and Miller outfalls near the upstream dam at Indian Bend Wash, and options for monitoring and/or mitigating discharges from the Papago Freeway (currently under construction). Figure 4-9 is a sketch of the existing and proposed stormwater features including the names, locations, and sizes of the affected outfalls. This figure was developed from Tempe drainage maps, CRSS channelization drawings, discussions with Tempe staff, and site reconnaissance.

A lake based on SRP supply, involving the lake as an urban SRP reservoir, may require greater isolation of existing storm drains from the lake than discussed below.

South Bank Bypass

A south bank bypass pipe is one option for intercepting and diverting discharges from south Tempe. The bypass extends from Rural Road to Grade Control Structure 4, collecting discharges from four existing outfalls: Farmer (72"), Ash (54"), Mill S. (24"), and Rural S. (66"). The bypass is approximately 6,700 feet long, assuming it is parallel and adjacent to the existing south bank improvements.

The bypass is designed to intercept the full design discharges from Farmer and Ash drains to avoid hydraulic grade conflicts near their outfalls. Examination of as-built drawings and discussions with Tempe engineering

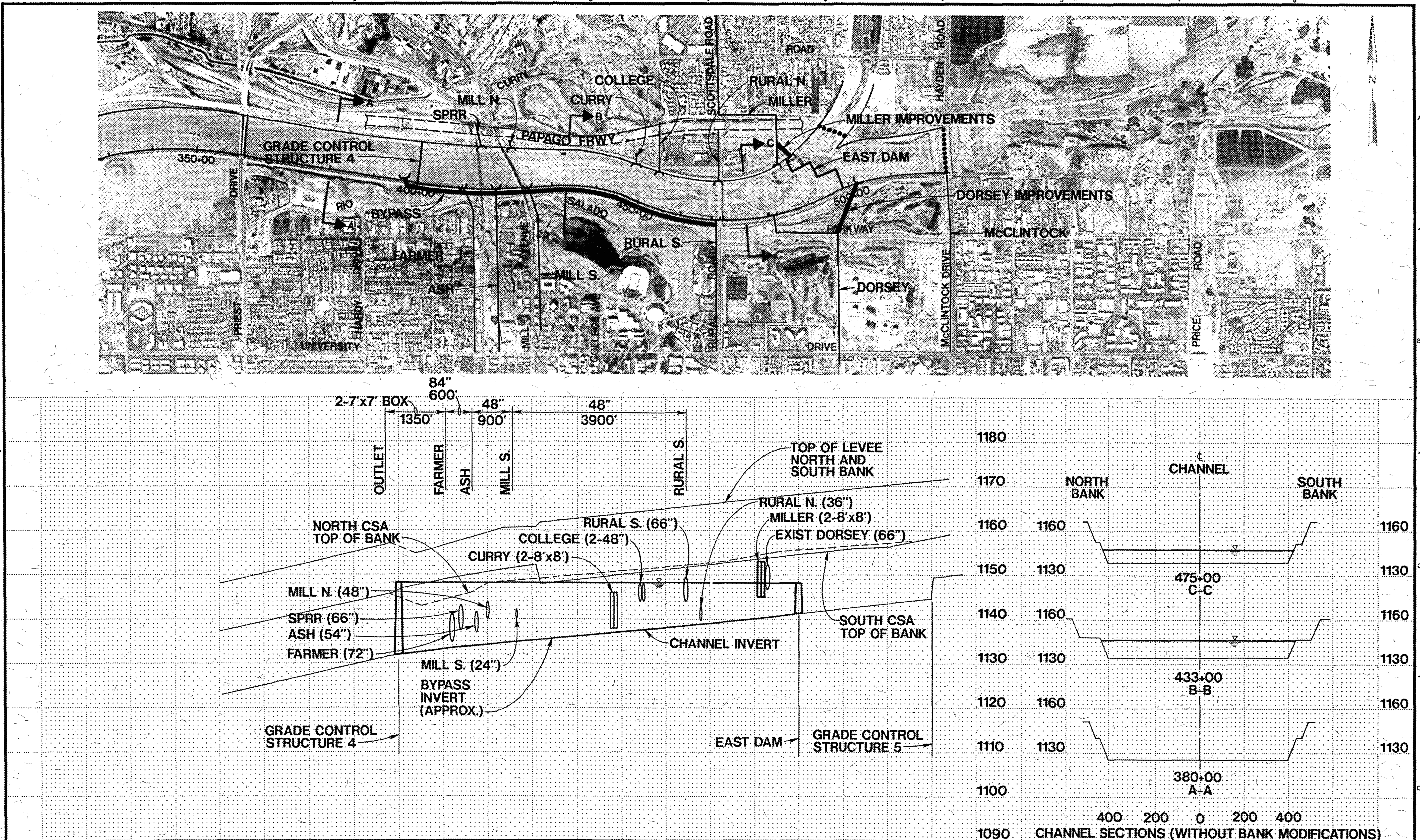


Figure 4-9
Stormwater Plan/Profile Detail

staff indicate that a proposed lake elevation of 1148.5 will submerge the Farmer and Ash outfalls, existing manholes on the outfalls, and existing and proposed street drains from Rio Salado Parkway, between Hardy Drive and College Avenue. The bypass extends east from Ash Avenue to Rural Road, diverting discharges from Mill South and Rural South outfalls. Diverting discharges equivalent to twice the average storm runoff will reduce the average pollutant load to the lake by about 80 percent as discussed in TM5.

The size of the proposed south bank bypass is based on estimates of storm discharges from south Tempe. Data relative to the actual and/or design discharges were not available, so discharges were estimated from outfall design drawings and statistical hydrology data presented in TM5. The capacities of Farmer and Ash drains are estimated at 355 and 195 cfs, respectively, assuming a full-flowing pipe. Bypass flow rates (twice the average storm discharge) from the Mill South and Rural South outfalls have been estimated at 6 cfs and 47 cfs, respectively. The bypass flow rates are based on an estimated average storm runoff of 97 cfs from south Tempe (TM5), distributed by area among all outfall pipes.

The capacity of the bypass is also a function of its slope. Tabulated pipe sizes (Table 4-6) assume a profile grade of 0.00146 ft/ft, which is equivalent to the slope of the river channel and slopes dictated by elevations of the existing outfalls. Neither the actual ground surface profile, nor the presence of conflicting structures, utilities, easements, etc. were evaluated when selecting the design slope. Additional information is necessary as part of preliminary design.

Table 4-6
Conceptual Bypass System
(100 Percent of Ash and Farmer Drains)

Segment	Length (ft)	Flow (cfs)	Slope Size (ft/ft)	Interceptor Size
Rural S. to Mill S.	3,900	47	0.00146	48" dia
Mill S. to Ash	900	53	0.00146	48" dia
Ash to Farmer	600	248	0.00146	84" dia
Farmer to G.C. #4	1,350	603	0.00146	2.7'x7'box

Significant cost savings may be realized by diverting only the first-flush flows from Farmer and Ash drains, estimated at 31 cfs and 54 cfs, respectively. Under these conditions, the bypass would consist of a 60-inch pipe between Farmer and Ash and a 72-inch pipe west of Farmer, but require reconstruction of existing manholes and street drains on Rio Salado Parkway. The cost and feasibility of these drainage modifications was not evaluated.

Diversion structures are required at the junction between existing drains and the proposed bypass. The diversion structure intercepts low flows while providing capacity for discharge of high flows directly to the lake. Figure 4-10

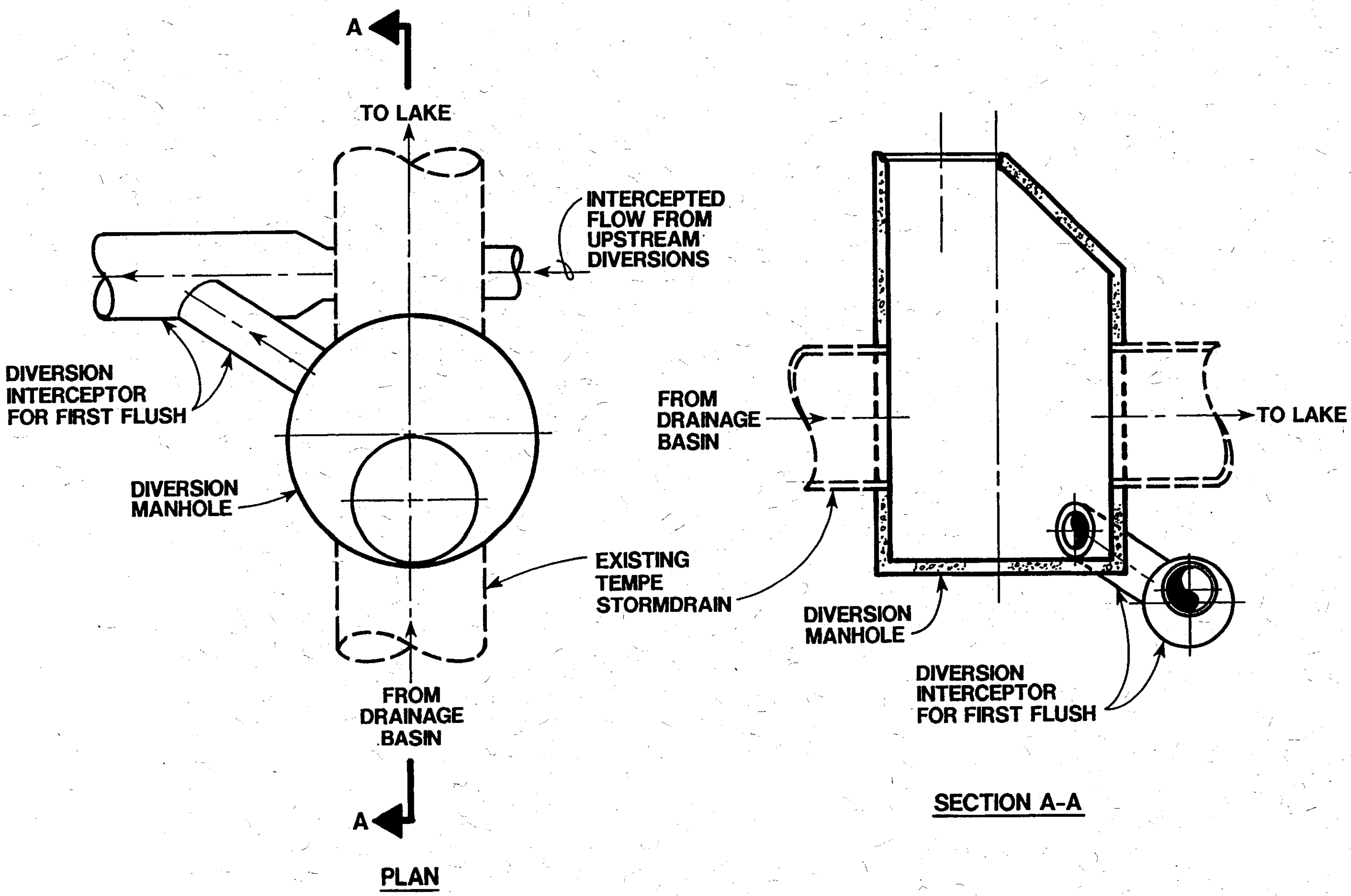


Figure 4-10
Stormwater/Diversion Structure

is a schematic of a typical diversion structure. Cost estimates assume that two structures are provided, one at Rural South, and the other at Mill South drains.

Dorsey Outfall Revisions

The Dorsey outfall is a 66-inch drain, outletting in the south bank, midway between Rural Road and McClintock Drive. The existing drain currently discharges to an unlined, open channel near the Karsten Golf Course at Rio Salado Parkway. The channel flows west approximately 1,100 feet, before turning north and passing through the south river bank improvements.

Reconstruction and realignment of the Dorsey outfall may be one option for eliminating direct discharges to the lake from the Dorsey drain, and reducing the size and length of the bypass pipe. Revisions to the Dorsey outfall include the installation of roughly 1,300 feet of new 66-inch pipe and a new outlet and headwall through the south bank (see Figure 4-9). Improvements to the Dorsey drain would result in discharges to the Salt River east of the proposed upstream dam. A preliminary evaluation of the existing and proposed outfall elevations indicates that the improvements are hydraulically feasible, however, land profiles, ownership, and the potential for conflicting structures or utilities were not investigated.

Miller Outfall Revisions

The 66-inch Miller outfall drains most of urban Tempe north of the Salt River. The Miller outfall currently discharges to the Salt River about 300 feet downstream of the proposed east (upstream) dam. Reconstruction of the Miller outfall was recently completed in conjunction with drainage improvements for the Papago Freeway. The outfall was enlarged from a 66-inch pipe to a double 8'X8' box culvert to accommodate the freeway drainage and providing additional capacity for Rural Road/East Papago Interchange.

One option for eliminating discharges from most of urbanized north Tempe and the Papago Freeway between Scottsdale Road and Indian Bend Wash (IBW) is the reconstruction and realignment of the Miller outfall. Improvements would include construction of a new 600 foot long open channel to direct runoff east of the upstream dam. A new double box culvert, headwall, and outlet is required through the north bank soil cement.

North Tempe and Papago Freeway

Five additional outfalls drain portions of urbanized north Tempe, the Papago Freeway, and undeveloped areas of Papago Park into the proposed Town Lake. The outfalls include the Southern Pacific Railroad (66"), Mill N. (48"), Curry (2-8'X8'), College (2-48"), and Rural N. (36").

Runoff from the Papago Freeway will be discharged to Town Lake through three outfalls. Between Scottsdale Road and IBW, drainage is discharged to the Miller Outfall discussed earlier. Freeway Drainage in the vicinity of College Avenue is discharged through one of the two 48-inch outfalls at College. Runoff west of College, in the vicinity of Mill Avenue, is discharge through the 48-inch Mill N. drain.

Runoff from the freeway corridor will likely be of poor water quality, particularly high in petroleum and rubber hydrocarbons, and suspended solids. However, the drainage area, and hence the volume of runoff from a typical storm is small. Assuming an approximate drainage area of 55 acres (Mill Avenue to IBW), a runoff coefficient of 90 percent, and an average storm of 0.42 inches (TM5), the Papago Freeway would generate 1.7 acre-feet of runoff to Town Lake. Runoff from the Papago Freeway represents about 3.5 percent of the total volume runoff from urbanized Tempe (51 acre-feet in TM5) during an average storm.

Options to prevent direct freeway discharges to the lake include the construction of a bypass device similar to the south bank improvements, or the construction of retention ponds on the north bank to capture low flows. Excess land acquired by ADOT may be available for basin construction between the freeway and Town Lake. A third option is to monitor the quantity and quality of freeway runoff, deferring construction options to a later date. The value of intercepting or bypassing freeway discharges depends on the intended use of the lake. More intensive uses (e.g. full or partial body contact, fishing, or urban SRP reservoir) may ultimately require freeway drainage improvements.

Stormwater Control Predesign Activities

The recommendations for stormwater control presented above are based on limited information on the design and performance of the existing stormdrain systems, the effluent water quality, right-of-way and utility constraints, and other design issues. During predesign these issues should also be investigated in greater detail. Changes that may be caused by the construction of the Papago Freeway should also be evaluated. The table, below, presents estimated stormwater control costs.

Table 4-7
Stormwater Control Costs

South Bank Bypass	$2,260,000
Dorsey Outfall	480,000
Miller Outfall	570,000
Subtotal	$3,310,000

Stormwater Management for SRP-Supplied Lake

Should the City wish to pursue implementation of a lake based on using the lake as an urban reservoir in the SRP canal system, further consideration is necessary of the potential for the complete removal of existing storm drains from the Salt River in the lake segment.

In general, SRP does not allow storm drains to discharge into the canal system. The degree to which this isolation must occur in the case of Town Lake would require further definitive agreements with SRP.

For purposes of this report, an additional level of isolation (compared to lakes based on using reclaimed water) is assumed. Compared to the stormwater control plan described above, greater isolation of stormwater is assumed to require a larger capacity diversion/bypass system on the south side of the lake, and a diversion/bypass system on the north side of the lake. These additional and greater capacity diversions have not been developed in detail, but the design would entail capacity to bypass design storm (flow) events equivalent to a return frequency similar to that of the flow events from Indian Bend Wash that can be retained above the upstream dam.

For planning purposes these additional and higher capacity storm bypass facilities have been estimated to cost $11,420,000 vs. $3,310,000 for the non-SRP-supplied lake.

Lake Shoreline

Construction of a consistent lake shoreline is appropriate for aesthetics, water quality maintenance, and safety.

As currently constructed, the Salt River bank protection has slopes of 1½ to 1 for the CSA and 3 to 1 for the alluvium plating (CSA cover). This could pose some difficulty in egress from the lake in circumstances of unauthorized swimming or boating emergency. In addition, the top elevation of the CSA follows the hydraulic grade line of the river channel (sloping downward from east to west) while the water surface of the lake will be uniform. The result is a varying point of interface between the CSA plating and the lake surface. The CSA plating is designed to be sacrificial at larger Salt River flow rates. Thus, after some period of years and Salt River flow events, the interface point between the lake surface and the shoreline could migrate closer to the CSA. This would present a situation where emergency egress from the lake would require an exhausted swimmer to climb a 1½ to 1 slope. This would be difficult and would pose a serious safety concern.

Various alternatives for creating shoreline retreats were developed and presented in TM8, the Town Lake Feasibility Study. These concepts have evolved into consideration for a continuous shoreline headwall of the

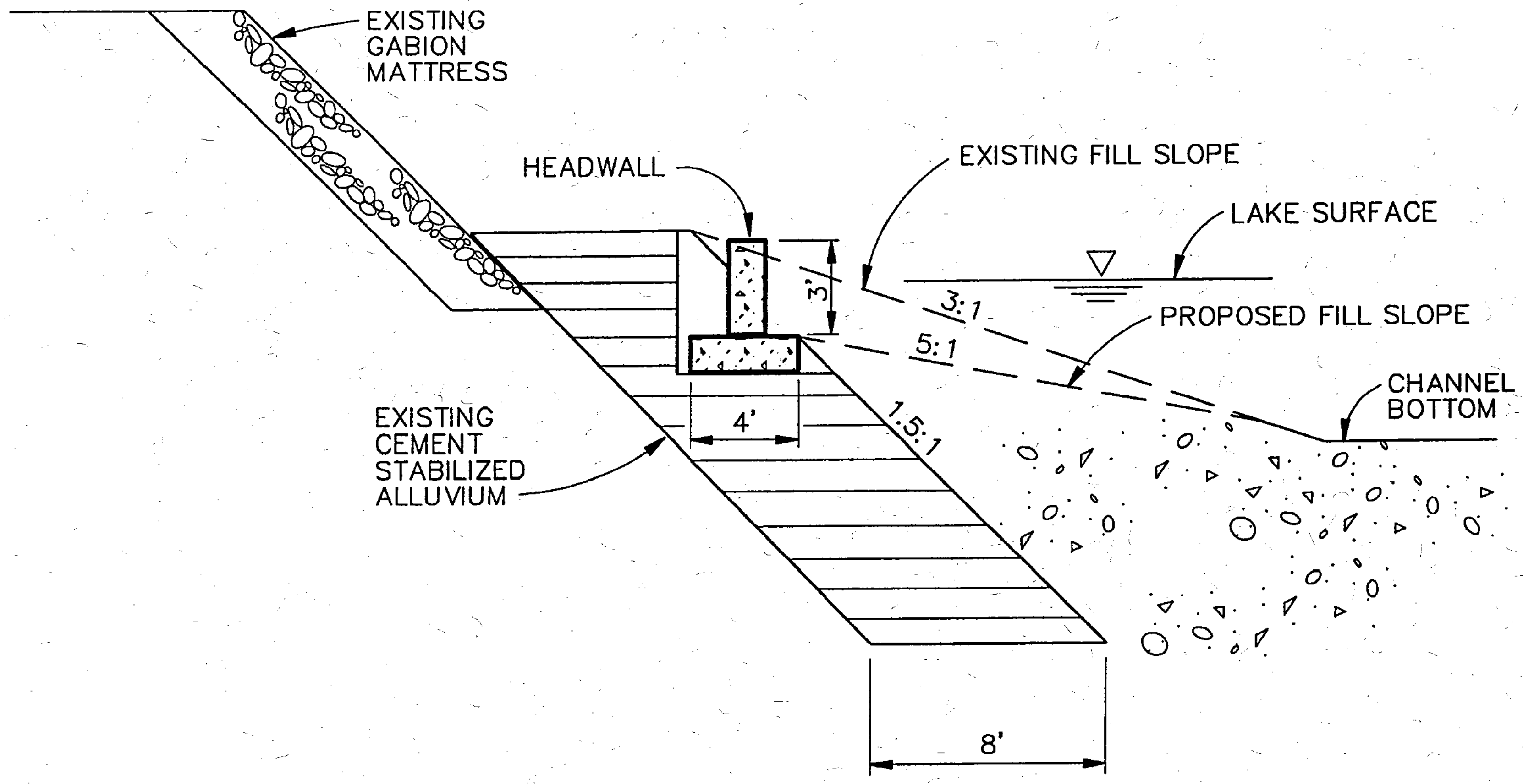


Figure 4-11
Lake Shoreline Headwall

configuration shown in Figure 4-11. This configuration provides a permanent interface for the "wet" shoreline and "dry" public and private developments that will be constructed along the perimeter of the lake. In addition, the consistent depth of water and underwater ledge should provide a stable area for further egress from the lake. The final design of the wall should consider steps and ladder arrangements for escape to a dry area above the lake. This underwater ledge area is similar to designs of other urban lakes, but much narrower. The lake should be generously signed with prohibitions against wading, swimming, and particularly diving.

The shoreline headwall will require removal of varying quantities of plating, cutting into the CSA, construction of the concrete headwall, and CSA backfill. The estimated cost is $2.37 million.

Section 5

Source Water Options

Potential Sources

The potential sources of water for the Rio Salado Town Lake are reclaimed wastewater, the Salt River, Salt River Project water, stormwater, and groundwater. Reclaimed water is the most probable source of water, either through direct reuse or indirectly through water exchanges. Direct reuse of reclaimed water occurs in supply alternatives that physically pipe the reclaimed water from the City's water reclamation facilities to Town Lake. Indirect reuse of reclaimed water occurs in supply alternatives that use aquifer storage and recovery (ASR) technology to transform the water in legal and technical terms from reclaimed water into groundwater. Indirect reuse is also considered in alternatives that are based on trading reclaimed water for other physical sources of water. Considering the options for direct and indirect reuse of reclaimed water, the potential sources of supply include the Salt River, Salt River Project, Central Arizona Project, urban stormwater, reclaimed water, and groundwater.

