7.0 ENGINEERING AND DESIGN FOR WATER REUSE

7.1 OVERVIEW

The following chapter presents the design and size of the facilities required to implement a conjunctive use project between the Fallbrook PUD and the Base. A major component of the conjunctive use project, the recycle and reuse program, includes all the facilities required to discharge and capture tertiary treated wastewater from the Fallbrook PUD's Wastewater Water Treatment Plant. Alternative 3, the diversion and recharge component of the conjunctive use project, is explained in detail in Chapter 7 of the Permit 15000 Study. These facilities include the installation of an Obermeyer Dam, increased capacity to the diversion structure, improvements to O'Neill ditch and the existing recharge ponds, construction of two additional recharge ponds, and installation of six new ground-water wells.

The design of the recycle and reuse component relied on the results from both the surface runoff modeling and ground-water modeling. In iterative process between modeling and design was utilized to optimize the size of the recycle and reuse components. For example, the sizing of the reservoir and related facilities was completed following results from the ground-water analysis. Many future model scenarios were constructed and analyzed in order to maximize the beneficial use of the Fallbrook PUD's additional water supply. Following the optimization of the ground-water component, surface water runoff models provided the engineers with the characteristics of the watershed upstream of the reservoir.

The design and sizing of the all facilities was completed for each of the three discharge scenarios: 1,500 AFY, 2,500 AFY, and 3,500 AFY. While all the results are presented in the Appendix, the following chapter only addresses the Alternative 10, 2,500 AFY scenario. The size and design of the recycle and reuse facilities are the same for each of the three scenarios of both Alternatives 9 and 10.

7.2 ENGINEERING FACILITIES

This section describes the type and size of the recycle and reuse facilities that were designed in connection with Scenario 2 (2,500AFY), the release of tertiary treated wastewater. The basic components of the recycle and reuse system include:

1. Pipeline from Outfall to the Treatment Wetland (Wetland Pipeline)
2. Treatment Wetland
3. Earth Embankment Dam and Storage Reservoir
4. Pipeline from Storage Reservoir to the Santa Margarita River (Reservoir Discharge Pipeline)

5. “Alternative 3” facilities associated with the Permit 15000 Study
    A. New Santa Margarita River Diversion Dam
    B. Capacity Improvements to O’Neill Ditch
    C. New Recharge Pond Nos. 6 and 7

Figure 7-1 shows the proposed locations of the major recycle and reuse system facilities. The facilities listed above were also designed for Scenario 1 (1,500 AFY) and Scenario 3 (3,500 AFY), using the assumptions and techniques described below. Design details of the recycle and reuse facilities associated with Scenarios 1 and 3 are presented in the Appendix.

A site visit and field investigation was performed by Stetson Engineers during September 2001 to support the engineering and design effort. The purpose of the visit was to visually examine the proposed pipeline alignments and the proposed wetland and reservoir sites. General observations were made of the vegetative cover, topography, existing structures and facilities, soils, and geology. The information obtained during the site visit will be included in the discussion of each major facility described below.

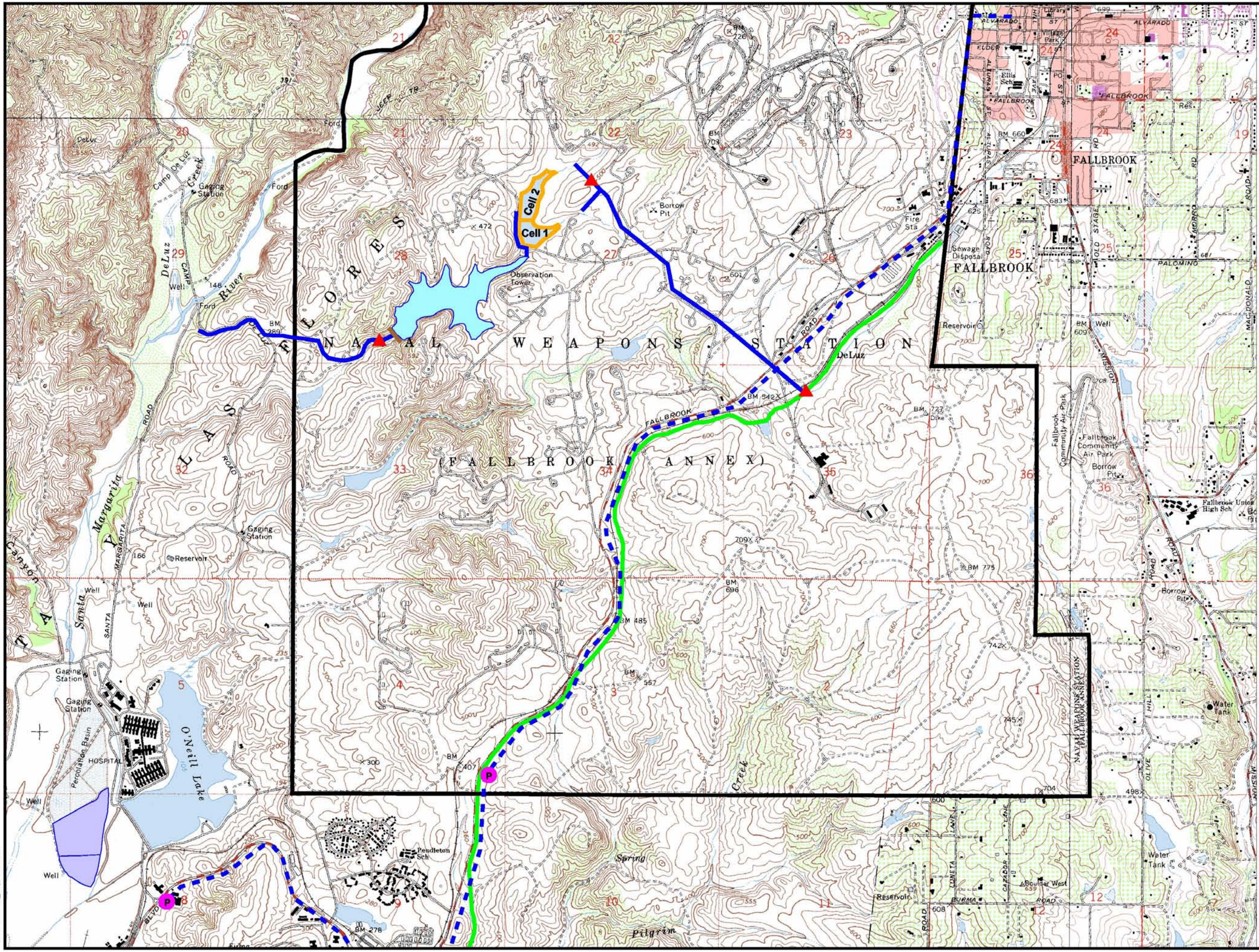
7.2.1 Pipeline from Outfall to Wetland (wetland pipeline)

A buried pipeline will be utilized to convey tertiary treated wastewater from the existing 16-inch ocean outfall pipeline to the proposed treatment wetland. The wetland pipeline was designed to operate entirely under gravity flow from the ocean outfall to the points of discharge just upstream of the treatment wetland. The wetland pipeline will be a 12-inch high density polyethylene pipe (HDPE) with invert elevations of 556 feet mean sea level (msl) at the outfall pipeline to 431 feet msl at the point of discharge. The total length of HDPE pipe required for the wetland pipeline will be approximately 9,000 feet.

7.2.1.1 Existing Site Conditions and Proposed Alignment (Wetland Pipeline)

The proposed wetland pipeline connects to the ocean outfall pipeline at a location approximately one mile southwest of the Fallbrook PUD WWTP and one quarter mile west of Ammunition Road. At this location, flow control valves will be installed on the existing outfall pipeline and proposed wetland pipeline to control the rate of flow to the treatment wetland. From its connection with the outfall pipeline, the wetland pipeline will travel in a northeast direction over mostly flat, sparsely vegetated terrain for approximately one half mile. At the northern end of this leg, the pipeline will cross two small ephemeral creeks located in a wide swale, at an elevation of 530 feet msl before following existing roads on the NWS. The road along the pipeline alignment is a well maintained gravel and earth road. The pipeline will follow this road for approximately one half mile before traveling cross-country towards the treatment wetland. At an elevation of approximately 480 feet msl, the main wetland pipeline divides into

FIGURE 7-1



PROPOSED PROJECT FACILITIES

ALTERNATIVE 10, SCENARIO 2

SUPPLEMENTAL FEASIBILITY STUDY FOR THE SANTA MARGARITA RIVER RECHARGE AND RECOVERY ENHANCEMENT PROGRAM

Proposed Reservoir
Proposed Treatment Wetland
Proposed Recharge Pond Nos. 6 & 7
Proposed Pipeline
Proposed Fallbrook Return Pipeline
Existing Outfall Pipeline
Proposed Dam
Fallbrook Naval Weapons Station
Proposed Pump Station
Proposed Flow Control Location

N W E S

0 0.25 0.5 Mile

STETSON ENGINEERS INC.

alt10scen2fac_c.apr 07/03/02

two segments which will deliver water to two natural drainages located upstream of the treatment wetland.

The natural drainages above the proposed treatment wetland are densely vegetated with plants ranging from small scrubs and grasses to medium growth trees. In order to facilitate installation of the wetland pipeline, the alignment will not follow the main section of drainage but will approach the natural drainages perpendicularly. The pipelines will discharge at an elevation of approximately 431 feet msl, above each of the two cells of the wetland.

The wetland pipeline alignment shown in Figure 7-1 was selected because: 1) the alignment provides the head required to facilitate gravity flow throughout the pipeline, 2) the alignment takes advantage of travel parallel to existing roads in order to facilitate the installation process, and 3) the alignment provides flow to both treatment cells of the wetland using a minimum pipeline distance.

7.2.1.2 Design Considerations (Wetland Pipeline)

The proposed wetland pipeline was designed to convey the projected average daily flow of reclaimed water released from the Fallbrook PUD while maintaining a flow velocity under 6 feet per second. The design flow rate for Scenario 2 was 3.5 cfs (2.3 MGD). A 12-inch pipeline was determined to provide the capacity requirement for projected average daily releases from the Fallbrook PUD WWTP with an allowance for greater operational flexibility and increased flows during peak release periods.

Valves and meters were incorporated into the design and cost of the wetland pipeline in order to allow for flow control, maintenance, and flow measurement. In addition to the control valves at the ocean outfall-wetland pipeline connection, a third control valve will be located near the treatment wetland to allow the control of water into one or both cells of the treatment wetland. Isolation valves to be installed in the ocean outfall pipeline and the wetland pipeline will allow the wetland pipeline and its appurtenant facilities to be taken out of service as necessary for maintenance, inspection, or repair. Air/vacuum release valves will be located at appropriate locations along the wetland pipeline to prevent potentially damaging vacuum forces that may occur. A venturi meter is included in the wetland pipeline design and cost estimate to accurately totalize flow and record reclaimed water deliveries to the treatment wetland. Pressure reducing valves will be installed on each of the pipe segments that discharge to the natural drainages above the treatment wetland to dissipate the energy of the reclaimed water before it flows to the treatment wetland cells. All valves will be placed in concrete vaults as necessary to allow for easy access and maintenance.

Concrete outlet structures designed to dissipate the energy of the reclaimed water will be constructed at both proposed wetland pipeline discharge locations. The reclaimed water will pass over a concrete weir in the outlet structure and through a section of rip rap prior to flowing down the natural drainage into the treatment wetland.

A general schematic of the wetland pipeline profile, proposed valve configurations, and their approximate locations along the pipeline is shown in Figure 7-2. Also included in Figure 7-2 is the estimated hydraulic grade line of the proposed wetland pipeline, at the design flow rate of 3.5 cfs, from the ocean outfall pipeline to the treatment wetland.

7.2.2 Treatment Wetland

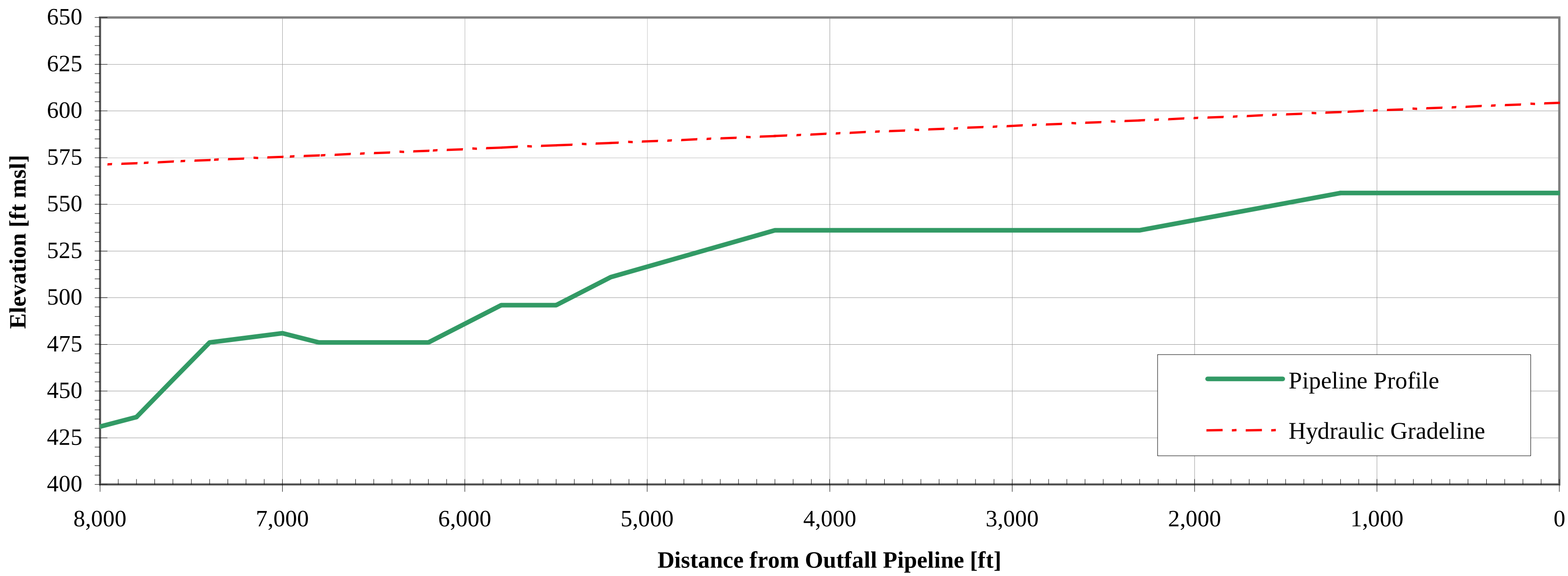
An 18-acre treatment wetland was designed for Scenario 2. The treatment wetland will receive reclaimed water at an average rate of 3.5 cfs and provide a hydraulic retention time (HRT) of at least 4 days to allow for denitrification prior to discharge into the a proposed storage reservoir.

The primary role of the treatment wetland is to "polish" the tertiary treated effluent released from the Fallbrook PUD WWTP. The wetland will reduce nitrate concentrations in the effluent water to a level that is acceptable for discharge to the storage reservoir and subsequently to the Santa Margarita River. The RWQCB requires that water discharged to the Santa Margarita River must have a total nitrogen concentration less than or equal to 1.0 milligrams per liter (mg/l). Nitrogen (NO_3-N) concentrations in the Fallbrook PUD treated wastewater effluent have been known to reach levels up to 15.0 mg/l, with average year 2000 reported concentrations of approximately 7.0 mg/l. It is necessary to maintain nitrate concentrations less than 10 mg/l for public health concerns and less than 1.0 mg/l to prevent excessive algal growth in the storage reservoir.

7.2.2.1 Existing Site Conditions (Treatment Wetland)

The site for the proposed treatment wetland is an existing water body named Depot Lake. Depot Lake occupies between 15 and 20 acres depending on water levels. A visual investigation of the site indicated that the lake is mostly open water with an estimated average depth of 10 feet. The deepest portion of the lake is in the south end and may reach a depth of 15 feet. The lake is formed by an earth levee located at the south end of the lake. The levee has a well maintained paved road along its crest and its outlet works consist of two uncontrolled concrete box culverts which pass under the roadway. The culverts are each 4 feet high by 11 feet wide and lead to a wide trapezoidal concrete spillway that drains into the heavily vegetated natural drainage below the dam. It was estimated that the invert elevation of the spillway is at least 10 feet above the current water surface. The box culvert and spillway on Depot Lake is the only

Generalized Profile and Schematic of Proposed Pipeline from Outfall Pipeline to Treatment Wetland
Scenario #2: 2500 AFY Release from Fallbrook



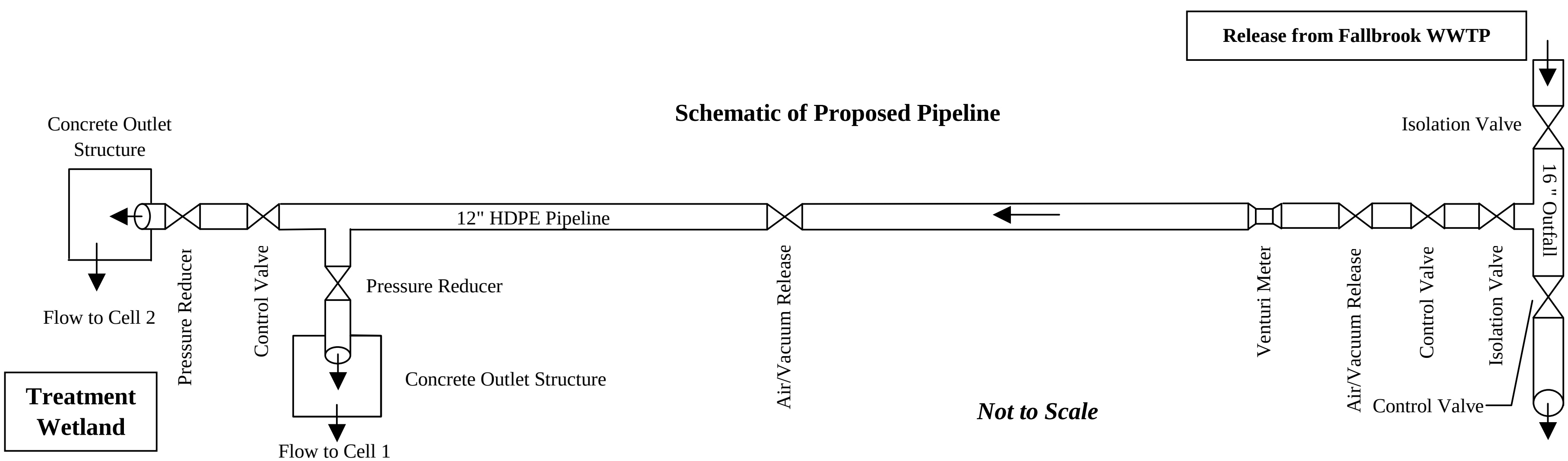


FIGURE 7-2

existing outlet and will be retained in the design of the proposed wetland for emergency spill purposes.

7.2.2.2 Design Considerations (Treatment Wetland)

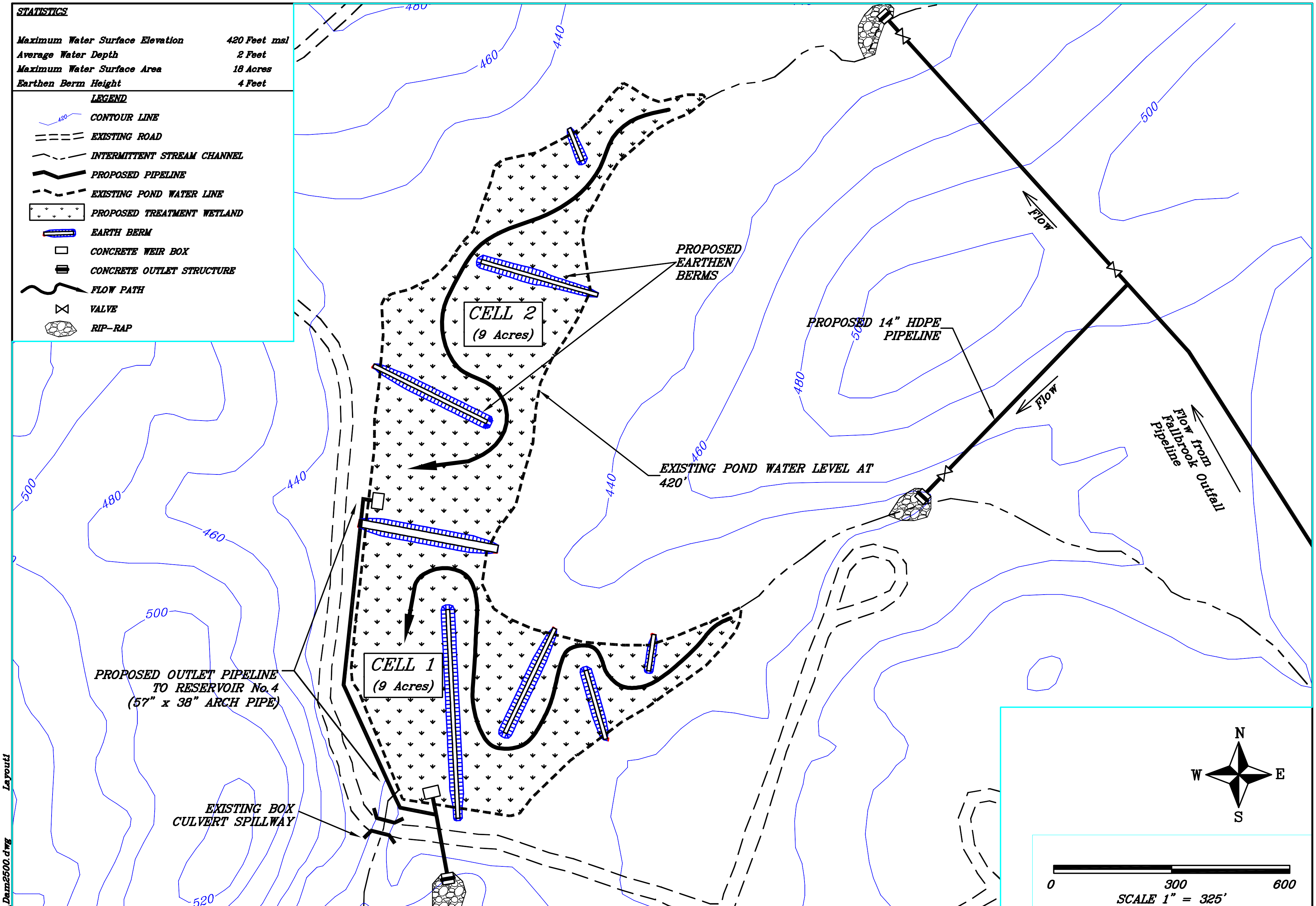
As previously mentioned, the proposed treatment wetland was designed with an average HRT of 4 days. The HRT is the total time required for water to flow from the wetland inlet to the wetland outlet. The treatment wetland was also designed to maintain a constant flow depth of 2 feet. The HRT and flow depth will provide the time necessary for adequate nitrogen uptake by the wetland plants. The HRT and design inflow rate were used to estimate the area needed to construct the treatment wetland. The total surface area for the Alternative 10, Scenario 2 treatment wetland is 18 acres, which corresponds to the approximate size of the existing Depot Lake.

The treatment wetland will be constructed with two individual cells, each approximately 9 acres in size, to allow for operational flexibility and ensure continuous operation of the treatment wetland should one of the two cells need to be temporarily removed from service. Both cells of the treatment wetland will have separate inlet and outlet structures and will be divided by a low earth levee to allow the cells to be operated independently from each other.

The water discharged from the treatment wetland pipeline will enter the treatment wetland via two natural drainages located upstream of each of the two cells of the wetland. The water will travel approximately 500 feet from the outlet structure of the pipeline, along the natural drainage, to the entrance of the wetland. There are no control structures at the wetland inlet because the wetland pipeline and its outlet structures were designed to regulate the flow entering each cell.

A single earthen berm will divide the treatment wetland to form two cells and prevent passage of water between the cells. Additional berms will be constructed to direct the flow path through the treatment cells. The berms will be 4 feet in height and will provide 2 feet of freeboard above the normal water surface of the wetland. The earthen berms are designed to prevent any short circuiting of flow through the treatment wetland. Figure 7-3 shows the details of the proposed treatment wetland including its two cells, berms, flow paths, and outlet structures.

The treatment wetland was designed to maintain a water surface elevation of 420 feet and an average flow depth of 2 feet. Excavation and grading is required to achieve the design water surface elevation and flow depth. The volume of fill required to set the bed of the treatment wetland at 418 feet was estimated using *Land Development Desktop* software and contours



Prepared for Fallbrook Public Utility District	STETSON ENGINEERS INC.	PROPOSED TREATMENT WETLAND SCENARIO #2 (2,500 AFY RELEASE FROM FALLBROOK) FALLBROOK SUPPLEMENTAL FEASIBILITY STUDY	EXISTING AND PROPOSED FACILITIES

DATE October 15, 2001
SCALE 1" = 325'
PROJECT No. 1922

FIGURE 7-3

F:\data\1922\CAD\Dam2500.dwg Layout1

digitized from USGS 7.5-minute topographic maps. It was estimated that approximately 56,000 cubic feet of soil will be needed to create the berms and grade the existing lake to a bed elevation of approximately 418 feet. The bed of the treatment wetland invert will be gently sloped toward the outlet of each of the treatment cells. The cells will be vegetated throughout except near the inlets and outlets where the bed invert is 3 feet lower to prevent vegetative growth and subsequent clogging of the structures.

Both cells of the treatment wetland are designed with a concrete weir outlet structure. The weir crest elevation is at 420 feet msl to ensure a flow depth of two feet. The weir outlet structures will each be 7 feet wide by 6 feet deep and 10 feet long. The weir outlet structures will be equipped with steel trash racks. The trash racks will prevent clogging by any floating debris or vegetation.