Each potential water source has unique considerations related to quantity and quality. Source water considerations include reliability, average annual volume, seasonal supply fluctuations, water quality, and legal and institutional issues.

All supply options included in this study are based on filling the lake by capturing receding Salt River flows. In other words, following any Salt River flow event that requires the lowering of the inflatable dams, the dams would be inflated to capture a pool of water behind the dam at the conclusion of the river flow event. Other sources of water considered herein are intended to serve as makeup water for lake evaporation and seepage losses, supply to other water features such as wetlands, and irrigation demands.

Source water considerations follow for each potential source. TM3 provides further information regarding these sources. In cases of factual differences, the information that follows supersedes that presented in TM3.

Salt River

Runoff in the Salt River has a high degree of annual and seasonal variability. Occasional, beneficial flows may be expected during winter months, but excess runoff is unusual during the summer. Beneficial flows are defined here as those which do not require the lowering of the dams. Beneficial flows typically range from zero in summer to as much as 760 ac-ft per month during the winter. Annually, the beneficial flow may be as high as 4,300 ac-ft.

Because of an unusually wet winter and ongoing modifications to Roosevelt Dam, the Salt River Project (SRP) has released more water than usual from Granite Reef Dam into the river during winter 1991 and spring 1992. The largest spills have been over 13,000 cfs, although the average has been approximately 2,000 to 3,000 cfs. Releases could continue into the summer of 1992. As construction continues on the dams over the next few years, releases may continue to be higher and more frequent than historical records would suggest.

Salt River water quality is generally high except for periodic high fecal coliform count. Historically, metals and nutrient concentrations have been low. Except when intercepted by infiltration and evaporation losses in the riverbed, spills over Granite Reef Dam are a direct source of water to Rio Salado.

These spills, considered "run of the river" water, are appropriable surface water supplies. They may be passed through the lake system as dilution and circulation without appropriation, however, capture of the water for consumptive use is subject to the appropriation process. This type of activity would likely result in protests by senior downstream appropriators.

Salt River Project Water

The direct use of SRP water is not a viable option since Rio Salado development is likely to occur outside SRP boundaries. Attractive indirect uses include an in-line reservoir or a water trade via the exchange of recharge credits associated with reclaimed water.

The quality of SRP water is reported to be similar to Salt River surface water, and assumed to be of identical water quality at its source. Surface water in SRP canals is frequently augmented with groundwater from SRP wells. The groundwater is frequently higher in TDS and nitrate levels than surface water. Agricultural return flows frequently contain detectable quantities of nutrients from fertilizers and toxics from pesticides. In addition, the quality of SRP water is affected by the conveyance system. Warm, shallow water moving slowly in open canals provides an opportunity for algae growth, aquatic weeds, and other water quality transformations.

Central Arizona Project

Central Arizona Project (CAP) water is considered a long-term potential source of water to Rio Salado. The City of Tempe has an annual CAP allocation of 4,315 ac-ft which equates to a monthly supply of 475 ac-ft. The quality of CAP water is among the highest of any potential source. It is not likely however that CAP water, without the construction of a dedicated pipeline, could reach the lake without mixing with SRP water. Thus the actual water quality of this "traded" water source would equal that of SRP water.

Urban Stormwater

Stormwater is a nuisance water source. The timing and volume of runoff is not easily controlled, and the water quality is poor. Runoff is expected to be high in suspended solids, nutrients, and metals and some organic chemicals.

Very little reliable data exist regarding the relationship between rainfall and runoff for small storm events in the Phoenix metropolitan area. Most available hydrologic information is for large storm events. In TM3, some simple watershed parameters were estimated for use in predicting average annual runoff. The calculations were cursory estimates at best; however, they provide order-of-magnitude predictions of the volume of urban runoff that may impact the lake. The estimated potential annual runoff volumes range from about 1,000 ac-ft. to 10,000 ac-ft.

Urban runoff is the least desirable source of water for the Rio Salado Project, both in terms of quantity, timing of flow, and quality. Water quality from storm drains varies widely. Existing data are based on single grab samples. The data show a large variability. No flow measurements were taken, so discharge rates and volumes at the time of sampling are not known.

In general, at the measured storm drains, discharges are high in suspended sediments (TSS) and associated metals. Nutrients (N and P compounds) are high as well, rivaling secondary wastewater effluent characteristics. Toxic organic compounds and pesticide residues have not been detected in Phoenix and Tempe area storm drains. Average storm drain metal values exceed ADEQ criteria for the protection of aquatic and wildlife for cadmium, copper, lead, and zinc. Arsenic levels could also be a problem.

Reclaimed Water

Reclaimed water is one of the most reliable sources of water to Rio Salado. The potential supply is assumed to be nearly constant, at up to 3,360 ac-ft per year (for the existing Kyrene WRF at 3.0 mgd), or 6,720 ac-ft per year for the proposed North WRF. The Kyrene WRF was designed for future expansion from 3.0 to 6.0 mgd. The actual flow being diverted to the Kyrene WRF during 1992 will average 2.6 to 2.8 mgd. Population growth or additional interceptor sewer diversions will be necessary to achieve 3.0 mgd or greater flows at the Kyrene WRF. Direct reuse of water reclaimed at these facilities has the disadvantage of having high nutrient levels (primarily phosphorus) that will promote algae and aquatic weed growth in the lake. If not under intensive management, the lake could develop seasonal aesthetic and odor problems. The supply alternatives based on direct reuse include considerations of additional treatment processes that could be constructed to reduce phosphorus concentrations in the reclaimed water, thus diminishing the potential aesthetic problems that could result from direct reuse.

Limited information is available regarding the phosphorus concentration in the Kyrene WRF reclaimed water during the plant's first three months of operation. Additional sampling and testing is ongoing. For planning purposes this report assumes that effluent phosphorus concentrations without additional treatment, would be in the range of 4 mg/l. A range of 2 to 5 mg/l has been observed by City staff.

Groundwater

The direct use of groundwater is not a viable option due to conservation constraints imposed by the Groundwater Management Act. Groundwater may be used indirectly through recharge and recovery operations or the exchange of recharge credits involving reclaimed water. As much as 4,603 ac-ft per year may be available from Tempe's existing wells (384 ac-ft/month). Groundwater is generally of high quality in terms of nutrients, especially phosphorous, compared to other sources. TDS levels, an indicator of inorganic contents, are moderately high.

Water Quality Issues

Detailed evaluations and water quality modeling were presented in TM7. Stormwater quality was discussed in TM5. The results of those studies are summarized and explained where appropriate in this section.

The water supply options described above vary in water quality. This section will summarize the potential effects on Town Lake of using the following sources of water:

- SRP water from the Tempe canals.
- Reclaimed wastewater from the Kyrene WRF. The proposed North WRF water quality is assumed to equal the Kyrene WRF.
- Kyrene WRF water following several levels of advanced phosphorus removal.
- Recovered groundwater. (The indirect use of recovered water for Town Lake would be made possible by recharge and recovery or by trading of reclaimed water for SRP water.)

The critical difference between these categories of water is in their probable effect on the growth of free-floating algae and/or attached aquatic weeds. Fertilization potential, as measured by the concentration of nitrogen and phosphorus compounds, varies greatly among the sources. Direct use of reclaimed water has the highest fertilization potential followed by reclaimed water, and then SRP water. Groundwater has the least potential for stimulating adverse levels of aquatic plant growth. Average water quality for these sources is given in Table 5-1.

Table 5-1
Source Waters for Town Lake

Parameter	Average Water Quality			
	Reclaimed Water[a]	SRP Water[b]	Recovered Water[c]	Storm-water[d]
Total Suspended Solids (TSS) (ppm)	2.00	15.80	0.01	43.18
Total Dissolved Solids (TDS) (ppm)	790	338	1,000	533
Total Phosphorus (TP) (ppm)	4	0.137	0.020	0.160
Total Nitrogen (TN) (ppm)	10	4.5	5	4,840
F. Coliform (cfu/100 mi)	2.2	10	0.010	1,642
Arsenic (ppm)	<0.005	0.003	0.009	0.005
Cadmium (ppm)	<0.005	<0.001	0.002	0.001
Chromium (ppm)	<0.010	0.005	0.010	0.014
Copper (ppm)	<0.010	0.008	0.025	0.025
Lead (ppm)	NA	0.003	NA	0.003
Mercury (ppm)	<0.0002	<0.0001	NA	<0.001
Selenium (ppm)	<0.0005	<0.001	0.004	<0.001
Silver (ppm)	NA	<0.001	NA	0.003
Zinc (ppm)	0.038	0.014	0.061	0.160

[a] Kyrene WRF projected quality
[b] From City of Tempe intake data
[c] From City of Tempe well data
[d] As developed in TM 5

NA = Data not available

In the hot climate of the low-elevation desert southwest, lakes have a long growing season and undergo extended periods where the warm surface water forms a stable layering in the lake known as stratification. During the summer stratified period, the natural cleansing processes of lake mixing and oxygenation are blocked from the deeper, cooler portions of the lake. As a result, nutrient enhancement of algal growth will create problems of oxygen depletion in deeper water, increased nutrient and metals release from the sediment (fueling further growth and possible toxicity), and occasional summer fish kills.

The average summer growing season conditions of the lake when filled with different source waters can be compared in Table 5-2. The water quality effects of stormwater additions from an average storm are included for each option as well. The ratios of available nitrogen and phosphorus in the source waters and stormwater indicate that algal growth for Town Lake will be most strongly controlled by the availability of phosphorus. Higher water clarity projected for a source-water alternative is strongly associated with a decreased potential for oxygen depletion and decreased probability of blue-green algae dominance. These factors taken together demonstrate a significant range in projected quality of lake water based on the different source water alternatives. The empirically derived relationships used to develop these water quality predictions are based on lake morphometry and nutrient inputs.

Additional treatment processes for greater levels of phosphorus removal could be applied to the Kyrene WRF. A high degree of phosphorus removal would be required to achieve noticeable lake benefits, however, significant direct benefits to lake water quality could be achieved by rigorously pursuing this option (Figure 5-1). Significant Kyrene WRF phosphorus removal (below 0.5 ppm total phosphorus) would yield comparatively high quality water as a primary source for Town Lake (i.e., compare Figure 5-1 with Secchi Depth values in Table 5-2). Noticeable improvements in lake water quality could be further expected if effluent phosphorus concentrations were reduced to below 0.5 ppm (Figure 5-1).

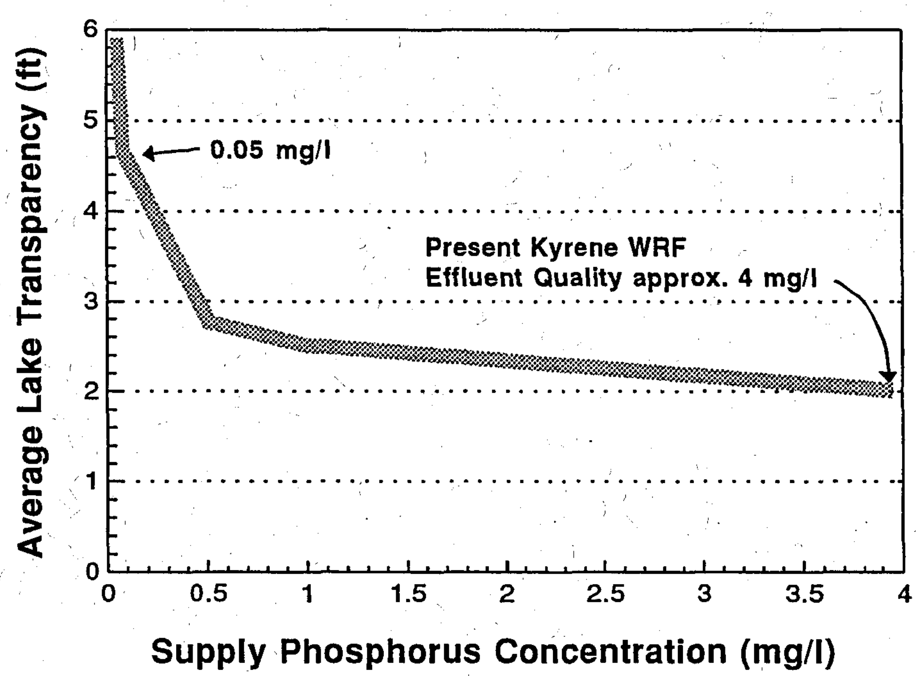


Figure 5-1
Lake Transparency Related to Supply Water Phosphorous Concentration

Table 5-2
Steady-State Conditions and Stormwater Effects
Effect of Different Source Waters

Parameter	Reclaimed Water	Reclaimed Water Plus Stormwater[c]	SRP Water	SRP Water Plus Stormwater[c]	Recovered Water	Recovered Water Plus Stormwater[c]
TSS (ppm)	0.020	5.9	0.1	6	<.001	5.8
TDS (ppm)	790	742	338	374	1000	913
Chlorophyll a (ppm)	0.304	0.269	0.041	0.055	0.017	0.038
Transparency (ft)	1.8	1.9	3.7	3.4	6.2[a]	4.3
Prob. anoxia	0.99	0.86	0.72	0.55	0.29	0.40
Prob. BG dominance	0.99	0.79	0.60	0.46	0.10	0.25
TP (ppm)	0.364	0.322	0.052	0.068	0.013	0.036
TN (ppm)	6.077	5.811	2.735	3.092	3.038	3.339
F. Coliform (cfu/100 ml)	2.2	261[b]	10	267[b]	0.010	259[b]
Arsenic (ppm)	<0.005	0.002	0.001	0.002	0.006	0.004
Cadmium (ppm)	<0.005	0.002	<.001	0.000	0.001	9,991
Chromium (ppm)	<0.010	0.005	0.001	0.003	0.007	0.006
Copper (ppm)	<0.010	0.008	0.002	0.006	0.018	0.014
Lead (ppm)	NA	0.001	<.001	0.001	<.001	0.001
Mercury (ppm)	<0.0002	0.000	<.001	0.000	<.001	0.000
Scicnium (ppm)	<.005	0.002	<.001	0.000	0.003	0.002
Silver (ppm)	NA	0.001	<.001	0.001	<.001	0.001
Zinc (ppm)	0.029	0.054	0,008	0.036	0.047	0.057

[a] Exceeds required clarity, MCHD swimming regulations
[b] Exceeds ADEQ full-body contact criteria
[c] New steady-state conditions following the addition of water from an average storm

Transparency

Transparency is used in this report to characterize the aesthetic and recreational values of Town Lake. Transparency is measured scientifically with a Secchi disk. A Secchi disk is a white and red round disk that can be placed at varying depths in the lake. If the disk is visible at a 2-foot depth, the interpretation is that the transparency is 2 feet. Transparency from the public's perspective will be related to the general clarity of the water and the presence or absence of algae. A lake with an average transparency of 6 feet, for example, may be perceived to be of higher quality than a lake with 1 foot of transparency.

Because of the empirical methods that are available to predict transparency, only average values are reported for equilibrium conditions. It should be expected that most water quality parameters will vary and that deviations from average transparencies will occur. If these deviations in transparency are on the order of 1 to 2 feet or more, then a lake with an average transparency of 2 feet could be expected to have near zero transparency at times.

The water quality of urban lakes in the Phoenix/Tempe area is variable but falls within the ranges shown in Figure 5-1 and Table 5-2. Area lakes, although usually smaller than Town Lake and of different morphometry, are maintained with the same variety of source waters (reclaimed effluent, recovered groundwater, etc.) under consideration for the Rio Salado project. Based on local experience, and as supported by water quality projections, the City can expect that Town Lake will experience one to several feet in transparency during the summer months with water quality variability influenced primarily by the differences in source water. The beneficial uses of Town Lake will be influenced by the water supply alternatives in that swimming will be inappropriate for any of the alternatives and boating and fishing could be influenced by lake fertilization and additions of fecal coliform bacteria. Stormwater may temporarily boost fecal coliform counts in Town Lake and preclude full body contact recreation. Aquatic weed or algae growth in a lake filled with reclaimed effluent could impair boating or fishing activities.

Local shallow, urban lakes, such as Town Lake experience a long growing season with the proven potential for objectionable growths of algae and aquatic weeds. Water quality is likely to be seasonally predictable, with the greatest plant density and worst water quality during the summer. However, the timing of specific water quality problems, such as floating mats of blue-green algae, shorezone growths of filamentous algae, mats of submerged aquatic weeds, or severe oxygen depletion (and resultant fish kills) cannot be accurately predicted. In the Phoenix area, these types of water quality problems are likely to occur with little warning. Effective management for Town Lake must be based on a continuous water quality monitoring program and response plan coupled with an active, ongoing management program.

Predesign Activities

The most critical predesign activities associated with water quality are to complete the laboratory tests necessary to evaluate the level of advanced phosphorus removal potentially available for the Kyrene WRF and to acquire more complete phosphorus data, in general, for the source waters. Groundwater from wells potentially available for recovered water supply should be tested for phosphorus and nitrogen concentrations. Kyrene WRF phosphorus levels are also imprecisely known and more effluent samples should be tested for phosphorus content.

Section 6
Supply Alternatives

During the summer of 1991 the study team developed 13 scenarios for delivering the principal sources of water to the lake. The principal sources were the existing Kyrene WRF, the proposed North WRF, and SRP canals. Information on these alternatives was presented in Workshop Two. At that time, City staff proposed two additional scenarios for water supply.

The 15 scenarios were developed from the three primary supplies, including options of aquifer recharge and recovery using surface spreading basins at various sites; well injection aquifer recharge and recovery; direct reuse of WRF product water; and several schemes for trading reclaimed water for surface water (SRP water).

During the workshop and follow-up meetings on the issue of supply options, it became apparent that:

- The lake supply options must be coordinated with ongoing City-wide water resource management planning. The City-wide water planning involves decisions regarding City participation in regional wastewater facilities (91st Avenue) in comparison with expanded and additional City-owned reclamation facilities. Reuse of physical water or traded water rights based on reclaimed water exchanges is critically important to the water conservation aspects of the Rio Salado project.

- The quantity of water required to maintain a lake was a key unknown. The water required was uncertain because (1) the location (size) of the lake had not been selected and (2) the estimates of seepage losses (lake infiltration losses) based on available data had a wide range of uncertainty. Overall, it was estimated that a lake would require from 1 to 12 mgd of supply water on an annual average basis. This wide range of required water had a significant impact on which supply options were feasible to consider, as well as an impact on overall water resource planning.

- City staff needed more information regarding the range of possible supply scenarios before a single plan could be recommended to the City Council for implementation.

The result of these circumstances led the City to conclude that field studies were necessary to better define the lake location and water requirements. These field studies have now been documented in TM8, the Town Lake Feasibility Study. TM8 provided the basis for the City's selection of a lake located between Grade Control Structure 4 and the confluence of Indian Bend Wash. TM8 also established the water demands for the preferred lake alternative which is the basis for continuing interest in the following supply alternatives. The City also concluded that the Engineering Report (this document) would not recommend a preferred supply alternative, rather it would present alternatives that could be evaluated as elements of the broader scale water resource planning effort. The City directed that the following water supply alternatives be developed in this Engineering Report.

Alternative 1—Direct Reuse from Existing Kyrene WRF

This alternative has the following variations:

1a. Direct reuse of existing plant reclaimed water
1b. Direct reuse of additionally treated reclaimed water

Alternative 2—Indirect Reuse from Existing Kyrene WRF

This alternative has the following variations:

2a. Basin ASR at Hardy Farm
2b. Injection ASR at Hardy Farm
2c. Basin Recharge at Hardy Farm/Recovery at Point of Use

Alternative 3—Direct Reuse of Proposed North WRF

This alternative has the following variations:

3a. Direct reuse of proposed plant reclaimed water
3b. Direct reuse of modified plant (additionally treated) reclaimed water

Alternative 4—Indirect Reuse of Proposed North WRF

This alternative is based on injection recharge technology at an injection site remote from the proposed North WRF site.

Alternative 5—SRP Urban Reservoir

This alternative is a "flow through" concept for SRP water, wherein the lake would be used as an equalizing reservoir in the SRP supply system.

This alternative has the following variations:

5a. "Flow through" equalizing reservoir concept.
5b. "Flow through" equalizing reservoir concept with partial supply to the Papago Water Treatment Plant.

Physical Components of Supply Alternatives

To further develop design concepts and estimates of capital, operation, and maintenance costs for the proposed water supply alternatives, five components of these alternatives are described in the following section. The physical facilities are:

- Supply pipeline from Kyrene WRF to lake
- AWT improvements at either Kyrene or North WRFs
- AWT additions to meet drinking water standards for injection well recharge/recovery
- Supply pipeline to the Papago WTP
- Other pipelines and pump stations

Where appropriate this report presents "Probable" costs and "Contingent" costs for project components listed above. Definitions of terms and limitations of cost estimates presented in this report are addressed in Section 10. In general, these estimates have been developed without benefit of detailed engineering, and are therefore approximate in nature. For example, in Alternative 5b, the pipeline cost that is estimated for the pipeline that delivers captured seepage water from the lake to the Papago WTP has a "Probable" cost that includes an assumption regarding the footage of pipeline that will probably, based on limited information, require rock excavation. The "Contingent" cost for this pipeline includes additional footage of rock excavation that could possibly occur, thus increasing the cost estimated for the pipeline. Additional geotechnical fieldwork, as part of the predesign phase of this pipeline would be useful in determining the engineer's estimate for the pipeline prior to bidding the project. In most cases it is not appropriate at this time to investigate and refine the "Contingent" cost estimates. These refinements should occur as the City reaches a decision regarding the preferred supply alternative.

A discussion of each component follows.

Supply Pipeline from Kyrene WRF to Lake

Alternatives 1a and 1b require a pipeline for conveyance of Kyrene WRF reclaimed water from the WRF to the lake. The Kyrene WRF is located on Guadalupe Road just east of Kyrene Road. The distance from the WRF to the lake is approximately 5 miles.

The options for routing the pipeline are:

- ***Railroad route.*** This alignment parallels the railroad tracks and uses the railroad right-of-way to the maximum extent possible between Guadalupe Road and 13th Street. North of 13th Street the route uses Farmer Avenue. This route is the shortest distance between the WRF and the lake and would also require the least surface restoration (pavement cutting and replacement). Extensive negotiations with the railroad may be required to permit the pipeline because it is somewhat unusual to request extensive parallel use of railroad right-of-way. Other than the costs associated with acquisition of railroad permits and right-of-way, the railroad route is the preferred alignment based on lower construction cost and traffic maintenance issues. However, the right-of-way cost or annual lease could be prohibitive.