From the outlet structures of the treatment wetland, the water will flow through a 57-inch by 38-inch arch corrugated metal pipe (CMP). The flow from Cell 2 will travel through an arch CMP and join the outlet flow from Cell 1 before passing through a proposed pipeline in the existing dam where. Riprap will be placed at the upstream end of the storage reservoir to dissipate energy in the water received from the wetland.

7.2.2.3 Nitrogen Removal

The removal of nitrogen, particularly nitrate (NO_3), is important because nitrate can be toxic to infants and it can be the limiting element of unwanted growths of algae in lakes and rivers. The treatment wetland was designed to reduce the levels of nitrate, the bioactive component of Total Nitrogen (TN). Total nitrogen includes inorganic and organic compounds, both soluble and insoluble. Typically, TN is removed as particulate matter that settles to the bottom of the wetland, while nitrate, a soluble compound, must be taken up by plants or bacteria. The plants will directly and indirectly remove nitrate in the proposed treatment wetland. Plants provide a source of organic carbon in the wetland sediments when they die and settle to the bottom of the wetland. Bacteria in the sediments then use the carbon to break down the nitrate in order to obtain the oxygen they need. The soluble nitrate is reduced to nitrogen gas, ammonia gas, and nitrous oxide which escape to the atmosphere.

An average nitrogen removal rate of 500 milligrams of Nitrogen per square meter of wetland surface per day (mg N m^{-2} d^{-1}) was used to estimate the performance of the proposed treatment wetland. Recent research in nitrogen removal rates in treatment wetland suggest that removal rates can range from 200 mg N m^{-2} d^{-1} in winter months to 1000 mg N m^{-2} d^{-1} in summer months. Nitrogen removal tends to be higher in the summer and early fall when the vegetation is growing and water temperatures are warm (Reilly et al, 2000). The vegetation in the proposed treatment wetland will consist of Cattails (*Typha sp.*) and Bulrush (*Scirpus sp.*), which have been proven to have nitrogen removal rates within the range mentioned above (Bachand & Horne, 2000). It has been found that nitrogen removal rates are typically low in the first few growing seasons following initial construction. After the plants become established, the removal rates are expected to reach up to 1000 mg N m^{-2} d^{-1} during the warmer seasons. Given the expected nitrate concentrations, as nitrogen, in the Fallbrook PUD treated effluent (7 to 15 mg/l) and the above design parameters, the nitrate concentrations in water leaving the treatment wetland are not expected to exceed 1.0 mg/l NO_3-N.

7.2.2.4 Phosphorus Removal

A number of studies have shown that treatment wetlands not only lead to a significant reduction in nitrogen, but also a decrease total dissolved phosphorus (P) as the water passes from the point of discharge. Phosphorus retention is considered one of the most important attributes of a constructed wetland that receives wastewater (Mitch and Gosselink 1993). P-removal in wetlands generally occurs through sedimentation and burial of particulate-P, or by uptake of soluble phosphate into plants and bacteria. A considerable portion of the phosphorus brought into a wetland is sorbed onto clay particles and thus indirectly made available to the biotic components of the wetland. Phosphorus removal rates may vary from year to year and from site to site. Studies have shown that nutrient retention in wetlands receiving wastewater may range from 46% to 80% as a function of loading (Mitch and Gosselink 1993).

Wetlands can serve as nutrient sinks for several years, although their assimilation capacity can become saturated for certain chemical constituents (Mitch and Gosselink 1993). Adaptive management strategies can prolong the ability of the wetland to maintain an optimal level of nutrient removal. Pilot studies and monitoring programs are instrumental in devising a management strategy that can effectively oversee the functioning of the treatment wetland based on site-specific characteristics.

The phosphorus cycle in lakes also involves organic and inorganic phosphorus in both soluble and particulate forms. Sorption and desorption of phosphate (PO_4) onto organic and inorganic particles surfaces dominates phosphorus chemistry in natural waters. Sorption of PO_4 occurs at the bottom of the lake as phosphorus binds onto sediments. Desorption from particles, during anoxic conditions (no available oxygen) releases biologically available phosphorus (BAP), which is available as PO_4 for phytoplankton growth (Horne and Goldman, 1994). The extent of sorption and desorption that occurs in a lake is a function of existing nutrient loading, lake depth, and internal mixing. Applied management strategies may allow for the trapping of sorbed phosphorus, thus reducing phosphorus levels at the reservoir outlet. Additional on-site studies are required to effectively implement a phosphorus reducing strategy.

7.2.2.5 Similar Projects

Wetland projects similar to that proposed here that are currently in various stages of development in California include, but are not limited to:

- Prado Wetlands – Orange County Water District
- River Road Constructed Treatment Wetlands – Orange County Water District
- Hayward Marsh – East Bay Regional Parks District & East Bay Dischargers Authority
- Arcata Marsh & Wetlands – Arcata, CA
- San Joaquin Wetland – Irvine Ranch Water District
- Lake Elsinore Wetlands – Elsinore Valley Municipal Water District

7.2.3 Dam and Storage Reservoir

7.2.3.1 Site Selection and Existing Site Conditions

Selection of the proposed storage reservoir and dam site was based primarily on a review of USGS topographic maps and the Permit 15000 Study prepared by Stetson Engineers. The 7.5-minute topographic maps were utilized to identify a dam site that would provide approximately 2,500 AF of storage while minimizing the size, and thus the cost, of the dam. The proposed dam and storage reservoir was previously selected as Reservoir No. 4 in the Permit 15000 Study.

A field visit was conducted in September of 2001 in order to view the dam and reservoir site. It was found that a fairly well maintained gravel and dirt road leads down the northwest side of the proposed reservoir and travels through the proposed dam site. The walls of the reservoir are well vegetated with grasses, shrubs, and small trees. At the proposed dam site, there is a bottleneck in the canyon with both sides rising quickly from the canyon floor. A visual inspection of the proposed abutments identified composed granite and exposed rocky material. The canyon is approximately 200 feet wide at the canyon floor at the dam site and widens just downstream of the proposed dam. There is evidence of a small ephemeral creek in running through the proposed reservoir and dam site. This creek will serve as the flow path of the reclaimed water as it enters from the treatment wetland. The area of the natural drainage area upstream of the dam is approximately 1,500 acres.

7.2.3.2 Dam Design

The preliminary design proposed for the storage reservoir dam is a zoned earth embankment dam rising approximately 99 feet above the streambed elevation of 292 feet msl to the dam crest elevation at 391 feet msl. A concrete overflow spillway, 520 feet in length located near the right abutment, allows for uncontrolled spills to the natural drainage below the dam. Regulated flow through the dam, with a design flow rate of 75 cfs, is obtained with a proposed steel pipe inclined inlet tower and outlet works located in the middle of the dam. The details of the dam design, including dam type selection are described in the following sections.

7.2.3.3 Dam Type Selection

In general, the primary function of a dam often suggests its most suitable type. Three general dam types were considered in design of the storage reservoir dam: earthfill, rockfill, and concrete. The proposed selection was made on the basis of dam size, foundation geology, spillway requirements, and cost. A zoned earthfill dam was selected for the storage reservoir because only preliminary geologic information is available for the dam site. This type of dam has a core made of impervious clayey material (Zone 1) and an outer shell of less impervious soils (Zone 2).

Earthfill dams are suitable for sites with unconsolidated foundation material, unlike concrete dams which require a solid rock foundation and abutments. Rockfill dams are appropriate for sites that have a readily available supply of rock material and have a rock or compacted sand and gravel foundation. The zoned earth dam was also selected because this reservoir does not require a large spillway as it will be emptied each year and not used for long-term storage.

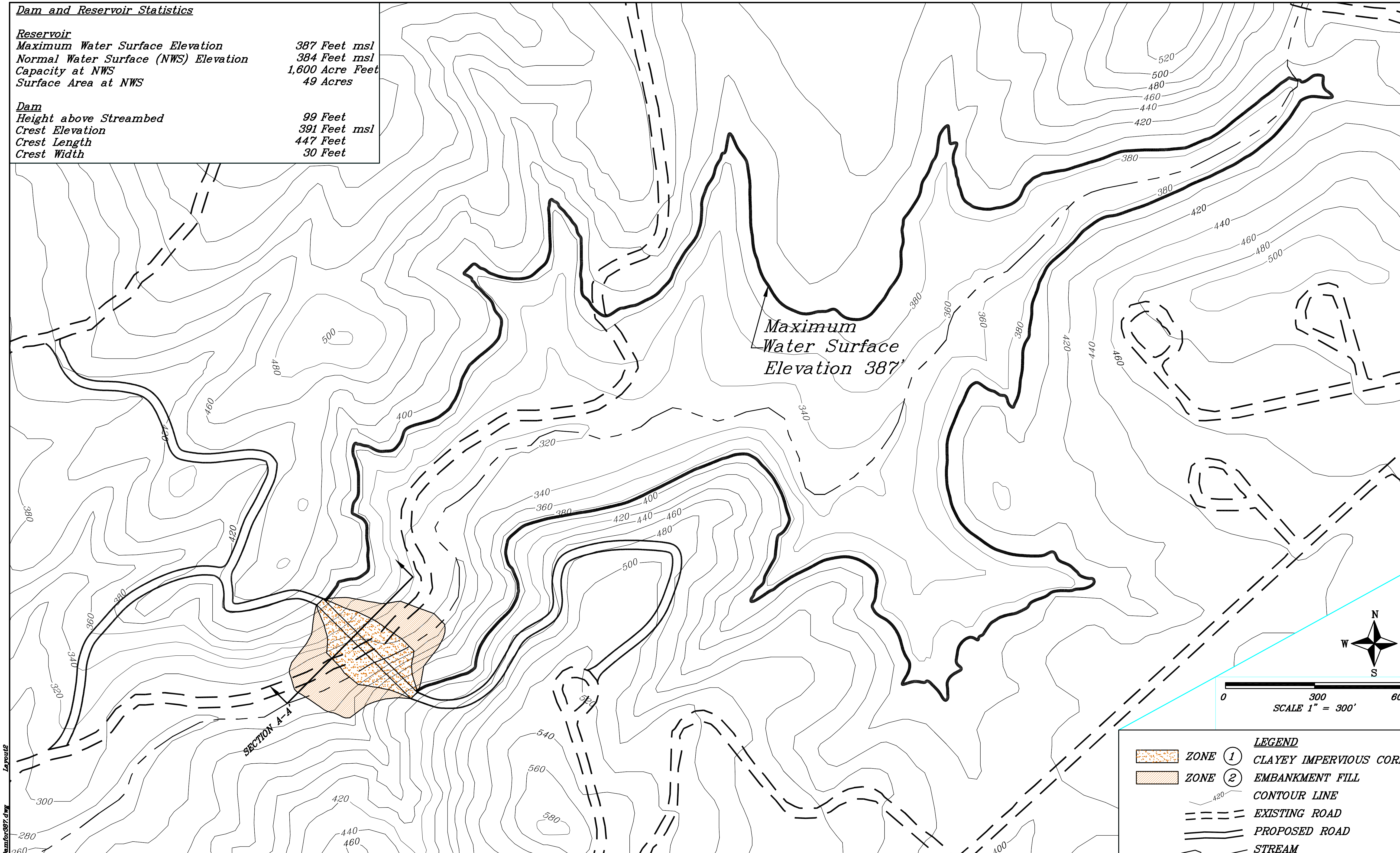
7.2.3.4 Reservoir Sizing

The storage reservoir was designed to provide storage for seven months of reclaimed water releases from the Fallbrook PUD. Because releases are occurring five months of the year, the necessary reservoir capacity for Scenario 2 is 1,600 AF. The dam was designed to provide four feet of freeboard over the maximum water surface elevation of 387 feet msl, thus yielding a crest elevation of 391 feet msl. The maximum water surface elevation is estimated at three feet above the normal water surface, 384 msl. This higher elevation is based on the result of expected wave effects on the surface of the lake. The storage reservoir and dam statistics are summarized in Table 7-1and shown in plan view and cross section in Figures 7-4 and 7-5.

FIGURE 7-4

Dam and Reservoir Statistics

Reservoir	
Maximum Water Surface Elevation	387 Feet msl
Normal Water Surface (NWS) Elevation	384 Feet msl
Capacity at NWS	1,600 Acre Feet
Surface Area at NWS	49 Acres
Dam	
Height above Streambed	99 Feet
Crest Elevation	391 Feet msl
Crest Length	447 Feet
Crest Width	30 Feet



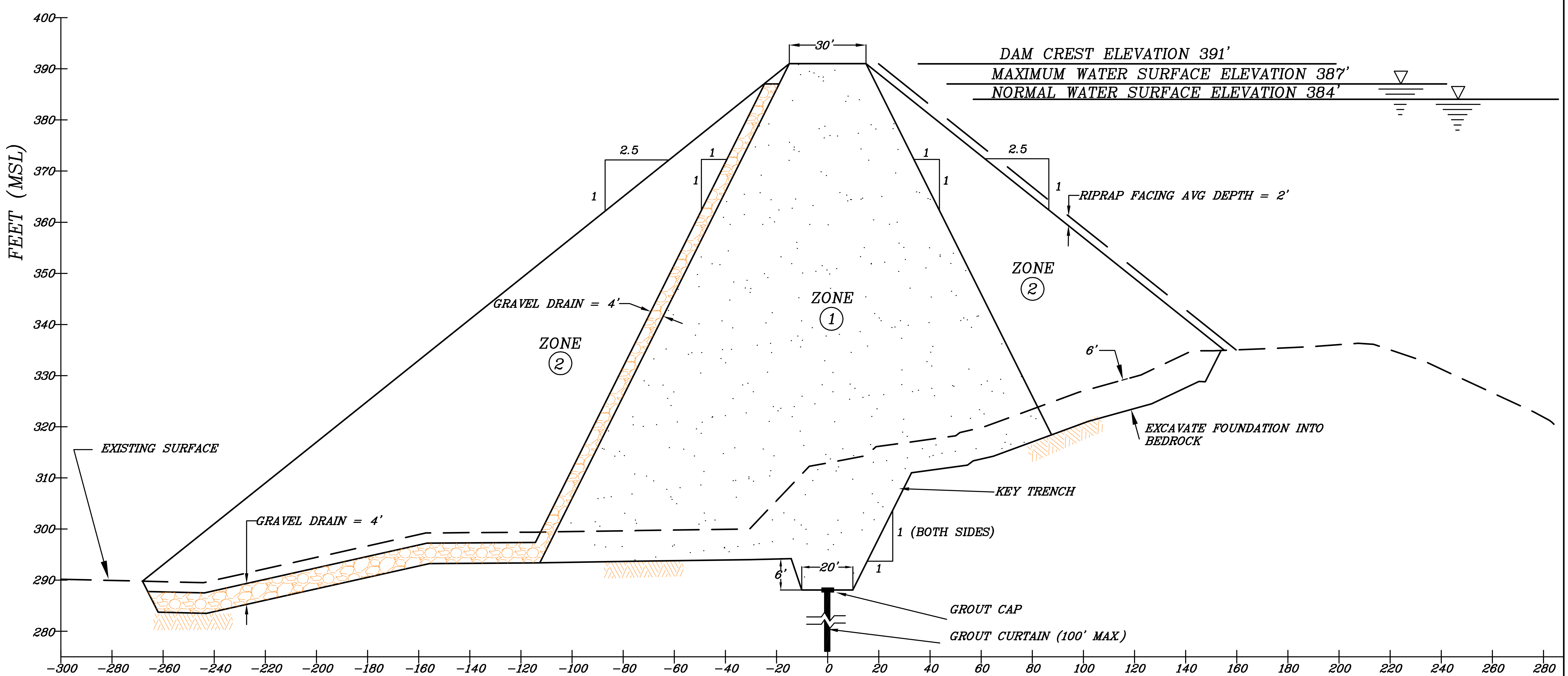
DESIGNED A. Richards / P. Luecking DRAFTED G. Trinidad CHECKED S. Reich	Prepared for Fallbrook Public Utility District	Stetson Engineers, Inc.	PROPOSED DAM AT RESERVOIR SITE No. 4 (2,500 AFY RELEASE FROM FALLBROOK) FALLBROOK SUPPLEMENTAL FEASIBILITY STUDY	PLAN VIEW OF PROPOSED DAM	DATE October 15, 2001 SCALE 1" = 300' PROJECT No. 1922
					SHEET OF

F:\data\1922\CAD\Damfor387.dwg Layout2

QUANTITY ESTIMATES

CLAYEY IMPERVIOUS CORE	143,400 CY
EMBANKMENT FILL	144,000 CY
GRAVEL DRAIN	12,000 CY
RIPRAP FACING	6,000 CY

SECTION A–A'



DISTANCE FROM CENTERLINE OF DAM AXIS

LEGEND

ZONE 1 CLAYEY IMPERVIOUS CORE

ZONE 2 EMBANKMENT FILL

Layout1

F:\data\1922\CAD\Damfor387.dwg

DESIGNED A. Richards / P. Luecking DRAFTED G. Trinidad CHECKED S. Reich	Prepared for Fallbrook Public Utility District	STETSON ENGINEERS INC. Stetson Engineers, Inc.	PROPOSED DAM AT RESERVOIR SITE No. 4 (2,500 AFY RELEASE FROM FALLBROOK) FALLBROOK SUPPLEMENTAL FEASIBILITY STUDY	CROSS SECTION OF PROPOSED DAM (OUTLETS WORKS NOT SHOWN)	DATE October 15, 2001 SCALE VERT: 1" = 20' HORZ: 1" = 40' PROJECT No. 1922
					SHEET OF

TABLE 7-1

SUMMARY OF SCENARIO 2 DAM AND STORAGE RESERVOIR STATISTICS

Dam Dimensions		
Crest Height above Streambed	99	feet
Crest Elevation	391	feet msl
Streambed Elevation	292	feet msl
Crest Length	447	feet
Crest Width	30	feet
Maximum Base Width	480	feet
Embankment Volume	287,400	cubic yards
Reservoir Statistics		
Maximum Water Surface Elevation	387	feet msl
Normal Water Surface (NWS) Elevation	384	feet msl
Capacity at NWS	1,600	acre feet
Surface Area at NWS	53	acres
Spillway - Uncontrolled concrete chute with Ogee crest		
Ogee Crest Elevation (NWS Elevation)	384	feet msl
Design Capacity	150	cfs
Outlet Works - 36-inch steel pipe		
Pipe Invert Elevation at Inlet	305	feet msl
Design Capacity	75	cfs

7.2.3.5 Clearing

The site visit to the proposed dam site found that there is a significant amount of vegetation present on the walls of the proposed reservoir. The vegetative cover will be cleared to prevent settlement of debris in the reservoir, clogging of the dam intake works, and fouling of the reclaimed water due to decomposition of large amounts of plant material. The plant material will be cleared, grubbed and removed from the reservoir site during dam construction.

7.2.3.6 Foundation Requirements

The foundation investigation and design can be considered of equal importance to the design of the dam section itself. A foundation investigation should include drilling to determine properties of the rock and subsoil such as compressive strength, shear strength, and permeability. For the purpose of this feasibility level study, it was assumed that the foundation conditions at the proposed dam site will be adequate for the construction of a zoned earthfill dam. Further investigations into the foundation material will be necessary prior to developing design specifications and a design level cost estimate.

Certain foundation design elements were evaluated and included in the design of the proposed dam. The footprint of the dam will be excavated to bedrock in order to provide a solid base for construction. This excavation will include stripping of topsoil and excavation of some rock material. A key trench will be cut in line with the dam crest, under Zone 1, to provide an additional level of strength and stability to the impervious core of the dam. The key trench will be excavated to a depth 6 feet below the rest of the foundation, it will be 20 feet wide and have sides sloped at a 1:1 ratio. The key trench will be filled with Zone 1 material. The exposed bedrock will be mechanically cleaned and exposed cracks will be filled with dental concrete to prevent water from easily flowing under the dam and potentially undermining the foundation.

7.2.3.7 Underseepage

A grout curtain will be constructed in the key trench along the entire length of the dam. The grout curtain provides a barrier to flow of water under the dam. The grout curtain is constructed by drilling holes approximately five feet apart along the entire length of the dam crest. The holes are then filled with grout, under pressure, in order to fill in all cracks and fissures extending from the drill holes. The close placement of the holes and pressurized grouting effectively creates an impervious wall, up to 100 feet deep, below the dam foundation.

A gravel drain, 4 feet in thickness, will be constructed on the downstream face of the Zone 1 material and under the downstream section of Zone 2 material. The gravel drain will allow for water moving through the pervious Zone 2 material to easily flow away from the dam structure itself, thus reducing the load on the structural Zone 1 component of the dam. The gravel drain is shown in Figure 7-5. A drain recovery system will be installed to capture this water and either return it to the reservoir or allow it to pass downstream of the dam.

7.2.3.8 Zoned Embankments

The proposed design of this zoned earth embankment dam includes two zoned areas. Zone 1 consists of a clayey, impervious material while Zone 2 is a pervious fill material that will consist of a composite of gravel and soil. Zone 1 is the structural component of the dam, while Zone 2 provides the mass and bulk of the dam necessary to retain the reclaimed water. Zone 1 forms the 30 feet wide crest of the dam and has sides sloped at a 1:1 ratio. Zone 2 begins just on either side of the dam crest and has side slopes with a 1:2.5 ratio. The zoned areas of the dam are shown in Figure 7-4.

The embankment volume for the two zones was estimated using the *Land Development Desktop* software and contours digitized from USGS 7.5-minute topographic maps. Approximately 288,000 cubic yards of soil will be required to construct the dam embankment. For Scenario 2, Zone 1 and Zone 2 each comprise about half the total volume required. Figure 7-6 shows the possible borrow sites for both Zone 1 and Zone 2 material.

7.2.3.9 Slope Protection

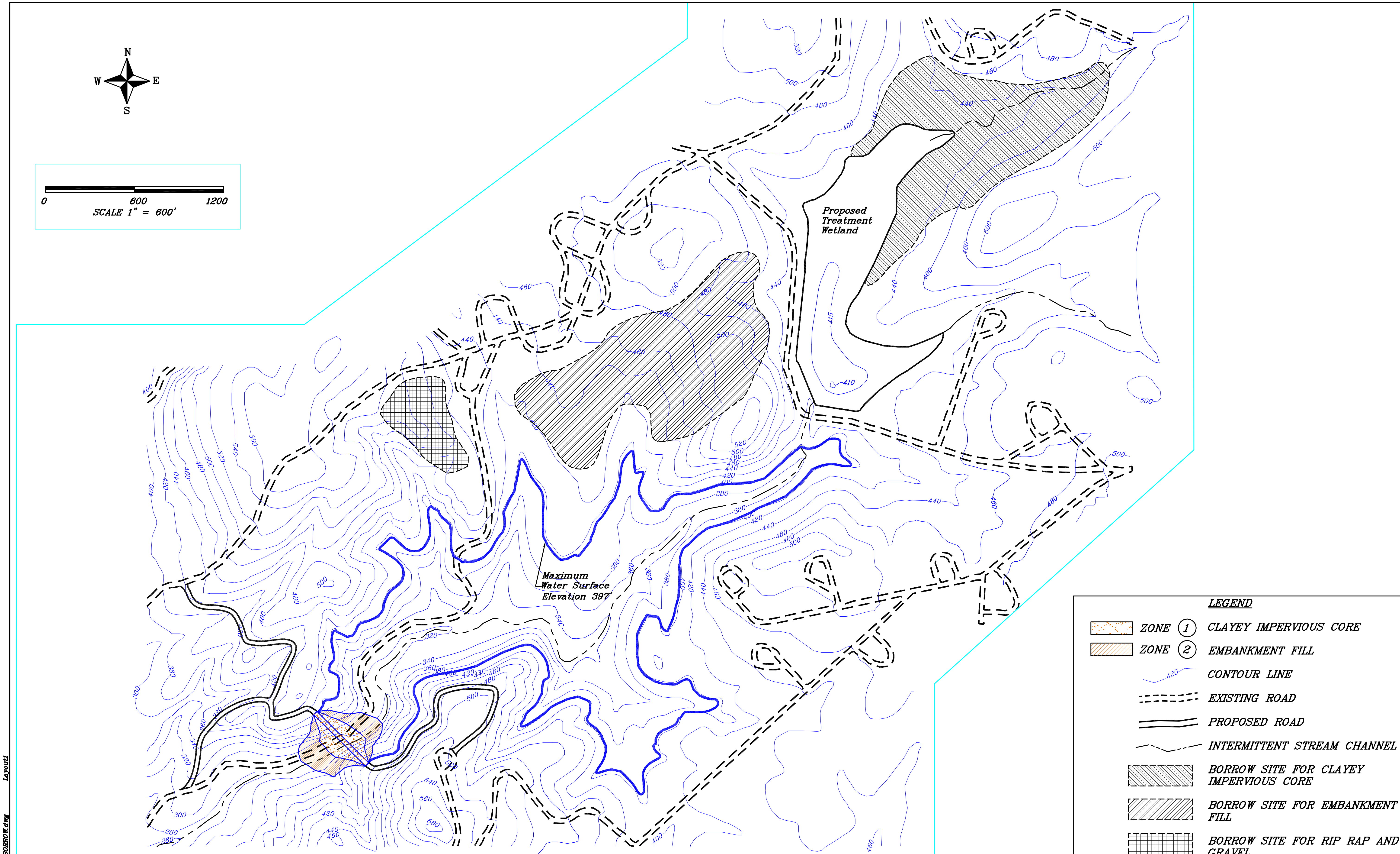
A rip rap facing will be placed on the upstream face of the dam to protect the dam structure from erosion due to fluctuating water levels in the reservoir and forces due to wind waves on the surface of the reservoir. The rip rap will be place to an average depth of 2 feet across the entire upstream face of the dam. A total of 7,000 cy of riprap slope protection will be required.