- ***Street route.*** This alignment has been evaluated in the event that negotiations with the railroad would result in unacceptably high permit and right-of-way costs. This alignment emphasizes use of City of Tempe street rights-of-way to the maximum extent possible, minimizing the need for right-of-way acquisition. Some consideration has been given to pipeline location outside of paved areas, but in general this analysis may include more pavement restoration than will actually be required in final design to establish an upper boundary condition for planning purposes.

No consideration is given here for a pipeline that conveys Kyrene WRF reclaimed water from the existing storm sewer outlet (near the I-10 crossing of the Salt River) to Town Lake. This is a distance of approximately 4 miles and has the disadvantage of potential contamination of reclaimed water due to stormwater and street drainage mixing with the supply to the lake.

This study assumes that the existing service pumps at the Kyrene WRF could be modified to meet the pumping conditions required to deliver reclaimed water to the lake. The lake is approximately 50 feet lower in elevation than the WRF. A detailed hydraulic analysis will be required during predesign to determine if portions of the pipeline should be designed for pressure- or gravity-flow conditions. An 18-inch-diameter pipeline is consistent with a future design flow capacity of 6 mgd. An allowance is also necessary to accommodate modifications to the pump control and distribution valves. Total estimated cost (see Table 6-1) for the pump station modifications is under $20,000.

Table 6-1 Construction Cost Estimate Railroad Route vs. Street Route	Railroad Route ($ mil)	Street Route ($ mil)
25,000 feet of 18-inch D.I.P. with 3 feet of cover	2.79	
30,000 feet of 18-inch D.I.P. with 3 feet of cover		3.86
Surface Restoration	0.26	1.20
Casing crossings at: Guadalupe Road Baseline Road Southern Avenue Broadway Road University Drive	1.10	0.63
Elevated Crossing at I-360	0.17	0.17
Total	4.32[a]	5.87

[a] Does not include cost of railroad right-of-way.

AWT Additions for Phosphorus Reduction

Alternatives have been considered for reducing the nutrient concentrations in the reclaimed water prior to discharge to the lake. The purpose of these alternatives is to consider ways for directly reusing Kyrene WRF water but achieve a higher lake water quality than would be possible with reuse of the reclaimed water "as is." The lake water quality models indicate that phosphorus may be selected as a controlling nutrient related to lake transparency. The relationship of phosphorus concentration to lake transparency was shown in Figure 5-1. By reducing the amount of phosphorus in the reclaimed water, the predictive model for transparency indicates improved water clarity.

The Kyrene WRF was designed to achieve Class H water standards. In Arizona, this standard allows for unrestricted agricultural use of reclaimed water. Phosphorus reduction is not usually a specific design objective for Class H water reclamation, however, some phosphorus reduction can be expected from the treatment processes that are in place. The design criteria for the WRF states a discharge limit (criteria) of 3 to 5 mg/l for total phosphorus. Additional phosphorus can be removed from the Kyrene WRF reclaimed water by a variety of processes. These additional treatment steps, or advanced waste treatment (AWT) additions have been developed for phosphorus removals down to 1 mg/l, 0.5 mg/l, and 0.05 mg/l.

The existing Kyrene WRF has a design capacity of 3 mgd. Actual flows during initial operation during 1992 are in the range of 2.6 to 2.8 mgd. The treatment facilities consist of screening, activated sludge with nitrogen removal facilities, and effluent filtration. Primary disinfection is provided by ultraviolet light. Waste sludge and scum are currently discharged to the 91st Avenue Wastewater Treatment Plant. The Kyrene WRF is designed to meet the discharge limits shown in Table 6-2.

Table 6-2
Kyrene WRF Water Quality Design Objectives

Parameter (mg/l, unless noted)	Design Discharge Limit (mg/l, unless noted)
BOD_5	2.0
TSS	2.0
Organic Nitrogen	1.4
Ammonia Nitrogen (N)	0.1
Nitrate - Nitrogen (N)	8.5
Total Nitrogen (as N)	10.0
Total Phosphorus (as P)	3-5
Turbidity (NTU)	1.0
Fecal Coliform (CFU/100 ml)	2.2
Enteric Virus (PFU, 40 l)	1
Alkalinity	147

Consideration is being given to using the effluent from the Kyrene WRF as the primary supply for the Town Lake project. Based on limnological modeling, it appears that phosphorus would be the limiting nutrient for this system. The following section identifies options for adding phosphorus removal facilities to the existing Kyrene WRF. Since the proposed North WRF is similar to the Kyrene WRF the proposed process additions could be considered appropriate for either plant. The North WRF has the advantage that should these processes be considered, they could be integrated into the project prior to construction.

Treatment Alternatives

Treatment alternatives have been studied to achieve effluent phosphorus concentrations of 1.0, 0.3, 0.1, and 0.05 mg/l. Treatment technologies considered for phosphorus removal include:

- Metal salt precipitation
- Lime precipitation
- Biological removal
- Continuous flow microfiltration
- Wetlands treatment
- ASR

Metal Salt Precipitation. Chemical precipitation using aluminum or iron coagulations is an effective means of phosphorus removal. While the exact coagulation reactions are complex, the primary reaction is combining orthophosphate with the metal cation. Aluminum ions combine with phosphate ions as follows:

$$A1_2(SO_4)_3 \bullet 14.3H_2O + 2PO_4^{=} = 2A1PO_4 + 3SO_4 + 14.3H_2O$$

The molar ratio for Al to P is 1 to 1 for this reaction. However, competing reactions, including reactions with alkalinity require a greater than stoichiometric alum dosage. Full scale operating experience at other facilities indicates that effluent phosphorus concentrations of 1.0 mg/l can be achieved at alum to phosphorus dosage of 1:1 (molar). The weight ratio of commercial alum to phosphorus is 9.7:1. To achieve additional phosphorus removal, molar ratios as high as 15:1 have been required.

In wastewater reclamation applications, alum or iron salts can be added directly to the aeration basins, upstream of the secondary clarifiers, or upstream of tertiary clarification. Because a large percentage of the phosphorus is contained within the biological floc, effluent filtration is required to reliably achieve a phosphorus concentration of less than 1.0 mg/l. Based on an assumed influent P concentration of 4 mg/l, estimates of an additional chemical sludge production are presented in Table 6-3 for varying alum dosages.

Table 6-3
Alum Sludge Production

Effluent Phosphorus Concentration (mg/l)	Alum Dosage Required (mg/l)	Alum Cost ($/day)	Chemical Sludge Produced	
			dry lbs/day	gpd*
1.0	50	$81	432	11,000
0.3	65	$106	552	14,000
0.1	70	$113	590	15,000
0.05	90	$146	726	18,500

* Volume of chemical sludge at WRF design WAS concentration of 4,770 mg/l.

Values for the chemical sludge produced only consider the $AlPO_4$ and $Al(OH)_3$ precipitates formed by the alum addition process. These values do not consider the possibility that solids removal efficiency may be improved by the addition of alum to the secondary process. Depending upon alum dosage and the original solids removal efficiency, some plants have experienced an increase of 20 to 40 percent in sludge production. Because the current solids removal efficiency for the plant is high, the overall impact on solids production will be primarily due to the chemical precipitates formed by alum addition.

Based on design criteria in the Kyrene WRF operations manual, the design solids production rate is 3.65 dry tons per day, or 183,000 gallons at the design waste activated sludge concentration of 4,770 mg/l. Based on the estimates of chemical precipitate presented in Table 6-3, the overall sludge production for the Kyrene WRF would be increased by 6 to 10 percent depending upon alum dosage.

Effect on Secondary Process

Alum or iron salt coagulants could be added to the secondary process or upstream of new tertiary clarifiers at the Kyrene WRF. Addition of alum to the secondary process will increase the percentage of inert solids in the secondary system, and may reduce the capacity of the secondary treatment system.

The Kyrene operations manual states that the design solids retention time (SRT) is 6.2 days. This value appears to include the volume of the anoxic zone. From Table 6-3, at an alum dosage of 70 mg/l, the mass of chemical sludge produced is 590 lbs/day. Therefore, the mass of inert solids in the secondary system will be increased by 3,660 pounds (6.2 days x 590 lbs/day). The design mixed liquor solids inventory is 40,700 pounds (including the anoxic zone). Therefore the effective solids retention time will be reduced by approximately 9 percent (3,660 ÷ 40,700).

The design SRT of 6.2 days could be maintained by increasing the mixed liquor suspended solids concentration to offset the addition of the inert chemical precipitates. To ensure no net reduction in SRT for biologically active solids, the mixed liquor suspended solids concentration would need to be increased from the design value of 2,495 mg/l to about 2,700 mg/l.

Based on the estimates of chemical sludge produced, it appears that operational modifications could be implemented to the Kyrene WRF secondary process that would permit the addition of alum to the secondary process, and still achieve the required SRT for nitrification.

Figure 6-1 presents a process flow schematic for Option A. Alum feed facilities are already in place at the Kyrene WRF. Chemical feed piping would need to installed to permit the addition of alum upstream of the aeration basins and upstream of the secondary clarifiers. Full-scale stress testing could be conducted to determine:

- Actual SRT required for nitrification
- Impact of alum on secondary process and sludge production
- Alum dosage required to achieve varying levels of effluent phosphorus concentration
- Secondary system performance and reliability at higher MLSS concentrations

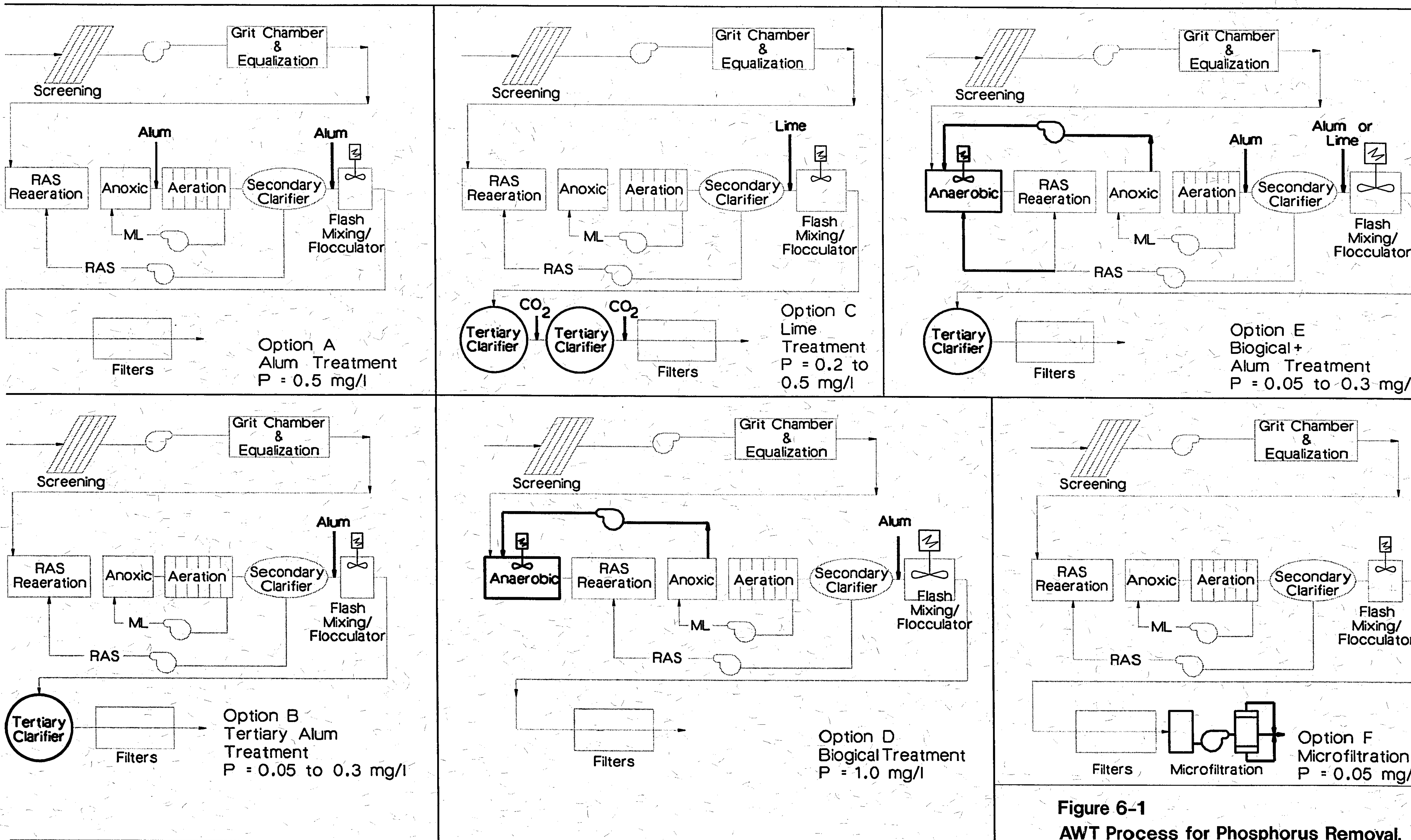


Figure 6-1
AWT Process for Phosphorus Removal, Options A Through E

If stress testing determines that the desired effluent phosphorus concentration cannot be achieved without adversely impacting the performance of the secondary process, tertiary clarifiers could be added downstream of the existing secondary clarifiers. At a design overflow rate of 600 gpd/ft^2, two 60-foot diameter clarifiers would be required. A flow schematic for this option is presented in Figure 6-1, Option B.

Lime Precipitation. Alum addition has been used to achieve phosphorus concentration of 0.05 mg/l in low alkalinity waters (e.g., Rock Creek, Oregon). Optimum removal efficiency is achieved at a pH of 6.5. Excessive alkalinity will require very high alum dosages to achieve this pH. Pilot scale evaluations of alum treatment by the City of Las Vegas, Nevada, indicate that the minimum phosphorus concentration that can be achieved for the relatively alkaline Colorado River water is 0.2 mg/l.

If it is determined that excessively high alum dosages are required to achieve the target phosphorus concentration, consideration should be given to lime treatment. When added to wastewater, lime increases the pH and reacts with carbonate alkalinity to precipitate calcium carbonate. The calcium ion also reacts with orthophosphate to form calcium hydroxyapatite. A pH in the range of 9.5 to 11.5 is required to remove the major fraction of phosphorus, and lime dosages of 150 to 300 mg/l are typical.

Recarbonation prior to filtration would be required to stabilize the wastewater. Recarbonation can be achieved in one or two stages. Excess lime is precipitated at a pH of 9.5, and carbonate is converted to bicarbonate for stabilization.

Figure 6-1, Option C, presents a process flow schematic for the lime treatment option. Based on the experience of other utilities, it is expected that an effluent phosphorus concentration of 0.2 mg/l can be achieved with this process. However, because of its complexity and higher cost relative to the alum precipitation options, no estimates of capital and operating costs have been developed.

Biological Removal. Modifications to the activated sludge process have been developed to permit biological removal of phosphorus. There are a number of variations including:

- Phostrip
- Modified Bardenpho
- A^2O process
- Capetown and modified Capetown processes
- Virginia Initiative Process (VIP)

All of these processes require the presence of an anaerobic zone for phosphorus removal. In the absence of oxygen, fermentation by facultative organisms produces acetate and other fermentation products. These products are preferred and readily assimilated by microorganisms capable of biological phosphorus removal. Because of their ability to assimilate these fermentation products, these microorganisms have a competitive advantage compared to "normal" activated sludge microorganisms.

To provide soluble BOD required for production of fermentation products needed by the phosphorus removing organisms, anoxic zone effluent is recycled to the anaerobic zone. The anoxic zone effluent has relatively low levels of nitrate, and relatively high levels of phosphorus are stored in the microorganisms recycled to the anaerobic zone. Stored phosphorus is released in the anaerobic zone, and metabolized by the phosphorus microorganisms. Because the mixed liquor entering the aerobic zone is relatively "starved" for phosphorus, enhanced removal of phosphorus is achieved in the aerobic zone.

There are a number of variations of the anaerobic/anoxic/oxic (A^2O) process. For the Kyrene plant, new complete mixed anaerobic zone(s) could be added upstream of the existing anoxic zones. The required detention time would be approximately one hour, and would require a 3 mgd recycle pump.

Figure 6-1, Option D, presents a process flow schematic for this option. This process is capable of reliably achieving an effluent phosphorus concentration of 1 mg/l. Alum feed would also be provided for phosphorus removal during process upsets or to reduce effluent phosphorus to less than 1 mg/l. If a very low concentration of phosphorus is desired, or if it is determined that addition of alum to the secondary process is undesirable, then tertiary clarifiers could be used to remove the alum precipitate (see Figure 6-1, Option E).

Continuous Flow Microfiltration. Continuous flow microfiltration (CMF) has been pilot tested at the Reedy Creek WWTP in Orlando, Florida, and has produced effluent phosphorus concentrations in the range of 0.05 mg/l. Pretreatment with alum, at dosages much less than stoichiometry would predict, are required for phosphorus removal. CMF is a patented technology that is owned by the Memtec America Corporation. This process is shown in Figure 6-1, Option F.

Conventional membrane technologies include reverse osmosis (RO), nanofiltration (NF), ultrafiltration (UF), and microfiltration (MF). Reverse osmosis membranes have played a significant role in polishing wastewater for sensitive applications such as aquifer storage and recovery, and recreational lakes. The largest of these installations is the 15 mgd RO system at Orange County (California) Water District's Water Factory 21. Reverse osmosis membranes are designed to remove ionic size particles, having molecular weights greater than 100. Microfiltration is design to remove much larger particles, having molecular weights in excess of 100,000 to 500,000, and particle sizes of 0.1 to 0.5 micrometers. Alum is used to precipitate and flocculate remaining phosphorus in the tertiary effluent to produce particles that can be removed by the CMF system.

Conventional membrane systems operate in crossflow mode to minimize membrane fouling and to suspend dissolved solids in the feed water. Crossflow mode requires that a significant fraction (10 to 75 percent) of the feed water bypasses the membrane. For this reason, their recovery rate is the range of 25 to 90 percent, whereby contaminants are concentrated and discharged as a reject or brine. CMF operates in direct flow, which reduces energy costs by as much as 60 percent. Conventional membrane systems usually require chemical cleaning to remove bacterial fouling and restore flow performance. The CMF membrane is cleaned through gas backwashing. This method eliminates bacterial fouling, and provides a flux rate of 0.5 to 0.9 gpm/sq. meter without the use of crossflow.

Facility requirements for the CMF system would be very similar to those required for a conventional membrane filtration system. Civil and structural requirements are minimal because of its modular skid-mounted construction. The majority of the facility costs will be associated with the CMF equipment. The largest modular unit has a membrane surface area of 900 square meters, and nominal capacity of 650,000 gpd. At the design flowrate of 3 mgd, 6 of these units would be required. Each unit has a footprint of 160 square feet. Actual site space requirements will be approximately 3 to 5 times the modular units footprints or 3,000 to 5,000 square feet.

Pilot testing would be required to determine design criteria such as flux rate, phosphorus removal efficiency, alum dosage, and operating pressure. Figure 6-1, Option F, presents a flow schematic for this process scheme.

Wetlands Treatment

The use of wetlands for post secondary treatment of wastewater can, in many instances offer a cost-effective alternative to more traditional treatment methods. For this project, constructed wetlands (WTS) were considered for polishing reclaimed water from the Kyrene or North WRF. As described earlier, the objective for this application would be to reduce nutrient loading in the lake, specifically phosphorus concentrations. While WTS has been shown to be effective at removing many constituents, removal rates for phosphorus are highly variable and somewhat unreliable. Assuming a flow rate of 3 mgd and an influent concentration of 4 mg/l, the WTS models predict that about 1,500 acres of wetlands would be required to reduce the TP concentration to 0.1 mg/l (loading rate of 2.76 $g/m^2/yr$).

Also, an aging effect has been detected in wetlands from 2 to 25 years after loading begins with significant decreases in removal efficiencies. One additional problem with WTS for TP reduction is the tendency of these systems to discharge or "burp" high concentration effluent occasionally, unpredictably, and without apparent cause. For these reasons, WTS was not considered further for source water nutrient reduction. Wetlands may, however have other uses and benefits for project components such as riparian zone mitigation.

Aquifer Storage and Recovery

ASR provides an excellent mechanism for reduction in phosphorus via either surface spreading basins or well injection. The recovered water could have phosphorus concentrations below 0.05 mg/l. Actual test results would have to demonstrate this capability. ASR is not considered further in this section as a treatment process in and of itself. Rather, the benefit of ASR (with phosphorus reduction) is credited in the supply alternatives that include an ASR component.

Cost Estimates for AWT (for Phosphorus Reduction)

Biological removal of phosphorus will reduce or eliminate alum purchase costs substantially. At the current cost for alum of $130 per dry ton, the unit cost for alum removal of phosphorus is $0.85 per pound of phosphorus removed (assuming a weight ratio of 13:1 is required to reduce phosphorus concentration to 1 mg/l). Assuming an influent phosphorus concentration of 4 mg/l, the yearly alum cost to provide an effluent phosphorus concentration of 1 mg/l is $23,000.

Sludge disposal is the other primary operating cost of alum treatment. Dewatering and disposal will be necessary if sludge cannot be discharged to the 91st Avenue WTP, or if the cost is prohibitive. Assuming 600 dry pounds is produced each day, approximately 1 yard of dry sludge (at 30 percent solids) will require disposal. At a tipping fee of $15 per yard, the annual disposal cost is about $6,000.

Further sampling is recommended to establish the validity of the assumed 4.0 mg/l phosphorus concentration in the reclaimed water.

All estimates of capital and operating costs have been prepared assuming that additional sludge produced by chemical addition can be discharged to the 91st Avenue WTP. No cost is included for sludge dewatering or disposal.

Option A. Option A would require the addition of the following facilities:

- Chemical feed piping to deliver alum to the aeration basins and secondary clarifier

Option B. Option B would require the addition of the following facilities:

- Two 60-foot diameter clarifiers
- Chemical sludge pumping station

The cost estimate for this option assumes that sufficient head is available for the new tertiary clarifiers to operate upstream of the existing filters without repumping.

Option C. Option C would require the addition of the following facilities:

- Lime feed system
- CO_2 storage and feed system
- Four 60-foot diameter clarifiers
- One 63,000 gallon recarbonation basin

Option D. Option D would require the addition of the following facilities:

- Two 62,500 gallon anaerobic basins
- Two anaerobic cell mixers (30 hp each)
- Anoxic mixed liquor recycle pumps (7.5 hp each)
- Piping gallery and piping to connect raw sewage piping to new anaerobic cells, and to connect new anaerobic basin to existing aeration basin.

Option E. Option E would combine the facilities required for Biological Removal (Option D) with the chemical treatment facilities described for Options B or C.

Anticipated process performance and estimated capital costs are presented in Table 6-4.

Option F. Option F would require the addition of the following facilities:

- 4,000 to 5,000 sq.ft. building
- Five 750,000 gpd skid mounted CMF units
- Air backwash system
- Surge tank and feed pumps

Table 6-4
Process Performance/Cost Summary

Option	Description	Estimated Construction Cost	O&M Cost	Effluent Phosphorus Concentration (mg/l)
A	Alum addition to existing secondary process	$10,000	$35,000	0.5
B	Alum addition and new tertiary clarifiers	$1,490,000	$60,000	0.05[a] to 0.3
C	Lime precipitation/tertiary clarifiers/recarbonation	$2,870,000	$90,000	0.2[a]
D	Biological removal	$835,000	$7,000	1.0
E	Biological removal/combined with: Option B Option C	 $2,325,000 $3,705,000	 $42,000 $67,000	 0.05[a] 0.05[a]
F	Continuous flow microfiltration	$4,600,000	$275,00 0	0.05[a]

[a] Pilot testing required for effluent phosphorus concentration goal less than 0.5 mg/l.

AWT Additions for Drinking Water Standards (Injection Well Recharge/Recovery)

Aquifer storage and recovery (ASR) is a component of supply alternatives 2b and 4. Aquifer storage and recovery of reclaimed water using well recharge technology is being considered in comparison with surface spreading basins. In instances where large parcels needed for surface spreading basins are not available or if the available land is not suitable for surface recharge, then it is appropriate to consider injection well technology. Compared to spreading basins, the design and operation of recharge wells are more sensitive to site-specific factors related to aquifer conditions, source water quality, groundwater quality, and the regulatory requirements related to water quality. Many of these factors have uncertainties that will require field investigations, laboratory analyses, geochemical modeling, pilot recharge well operations, and finally negotiations with regulating agencies to develop the design criteria for a full-scale facility. The sensitivity of these factors and the associated uncertainties greatly affects the facility requirements and results in a wide range of possibilities for a reuse plan using ASR via recharge wells.

The primary elements of an ASR system using recharge wells are:

- Pretreatment facilities
- Recharge wells
- Recovery wells
- Connecting pipelines

These facilities are described in further detail in the following sections.

Pretreatment Facilities

Well recharge requires that the water being injected must not degrade the water quality of the receiving aquifer or cause an unacceptable rate of clogging in the recharge wells. Since the aquifers underlying Tempe are drinking water quality, the reclaimed water should meet drinking water standards at the time it is injected into the aquifer. Additionally, the reclaimed water must have a concentration of suspended solids low enough to reduce the rate of clogging to an acceptable level. Another requirement for operation of recharge wells is maintaining a residual of disinfectant in the source water to control microbial growth in the well during injection. Chlorine is typically the disinfectant chosen. Disinfection facilities located at the plant are preferred from a capital cost and operations standpoint, although disinfection facilities located at each injection well is an alternative. In either case, provisions to maintain a disinfectant residual in the well between periods of recharge is also important.
Five pretreatment alternatives have been considered for purposes of establishing the range of possibilities associated with well recharge. The five alternatives are:

- Additional disinfection
- Granular-activated carbon (GAC) adsorption with disinfection

- Lime treatment, GAC, and disinfection
- Nanofiltration and disinfection
- Lime treatment, reverse osmosis (RO), and disinfection

All five alternatives incorporate the existing treatment processes including filtration. These five alternatives, respectively, represent the minimal realistic, probable, and contingent possibilities for pretreatment requirements.

Option A—Additional Disinfection Prior to Injection. The effluent quality at the Kyrene WRF and North WRF may meet drinking water standards with the single addition of disinfection. Current discharge requirements of turbidity of less than 1.0 NTU and total nitrogen (as N) less than 10.0 mg/l also meet the drinking water standards. Discharge requirements for pathogens (fecal coliform and enteric virus) are low and it is possible that the standards for drinking water could be met with additional disinfection prior to injection. It is unknown whether the effluent can consistently meet the standards for trace inorganic (primarily heavy metals) or trace organic substances (primarily volatile organic compounds), but it is uncommon for municipal wastewater to exceed drinking water standards for trace substances in reclaimed water where municipalities have pretreatment requirements for industrial dischargers. Industrial pretreatment requirements are particularly important for the effluent produced at the North WRF which will have a larger share of industrial dischargers. To obtain approval from regulatory agencies for this alternative the agencies will likely require assurances that sufficient controls on dischargers or contaminant barriers exist in the treatment process to ensure that exceedance of the standards does not occur. The major concern is whether there are enough safeguards built into the system to ensure that drinking water standards are continuously met at the point of injection even if occasional upsets occur in the quality of the sewage influent or in individual processes within the treatment system. Another concern will be whether the disinfection process would produce disinfection by-products (DBPs) (such as trihalomethanes) to exceed the forthcoming requirements of the EPA for DBPs.

Option B—GAC Adsorption and Disinfections. In this option, GAC is used as a filter media similar in concept to a rapid sand filter. The GAC acts to attract very fine solids from the process stream. The contractors are designed without backward provisions and the GAC life in the adsorption process is much greater than in the adsorption process.

Option C—Lime Treatment, GAC, and Disinfection. The precedent set at existing injection recharge well facilities using reclaimed water is to include lime treatment, filtration, and granular-activated carbon in the treatment process. Lime treatment removes trace inorganics (heavy metals) and phosphorus, and is highly effective at killing virus and bacteria due to high pH levels. The GAC process is effective at removing soluble organic materials, typically the refractory organics, left behind from the other treatment processes, such as pesticides, herbicides, synthetic organics, humic acids (trihalomethane precursors) and detergents. In addition, GAC can remove trace metals, chlorinated hydrocarbons, and organic phosphorus compounds.

Total Organic Carbon (TOC) concentrations will be reduced to approximately 1 to 5 mg/l which reduces the potential for producing disinfection byproducts. This process system is proven technology for recharge wells, even for cases where the recovered water is used for potable purposes.

Option D—Nanofiltration and Disinfection. Nanofiltration is emerging technology similar to reverse osmosis. With nanofiltration the membranes pass higher molecular weight molecules than RO membranes and operate at lower pressure.

Nanofiltration is currently being tested at the Kyrene WRF with encouraging preliminary results.

Option E—Lime Treatment, RO, and Disinfection. New regulations being promulgated in California for injection well recharge are requiring reductions of TOC concentrations to <2.0 mg/l for reclaimed water. TOC is used as an indicator parameter for organic substances. The only proven technology for reliably achieving such a low concentration of TOC with reclaimed water is RO. Therefore, RO would be considered for contingency purposes, in case ADEQ should adopt a similar TOC standard as California. RO is a demineralization process using membrane technology which removes about 95 percent of dissolved inorganic and organic substances. Since RO is effective at removal of such a wide range of contaminants it can be considered as a backup to the treatment processes used earlier in the system.

Pretreatment Cost Estimates. Capital and operating costs for the five pretreatment alternatives are shown in Table 6-5.

Table 6-5
Estimated Costs for Injection Pretreatment Alternatives

Alternative	Anticipated Effluent TOC mg/l	Estimate for 3.0 mgd Facilities ($ million)	
		Construction	Operation
Option A Additional Disinfection	> 10	0.12	0.05
Option B GAC Adsorption, and Chlorination	5 to 10	5.00	1.14
Option C Lime, GAC, and Chlorination	1 to 5	10.75	1.90
Option D Nanofiltration, and Chlorination	1 to 3	3.95	0.82
Option E Lime, RO, and Chlorination	< 1	18.85	2.04

Recharge Wells

Recharge wells must be located away from known sources of groundwater contamination, where aquifer conditions provide a suitable thickness of saturated granular material (to accept the water), where adequate aquifer storage space is available, and outside the immediate capture zone of production wells pumping water for potable purposes. The wells must be spaced far enough apart to prevent excessive hydraulic interference during injection. Each well site must have access for well drilling equipment and room for permanent water disinfection facilities.

For planning purposes, recharge wells for the Kyrene WRF have been sited in the vicinity of the City's "Hardy Farm" property, approximately 1 mile south of the WRF.

Further hydrogeological investigation and negotiations with ADEQ may allow siting at the Kyrene WRF property. The studies and negotiation would be in regard to impacts on a known groundwater contamination plume just north of the site. Recharge wells for the North WRF could be located within the 3-square-mile area bounded on the east and west by Rural Road and Priest Drive, and on the north and south by Broadway Road and the Superstition Freeway. These locations are sufficiently distant from known aquifer contamination and existing production wells that ADEQ APP permitting will be likely. In addition, available information indicate suitable aquifer conditions, reasonable depths to water, and sufficient aquifer thickness may be present here.

Construction of recharge wells is similar to production wells except that the well casing and perforated casing must be non-corrosive materials due to the corrosive effects of the disinfectants (typically chlorine) in the injection water. The casing openings and filter pack grain size are also typically larger than for production wells. The recharge wells must be equipped with conductor pipes for recharging and vertical turbine pumps for redevelopment. Pumping and surging for redevelopment will be required at regular intervals to mitigate the effects of clogging. The frequency of redevelopment will depend on quality of the recharged water, aquifer conditions, and the recharge rate. Typical frequencies for redevelopment range from weekly to once every three months. Finding a means to dispose of the water pumped during redevelopment will be an important factor in well site selection and development. Typical ways of disposal could include discharge to sanitary sewers, storm drains, dry wells, or small percolation basins.

Recharge Well Cost Estimates. Estimates of costs for a recharge wellfield having a total capacity of 3.0 mgd for both the Kyrene WRF and North WRF locations have been prepared. Recharge rates for the wells are estimated at one-half their expected yield during pumping. An additional well is included at each location for operation during redevelopment of other wells and for standby purposes. Recharge well estimates include the costs for well construction, pumping and electrical equipment, onsite piping and appurtenances, offsite piping for pumped water disposal and site development and fencing. For offsite piping it was assumed that a connection to a sewer or storm drain was made within 800 feet of the site. An automated system for maintaining a disinfectant residual in the well during periods of downtime is also included in the costs. Estimates for monitoring well facilities were made based on five monitoring wells constructed in each primary aquifer for each group. Monitoring wells are assumed equipped with a locking vandal-proof cover, water level recording equipment, and permanent pumping equipment installed for collecting water samples. Each monitoring well location will consist of a nest of two wells, screened at two different intervals (one in each of two primary aquifers).

For the cost of operations, a redevelopment schedule of one hour of pumping once every week is assumed for each well. Also included in operations is the maintenance of pumping, electrical, and instrumentation and control equipment. The costs for monitoring assumes quarterly sampling and laboratory testing of the recharge source water and groundwater at each nest of monitor wells (two samples per well).

Facility size assumptions and estimated capital and operations costs for a 3.0 mgd injection well and monitoring system are shown in Table 6-6. The number of injection wells would be determined by detailed fieldwork (drilling and aquifer testing) and City preferences for operation. For planning purposes, it is assumed that an injection well system would have sufficient capacity and operational flexibility to inject all Kyrene WRF water on a steady state basis without the need to divert reclaimed water for storm sewer disposal.

Table 6-6
Estimated Costs for 3.0 mgd Recharge Well System

	Probable	Contingent
Number of Injection Wells	4	5
Recharge Well Casing Diameter/Depth	16-inch/500 feet openings @ 250 to 500 ft.	16-inch/500 feet openings @ 250 to 500 ft.
Recharge Well Capital Costs	$1,323,000	$2,200,000
Annual Recharge Well Operations Costs ($)	$61,000	$72,000
Number of Monitor Wells	8	10
Monitor Well Diameter/Depth	4 @ 6-inch/300 feet 4 @ 6-inch/500 feet	5 @ 6-inch/300 feet 5 @ 6-inch/500 feet
Monitor Well Construction Costs ($)	$486,000	$608,000
Annual Monitor Well Operations Costs ($)	$35,000	$42,000

Recharge Basins

The City is conducting field studies at the "Hardy Farm" property to evaluate the site for long-term aquifer storage using surface spreading basin technology. At the time of this writing, the preliminary results are encouraging for full-scale recharge facilities in the range of 3.0 mgd capacity or more.

Apart from the land cost (or opportunity cost in dedicating the land to recharge), surface spreading basins have many advantages over injection well recharge technology. For surface spreading, no additional treatment steps would be necessary. In addition, as the reclaimed water passes through the upper soil media, additional water quality transformations occur that further improve the quality of the reclaimed water.

If the annual water balance between recharged quantities and recovered quantities is critical, evaporation losses need to be given close attention.

The outcome of the field investigations will determine the number and size of the recharge cells as well as depth of basin construction and other physical characteristics. A range in capital and operation and maintenance costs for recharge basins in shown in Table 6-7.

Table 6-7
Estimated Costs for Recharge Basin (Surface Spreading)

Component	Probable Cost ($ million)		Contingent Cost ($ million)	
	Capital	O&M	Capital	O&M
Spreading Basins	0.42	0.02	0.67	0.02
Monitoring Wells	0.35	0.02	0.49	0.03
Total	0.77	0.04	1.16	0.05

Recovery Wells

The location and well site requirements for recovery wells are similar to those for the recharge wells discussed previously, except disinfection facilities or piping for redevelop flows are not required. The construction requirements for the recovery wells would be similar to production wells used by the City. To allow flexibility to meet peak demands, a peaking factor of 2.0 is assumed for planning purposes. This means a 3.0 mgd injection system would be equipped with a 6.0 mgd recovery system.

Recovery Well Cost Estimates for 6.0 mgd. Estimates of a recovery wellfield having a total capacity of 6.0 mgd for both the Kyrene WRF and North WRF locations have been prepared. Recovery rates are estimated based on pumping rates typical for production wells in the area. An additional well is included at each location for standby purposes. Recovery well estimates include the costs for well construction, pumping and electrical equipment, onsite piping and appurtenances, and site development and fencing.

Estimated cost of operations includes the power costs for pumping and routine maintenance of pump and wellhead equipment.

Facility size assumptions and estimated capital and operations costs for a 6.0 mgd recovery system are shown in Table 6-8.

Table 6-8
Estimated Costs for 6.0 mgd Recovery Well System

	Probable	Contingent
Number of Recovery Wells	4	5
Recovery Well Casing Diameter/Depth	16-inch/500 feet	16-inch/500 feet
Construction Costs ($)[a]	$1,050,000	$1,640,000
Annual Operations Costs ($)	$125,000	$135,000

[a] See text for recovery wells located near Town Lake.

Potential Recovery Well Cost Savings. Potential cost saving measures for recovery wells may be possible if two or more dual-purpose injection/recovery wells are located within the same well site. Each well would be equipped for injection and for pumping into the supply system. Opportunities to share electrical switch gear, piping, and metering equipment would be available. Additional cost savings could be realized if the number of wells could be reduced. Further hydrogeological fieldwork is necessary to verify these savings potentials.

Another savings measure may be to redevelop existing wells for recovery purposes. Existing wells closer to Town Lake or other points of use could further reduce project costs by reducing the footage of transmission pipeline that is necessary. If wells closer to the lake are considered, it is also appropriate to consider the design capacity of the recovery wells in relation to the point of use demand. In this case, since the peak demand is 3.1 mgd for the lake and associated landscaping, the recovery wells could be sized accordingly. The City's existing well Number 1 (near College Avenue and 15th Street) may be a candidate for redevelopment as a recovery well. This well, in addition to one new well, may satisfy the peak demand. This approach has been employed in developing Supply Alternative 2c. In this approach, the recovery well costs would be $300,000 and the pipeline costs would be reduced from $5.87 million to $1.76 million.

Supply Pipeline to Papago WTP

Alternative 5b incorporates a pipeline for transmission of recovered Town Lake seepage losses to the Papago WTP.

This transmission system includes a central wet well storage reservoir that collects water from the seepage recovery wells, a booster pump station, and pipeline to the Papago WTP. This pipeline has been sized at 36 inches in diameter and is approximately 8,000 feet in length. This route is within City right-of-way. This area of the City is known for shallow rock and higher pipeline excavation costs associated with the rock.

The estimated cost of the collection and conveyance system is shown in Table 6-9. The cost for the seepage recovery wells is not included. The recovery well system is further described in Section 4. The well system is an additional \$2.61 million in capital and \$0.2 million per year in operations and maintenance. The contingent estimates assume a greater quantity of rock excavation.

Table 6-9
Estimated Costs for Collection and Conveyance System

Component	Probable Cost ($ million)		Contingent Cost ($ million)	
	Capital	O&M	Capital	O&M
Wet Well and High Lift Pumps	2.33	0.17	2.33	0.17
Pipelines	2.95		3.15	
Total	5.28	0.17	5.48	0.17

Pipelines and Pump Stations

Many of the alternatives require pipelines and pipe networks to connect the various major components. For planning purposes, these estimates for pressure pipes ranging from 12 inches to 18 inches are \$75 per foot for pipelines with minor surface restoration, and \$95 per foot for pipelines in developed areas (more significant surface restoration). Additional allowances are included for crossings of major streets, canals, utilities, and freeways.

Pump station costs have been estimated using a capital cost factor of \$2,000 per installed horsepower of pumping capacity, with a minimum cost of \$20,000. Operating cost is based on electrical energy cost of 7 cents per kilowatt hour plus consideration of annual labor requirements for maintenance.

Supply Alternative Evaluations

Alternative 1a—Direct Reuse from Kyrene WRF

This alternative uses the reclaimed water from the existing Kyrene WRF as the primary source of supply for the lake. Water from the Kyrene WRF is conveyed to the lake via a 5-mile pipeline. Since this alternative does not include water storage, the peak demands for lake supply (evaporation and seepage losses) must be met by the Kyrene WRF. On a summer day, the peak lake demand is about 3.1 mgd compared to the interim design capacity of the Kyrene WRF at 3.0 mgd (future 6.0 mgd). During the startup periods of 1992, actual flowrates to the Kyrene WRF ranged between 2.6 to 2.8 mgd. Additional flow could be diverted to the Kyrene facility, but with demands for water in addition to those of Rio Salado this supply scheme is not adequate for the peak summer months.

Because nutrients (nitrogen and phosphorus) are highest in reclaimed water compared to other sources, this alternative results in the lowest lake transparency (water quality). Lake transparency is predicted to average 2 feet during midsummer with excursions to near transparency. Significant problems associated with algae and aquatic weeds are likely with this water source.

The estimated costs for this alternative are shown in Table 6-10. A generalized layout is shown in Figure 6-2.

Table 6-10
Alternative 1a—Direct Reuse from Kyrene WRF

Supply Component	Probable Cost ($ million)	
	Capital	O&M/yr
Pipeline	5.87	~
Pump Station	.02	~
Total	5.89	~

The probable cost scenario described above is based on the "street" pipeline alignment. If easement costs associated with the railroad alignment are less costly the alternative railroad pipeline route could be considered.

Alternative 1b—Direct Reuse of Kyrene WRF Water After Additional AWT

This alternative is the same as Alternative 1a except that additional treatment processes have been added at the Kyrene WRF to reduce phosphorus concentrations in the reclaimed water. After additional treatment, reclaimed

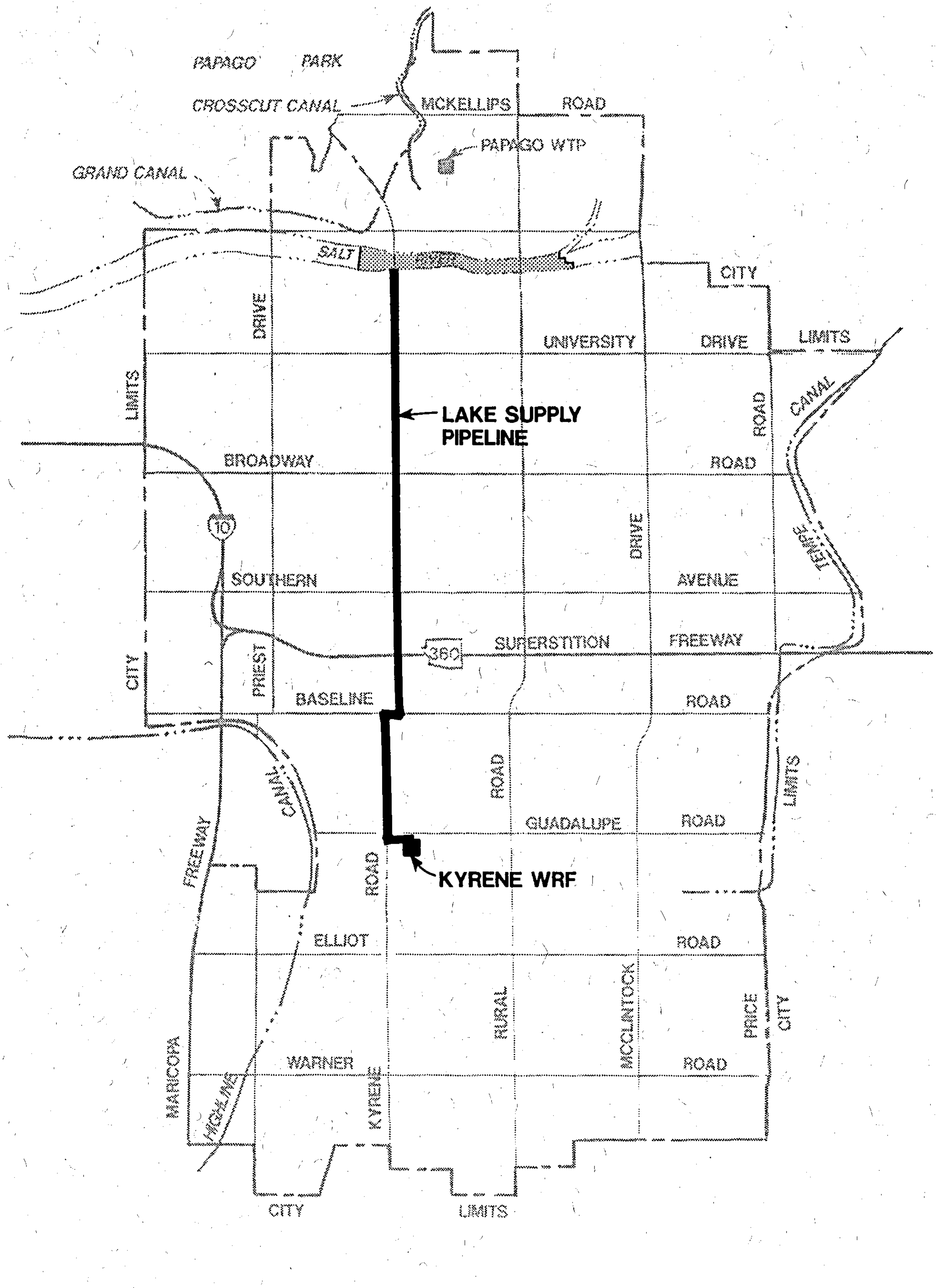


Figure 6-2
Water Supply Alternative 1

water is piped to the lake. There are five subalternatives producing varying levels of phosphorus in the reclaimed water. Each increment of phosphorus reduction improves the water quality (transparency) of the lake. The most significant improvement in lake water quality would result from processes to reduce phosphorus concentration to below 0.05 mg/l (Figure 6-2).