7.2.3.10 Intake / Outlet Works

The designed intake structure is an inclined steel pipe inlet tower with three inlet structures placed along the upstream face of the dam. The lowest intake structure is 36 inches in diameter while the remaining two are 24 inches in diameter. Multiple intakes allow the operator to draw water from multiple and or various depths in the reservoir. The intake structures will convey the water to a 36-inch concrete encased steel outlet pipeline that passes through the dam. The design capacity of the outlet works is 75 cfs and the invert of the dam outlet pipeline inlet is 305 feet msl. The dam outlet pipeline exits the dam and passes under the concrete spillway where it connects to the reservoir discharge pipeline at an elevation of approximately 295 feet msl. Details of the reservoir discharge pipeline to the Santa Margarita River are presented in a Section 7.2.4. A schematic of the proposed outlet works and spillway is shown below in Figure 7-6.

7.2.3.11 Spillway

The spillway is an uncontrolled concrete chute with and ogee crest located on the right abutment of the dam. Initially water will enter the spillway approach channel and then flow over the ogee crest at an elevation of 384 feet msl. The concrete channel is approximately 80 feet long, 40 feet wide and 8 feet deep. After passing over the ogee crest, the water will enter the main spillway channel, approximately 10 feet wide, 8 feet deep and 520 feet long. The spillway passes under the road that travels along the dam crest. A bridge will be constructed over the spillway at this location. Near the end of the spillway chute, the channel widens and opens into a concrete stilling basin. The total elevation drop is approximately 94 feet from 384 feet msl at the ogee crest to 290 feet msl at the stilling basin.



DESIGNED A. Richards / P. Luecking DRAFTED G. Trinidad CHECKED S. Reich	Prepared for Fallbrook Public Utility District	Stetson Engineers, Inc.	PROPOSED DAM AT RESERVOIR SITE No. 4 SCENARIO #3 (3,500 AFY RELEASE FROM FALLBROOK) FALLBROOK SUPPLEMENTAL FEASIBILITY STUDY	POTENTIAL BORROW SITES	DATE October 15, 2001 SCALE 1" = 600' PROJECT No. 1922

SHEET
OF

P:\data\1922\CAD\BORROW.dwg Layout1

The stilling basin is designed to rapidly dissipate the energy of the water acquired in the spillway chute. The stilling basin is 100 feet long and widens from 10 feet at its inlet to 20 feet at its outlet. The water exits the stilling basin and passes through a section of rip rap designed to further dissipate energy and prevent erosion of the natural drainage downstream. The spilled water will be allowed to flow uncontrolled along the natural drainage toward the Santa Margarita River.

The spillway is designed to pass 150 cfs based on an estimated average daily watershed runoff of 145 AF and peak daily watershed runoff of 230 AF. Under normal operating conditions, the spillway will not be utilized. However, if the reservoir is near capacity at the time of a major storm event, the spillway will prevent overtopping of the dam. A schematic of the spillway and stilling basin is shown in Figure 7-7.

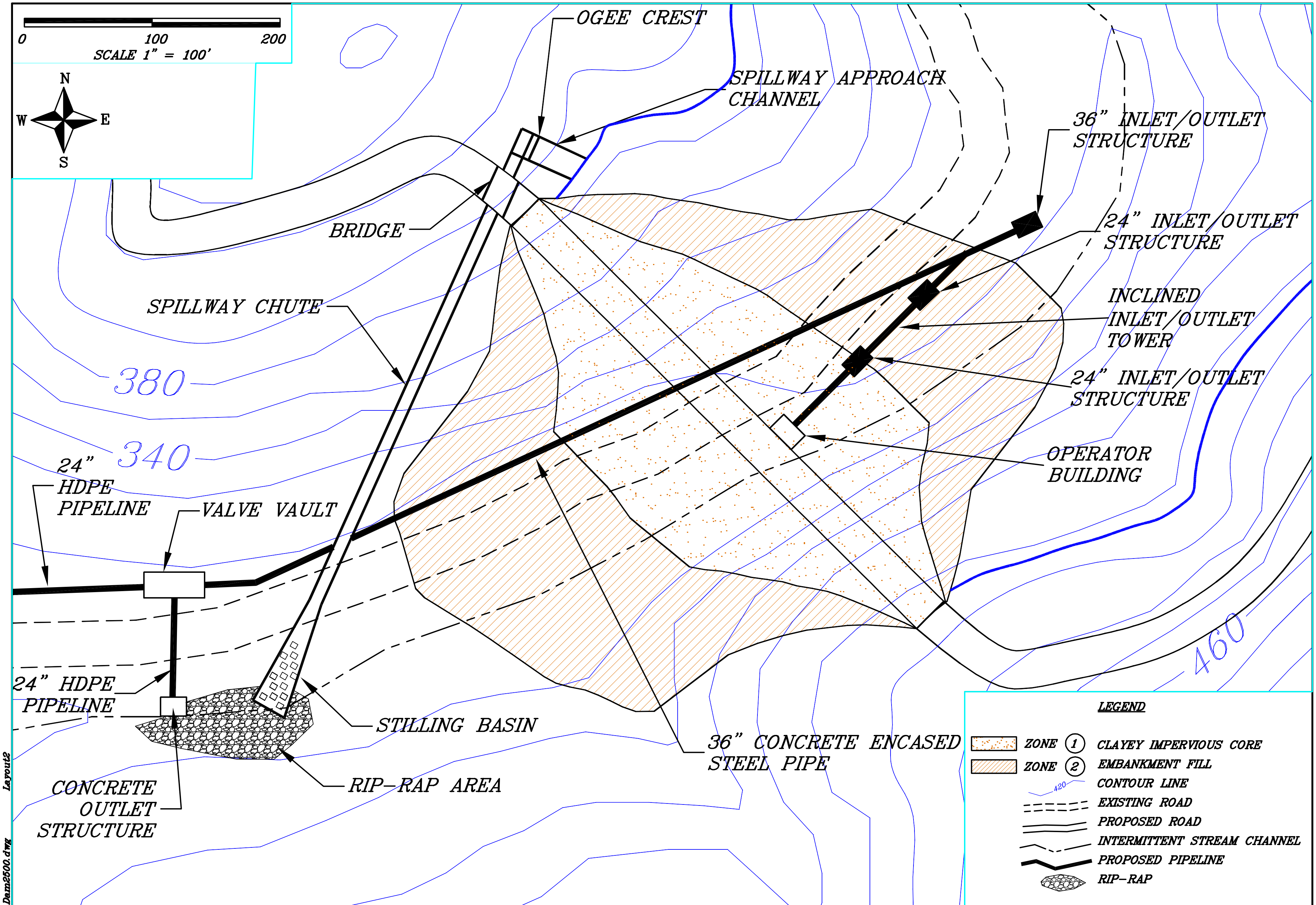
7.2.3.12 Associated Facilities

In addition to the major components described above, the dam design includes construction of access roads to be used during and following construction. The dam is in a relatively remote location, so power will need to be brought to the site. Instrumentation is necessary to efficiently operate the dam, provide for data collection, and to monitor ground-water levels surrounding the dam. Ancillary structures include fencing to restrict access to the reservoir, site drainage, and some areas of erosion control.

7.2.3.13 Diversion to Natural Drainage

The natural drainage located downstream of the dam site currently supports a stable growth of vegetation ranging from scrub grasses to large trees. An undetermined amount of water will be allowed to pass along this drainage either via the spillway or by prescribed releases from the dam in order to prevent destruction of the existing ecology of this drainage. All water that passes through the spillway will enter this drainage and its volume will be estimated by measuring the water depth above the ogee crest during spill events.

Water will also be diverted to the natural drainage directly from the dam outlet pipeline. After passing under the spillway, the dam release pipeline enters a valve box and connects to the reservoir discharge pipeline. A 24-inch HDPE natural discharge pipeline branches off of the main line and will provide flows to the natural drainage below the dam. The reservoir discharge pipeline design includes a Venturi meter to record and monitor the amount of water allowed to pass to the natural drainage.



F:\data\1922\CAD\Dam2500.dwg Layout2

Prepared for Fallbrook Public Utility District

STETSON ENGINEERS INC.

PROPOSED DAM AND OUTLET WORKS
SCENARIO #2 (2,500 AFY RELEASE FROM FALLBROOK)
FALLBROOK SUPPLEMENTAL FEASIBILITY STUDY

SPILLWAY AND OUTLET PIPELINE WORKS

DATE October 15, 2001
SCALE 1" = 100'
PROJECT No. 1922

FIGURE 7-7

7.2.4 Pipeline from Storage Reservoir to Santa Margarita River (reservoir discharge pipeline)

A buried pipeline will be utilized to convey reservoir releases to the Santa Margarita River. The reservoir discharge pipeline was designed to operate entirely under gravity flow from the reservoir to the river. The pipeline is a 20-inch HDPE with invert elevations of 295 feet msl at the connection to the 36-inch dam outlet pipeline and 130 feet msl at the Santa Margarita River. The total required length of the 20-inch HDPE pipeline is 5800 feet.

7.2.4.1 Existing Site Conditions and Proposed Alignment (Reservoir Discharge Pipeline)

The natural drainage downstream of the dam is choked with dense vegetation. Because of increased costs, and possible environmental impacts associate with clearing the drainage it was determined to convey the water discharged from the reservoir to the river via a buried pipeline. The reservoir discharge pipeline begins at the juncture with the dam outlet pipeline and the natural drainage pipeline. At this location a flow control valve will be installed on the reservoir discharge pipeline to control the rate of flow to the river. The alignment of the reservoir discharge pipeline is through a less densely vegetated area located above the main conveyance of the natural drainage. The alignment initially follows naturally declining terrain before rising to an elevation of 281 feet msl in order to pass over a long, relatively flat saddle and then again proceeds downhill toward the river. Following the saddle, the reservoir discharge pipeline will parallel De Luz Road until it empties into the Santa Margarita River.

7.2.4.2 Design Considerations (Reservoir Discharge Pipeline)

The proposed reservoir discharge pipeline was designed to convey the maximum daily release from the storage reservoir. This flow rate was estimated to be 8.7 cfs using the daily reservoir inflow rate and the flow rate necessary to steadily empty a full reservoir over a five month period. The required inside pipe diameter was designed to maintain a pipe flow velocity of less than 6 feet/sec. The design pipe size was then chosen based on the available standard size HDPE pipe with the required inside diameter. The static pressure requirement for this pipeline was calculated to be 113 psi, 160 psi HDPE was selected for the design.

The difference between the surface elevation of the storage reservoir and the pipeline outlet provides 258 feet to 179 feet of head when the reservoir is full and empty, respectively. This creates a large variance in the required operation of the pipeline. The pipeline was designed to ensure gravity flow for both the full and empty reservoir conditions.

Several valves were incorporated into the design of the reservoir discharge pipeline in order to allow for flow control, necessary maintenance, flow measurement, and to prevent damaging vacuum forces in the pipeline. A general schematic of the proposed valve configurations and their approximate locations along the pipeline is shown in Figure 7-8. Also included in the figure is the estimated hydraulic grade line of the reservoir discharge pipeline, for both a full and empty reservoir, at the design flow rate of 8.7 cfs.

In addition to the flow control valve at the dam outlet-reservoir discharge connection, one isolation valves is provided to close off flow to the juncture of the pipelines. Air/vacuum release valves are provided at appropriate locations to prevent potentially damaging vacuum forces that may occur. Two Venturi meters located below the junction of the three pipelines, one on the reservoir discharge pipeline and the other on the natural drainage pipeline, will measure flow to the Santa Margarita River and the natural drainage. Pressure reducing valves will be installed at the downstream end of the reservoir discharge pipeline and the natural drainage pipeline to dissipate the energy of the water before it enters the outlet structures. All valves will be placed in concrete vaults in order to allow for easy access and necessary maintenance.

Both the reservoir discharge pipeline and the natural drainage pipeline empty into an enclosed concrete outlet structure designed to dissipate the energy of the flowing reclaimed water. The reclaimed water will then pass over a concrete weir contained in the outlet structure and through a section of rip-rap prior entering the Santa Margarita River or the natural drainage.