The estimates below (Tables 6-11 and 6-12) are based on phosphorus reduction to 0.5 mg/l and 0.05 mg/l.

Table 6-11
Alternative 1b—Phosphorus Reduction to 0.5 mg/l

Supply Component	Probable Cost ($ million)	
	Capital	O&M/yr
AWT	0.01	0.04
Pipeline	5.87	
Pump Station	0.02	~
Total	5.90	0.04

Table 6-12
Alternative 1b—Phosphorus Reduction to 0.05 mg/l

Supply Component	Probable Cost ($ million)		Contingent Cost ($ million)	
	Capital	O&M/yr	Capital	O&M/yr
AWT	2.32[a]	0.04	4.60[b]	0.07
Pipeline	5.87		5.87	
Pump Station	0.02	~	0.02	~
Total	8.21	0.04	10.49	0.07

[a]Option E—Biological Removal with Alum
[b]Option F—Microfiltration

Alternative 2a—Kyrene WRF Indirect Reuse–Basin ASR at Hardy Farm

This alternative is based on piping Kyrene WRF reclaimed water approximately 1-1/2 miles south to the City's Hardy Farm site for surface spreading aquifer storage. Aquifer storage at this site is currently being investigated for feasibility. Initial phase results are encouraging for recharge in excess of 3.0 mgd. This alternative contemplates that recovered water would be pumped within 1/2 mile of the spreading basins. Thus recovered water would be pumped from the Hardy Farm site to the lake and other points of use. The recovery system is sized for 6.0 mgd. A generalized layout is shown in Figure 6-3.

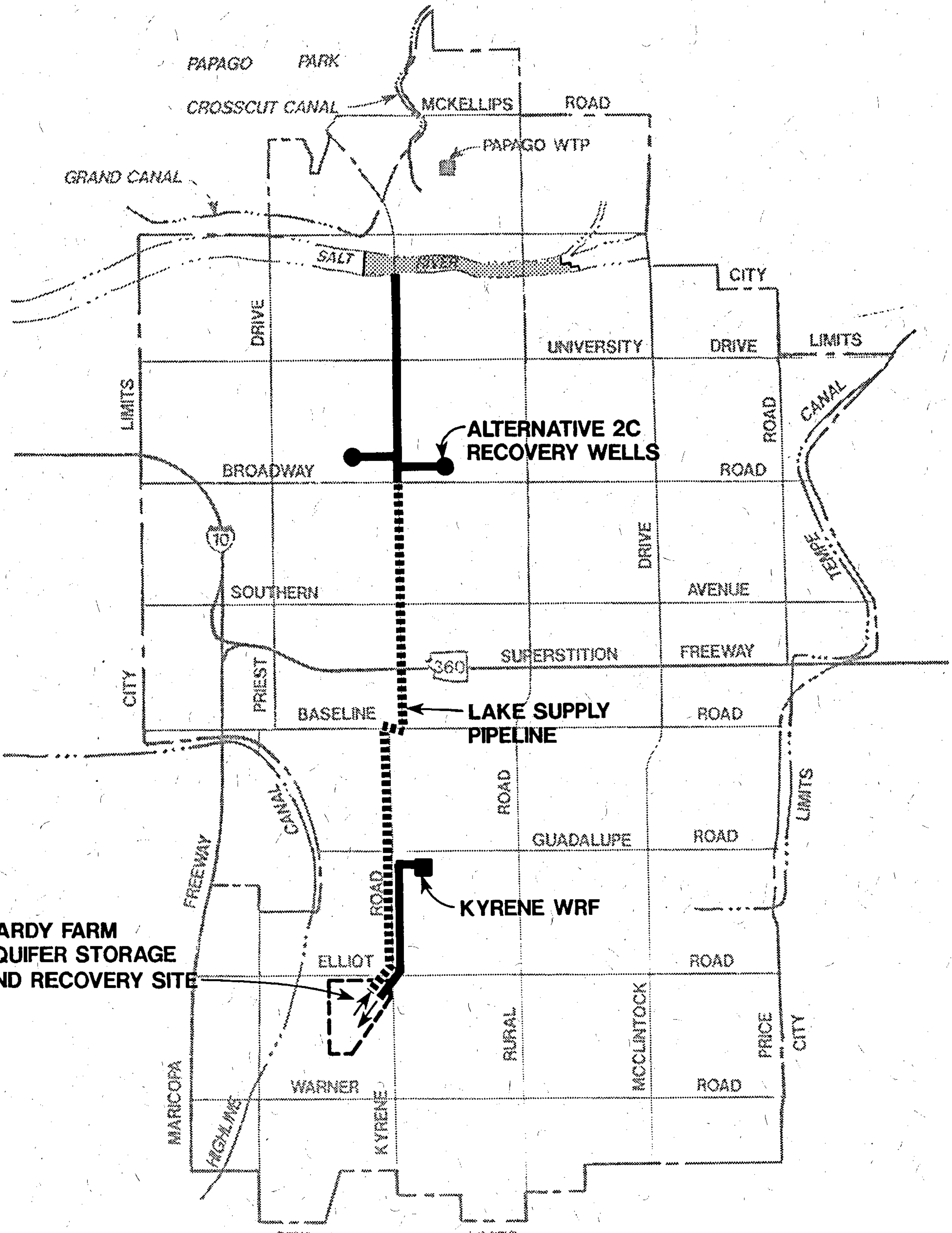
Aquifer storage and recovery of reclaimed water provides for equalization of the winter to summer water demands of the lake and irrigated areas. With ASR, recharge could occur at a relatively steady rate over the year, while recovery (aquifer pumping) would occur at variable rates commensurate with the demand for water on any given day. In any year, the quantity pumped could not exceed the amount recharged. The recovered water would yield relatively high quality lake conditions, due to soil treatment and aquifer adsorption. Summer transparency could be among the best of any of the supply alternatives.

Table 6-13
Alternative 2a—Indirect Reuse of Kyrene WRF

Supply Component	Probable Cost ($ million)		Contingent Cost ($ million)	
	Capital	O&M/yr	Capital	O&M/yr
Loop Pipeline: WRF to ASR Site	1.65		1.65	
Spreading Basins	0.42	0.02	0.67	0.02
Monitoring Wells	0.35	0.02	0.49	0.03
Recovery Wells	1.05	0.13	1.05	0.13
Pump Station	0.04	~	0.04	~
Pipeline: WRF to lake	5.87		5.87	
Total	9.38	0.17	9.77	0.18

Alternative 2b-Kyrene WRF Indirect Reuse-Injection ASR at Hardy Farm

This alternative is the same as Alternative 2a except that injection well aquifer recharge technology is proposed instead of surface spreading basins. Injection wells require very little land area compared to surface spreading basins. Injection wells may require additional reclaimed water treatment prior to injection.



NOTE:
LAKE SUPPLY PIPELINE FROM HARDY FARM
TO BROADWAY ROAD NOT USED IN ALTERNATIVE 2C.

Figure 6-3
Water Supply Alternative 2

Table 6-14 Alternative 2b—Injection ASR at Hardy Farm				
Supply Component	Probable Cost ($ million)		Contingent Cost ($ million)	
	Capital	O&M/yr	Capital	O&M/yr
Loop Pipeline: WRF to ASR Site	1.65		1.65	
30 mgd Injection, 6.0 mgd Recovery and Monitoring Wells at Hardy Farm	2.86	0.22	4.45	0.25
Pump Station	0.04	~	0.04	
Pipeline: ASR Site to lake	5.87		5.87	
Pretreatment	3.95[a]	0.82	10.75[b]	1.90
Total	14.37	1.04	22.76	2.15

[a] Based on injection pretreatment Option D
[b] Based on injection pretreatment Option C

The contingent cost associated with this alternative relates to the need for additional treatment processes at the Kyrene WRF. Injection well clogging rates, and the frequency of redevelopment, is related to the quality of the water injected. The need for additional treatment can only be determined by pilot studies.

It is expected that recovered water from the ASR facilities would be of higher quality than the injected water. The water is improved in the saturated soils due to chemical adsorption processes. These chemical transformations over time could produce recovery water quality similar to groundwater. As a result, lake water quality (transparency) could be the among the best of any of the supply alternatives.

Alternative 2c–Kyrene WRF Indirect Reuse–Basin Recharge at Hardy Farm/Recovery at Point of Use

Alternative 2c is similar to 2a except that the pipeline from the Hardy Farm recharge site (surface spreading basins) to the lake is eliminated. Instead, the Hardy Farm site is used only for recharge, and recovery wells are located closer to the lake. This eliminates approximately four miles of pipeline. In addition, these recovery wells are sized for the maximum Rio Salado demand (3.1 mgd in July) instead of 6.0 mgd in Alternatives 2a and 2b. Thus, costs for recovery wells dedicated to supplying other demands would accrue to their respective projects.

The estimated costs for Alternative 2c are listed in Table 6-15. The difference between the Probable and Contingent estimates for the recovery wells (located north of Broadway Road near Mill Avenue) is due to the possibility of redeveloping one of the City's existing wells versus constructing a new well. The probable cost is based on one existing well and one new well. The contingent cost is based on two new wells.

This alternative provides for a highly transparent lake water quality, which would be among the best of any of the supply alternatives.

Table 6-15
Alternative 2c–Basin Recharge at Hardy Farm/Recovery at Point of Use

Component	Probable Cost ($ million)		Contingent Cost ($ million)	
	Capital	O&M	Capital	O&M
Pipeline to Basins	0.90		0.90	
Basins and Monitoring and Infrastructure	0.77	0.04	1.16	0.05
Recovery Wells	0.30	0.06	0.50	0.06
Pipeline (Recovery Wells to Lake)	1.61		1.61	
Total	3.58	0.10	4.17	0.11

Alternative 3a—Direct Reuse of North WRF

The process flow schematic for the proposed North WRF is similar to the existing Kyrene WRF. Lake water quality is therefore assumed to be relatively poor and equal to that of the supply alternative using the Kyrene WRF. The primary difference between Alternatives 1a and 3a is the much shorter pipeline. A generalized layout is shown in Figure 6-4.

Since this alternative does not include flow equalization, the City's water resource planning for the North WRF must consider uses for North WRF water beyond the requirements of the lake.

Table 6-16
Alternative 3a—Direct Reuse of North WRF

Supply Component	Probable Cost ($ million)	
	Capital	O&M/yr
Pipeline	0.28	~
Total	0.28	~

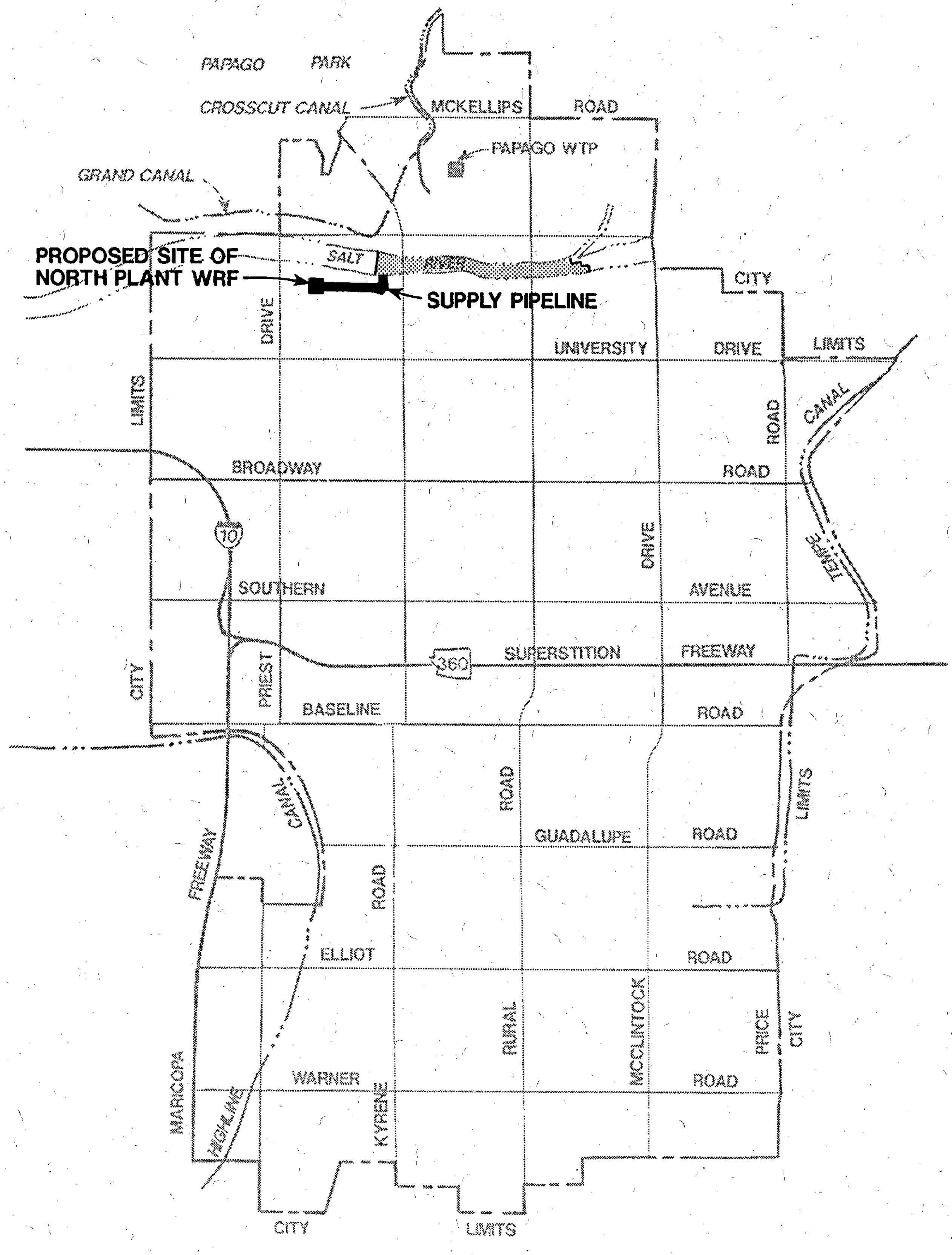


Figure 6-4
Water Supply Alternative 3

Alternative 3b—Direct Reuse of North WRF Water After Additional AWT

This alternative produces lake water quality similar to Alternative 1b. Additional treatment processes are proposed to further reduce phosphorus concentrations in the reclaimed water below levels that are achievable with the North WRF as currently proposed. Since it may be possible to incorporate the treatment process modifications into the plant design prior to bidding and construction of the facilities it is expected that the modifications at the North WRF would be less costly than modifications to the existing facilities at the Kyrene WRF. Further, since the processes could be integrated into the design prior to construction, this alternative focuses on reducing phosphorus to the lowest concentration of 0.05 mg/l.

Table 6-17
Alternative 3b—North WRF with Phosphorous Reduction

Supply Component	Probable Cost ($ million)		Contingent Cost ($ million)	
	Capital	O&M/yr	Capital	O&M/yr
Pipeline	0.28		0.28	
AWT	2.33[a]	0.04	4.60[b]	0.28
Total	2.61	0.04	4.88	0.28

[a]Based on phosphorous reduction Option E
[b]Based on phosphorous reduction Option F

Alternative 4—Indirect Reuse of North WRF After Injection ASR

This alternative is similar to Alternative 2b in the potential for yielding lake water quality with relatively high transparency. The difference between the alternatives is (1) the source of reclaimed water is the proposed North WRF instead of the existing Kyrene WRF, and (2) the injection well site would be north of the Superstition Freeway rather than the Hardy Farm site. A generalized layout is shown in Figure 6-5.

Table 6-18 Alternative 4—Indirect Reuse of North WRF After Injection ASR				
Supply Component	Probable Cost ($ million)		Contingent Cost ($ million)	
	Capital	O&M/yr	Capital	O&M/yr
Loop Pipeline: WRF to ASR Site	6.97	~	6.97	~
Injection, Recovery & Monitoring Wells	2.86	0.22	4.45	0.25
Pretreatment	3.95[a]	0.82	10.75[b]	1.90
Total	13.78	1.04	22.17	2.15

[a] Based on injection pretreatment Option D
[b] Based on injection pretreatment Option C

Alternative 5a—SRP Urban Reservoir

This alternative uses SRP water from the Tempe Canal as the source of supply for the lake. In this alternative the lake is considered an equalizing reservoir on the SRP canal system. Water would flow into the lake from the Tempe Canal and be stored in the lake as needed prior to release to the Grand Canal. SRP has indicated that the canal system could benefit from the ability to move water from the Tempe Canal to the Grand Canal. SRP has also indicated that there would be additional benefit associated with using the lake in an equalizing mode giving SRP the ability to instantaneously withdraw up to 35 cubic feet per second (cfs) for up to 6-hour periods to supply the Grand Canal. Because of the large volume of water in the lake compared to this rate of demand, water levels in the lake would only have to fluctuate a few inches to accommodate this equalizing use. A generalized layout is shown in Figure 6-6.

Water quality in the lake would be the same as the SRP canal system.

Table 6-19 Alternative 5a—SRP Urban Reservoir		
Supply Component	Probable Cost ($ million)	
	Capital	O&M/yr
Tempe Canal and Pipeline Turnout	5.83	~
Grand Canal Turnout and Lift Station	2.24	0.01
Total	8.07	0.01

Because the lake is created using SRP water, the City would be required to remove as much storm drainage into the lake as possible. SRP has an ongoing program to eliminate stormwater entry into the canal system.

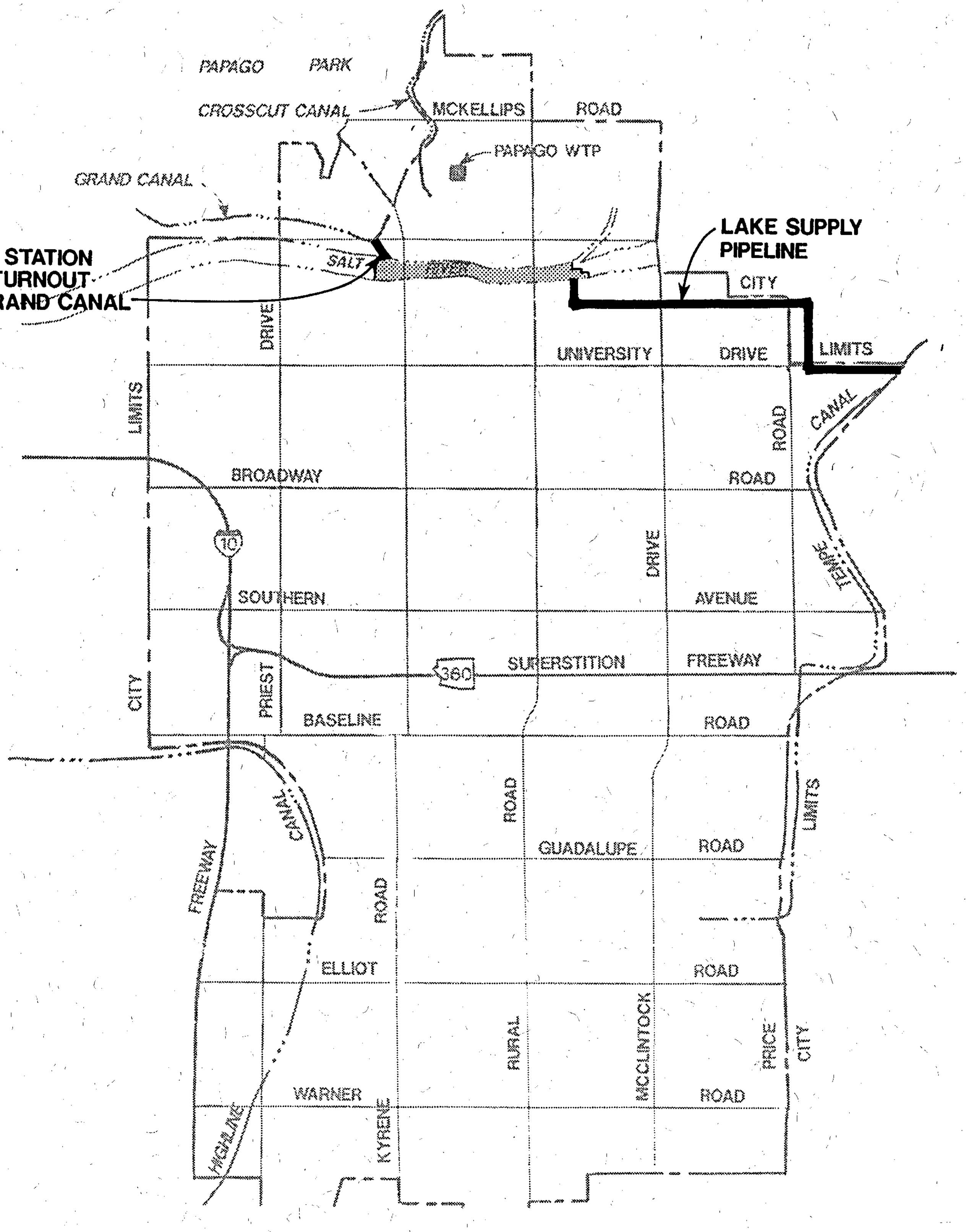


Figure 6-6
Water Supply Alternative 5

Existing storm drains that currently discharge to the Salt River channel (within the reach of the lake) may have to be completely rerouted.

Unresolved issues with this alternative are the source and mechanism for providing water to SRP to replace lake losses, operations, and maintenance.

Alternative 5b—SRP Urban Reservoir with Partial Supply to Papago WTP

This alternative is the same as Alternative 5a except that the lake seepage control system is a network of Ranney and conventional wells that intercept lake seepage losses, and the collected water is pumped to the Papago WTP. At times when the Papago WTP is not operating, the collected seepage would be returned to the lake.

Table 6-20
Alternative 5b—SRP Urban Reservoir with Partial Supply to Papago WTP

Supply Component	Probable Cost ($ million)		Contingent Cost ($ million)	
	Capital	O&M/yr	Capital	O&M/yr
Tempe Canal Turnout and Pipeline	5.83		5.83	
Grand Canal Turnout and Lift Station	2.24	0.01	2.24	0.01
Seepage Recovery System	See seepage discussion in Section 4			
Collector Pump Station and Pipeline to Papago WTP	5.28	0.17	5.48	0.17
Total	13.35	0.18	13.55	0.18

The contingent cost scenario for this alternative includes allowances for additional rock excavation for the pipeline from the collector system to the Papago WTP. The seepage recovery system costs are an additional $2.61 million capital cost and $0.2 million operations and maintenance.

Alternative Evaluation Matrix

Table 6-21 summarizes information presented in this section plus Sections 3, 4, and 5, related to the supply alternatives. The parameters are:

- **Potential beneficial uses.** The anticipated range of beneficial uses that will be supported by the lake depends on the ability of each

Table 6-21
Alternative Evaluation Matrix

Evaluation Parameter	Supply Alternative*									
	1a	1b	2a	2b	2c	3a	3b	4	5a	5b
Potentially available beneficial lake uses: Boating Fishing Swimming	yes maybe no	yes maybe no	yes yes maybe	yes yes maybe	yes yes yes	yes maybe no	yes maybe no	yes maybe maybe	yes yes no	yes yes no
Probable Average Lake Transparency (ft)	1.9	2.5	6.2	6.2	6.2	1.9	2.5	6.2	3.7	3.7
Water Supply Cost ($x1,000,000) Capital O&M	5.89 0.02	8.21 0.04	9.38 0.17	14.37 1.04	3.58 0.10	0.28 -	2.61 0.04	13.78 1.04	8.07 0.01	13.35 0.18
Major Permits Anticipated: APP Reuse NPDES	yes yes yes	yes yes yes	yes yes yes	yes yes yes	yes yes yes	yes yes yes	yes yes yes	yes yes yes	maybe maybe maybe	yes maybe maybe
Permitting "complexity" Low, Moderate, High	L	L	M	H	H	L	L	H	M	H
Appropriate level of stormwater protection: Low, Moderate, High	L	M	M	M	M	L	M	M	H	H
Degree of SRP participation required	L	L	L	L	L	L	L	L	H	H
Water rights trades required to effect supply	no	no	no	no	no	no	no	no	maybe	maybe
ASR provides seasonal equalization	no	no	yes	yes	yes	no	no	yes	NA	NA
Seepage control option Cutoff Wall (C), Liner (L), Recovery Wells (R)	C,L,R	C,L,R	C,L,R	C,L,R	C,L, R	C,L,R	C,L,R	C,L,R	C,L,R	R
Lake water quality management requirements Low, Moderate, High	H	M	L	L	L	H	M	L	M	M
Flexibility and Reliability Low, Medium, High	M	M	H	H	H	M	M	H	L	L
Public Perception and Acceptance	L	M	H	H	H	L	M	H	H	H

***Alternatives**
- 1a Direct Reuse from Kyrene WRF
- 1b Direct Reuse of Kyrene WRF Water After Additional AWT
- 2a Indirect Reuse of Kyrene WRF After Surface Spreading ASR at Hardy Farm
- 2b Indirect Reuse of Kyrene WRF After Injection ASR at Hardy Farm
- 2c Indirect Reuse of Kyrene WRF–Surface Spreading Recharge at Hardy Farm & Recovery at Point of Use
- 3a Direct Reuse of North Plant WRF
- 3b Direct Reuse of North Plant WRF After Additional AWT
- 4 Indirect Reuse of North Plant WRF After Injection ASR
- 5a SRP Urban Reservoir
- 5b SRP Urban Reservoir with Partial Supply to Papago WTP

source to meet regulatory and aesthetic constraints.

- **Probable average lake transparency.** The transparency of the water is used here as an indicator of general water quality. It is implicitly assumed that most other narrative quality measures such as number and extent of algal blooms, odors, fish kills, as well as numeric parameters, are relatively well predicted by the transparency.

- **Costs.** Conceptual-level costs described in previous sections are summarized. Both capital and basic operations and maintenance costs are shown.

- **Anticipated permits.** The major permits, requiring significant investments of time and/or expense to coordinate are the APP, reuse, and NPDES permits. In addition, the general anticipated level of complexity of the permit process for each alternative is evaluated. This list is not exhaustive and other permits may actually be required.

- **Recommended level of stormwater management and costs.** The risks associated with stormwater discharges into the lake reflect the "costs" of episodic non-support of designated lake uses and the sensitivity of the lake to the pollutant loading caused by the stormwater.

- **Degree of SRP participation required/water rights trades.** This parameter reflects the degree to which the source water is subject to control by outside interests. Participation by SRP, Mesa, Scottsdale, or an Indian Community, entails constraints on the use of the source to meet the need of that entity.

- **ASR storage.** ASR provides a storage mechanism that facilitates seasonal and operational storage. This storage would allow the City more flexibility in managing the resource.

- **Seepage control options.** This parameter illustrates constraints on the seepage control methods applicable with each source.

- **Lake water quality management.** The appropriate choices of options for managing the quality of the lake water vary with the source, uses, and sensitivity to variations in quality. This parameter is a general indication of the intensity of management required and therefore the relative costs.

- **Flexibility/reliability.** The flexibility and reliability of each source depends on the susceptibility of that source to shortage, outage, variation in quality and regulatory/institutional control.

- **Public acceptance.** Public perception regarding issues such as the quality of effluent and the responsible use of water resources is difficult to estimate. This parameter reflects possible sensitivity of the source alternatives to public perception.

Section 7
Lake Management

Maintaining an attractive, stable lake environment that consistently delivers the greatest benefits to Tempe, requires a proactive management program. The program should be flexible, with a variety of management tools, and it should be responsive to changing water quality conditions. An effective management program will mitigate the natural degradation of Town Lake, including oxygen depletion, stratification, algae and weed growth, nutrient and metals concentrations, unpleasant odors, and the accumulation of shoreline trash. A water quality monitoring program is essential to anticipate and alleviate undesirable conditions.

Six management techniques are described in this section. Most of these techniques have been implemented at existing lake features in the Phoenix metropolitan area, and are considered appropriate for Town Lake:

- Artificial circulation
- Hypolimnetic withdrawal
- Dilution and flushing
- Mechanical harvesting
- Chemical control
- Fish population

Additional components of the plan should include preventive techniques such as control of resident and migratory waterfowl populations, landscaping to minimize runoff and erosion, trash collection, and fisheries management. All of the techniques described in this section are appropriate, regardless of the primary and maintenance water source. The frequency, intensity, and cost of management activities will be tailored to the project, depending upon the characteristics of the source water and the intended uses of the lake.

All the water supply alternatives under consideration for Town Lake will support aquatic vegetation and free-floating algae. The estimates of total system productivity indicate that some options (direct use of reclaimed water) will have a large potential for creating objectionable over-fertilization of the lake. It should be recognized that management techniques are not likely to influence Town Lake water quality to the same degree as the choice of primary water supply. Management and restoration techniques can mitigate predictable problems and respond to changing conditions but, for the most part, they will have little influence on the overall productivity of the system. Nutrient content, especially phosphorus and nitrogen, of the source water will dictate the average quality and appearance of Town Lake more than any other single factor.

Description of Lake Management Techniques

A brief discussion of lake management techniques is provided below. Advantages and disadvantages of the technique are discussed, and conceptual cost estimates are provided where appropriate.

Artificial Circulation

Algae growth typically occurs in warmer, sunlit water near the surface of a lake. As the biomass grows, some of the algae settles into darker, more stagnant water below the surface (hypolimnion). Bacterial decay and organic bottom mud deplete the dissolved oxygen content of the water, resulting in undesirable odors. In the Phoenix area, stagnant, stratified lakes may also contribute to extensive fish kills during times of high productivity (spring, summer, fall).

The objectives of artificial circulation are to prevent stratification of the water column and improve aeration and chemical oxidation in the lake. Mixing also reduces algal production by diminishing the penetration of sunlight near the surface.

Tempe wind data and estimates of natural circulation suggest that wind mixing alone is inadequate to provide continuous lake circulation. One method of artificial circulation can be achieved through air-lifting, which is common in Phoenix area lakes. Air-lifting is accomplished by injecting compressed air into the deepest portion of the lake where it usually affords the greatest rate of mixing as air bubbles rise to the surface. Air injection in shallow water provides limited benefits.

Some degree of aeration is recommended for each of the Town Lake water supply options. The number of air diffusers, and the volume of air supplied to the lake will be determined during preliminary design, after the source water and intended uses of the lake have been determined. One concept for Town Lake was developed with the goal of mixing portions of the lake that exceed 10 feet in depth, or an area roughly bounded by Mill Avenue to the east and Grade Control Structure 4 to the west. The design consists of a grid of 21 air diffusers, each spaced approximately 300 feet apart. Roughly 7,000 feet of 2- to 4-inch flexible pipe is required to supply the air for this system.

At least two engineering options are available for installing the air piping network. Typically, low cost, flexible plastic pipes and in-line diffusers are anchored to the lake bottom. The installation is above-ground, and therefore relatively quick and inexpensive. A notable drawback is the system's susceptibility to damage from drawdown of Town Lake during storm events or major reservoir releases. High maintenance and repair costs may be attributed to the frequency of repair. One estimate of capital costs for a sacrificial air injection system in Town Lake is $50,000, but considerable refinement is necessary based on site-specific information.

Alternatively, pipes installed below the scour depth of the river will provide a permanent air distribution system. Initially, a permanent system is far more expensive, but requires less labor and expense for operations and maintenance.

Hypolimnetic Withdrawal

Hypolimnetic withdrawal removes deeper, nutrient-rich, and oxygen depleted water from the lake bottom. Removing the hypolimnetic water decreases the residence time of the hypolimnion, thereby increasing the oxygen content at the sediment-water interface, and decreasing the internal phosphorus loading. The technique has been applied successfully in Europe and the United States. Observed water quality improvements include reduced internal loading from sediments, increased oxygen concentrations, and increased transparency.

Benefits of this technique assume that portions of the lake is deep enough to stratify and form a hypolimnion. The potential for success is greatest for stratified lakes with high internal loading of phosphorus. Although Town Lake is expected to stratify during the summer near the deepest part of the lake, artificial circulation by aeration supplemented by hypolimnetic withdrawal should effectively counter this tendency.

Hypolimnetic withdrawal is attractive because of the simplicity of design and operation. At Town Lake, the hypolimnetic withdrawals could supply water features below the dam, or serve as a source of water for landscape irrigation. An engineering concept for Town Lake consists of a transverse collection pipe anchored to the dam foundation, drawing water from behind the dam near the lake bottom. The pipes could penetrate the dam and provide water for downstream features, or deliver water to a sump for irrigation pumps.

Dilution and Flushing

Introducing a source of low-nutrient water to a eutrophic lake, whether on a continuous or periodic basis, acts to dilute the concentration of nutrients and flush out algal cells. The addition of low-nutrient water reduces nutrient concentrations and the potential for algal production. By increasing the fresh water input, a flushing action may occur, and at high rates may act to scour nutrient-laden or contaminated sediments from the lake bottom.

Observed benefits of dilution include a reduction in phosphorus and chlorophyll concentrations. The benefits of dilution and flushing are immediate and proven effective. Supply Alternative 5, involving SRP, is the only realistic option for sufficient source capacity to provide a flushing action.

At Town Lake, high-quality surface water from the Salt River watershed may be available for dilution and flushing. Salt River water could be supplied to the lake in two ways: run-of-the river releases below Granite Reef Dam, or delivery of SRP water via SRP's existing irrigation distribution canals. Run-of-the-river releases are infrequent and undependable, but most often occur in February and March. Excess flows in the river could be used to dilute the lake during routine reservoir releases, and flush bottom sediments during major storm events. As the storm hydrograph recedes, low-nutrient water could be captured and impounded behind the inflatable west dam. One disadvantage of this scheme is that excess flow in the Salt River is rare during summer months when it may be most beneficial to the quality of Town Lake.

A highly dependable source of low-nutrient water may be available from SRP's Tempe Canal near University Drive in east Tempe. One design concept for Town lake includes a new distribution pipe from the canal to the lake (see Water Supply Alternatives 5a and 5b).

Mechanical Harvesting

Harvesting of aquatic plants removes undesirable vegetation that either interferes with the lake's recreational and aesthetic benefits, or may be undesirable habitat for wildlife. Aquatic weed growth is a common lake management problem in the Tempe area.

The basic steps in harvesting aquatic vegetation are cutting, or separation of vegetation, collection of plant material, processing and storage, transportation to the shore, and disposal. Harvesting of the vegetation can occur either in a single-stage harvest by one machine or in multiple stages where cutting, collection, transport, and disposal are conducted by separate equipment. The factors affecting aquatic plant harvesting depend on site-specific characteristics; the type, density, and distribution of vegetation; public perception; and financial resources.

Some of the technologies available for mechanical control of submerged aquatic vegetation include aquatic plant fragment barriers, lake-bottom barriers, hydraulic dredging, diver-operated dredging, rototilling, and harvesting. The mechanical harvester is essentially a submerged mower, towed by boat or barge. Conveyer belts stockpile the weeds onboard for offsite disposal.

At Town Lake, the City will likely need a mechanical harvester, or subcontract this activity on a continuous basis. The primary benefits of mechanical harvesting consist of the removal of nuisance, and undesirable weeds, biomass, and nutrients. Drawbacks, however, include the potential spread of undesirable plant species, possible harm to fish and waterfowl populations, and labor and equipment costs. Lake features such as loading docks and truck access ramps are necessary to facilitate weed removal.

Capital and operations costs will fluctuate seasonally, and depend to a greater extent on source water quality. An estimate of annual expense ranges from $30,000 to $180,000 dollars.

Chemical Control

Chemical control of water features is commonly practiced in the Phoenix/Tempe area. With proper chemical applications, nuisance macrophytes can be killed, controlled, and maintained at acceptable population densities with minimal potential for human or wildlife toxicity. Herbicide treatments are a rapid, effective short-term management technique for temporarily reducing nuisance vegetation. Table 7-1 summarizes common aquatic weed species and responses to herbicides.

Algal growth can occur quickly and the appropriate response depends on species and the extent of algal blooms. Chelated copper compounds are most effective against free-floating and filamentous algae, whereas a variety of other organic herbicides are effective against specific aquatic weeds. Floating aquatic vegetation can be controlled with 2,4-D (2,4-dichlorophenoxyacetic acid), diquat (6,7-dihydrodipyrido[1,2-$_a$:2,1-c] pyrazinediium ion), and endothall (7-oxabicyclo[2.2.1] heptane-2,3-dicarboxylic acid). Emergent broadleaf vegetation can be controlled with 2,4-D, dalapon (2.2-dichloropropionic acid), and glyphosate (N-(phosphonomethyl)glycine). Submerged aquatic vegetation can be controlled 2,4-D, copper sulfate, copper carbonate, organic compounds of copper, diquat, dichlobenil (2,6-dichlorobenzonitrile), and endothall. The effect of these herbicides on floating, emergent, and submerged vegetation can be species-specific in certain instances, and less effective on others.

Although herbicides and plant growth regulators are relatively non-persistent in natural environments, these chemicals do cause changes in aquatic ecosystems. Impacts from these chemicals must be considered for their toxicity to the target species, relative toxicity to non-target species, fate of residues and their significance to water, fish and public health, and conditions that affect toxicity, efficacy, and persistence. Synergistic and antagonistic activity of carriers, metabolites, and degradation products should also be considered. Public perception and environmental risks associated with chemical applications dictate that chemical control should be a last resort at Town Lake.

The U.S. Environmental Protection Agency (EPA) regulates the application of pesticides and establishes maximum contaminant levels (MCLs) for residual

Table 7-1
Common Aquatic Weed Species and Their Responses to Herbicides
(Adapted from Nichols, 1986)

	Diquat	Endothal	2,4-D	Glyphosate (Rodeo)	Fluridone (Sonar)
Emergent Species					
Alternanthera philoxeroides (alligatorweed)			C	C	C
Dianthera americana (water willow)			C		
Glyceria borealis (mannagrass)	C	NC	NC		
Phragmites spp (need)				C	
Ranunculus spp (buttercup)			C		
Sagittaria sp (arrowhead)	NC	NC	C		C
Scirpus spp (bulrush)	NC	NC	C		C
Typha spp (cattail)	C	NC	QC	C	QC
Floating Species					
Brasenia schreberi (watershield)	ND	NC	C		NC
Eichhornia crassipes (water hyacinth)	C[a]		C		NC
Lemna minor (duckweed)	C	NC	NC		NC
Nelumbo lutea (American lotus)	NC	NC	QC	NC	
Nuphar spp (spatterdock)	NC	NC	C	C	C
Nymphaea spp (waterlily)	NC	NC	QC	C	C
Submerged Species					
Ceratophyllum demersum (coontail)	C	C	C		C
Chara spp (stonewort)	NC[b]	NC[b]	NC[b]	NC[b]	NC[b]
Elodea spp (elodea)	C	QC	NC		C
Hydrilla verticillata (hydrilla)					C
Myriophyllum spicatum (milfoil)	C	QC		NC	C
Najas flexilis (naiad)	C	QC	NC	NC	C
Najas guadalupensis (southern naiad)		QC	NC		C
Potamogeton amplifolius (large-leaf pondweed)	QC	C	NC		
P. crispus (curly-leaf pondweed)	C	C	NC		
P. diversifolius (waterthread)	NC	C	NC		
P. natans (floating leaf pondweed)	C	C	C		
P. pectinatus (sago pondweed)	C		C	NC	C
P. illinoiensis (Illinois pondweed)					C

[a] Plus chelated copper sulfate
[b] Controlled by copper sulfate

Legend:

C = Controlled
NC = Not Controlled
BLANK = Information Unavailable
QC = Questionable Control

Source: From U.S. Environmental Protection Agency (1988) pesticide concentrations in drinking water. State agencies such as the ADEQ may impose more stringent standards than the federal regulations. All of the applicable laws should be reviewed before chemical treatment is initiated. At Town Lake, licensed professionals may be required to apply the chemicals.

Furthermore, chemical treatments of Town Lake must be compatible with intended use of the lake. Potable reuse of lake seepage losses, such as the proposal to deliver water to the Papago WTP, may not be compatible with chemical treatments. Town Lake as an in-line reservoir for SRP water may require conformance with SRP's treatment policies.

The cost of chemical treatments in Town Lake depends on the type of herbicide, the dosage, and the frequency of application. Each of these factors must be evaluated on a case-by-case basis. However, one chemical treatment of Town Lake might cost from $75,000 to $150,000 dollars.

Fish Populations

Herbivorous fish species in Town Lake may be beneficial for algae and aquatic plant management. Plant-eating *Tilapia* species frequent the Salt River drainage and are stocked in the Phoenix/Tempe area for algae and weed control. *Tilapia* should be stocked and managed in Town Lake if fisheries are consistent with desired lake uses. Mosquito fish may also be beneficial for insect control.

Sport fish may not be compatible with beneficial herbivorous fish. For example, largemouth bass should probably not be stocked as they often tend to eliminate other species. *Tilapia*, bluegill, sunfish, and catfish could all be sustained in Town Lake, as they are in other Tempe area lakes. Smaller forage fish will probably invade the system with the Salt River flows. A fisheries program in Town Lake should be consistent with the recommendations of the Arizona Game and Fish Department.

Town Lake Management Plan

Each of the lake management techniques described in the previous section is recommended for phased implementation, regardless of the source water. These primary management components will contribute to the following goals:

- Control the production of aquatic weeds and algae
- Maintain visual and recreational appeal
- Anticipate adverse water quality impacts
- Achieve the highest recreational and economic returns
- Avoid health concerns and negative public perception

In addition to these continuous management activities, a comprehensive management plan should include a combination of preventive measures, routine maintenance, and provisions for quick response to rapidly changing lake conditions.

Preventive Measures

Several components of the lake management program are meant to be preventive. Preventive activities include landscape design and maintenance that emphasizes erosion control, low fertilizer use, and minimal production of organic debris. Control of resident and migratory waterfowl, and a comprehensive water quality monitoring program are also preventive measures.

Irrigated landscaping is an integral part of the Rio Salado project. The landscape plans should enhance the aesthetic appeal of the project, while contributing to prudent water quality management. Strip parks and public use areas adjacent to the lake should be designed to retain localized stormwater runoff, thereby avoiding direct discharges to the lake. Seeding and maintenance of undeveloped properties and the construction of retention basins may limit erosion potential. Fertilizers are among the most significant sources of nutrient loading in receiving waters, so an emphasis on low-use or alternative soil supplements is desirable. Finally, landscape plans should avoid the use of species that may contribute organic debris such as leaves, branches, and lawn clippings to the lake. At a minimum, floating debris is unsightly.

Waterfowl populations, although an integral part of aquatic wildlife, can be detrimental to urban lakes. They muddy and destroy lakeside vegetation and lawns and contribute to the overfertilization of the lake. A program of public education and active management should be implemented to discourage the feeding of domesticated ducks and geese. The use of the lake by migratory waterfowl can be encouraged through the development of a shallow area with emergent aquatic vegetation, but extensive populations of resident birds should be discouraged. The Arizona Game and Fish Department is a resource for developing waterfowl management plans.

Water quality monitoring is integral to the lake management program and must be considered ongoing and preventive. Public health concerns such as waterborne pathogens (fecal coliform) and lake nutrient levels, as measured by water clarity and observations of algal growth, should be monitored weekly or as conditions warrant. The program will require trained personnel operating water quality sampling equipment and field meters from a boat. Results should be charted and reviewed in real time to be effectively used in lake management decisions.

Trash Accumulations

Floating debris of all kinds, man-made and plant materials, are a common problem of urban lakes. A management program of straining debris from the lake will prevent unsightly accumulations along windward shores. Removal may be by hand netting or mechanical screens towed from a boat. The trash cleanup program should be routine and continuous. Trash racks should be used on any stormwater inputs.

Minimal Intrusion

The management of Town Lake will need to be continuous and frequent, yet must not interfere with public use of the lake. Mechanical weed harvesting or aeration can be accomplished in locations away from most public recreation. Pumping systems should be designed for minimal public perception. Chemical control can be done in a manner to minimize disruption of normal lake activities. All management programs should be designed to minimize impacts to fish or wildlife as well. A program of public involvement through notices and newsletters can help ensure positive and informed attitudes towards lake management.