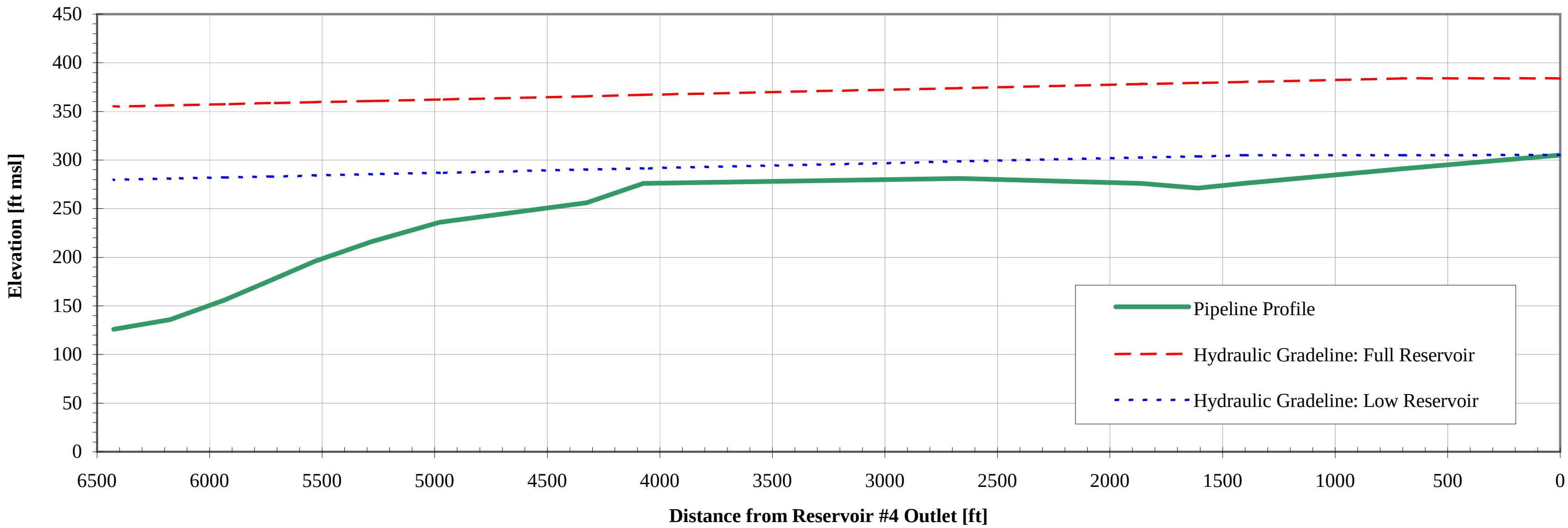
7.2.5 Pumping Wells

Ground-water pumping wells will be used to extract the water from the aquifers in the Upper Ysidora and Chappo subbasins. The ground-water well’s production capacity were based on actual capacity of the existing wells now in use on the Base. The water will be pumped from each well to the advanced treatment facilities before being conveyed to either the Base’s potable water supply or the Fallbrook PUD.

7.2.6 Fallbrook Return Pipeline

A pipeline was designed to convey treated potable water from Camp Pendleton to the town of Fallbrook. The water will be pumped from an elevation of 120 feet msl near Lake O’Neill to an elevation of 800 feet msl at the connection to the Fallbrook distribution system located on the northeast side of Fallbrook. Two pump stations, one primary and one booster will be used to lift the water 680 feet over a total distance of 9.75 miles. The Fallbrook return pipeline was designed separately for Alternative 9 and Alternative 10 conditions. The pipeline was sized based on conveying the estimated total annual pumping indicated in the ground-water

Generalized Profile and Schematic of Proposed Pipeline from Reservoir #4 to the Santa Margarita River
Scenario #2: 2500 AFY Release from Fallbrook



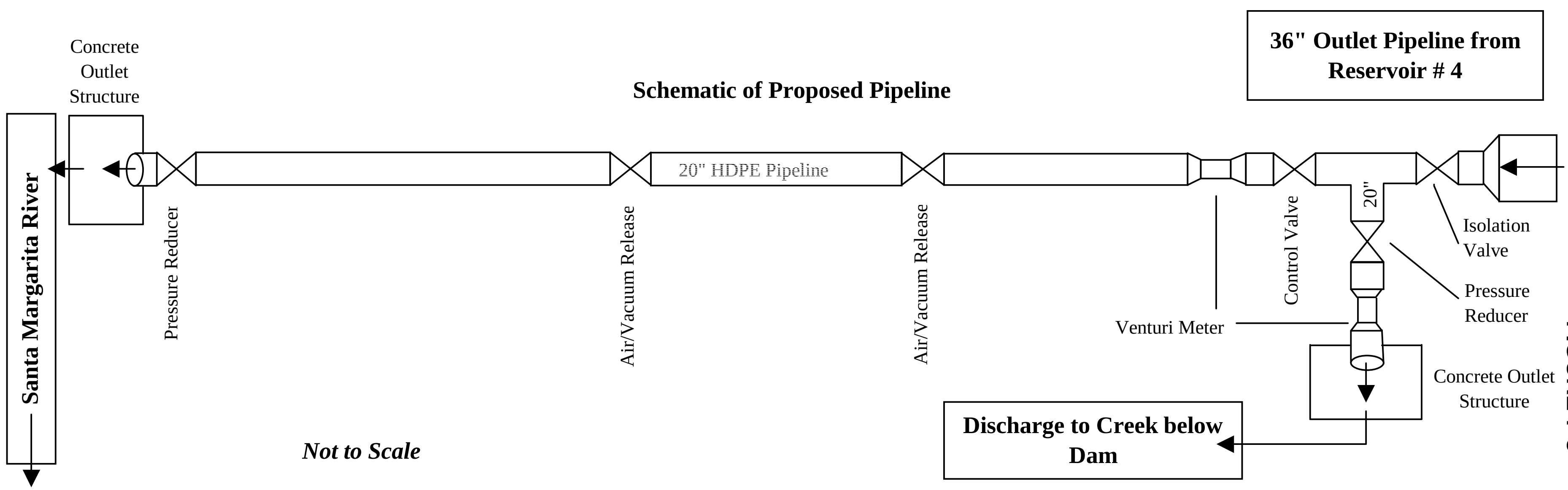


FIGURE 7-8

model results, over a period of six months. Table 7-2 below shows the flow and pumping requirements for each Alternative under Scenario 2 conditions.

TABLE 7-2

FLOW AND PUMPING REQUIREMENTS OF THE RETURN PIPELINE TO FALLBROOK

	Scenario 2 Alternative 9	Scenario 2 Alternative 10
Annual Fallbrook PUD Project Yield (AF)	5,150	7,410
Monthly Pumping Requirement (AF)[1]	900	1,200
Required Pumping Rate (gal/min)	6,450	9,300

1. Annual yield pumped in a period of six months

7.2.6.1 Proposed Alignment (Fallbrook Return Pipeline)

The alignment of the conveyance pipeline is similar to the selected alternative in a 1994 Conjunctive Use Study prepared by NBS/Lowry Engineers for the Fallbrook PUD. The pipeline begins near the intersection of Santa Margarita Road and Vandergrift, on Camp Pendleton. The buried pipeline will then parallel Ammunition Road through Camp Pendleton and the NWS until it approaches the Fallbrook PUD WWTP. Here the pipeline heads almost due north along the NWS boundary until it intersects Alvarado Street in Fallbrook. The pipeline will then travel east through Fallbrook and exit on Mission Road where it will connect to the existing Fallbrook system approximately one half mile east of Fallbrook.

8.0 ALTERNATIVES

8.1 OVERVIEW

This study focuses on developing a conjunctive use program, through the reuse and recycling of tertiary treated wastewater, as a supplement to the Permit 15000 Feasibility Study. Alternatives that were developed in the Permit 15000 Study addressed improvements to the diversion and recharge facilities, already existing on the Base. Alternatives 9 and 10, presented below, have been developed based on the information and analyses completed in the prior study. Although the previous study did not directly address a conjunctive use program between the Fallbrook PUD and the Base, it did address the need for the Base to partner with another party to provide for the capacity to fulfill the conjunctive use pumping program.

Alternatives 9 and 10 address the facilities required to support an alternative water supply to the existing sole source of supply from the Santa Margarita River. Alternative 9 addresses the facilities required to convey, treat, and reuse tertiary treated wastewater effluent from the Fallbrook PUD's WWTP. Alternative 10 addresses the identical facilities, but also includes the enhanced diversion facilities outlined in Alternative 3 of the Permit 15000 Study. The purpose for the development of Alternative 9 was to determine the yield of the wastewater release and reuse component, with respect to No Project (Alternative 1). Because maximum historical pumping was a component of the No Project alternative, Alternative 1A was developed in order to maximize pumping under "no project" conditions.

8.2 ALTERNATIVE 9

Alternative 9 includes the wetland pipeline, treatment wetlands, storage reservoir, reservoir discharge pipeline, and conveyance facilities described in detail in Chapter 7. The purpose of Alternative 9 is to estimate the yield of releasing tertiary treated wastewater from the Fallbrook PUD WWTP to the Santa Margarita River. Under this scenario, treated wastewater is released to a treatment wetland, stored in a temporary storage reservoir, then discharged to the Santa Margarita River for maintenance of riparian habitat during the dry summer and fall months. The release of these waters allow pumping levels to be sustained during the dry season, providing a median annual yield of 14,000 AFY, 2,150 more than compared to Alternative 1A. Accounting for evaporation, transpiration, and seepage, almost 90% of the water released from the wastewater treatment facility is captured and available for reuse (Scenario 2).

This chapter presents the results of a conjunctive use project with a recycling program sized for the annual release and reuse of 2,500 AFY of tertiary treated wastewater (Scenario 2).

The Appendix provides the modeling results for a recycling and reuse program sized for 1,500 AFY and 3,500 AFY, Scenarios 1 and 2, respectively.

8.2.1 Surface Water Analysis for Alternative 9

The surface water analysis for Alternative #9 integrated the Reclaimed Water Reservoir Operations Model (RWROM) (described in Chapter 6) with an existing Reservoir Operation Model for the Camp Pendleton Facilities (hereafter referred to as the Camp Pendleton Reservoir Operations Model – CPROM). The RWROM simulates flow near the confluence of the Santa Margarita River and De Luz Creek. When added to the estimated flow in the Santa Margarita river, the total estimated flow provides input to the CPROM. The resulting output from the CPROM provides streamflow and diversion data for the ground-water model.

8.2.1.1 Reclaimed Water Reservoir Operations Model (RWROM)

A Reclaimed Water Reservoir Operations Model (RWROM) was developed to simulate all surface water influencing the water recycle and reuse component, noting losses, gains, storage, and releases. The water recycle and reuse component begins with the release of tertiary treated wastewater from the Fallbrook PUD WWTP. The wastewater releases flow from the Fallbrook PUD WWTP 1, through delivery pipelines, then through the natural channel into a treatment wetland, eventually terminating in a storage reservoir. The water is stored in the reservoir for a period up to seven months before it is released via pipeline into the Santa Margarita River. Precipitation, evaporation and transpiration all play an important role in accounting for the effective losses in the water budget. A summary of the results of the RWROM tracking tertiary treated wastewater from the Fallbrook PUD WWTP to the Santa Margarita River is shown in Table 8-1.

TABLE 8-1:

SUMMARY OF RESULTS FROM RECLAIMED WATER RESERVOIR OPERATIONS MODEL (RWROM)

Average Annual	Scenario 1 1500 AFY	Scenario 2 2500 AFY	Scenario 3 3500 AFY
Release from FPUD WWTP	1,500	2,430	3,230
Release to Santa Margarita River	1,330	2,175	2,900
Effective Losses	170	255	330
Effective Losses (%)	11 %	10 %	10 %

Note: All values are average values based on the 20-year model run.

The quantity of water released to the Santa Margarita River is considered to be available to provide flows for riparian habitat maintenance. All releases from the storage reservoir remain in-stream and are not diverted to the recharge ponds.

8.2.1.2 Camp Pendleton Reservoir Operations Model (CPROM)

Camp Pendleton plays an integral role in the water reuse cycle, by instituting a conjunctive use program which seeks to optimize the use of both surface and ground-water resources in the Santa Margarita River Basin. Based on Alternative 9 conditions, Camp Pendleton facilities include a sheet pile diversion structure on the Santa Margarita River, Lake O'Neill, a series of five percolation ponds, and an array of ground-water wells and pumping facilities. A detailed description of the CPROM model and the results can be found in Chapter 7 (Alternative 1) of the Permit 15000 Study. A brief description of the model parameters and Camp Pendleton facilities that are relevant to this study, is provided within this section.

Streamflow at Model Boundary

A spreadsheet model was used to reconstruct the surface flow at the Model boundary for Model Years 1 through 20 (1980-1999). Due to missing data at many of the gage locations, the entire period of record for each gage was reviewed in order to estimate flow at the model boundary. The period of record was divided into 3 parts due to the non-continuous data set. For water years 1925 to 1980, the total streamflow at the Model boundary was calculated based on adding the observed streamflow from the Fallbrook gage to the simulated streamflow contribution from De Luz Creek. For water years 1981 to 1989, the peak flows during precipitation events were determined by the Soil Conservation Service (SCS) method for calculating surface runoff, and the baseflow was simulated using the natural flow at the Gorge modeled using the EPA's Hydrological Simulation Program-Fortran (HSPF) for calculating surface runoff. For water years 1990 to 1999, the observed streamflow values at the Fallbrook PUD sump gage, Sandia Creek, and De Luz Creek were added together to approximate the flow at the Model boundary. A summary of the calculated streamflow in the Santa Margarita River at the model boundary is shown in Table 8-2.

TABLE 8-2

STREAMFLOW AT THE MODEL BOUNDARY

Simulated Period MY 1 - 20	Historical Flow in the Santa Margarita River (AF)	Additional Augmented Flow (AF)	Total Flow Santa Margarita River (AF)
20-year Total	1,067,800	49,300	1,117,100
Average	53,400	2,500	55,900
Median	27,700	2,500	30,700
Minimum	9,300	1,500	10,700
Maximum	224,700	4,000	226,200

Note: Historical flow in the Santa Margarita River is based on WY 1980-1999 historical records.

Lake O'Neill

Lake O'Neill is a 1,200 acre-foot reservoir located on Fallbrook Creek, a minor tributary to the Santa Margarita River. Most of the water stored in the lake is diverted from the nearby Santa Margarita River. The Lake O'Neill dam and the diversion ditch from the Santa Margarita River were constructed in 1883 as part of the farm irrigation system. Since acquisition by the U.S. Government for Camp Pendleton, Lake O'Neill has been used for recreation, training purposes, and subsequent ground-water recharge (Leedshill and Herkenhoff, 1988).

Percolation Ponds

There are five recharge ponds located off the diversion channel from the Santa Margarita River. These ponds permit water to recharge the ground-water system. The reservoir operations model calculates the daily flow of water into the recharge ponds, the net effect of precipitation and evaporation, the volume of water infiltrating into the ground, and finally the volume of water, which spills out of the last pond.

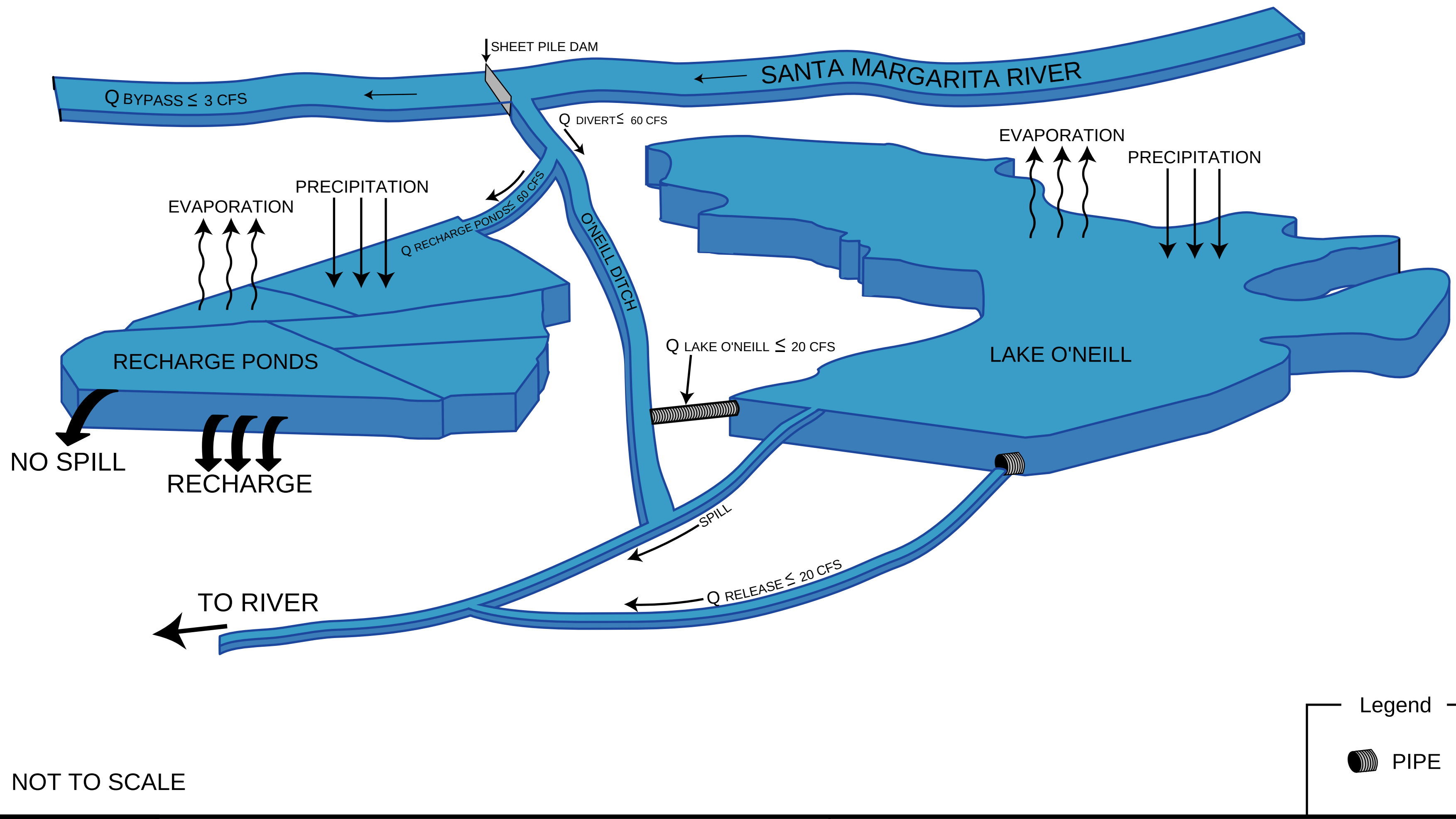
8.2.1.3 Alternative 1a Reservoir Operation Model

The reservoir operations model used for Alternative 1a estimated the rate of diversion from the Santa Margarita River to both the recharge ponds and Lake O'Neill. Limitations to the diversion rate from the Santa Margarita River accounted for in the reservoir operations model included not only the available water supply and physical limitations of the existing diversion facilities, but also such factors as available water rights, recharge pond infiltration rates, rainfall, evaporation, and spill from both the ponds and the lake. The Alternative 1a reservoir operations model also accounted for augmented surface flows and increased diversion efficiencies due to the maintenance and repair projects recommended in Chapter 6 of the Permit 15000 Study. Results from the model analysis were used by the ground-water model to estimate recharge at the ponds, streamflow past the diversion point, and releases from Lake O'Neill.

When combined with the RWROM, the CPROM portion of the Alternative 1a does not change. Thus, the Alternative 9 CPROM gives the same results as the Alternative 1a CPROM with the exception of the streamflow at the model boundary, which will increase by the amount of reclaimed flow released from the reservoir.

8.2.1.4 Alternative 9 Reservoir Operation Model

A schematic diagram of the reservoir operations model is shown in Figure 8-1. The Alternative 9 Diversion Schedule to Lake O'Neill and the Recharge Ponds is shown in Table 8-3. During periods of diversion, 3 cfs remains in the Santa Margarita River while the remaining surface flow may be diverted to either Lake O'Neill or to the recharge ponds. The simulated diversion to Lake O'Neill is limited to 20 cfs or less, while the maximum simulated diversion to the recharge ponds is 60 cfs.



NOT TO SCALE

STETSON ENGINEERS INC.

Surface Water Analysis
Resevoir Operations Model

Alternative 9
No Project

October 15, 2001

FIGURE 8-1

TABLE 8-3

ALTERNATIVE 9 DIVERSION SCHEDULE TO LAKE O'NEILL AND RECHARGE PONDS

Month	Activity	Rate	Limit	Water Right
Diversions to Lake O'Neill				
Nov	Drain	$Q_{release}$ <= 20 cfs	Min Volume = 100 AF	Pre-1914 Water Right
Dec to Jan	Fill	$Q_{lake\ O'Neill}$ <= 20 cfs	Max Volume = 1,200 AF	Permit 15000
Feb to May	Precip & Evap	Q_{spill} = f(precip & evap)	N/A	
June to Oct	Fill	$Q_{lake\ O'Neill}$ <= 20 cfs	No spill of Pre-1914 water	Pre-1914 Water Right
Diversions to Recharge Ponds				
Nov	Fill w/ 100% Q_{divert}	$Q_{recharge\ ponds}$ <= 60 cfs	No Spill	License 21471 A
Dec to March	Fill w/ $Q_{divert} - Q_{lake\ O'Neill}$	$Q_{recharge\ ponds}$ <= 60 cfs	No Spill	License 21471 A
May to June	Fill w/ Q_{divert}	$Q_{recharge\ ponds}$ <= 60 cfs	No Spill	License 21471 A
July to Sept	No Diversion	$Q_{recharge\ ponds}$ = 0 cfs	N/A	N/A
Oct	Fill w/ Q_{divert}	$Q_{recharge\ ponds}$ <= 60 cfs	No Spill	License 21471 A

The simulated annual diversion to Lake O'Neill and the recharge ponds, under the Pre-1914 and Permit 15000 Water Rights is shown in Table 8-4.

TABLE 8-4:

Alternative 9 - Diversions to Recharge Ponds and Lake O'Neill (AFY)

Model Year	Pre-1914 Water Diverted to Lake O'Neill from Jun 1st-Oct 31st (AFY)	Permit 15000 Water Diverted to Lake O'Neill from Dec 1st – Jan 31st (AFY)	Alternative 9 Diversions to Recharge Ponds (AFY)	Total Diversions from the Santa Margarita River (AFY)
Total	**28,200**	**22,300**	**155,200**	**205,700**
Average	1,400	1,100	7,800	10,300
Median	1,500	1,100	8,000	10,600
Min	9,00	1,100	3,900	5,900
Max	1,500	1,100	11,300	13,900

Table 8-5 summarizes the results of the Alternative 9 reservoir operations model. Monthly values calculated in the reservoir operations model for streamflow at the model boundary, diversions from the river, and recharge to ground water at the percolation ponds, are used as monthly input to the ground-water model to show the seasonal variation that takes place under the different hydrologic conditions.

TABLE 8-5:

Alternative 9 – Scenario 2
Maintenance and Repair Items with Augmented and Reuse Flows

20-year Simulated Period	Augmented Flow at Santa Margarita River (AF)	Reclaimed Water in River (AF)	Total Diversion Max 60 cfs (AF)	Total Diversion to Lake O'Neill (AF)	Permit 15000 Diversion to Lake O'Neill (AF)	Diversion to Recharge Ponds (AF)	Recharge to Ground Water (AF)
20-yr Total	1,117,110	43,501	205,749	50,522	22,299	155,228	154,364
Average	55,860	2,150	10,287	2,526	1,115	7,761	7,718
Median	30,740	2,214	10,595	2,613	1,118	7,962	7,924
Min	10,730	2,025	5,870	2,001	1,059	3,870	3,852
Max	226,230	2,252	13,921	2,639	1,138	11,304	11,268

8.2.2 Ground-Water Analysis for Alternative 9

The ground-water model analysis for Alternative 9 compares the simulated results from the release of reservoir water and the addition of four new production wells to Alternative 1a, the adjusted baseline model run.

- Alternative 1a includes:
  - maintenance and repair of the existing diversion and recharge system on the Base,
  - augmented streamflow from the recent settlement with RCWD,
  - 60 cfs capacity in the diversion system,
  - optimized water management which yields
    - average annual diversion of 2,530 AFY to Lake O'Neill,
    - average annual diversion of 7,760 AFY to the five existing recharge ponds,
    - ground-water production of 11,850 AFY during normal or above normal streamflow years.

- Alternative 9 incorporates
  - Alternative 1a baseline condition,
  - increased streamflow in the Santa Margarita River from the 5 month release of reservoir water between July and November, generated from scenario 2 of Fallbrook PUD reclaimed water,
  - the addition of two new ground-water production wells yielding 2,150 AFY.