Maintain Flexibility

The Town Lake management program should be dynamic, and responsive to monitoring in an unpredictable, newly created environment. Although traditional water quality problems will be encountered, Town Lake is unique in morphometry, hydrology, and source water, and other site-specific characteristics. New reservoirs commonly undergo an evolution in species and system productivity in response to management and invasions of plants and animals over several years. Likewise, Town Lake will evolve and remain dynamic for several years. Variable conditions will persist due to its unique location, combination of source water, maintenance water, stormwater inputs, and flushing from river flows. As such, the management program must be continuously responsive to lake conditions and should have a number of components in place (e.g. aeration, weed control, harvesting) for use with any of the maintenance water options. Chemical treatments and the unique option of lowering Town Lake to flush the system should be reserved for particularly intractable or severe water quality problems.

Cost

Annual costs for the management of Town Lake may fluctuate drastically depending upon the source water quality, season, temperature, stormwater discharges, public perception, and beneficial uses. The actual costs will not be apparent until the lake is created and maintained over a period of 5 to 10 years. A reasonable expectation for the range of annual maintenance costs is from $200,000 to over per $500,000 per year.

Section 8

Alternative Lake Implementation Concepts

Elements of this project were described in Sections 4 and 6. These elements include the dams, methods of seepage control, creation of a shoreline for the lake, pipelines to transport water to the lake, wells for recovering reclaimed wastewater stored in an underground aquifer, canal turnouts, and possible treatment process additions at either the existing Kyrene WRF or the future North WRF. These project elements can be selected to create numerous concepts for the final "total project" alternative.

This section presents four concept plans as examples of how the various elements of the project could be selected to provide insight into the estimated capital and annual costs associated with a "complete" lake and supporting infrastructure. These four concept plans are based on the primary differences between the sources of water supply as follows:

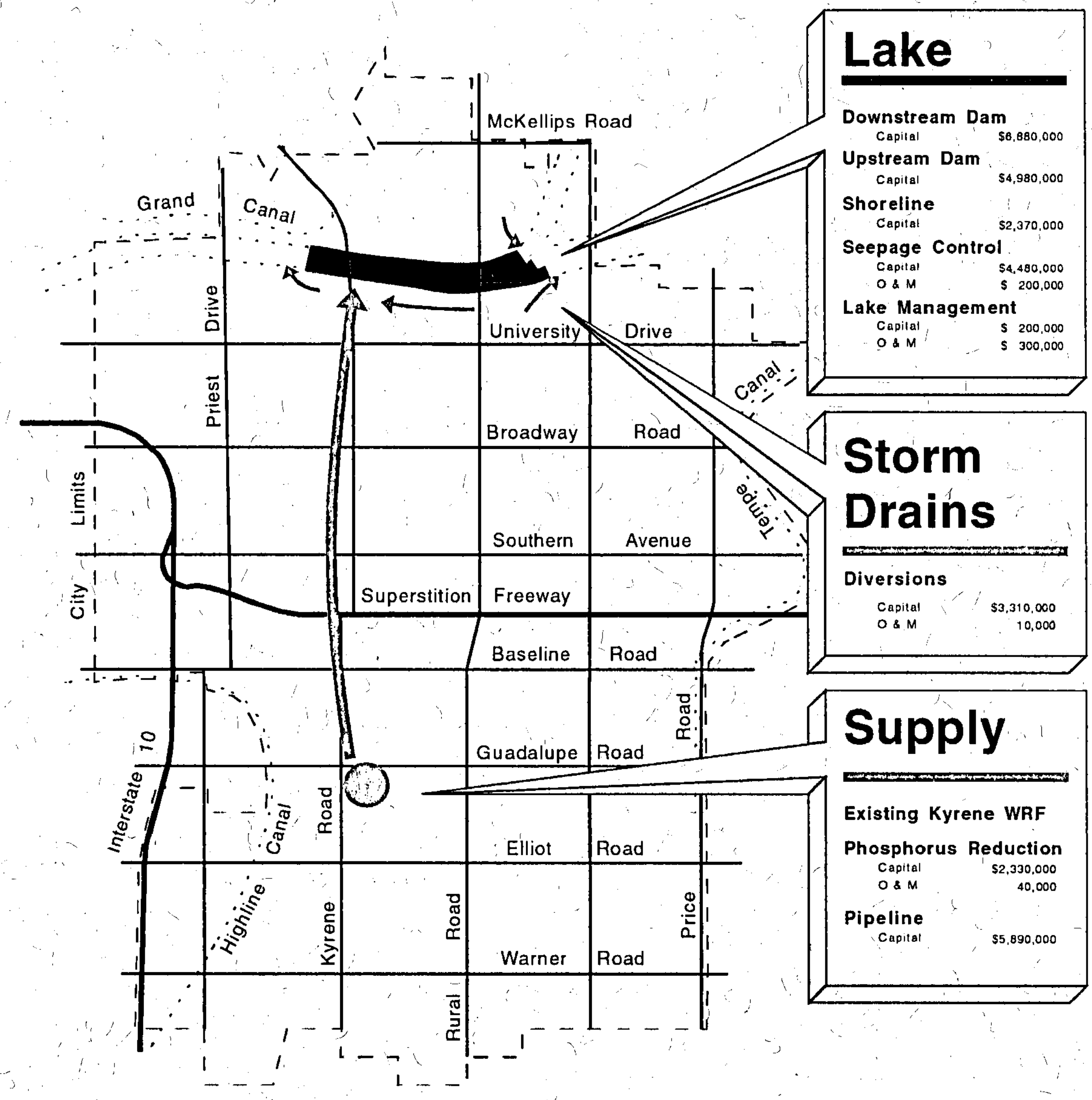
Concept 1. This concept uses the existing Kyrene WRF plus additional treatment focused on reducing the phosphorus content of the reclaimed water. The additionally treated reclaimed water is piped to the lake.

Concept 2. This concept uses the existing Kyrene WRF whereby the reclaimed water is stored in an aquifer via surface recharge techniques on city-owned land south of Elliot Road near Kyrene Road (the Hardy Farm site). The reclaimed water is recovered using wells north of Broadway Road near Mill Avenue, then piped to the lake.

Concept 3. This concept is based on supply from the future North WRF located south of the Rio Salado Parkway near Priest Drive.

Concept 4. This concept uses SRP water supplied from the SRP Tempe Canal. In this concept the lake would function as an SRP transport system, allowing movement of SRP water from the Tempe Canal to the Grand Canal, and also function as an equalizing reservoir in the SRP system.

These alternative "complete" lake projects are illustrated in Figures 8-1 through 8-4.



Concept 1

Capital $ 30,440,000
O & M $/yr. 550,000

Figure 8-1

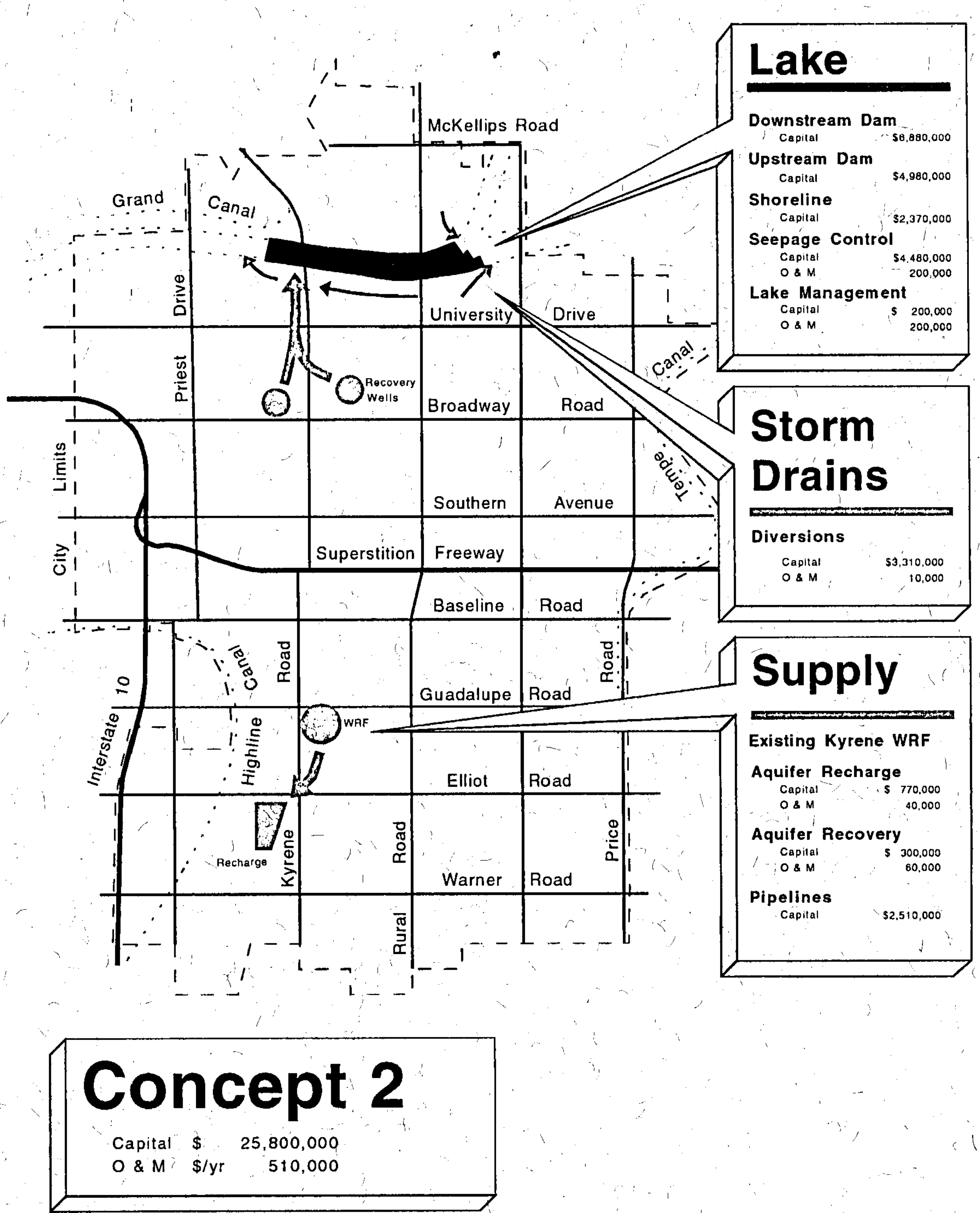

McKellips Road
Grand Canal
University Drive
Broadway Road
Southern Avenue
Superstition Freeway
Baseline Road
Guadalupe Road
Elliot Road
Warner Road
Priest Drive
City Limits
Interstate 10
Highline Canal
Kyrene Road
Rural Road
Price Road
Tempe Canal
Recovery Wells
WRF
Recharge
Lake
Downstream Dam
Capital $6,880,000
Upstream Dam
Capital $4,980,000
Shoreline
Capital $2,370,000
Seepage Control
Capital $4,480,000
O & M 200,000
Lake Management
Capital $ 200,000
O & M 200,000
Storm Drains
Diversions
Capital $3,310,000
O & M 10,000
Supply
Existing Kyrene WRF
Aquifer Recharge
Capital $ 770,000
O & M 40,000
Aquifer Recovery
Capital $ 300,000
O & M 60,000
Pipelines
Capital $2,510,000
Concept 2
Capital $ 25,800,000
O & M $/yr 510,000

Figure 8-2

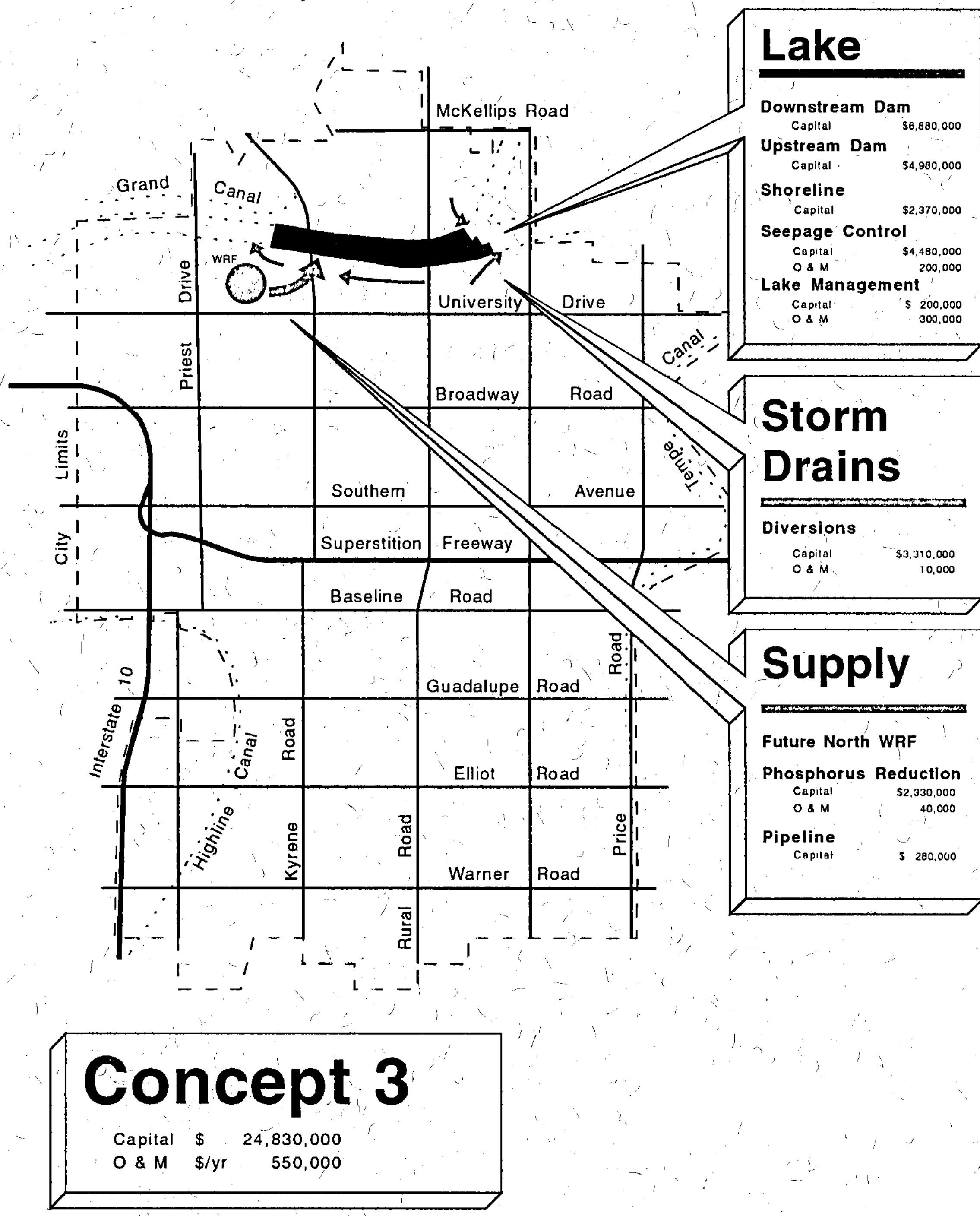

McKellips Road
Grand Canal
WRF
Drive
Priest
University Drive
Canal
Broadway Road
Limits
City
Southern Avenue
Tempe
Superstition Freeway
Baseline Road
Road
Interstate 10
Guadalupe Road
Canal
Road
Elliot Road
Highline
Kyrene
Road
Price
Warner Road
Rural
Lake
Downstream Dam
Capital $6,880,000
Upstream Dam
Capital $4,980,000
Shoreline
Capital $2,370,000
Seepage Control
Capital $4,480,000
O & M 200,000
Lake Management
Capital $ 200,000
O & M 300,000
Storm Drains
Diversions
Capital $3,310,000
O & M 10,000
Supply
Future North WRF
Phosphorus Reduction
Capital $2,330,000
O & M 40,000
Pipeline
Capital $ 280,000
Concept 3
Capital $ 24,830,000
O & M $/yr 550,000

Figure 8-3

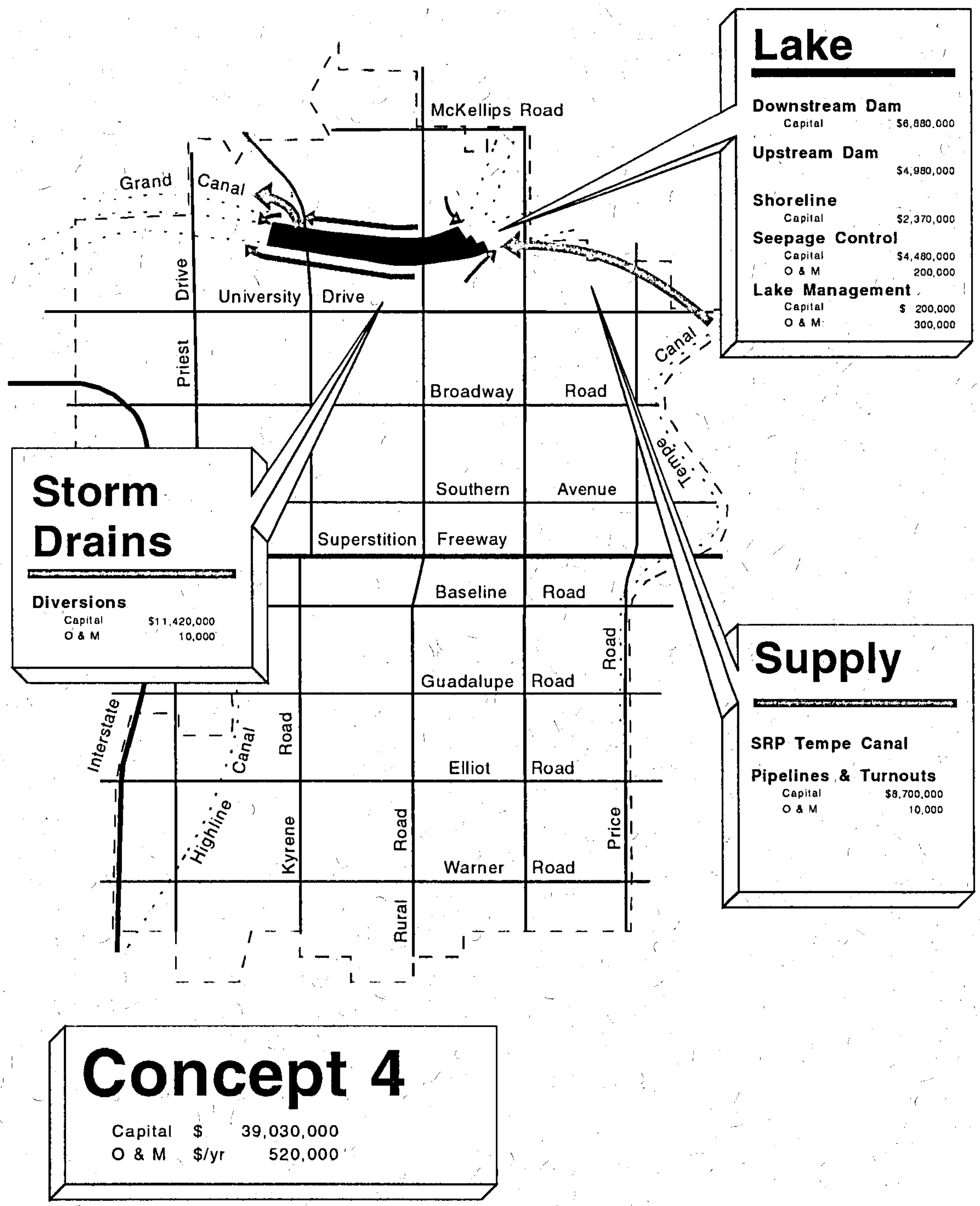

McKellips Road
Grand Canal
University Drive
Priest Drive
Broadway Road
Southern Avenue
Superstition Freeway
Baseline Road
Guadalupe Road
Elliot Road
Warner Road
Interstate
Highline Canal
Kyrene Road
Rural Road
Price Road
Tempe Canal
Lake
Downstream Dam
Capital $6,880,000
Upstream Dam
$4,980,000
Shoreline
Capital $2,370,000
Seepage Control
Capital $4,480,000
O & M 200,000
Lake Management
Capital $ 200,000
O & M 300,000
Storm Drains
Diversions
Capital $11,420,000
O & M 10,000
Supply
SRP Tempe Canal
Pipelines & Turnouts
Capital $8,700,000
O & M 10,000
Concept 4
Capital $ 39,030,000
O & M $/yr 520,000

Figure 8-4

These concept plans have project components that are common to all four concepts. Common elements are:

- **Lake construction techniques.** All four concept plans are based on a lake created by the construction of two dams. A downstream dam impounds the water that creates the lake. And an upstream dam creates a defined upstream boundary for the lake, plus serves as a means for limiting the amount of stormwater runoff that enters the lake from Indian Bend Wash and other storm drains via the Salt River upstream of the lake. The upstream dam, as a defined upstream boundary, serves to maintain deeper water at the boundary of the lake which should enhance water quality conditions in this area of the lake.

- **Lake shoreline.** All four concept plans employ revisions to the cement stabilized alluvium bank protection to provide for a uniform lake "shoreline." This shoreline consists of a low wall that provides a uniform water depth of approximately 18 inches along the north and south shorelines. The wall, when finally designed, should consider safety issues regarding unauthorized swimmer egress or emergency egress from the lake. The wall also provides a dry interface for construction of developer or City-owned improvements along the north and south banks. The lake should be generously posted with signs prohibiting swimming, wading, and most certainly diving into the lake from any point along the lake's perimeter.

- **Seepage control.** All four concept plans are based on using pumped seepage control as the primary method of controlling seepage losses. This approach uses slurry cutoff walls at the west end of the lake from the downstream dam location east to the vicinity of Mill Avenue. The area from approximately Mill Avenue to the upstream dam the seepage would be controlled with wells along the perimeter of the lake that would intercept the infiltration (bottom and side water losses) and return it to the lake. This method of seepage control has the lowest capital cost and lowest life cycle cost compared to the construction of a continuous liner system. This approach to seepage control is somewhat innovative and will require detailed hydrogeological fieldwork and analysis supporting the Aquifer Protection Permit. If this approach is ultimately not allowed by the permitting agencies, the more conventional, and more costly liner system would have to be used.

Discussion of Concept Plans

Each of the Concept Plans has unique advantages and disadvantages that must be considered before any one can be selected as the preferred plan for implementation.

Concept 1

By using the existing Kyrene WRF as the source of supply and piping the water directly to the lake, this concept will be readily accepted as a wastewater reclamation, water conservation project. This concept provides for additional treatment processes to reduce the nutrient content (phosphorus) of the reclaimed water such that the overall lake water quality will be acceptable. Reducing the phosphorus concentration of the supply water will help suppress algae growth and aquatic weeds, allowing reasonable lake maintenance procedures and levels of effort.

This concept requires that the Kyrene WRF have sufficient capacity to deliver the water requirements needed during the maximum month. During the month of July, maximum water demands have been estimated to be 3.1 million gallons per day (mgd). Currently, the Kyrene WRF does not produce sufficient water to supply this demand. Initial operating results (1992) indicate the plant is operating at approximately 2.6 mgd. The interim design capacity of the plant is 3.0 mgd (with future expansion to 6.0 mgd). Since the Kyrene WRF also provides reclaimed water to other sites, the Kyrene WRF currently does not have the capacity to supply all the water demands. Once flows reach the designed 3.0 mgd, the Kyrene WRF would still only marginally meet the summer demands. Since the Rio Salado demands vary from a summer maximum to a winter minimum of approximately 1.0 mgd, there are operational issues associated with the Kyrene WRF that would also need to be considered.

This concept requires a pipeline from the Kyrene WRF to the lake of approximately 5 miles. The estimated cost is based on an alignment following city rights-of-way. Construction of this pipeline will be disruptive and costly.

Concept 2

This concept is based on indirect reuse of the existing Kyrene WRF reclaimed water. This is considered indirect reuse because the reclaimed water is first stored in an aquifer via surface recharge methods, then withdrawn from the aquifer using conventional wells and piped to the lake.

This approach has several advantages over Concept 1. In this case, the Kyrene WRF can be operated at a steady rate, for instance, 2.6 mgd currently or 3.0 mgd (design) at some point in the future. Water is recharged to the aquifer at this steady rate. However, water is withdrawn via wells at varying rates depending on demand. The system of recharge and recovery could be operated such that the aquifer inputs and outputs would be equal on an annual average basis. This means that during the winter months when demands are low, excess water would be stored ("banked") in the aquifer. During the summer months, when demands exceed the design capacity of the Kyrene WRF (3.0 mgd), the peak demands would be supplemented with water that had been banked in the aquifer.

There are two additional advantages of aquifer storage and recovery (ASR) over direct reuse options. (1) Water can be recharged in one location and withdrawn at another location within certain regulatory constraints; and (2) the ASR provides an additional polishing step in improving the water quality of the recovered water, hence a potentially higher water quality goal for the lake. This concept is based on recovery wells located north of Broadway Road. This approach eliminates approximately four miles of expensive pipelines (compared to Concept 1), and provides for the opportunity to redevelop existing wells in lieu of constructing new recovery wells. To be conservative, this concept is based on using one existing well and constructing one new recovery well.

This concept provides for the highest water quality possible for the lake. This is the only Concept Plan that has the possibility of consistently meeting the high water transparency requirements for a lake that could be used for swimming.

The value of the land used for the surface recharge basins is not included in the estimated costs for this alternative.

Concept 3

This plan is identical to Concept 1 except that the supply is based on the future 6.0 mgd North WRF located near Priest Drive and the Rio Salado Parkway. At 6.0 mgd, more than enough water would be available to meet the peak month water demands of 3.1 mgd. A short distance of pipeline would be needed to convey reclaimed water (with added treatment for phosphorus) to the lake. This concept produces a lake water quality identical to Concept 1. Since the North WRF could potentially be treating wastewater from service areas containing industrial dischargers (versus the Kyrene WRF which is largely residential), some additional water quality monitoring could be required.

This concept is only viable if and when the North WRF is constructed.

Concept 4

This concept is based on the lake functioning as an urban reservoir in the SRP canal system. In addition, the lake would allow SRP to move water from the Tempe Canal to the Grand Canal, providing a diversity that is not currently possible. As an equalizing reservoir, water would enter the lake from the Tempe Canal at a relatively steady rate and be withdrawn and pumped into the Grand Canal at variable rates. These withdrawals to the Grand Canal could cause up to one-foot fluctuations in the lake water surface elevation.

The cost estimated for storm drain diversions and rerouting is higher in this plan than in Concepts 1, 2, and 3 because it is likely that greater care may be required to isolate the lake (now part of the SRP canal system) from storm drains. In general, SRP does not allow urban storm drains to be discharged to their canals. This issue would likely require a written negotiated understanding between the City and SRP in so far as the issue could become further complicated by NPDES requirements for stormwater discharges.

As part of the SRP canal system, this plan assumes that the net seepage and evaporation losses would be borne by SRP as incidental to the transportation losses inherent in the remainder of the SRP system.

Section 9
Implementation Plan

This section discusses implementation of the project beyond the feasibility study phase represented in this Engineering Report.

Preferred Plan Selection

The information contained in this report will be used by the City in ongoing water resource management planning to select a preferred source of supply to the Rio Salado Town Lake. Among the most significant issues affecting the decision on source of supply is the City's level of continued participation and use of the 91st Avenue regional wastewater treatment system. This decision affects the expansion of the Kyrene WRF and construction of other wastewater reclamation facilities. Decisions on water reclamation facility construction is also conversely related to the question of how much water is required to sustain a lake. Over the life of this Engineering Report, this latter question (How much water is required to create a lake?), was resolved by TM8, the Town Lake Feasibility Study as modified by this Engineering Report. With the water demands for the lake established, the City is now in the position to make the related decisions regarding the quantity of reclaimed water that is (or will be made) available.

Thus, selection of the preferred supply option is the first step in preparing a specific project proposal for initiation of the environmental permitting process. Once a specific plan is selected, then geotechnical and predesign activities can be performed to support the technical requirements of the environmental permits.

Geotechnical investigations could be considered for immediate implementation to the extent that the geotechnical results might aid in further defining the preferred plan. This would be true, for example, in the case of further hydrogeotechnical investigations that would more conclusively determine the feasibility and cost associated with the alternative lake seepage control technologies.

Unresolved Issues

Key unresolved issues that must be addressed in the context of the City's overall water resource management planning for Rio Salado are:

- Current and future availability of reclaimed water for Rio Salado in relation to the City's continuing role in the 91st Avenue regional wastewater treatment system.

- Salt River Project's interest in a defined role in the Rio Salado project. This role could vary from a limited role in operating the dams, to a larger role of SRP supply, ownership, maintenance, and operations of the lake for purposes of creating an SRP urban reservoir.

- Current state regulations and SRP's associated policies prohibit using the SRP canals for conveyance of reclaimed water. This prohibition is on a legal-administrative basis and has no bearing on the quality of the reclaimed water. Reclaimed water can often be of higher quality than SRP water. This prohibition affects water trades, and water rights trades for supply alternatives that involve indirect reuse of Kyrene WRF reclaimed water. At present, Kyrene WRF reclaimed water would have to be piped to a dedicated agricultural user rather than simply discharging to the nearest canal to affect an agricultural exchange.

- Level of stormwater protection required by SRP for the supply options incorporating the lake as an integral part of the SRP canal system.

- Use of the Southern Pacific Railroad right-of-way for the pipeline between the Kyrene WRF and the lake is a significant cost factor affecting the alternatives that need a piped route from the Kyrene WRF to the lake. The viability of this route could be established using the information presented in this report by City right-of-way staff in discussions with the railroad. This report has estimated an alternative, but higher cost route for planning purposes herein.

Implementation Strategy

The major technical issue affecting implementation of Rio Salado relates to the technical basis for obtaining the required environmental permits, most importantly, the ADEQ APP. A successful permitting strategy should be based on "zero" impact on the hydrogeology of the project area. All planning to date has been performed with this approach in mind. This approach is intended to conserve water and to isolate the lake from the surrounding aquifer systems. Hazardous waste sites, both Superfund and non-Superfund sites exist in the vicinity of the project. Varying levels of data exist concerning these sites. In general however, it is widely accepted that owners of water projects should not adversely alter groundwater conditions in the vicinity of landfills, thereby exacerbating potential groundwater contamination problems.

By maintaining a strict program of lake isolation from the local groundwater, the project would be viewed most favorably by the permitting agencies.

The financial plan for implementing the Rio Salado project is being addressed independent of this project by City staff.

Project Delivery

The continuing phases of the project, from an engineering reference, are:

- Preferred Plan Selection
- Geotechnical Investigations
- Predesign
- Pilot Testing (Optional for Some Supply Alternatives)
- Preliminary Operations Plan
- Preliminary Safety Plan
- Environmental Permitting
- Design
- Construction Permitting
- Bid and Award of Construction Contracts
- Construction
- Final Operations Plan
- Final Safety Plan
- Startup and Operations

Each of these activities, in the context of Rio Salado are briefly described in the following pages.

Preferred Plan Selection

This Engineering Report describes five alternative water supply plans. Most of the plans have two variations, for a total of nine primary supply scenarios. Some of these nine scenarios have additional considerations of varying levels of treatment (for phosphorus removal for example) prior to lake supply. In addition, there are three viable technologies for controlling lake seepage losses. Among the possible combinations and permutations of these project components a "Preferred Plan" must be selected. The "Preferred Plan" will form the basis for all following technical phases of the project. Section 8 of this report provided four examples of Concept Plans that either in part or combination could become the preferred plan.

Geotechnical Investigations

Further geotechnical investigations are required to (1) provide documentation for the environmental permits (ADEQ APP and ADWR Reuse Permit); (2) establish foundation requirements for the dams; (3) establish design data for pipeline construction, (4) further define the cost-effectiveness of the alternative seepage control technologies; and (5) provide other geotechnical information supporting the design of the project.

The most critical geotechnical work is independent of the preferred water supply alternative selection and could be implemented at any time. The hydrogeotechnical work associated with seepage control will require about six months of exploration and analysis. Thus, when the project predesign-design schedule is set, this work should be programmed accordingly.

Predesign

Predesign of the lake seepage control system, the dams, and pipelines should be scheduled with the supporting geotechnical studies. Some predesign will be necessary to support the environmental permitting process. Most agencies require sufficient engineering drawings and specifications to adequately define the proposed project at the time of permit application. The permitting process cannot usually be completed without final engineered drawings and specifications.

Pilot Testing (Optional for Some Supply Alternatives)

Pilot testing will range from desirable (but optional) for some of the supply alternatives, to required for others as part of permitting requirements. More testing of phosphorus content of the Kyrene WRF reclaimed water could be performed immediately with modest cost. Testing of alum treatment schemes for phosphorus reduction could also be performed at modest cost. Other pilot testing at the Kyrene WRF may be desirable if Kyrene is selected as the preferred source of supply (on direct reuse basis). Pilot testing could also be applicable to supply alternatives involving well recharge and recovery systems.

If appropriate, pilot testing should be performed as part of the predesign phase activities. Testing is ongoing regarding the feasibility of using the Hardy Farm site for surface spreading recharge basins. Preliminary data are encouraging.

Preliminary Operations Plan

After the preferred supply plan is selected a preliminary plan for operation of the dams and supply system would be useful for permitting, detailed coordinating and possible contractual arrangements with SRP, and further establishing design criteria for automated control systems if such systems are preferred.

Preliminary Safety Plan

A preliminary safety management plan for the lake could be developed in conjunction with the Operations Plan or as a separate document. The safety plan would address security for the inflatable dams, fencing, swimming prohibitions, search and rescue requirements, safety patrols, fishing prohibitions, boating rules and regulations, considerations for participant and spectator viewing of water sport events, evacuation procedures, "failsafe" dam deflation modes, and other security issues.

Environmental Permitting

Selection of the preferred plan will determine what regulatory agencies and permits will be required to implement the project. The process of environmental permitting should begin soon after the preferred plan is selected with pre-application meetings (with agencies that have the pre-application process). Following the pre-application meetings, information from the predesign phase will be necessary to complete the permit applications. The recommended approach to environmental permitting is to focus first on the Aquifer Protection Permit (ADEQ) followed by the Reuse Permit (ADWR), then all other permits.

The preferred plan will also determine the complexity of the permitting program. For example, a plan that incorporates Ranney well collectors for lake seepage control will require more documentation and permit negotiations than lake seepage control based on a constructed liner system. Also, any plan that incorporates ASR will require more permitting effort than a plan that does include ASR.

Once the permitting process is started it should continue aggressively through completion of the project. Most of the permits will not be issued in final form until after construction is complete. For this reason, funding for the defined elements of the preferred plan should be identified at the outset of the permitting process. Extraordinary delays during the predesign, design, construction phases, could result in a very frustrating and inefficient permitting experience.

The following is a summary of the environmental permits.

Environmental Permits

A variety of permits are required to implement the Rio Salado project. Some of the permits may or may not be required, depending on the source water used to fill Town Lake. Following is a description of the permits and their applicability to the project.

National Pollutant Discharge Elimination System Permit. The National Pollutant Discharge Elimination System Permit (NPDES) is administered by the U.S. EPA. ADEQ prepares the preliminary draft permit using technical information supplied by the applicant.

A NPDES permit is required for point source discharges to waters of the U.S. Both EPA and ADEQ consider the Salt River to be waters of the U.S. This permit is required for all sources of water that contain any contaminants. All of the potential water sources would require a NPDES permit except Salt River water. The water discharged will have to meet the requirements of Title 18, Chapter 1, Article 1, Water Quality Standards for Navigable Waters.

Section 404 Permit. A Clean Water Act Section 404 Permit is required when altering or disturbing an area greater than one acre within the 100-year floodplain of a water of the U.S. The 404 Permit is administered by the U.S. Army Corps of Engineers with input from various federal and state agencies. This permit will be required for construction of Town Lake and is independent of the water source.

Reclaimed Wastewater Reuse Permit. A reclaimed Wastewater Reuse Permit, administered by ADEQ, is required when reclaimed wastewater is used for beneficial purposes. For the Rio Salado project, a reuse permit is required if reclaimed water is used for landscape or turf irrigation outside the 100-year floodplain. Any use of reclaimed water within the 100-year floodplain requires a NPDES permit. An annual water balance must be prepared to obtain a Reuse Permit.

Aquifer Protection Permit. An Aquifer Protection Permit (APP) is required for surface impoundments (Town Lake) and underground storage and recovery projects. The water source used will affect the monitoring requirements of the APP. APPs are issued by ADEQ. A hydrogeology report is required to obtain an APP.

Dam Safety Permit. Dam Safety Permits (DSPs) are required for construction of dams that exceed 6 feet in height and 50 acre-feet of storage. DSPs will be required for construction of the dams that form Town Lake. ADWR administers the dam safety program in Arizona. ADWR reviews dam designs prior to issuing the permit. The water source will not affect the permits.

Underground Storage and Recovery Permit. An Underground Storage and Recovery (USR) Permit is required when water is recharged, stored underground, and then recovered. During the USR Permit processing, ADWR coordinates with ADEQ who will be processing the APP concurrently. The hydrogeology report prepared for the APP is submitted to ADWR for the USR Permit. Reclaimed water is the only water source currently being considered for underground storage and recovery.

Appropriation Permit. An Application for a Permit to Appropriate Surface Waters (Appropriation Permit) is required if Salt River water is stored within Town lake. ADWR administers the appropriation of surface waters. Use of other water sources in Town Lake would not require an Appropriation Permit.

Aquatic Wildlife Stocking Permit. To stock Town Lake with fish, an Aquatic Wildlife Stocking Permit is required from the Arizona Game and Fish Department. The permit will describe the types and amounts of fish allowed in Town Lake.

Design

The elements of the preferred plan should be evaluated for appropriateness for separate design and construction scheduling. For example, the dams (upstream and downstream) could be separate projects or combined. Likewise, the seepage control system could either be included or separated from the construction contract for the dams. It may also be appropriate to consider two contracts for each dam, one for foundations and concrete structures, and the second contract for inflatable dam purchase, delivery, and installation. Separating the inflatable dam purchase could accommodate a long lead time in the manufacture of the dam plus eliminate general contractor markups and profit that could increase the overall cost of the dam(s). Other elements of the preferred plan such as pipelines, AWT processes at the water reclamation facilities, and ASR systems could be designed and implemented under separate contracts.

Construction Permitting

Pipelines associated with the preferred plan could require permits from the Southern Pacific Railroad, ADOT, and coordination with telephone, gas, and electric utilities. The dams will require a permit from the ADWR, and coordination and permits from Maricopa County and the Corps of Engineers. All facilities will be subject to construction permitting in one form or another. Definition of all the construction permits will follow selection of the preferred plan.

Bid and Award of Construction Contracts

All project elements can follow standard City of Tempe capital projects bidding and award policies and procedures. In considering sources of grant and loan funds from state and federal agencies, the City should consider implications of these programs on the design and construction phases. For example, some state and federal programs have had prohibitions on owner preferences for equipment selection and foreign equipment. Since manufacturers of air inflatable dams are limited, these types of restrictions could have significant impact on the materials that could be designed into the project.

Construction

Individual elements of the preferred plan could have construction durations ranging from 6 months to 18 months, depending on how the individual projects are defined for the bidding and construction phase. Overall, it is probable that the construction phase could be limited to an 18-month envelope, but 24 months is a more appropriate time frame for planning purposes until after the preferred plan is selected.

Final Operations Plan

The Final Operations Plan would be a refinement of the Preliminary Operations Plan and would include any changes that became necessary during the final phases of design, permitting, or dam operations agreements with SRP.

Final Safety Plan

Similar to the Final Operations Plan, the Final Safety Plan would incorporate any changes that followed the Preliminary Safety Plan. In actual practice, the Final Safety Plan would probably require periodic updating for some time after construction of the facilities is completed to accommodate adjustments for actual operations experience.

Startup and Operations

Startup of the individual elements of the preferred plan will be similar to the City's normal practice for the existing water and wastewater facilities. The equipment that operates the inflatable dams is typical of the mechanical equipment that City staff are familiar with as part of the existing water and wastewater facilities. The unusual aspect of startup and operations will be the timing and sequencing of operations that inflate and deflate the dams in conjunction with Salt River flows. Under any of the preferred plans, operation of the dams could be performed by Salt River Project.

Section 10
Cost Opinions

The opinions of cost shown in this report, and any resulting conclusions on project feasibility or budget requirements, have been prepared for guidance in project evaluation and implementation from the information available at the time the opinion was prepared. The final costs of the project and resulting feasibility will depend on actual labor and materials costs, competitive market conditions, actual site conditions, final project scope, implementation schedule, continuity of personnel and engineering, and other variable factors. As a result, the final project costs will vary above and below the opinions of cost presented herein. Because of these factors, project feasibility, benefit/costs ratios, risks, and funding needs must be carefully reviewed by the City prior to making specific financial decision or establishing project financial budgets to help ensure adequate funding.

Order-of-Magnitude Cost Estimating Methodology

This is an estimate made without detailed engineering data. Some examples would be: an estimate from cost capacity curves, an estimate using scale up or down factors, and an estimate based on a ratio of cost comparing the cost of one facility to another. Costs are based in general price levels for labor and materials delivered during 1992. Most estimates prepared in this report are consistent with Order-of-Magnitude methods.

Budget Level Cost Estimating Methodology

This budget applies to the City's budget and not to the budget as a construction budget control document. Preparation of a budget estimate requires the use of flow sheets, layouts, and equipment details plus input from the City regarding allowances and contingencies that the City normally expects to include in projects of a given type or level of risk.

Definitive Level Cost Estimating Methodology

This is an estimate prepared from very defined engineering data. The engineering data includes as a minimum, 85- to 95-percent complete plot plans and elevations, piping and instrument diagrams, one line electrical diagrams, equipment data sheets and quotations, structural sketches, soil data and sketches of major foundations, building sketches and a complete set of specifications. Typically a definitive estimate would be made from "Approved for Construction" drawings and specifications. None of the estimates presented in this report have been prepared using detailed plans and specifications.

Probable Cost Scenarios

Based on information available, and data that could be reasonably generated as part of this Engineering Report, the probable cost scenarios are based on Order-of-Magnitude and Budget Level estimating methodologies. Further, the Probable Cost Scenarios have been established to reflect engineering opinions regarding the likely cost for the component estimated. Where there is some expectation that a higher cost could result due to information not currently available this report includes Contingent Cost Scenarios.

Contingent Cost Scenarios

Some components of the project are sensitive to rock excavation, negotiated permit conditions that could require additional factors of safety or equipment redundancy, and other conditions that are not possible to fully define at this level of study. In situations where these circumstances are recognized, at least for planning purposes, a higher cost outcome has been presented. Alternatives and project elements that might be affected by these higher cost outcomes are discussed in the text with companion Probable and Contingent cost estimates. The Contingent Cost Scenarios reflect a high degree of engineering judgement and experience with representative similar projects, but do not guarantee a maximum cost based on the limitations of the Order-of-Magnitude and Budget Level estimating methodologies.

Allowances

All estimates include allowances for design and construction unknowns (contingency), bonds, insurance, administration, and engineering. Contingencies range between 20 and 30 percent depending on the type of facility estimated. The contingency is generally lower for pipelines and higher for ASR and treatment plant construction. A flat 15 percent has been included for engineering. Engineering will, of course, vary below and above the 15 percent level for separate elements of the project but the overall average is reasonable for the level of decisionmaking required to select among the various alternatives.

Glossary of Abbreviations

ac-ft	Acre feet
ADEQ	Arizona Department of Environmental Quality
ADWR	Arizona Department of Water Resources
ADOT	Arizona Department of Transportation
APP	Aquifer protection permit
ASR	Aquifer storage and recovery
CAP	Central Arizona Project
CFM	Cubic feet per minute
cfs	Cubic feet per second
CMF	Continuous flow microfiltration
CSA	Cement stabilized Alluvium
DBPs	Disinfection by-products
EPA	Environmental Protection Agency
FCDMC	Flood Control District of Maricopa County
GAC	Granular activated carbon
IBW	Indian Bend Wash
MCHD	Maricopa County Health Department
mg/l	Milligrams per liter
mgd	Million gallons per day
N	Nitrogen
NPDES	National Pollutant Discharge Elimination System
P	Phosphorus
PLC	Programmable logic controller
RO	Reverse osmosis
SCADA	Supervisory Control and Data Acquisition
SRP	Salt River Project
SRPMIC	Salt River Pima-Maricopa Indian Community
SRT	Solids retention time
TDS	Total dissolved solids
TM	Technical memorandum
TOC	Total organic carbon
TSS	Total suspended solids
USBR	U.S. Bureau of Reclamation
USR	Underground Storage and Recovery
WAS	Waste activated sludge
WRF	Water (wastewater) reclamation facility
WTP	Water treatment plant
WTS	Constructed Wetlands