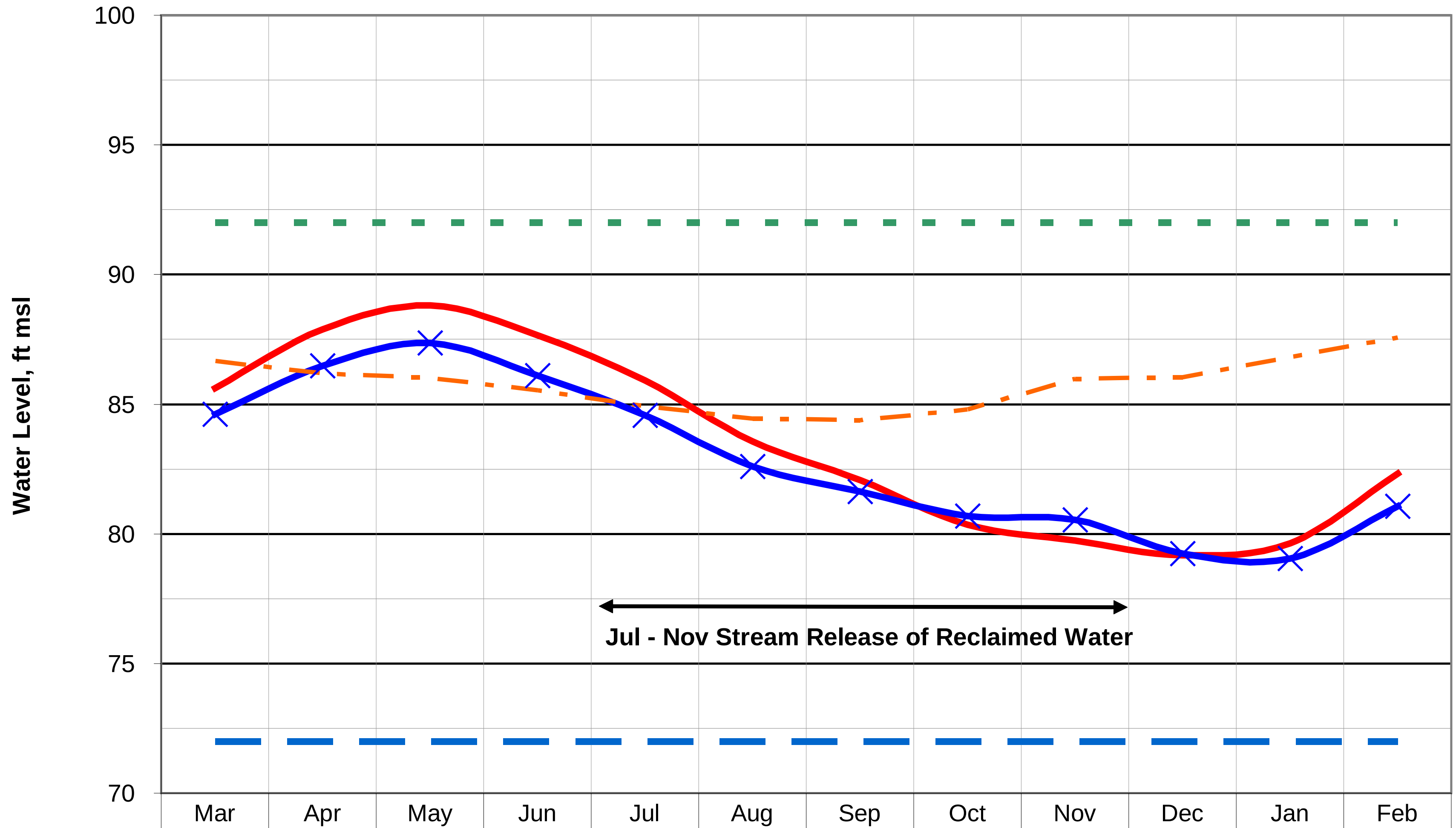
Implementation of Alternative 9 is estimated to yield a total of 14,000 AFY from the ground-water basin during normal or above normal hydrologic conditions. As in all of the alternatives considered for this study and the Permit 15000 Feasibility Study, water from Fallbrook Creek was modeled as passing through Lake O'Neill and discharging into the Lake O'Neill release canal.

8.2.2.1 Modeled Stream with Reservoir Release

The MODFLOW stream package was used to model the gaining and loosing stream segments of the Santa Margarita River on a monthly basis for 20 years. The stream package simulates the location and quantity of water that infiltrates through the streambed into the ground-water aquifer and vice versa from the aquifer into a gaining stream. The flow at the model boundary, near the Base hospital's current location, accounted for seasonal flows, extended wet and dry hydrologic cycles, future augmented flows for the Rancho California settlement, and estimated reservoir releases originating from Fallbrook PUD reclaimed water.

For the purpose of assessing the influence of the Alternative 9 scenario, the Fallbrook PUD reservoir releases were timed to occur from July 1 through November 30 each year. The five month, July to November release, was determined by viewing the results from different release scenarios on a 20-year monthly average graph and choosing the best solution for all years that minimized the ground-water level decline in late summer. The release scenarios ranged from 5 to 7 months and started from May through July. Figure 8-2 compares the water level results between Alternatives 1a and 9 on a 20-year, average monthly basis. The additional pumping in Alternative 9 lowers the high water levels in the spring months, while the additional stream release from July through November raises the water levels in the late summer months compared with Alternative 1a. In practice, actual day-to-day management would probably differ, depending on ground-water levels in the basin, projected water demand, and pattern of precipitation events during winter months. For purposes of this study, the release time was set as constant so that other varying influences could be monitored.

Average Monthly Simulated Water Levels @ Well 10/4-7J1, Upper Ysidora, MY 1-20



	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb
Alt 1a	85.61	87.89	88.82	87.66	85.91	83.56	82.07	80.35	79.74	79.17	79.65	82.33
Alt 9	84.61	86.47	87.36	86.11	84.56	82.60	81.63	80.69	80.53	79.24	79.05	81.06
Ground Surface	92.00	92.00	92.00	92.00	92.00	92.00	92.00	92.00	92.00	92.00	92.00	92.00
max Ext Depth	72.00	72.00	72.00	72.00	72.00	72.00	72.00	72.00	72.00	72.00	72.00	72.00
observed	86.68	86.18	86.01	85.54	84.93	84.44	84.37	84.80	85.98	86.04	86.82	87.56

FIGURE 8-2

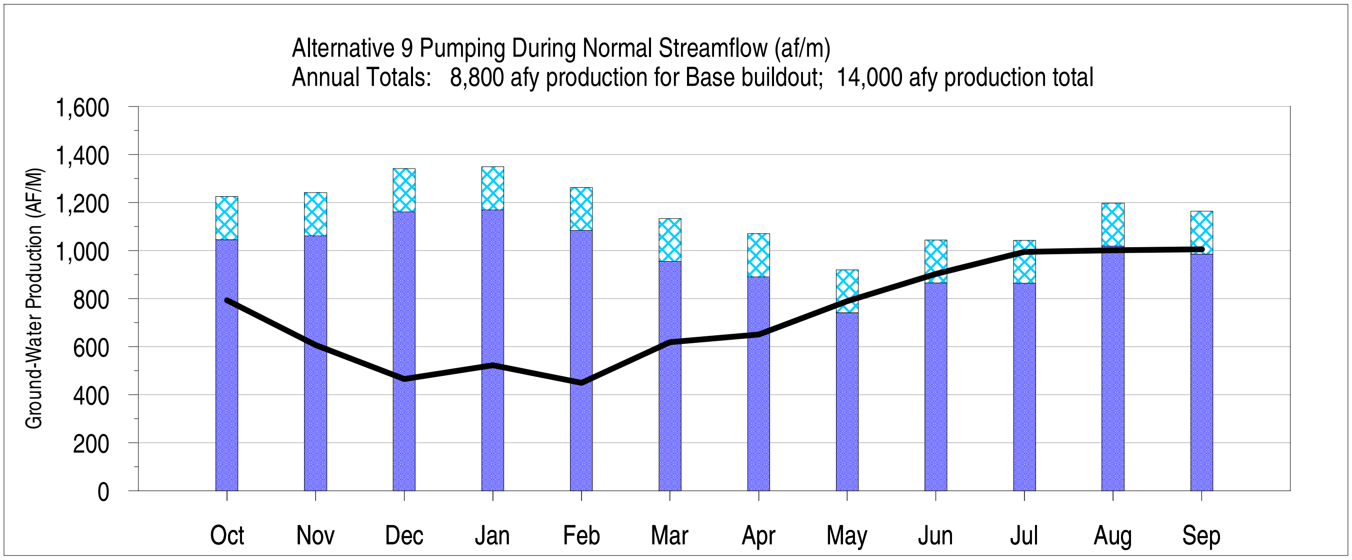
8.2.2.2 Ground-Water Production

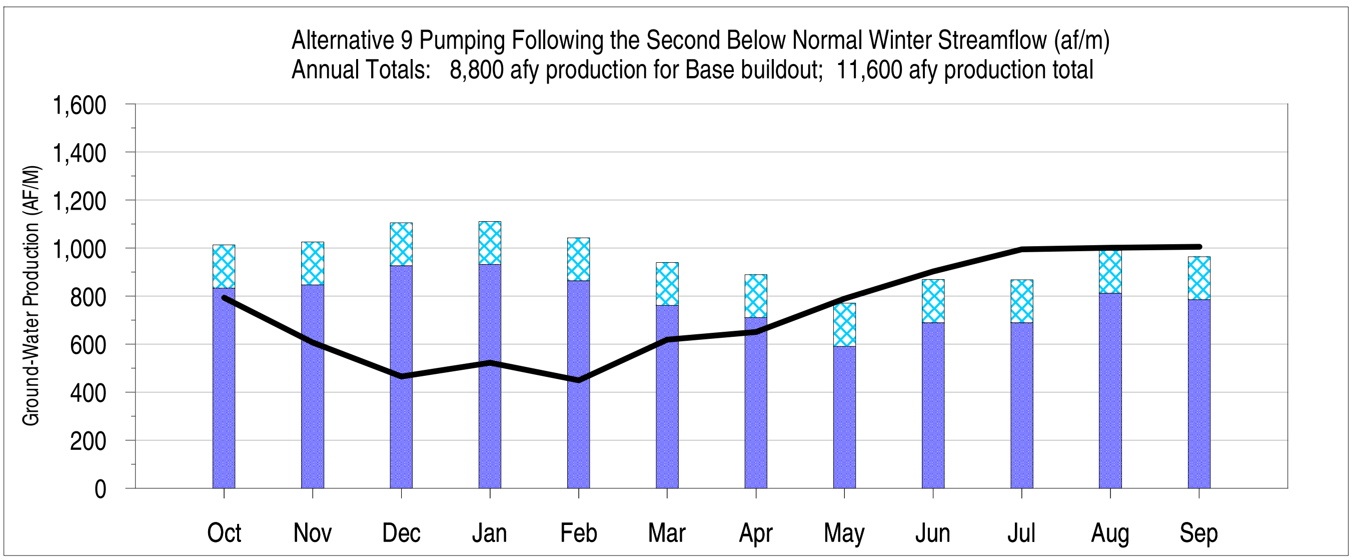
Ground-water pumping under Alternative 9 combines the 80% F3 pumping of Alternative 1a with two additional wells pumping an amount equivalent to the reservoir release, adjusted for evaporative losses. The 80% F3 pumping schedule considered in Alternative 1a was established under the Permit 15000 Feasibility Study to minimize impacts of ground-water level drawdown on riparian vegetation. This conjunctive use pumping schedule has been designed to lower the ground-water levels in the aquifer during the dormant winter season in order to have capacity in the aquifer to capture wintertime flow events and minimize mounding at the recharge basins. Based on this schedule, pumping rates are greatest during the winter and curtailed during the summer to help protect the riparian habitat. The 80% F3 pumping schedule also incorporates a dry year management plan that reduces pumping during consecutive dry years to minimize the impact at the time of drought conditions. Ground-water production is reduced by 2,400 AFY (to 80% of full pumping) commencing with the summer months following the second below normal winter/spring streamflow. If the below normal streamflow continues through a third consecutive winter/spring, ground-water production will be curtailed by an additional 2,400 AFY (to 59% of full pumping) until normal or above normal streamflow conditions return. Figure 8-3 compares the 80% F3, Fallbrook PUD release water, and Base full build-out monthly pumping schedules during different conditions in Alternative 9.

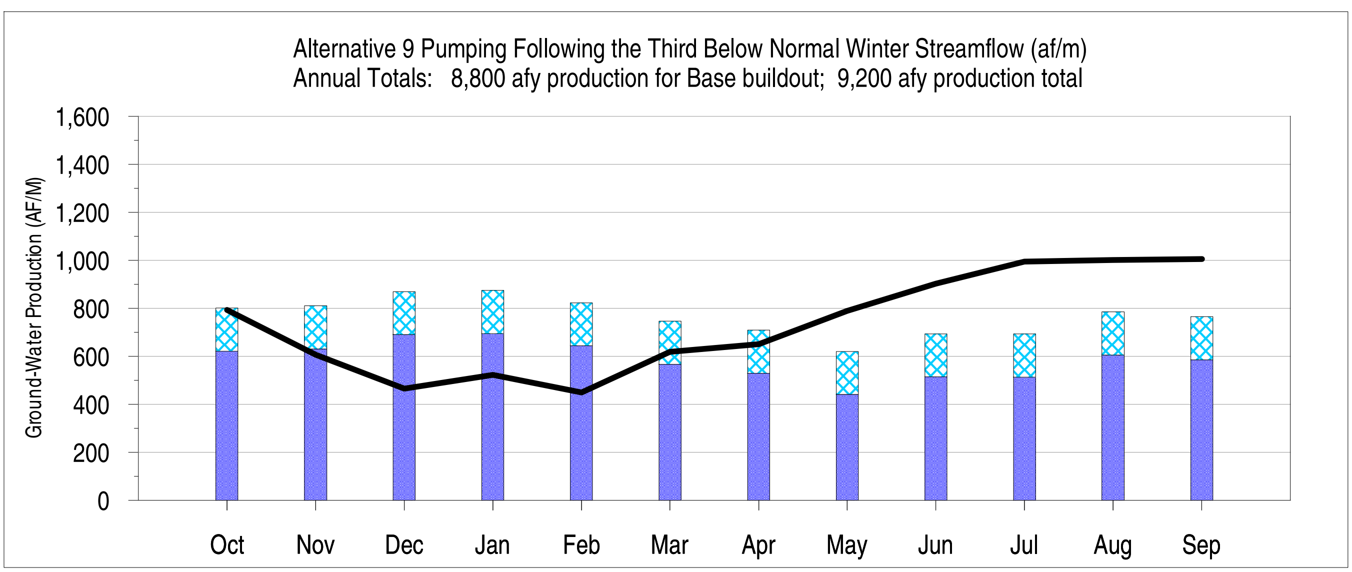
Alternative 9, scenario 2 allows for a median increase of 2,150 AFY of pumping in addition to the median 11,850 AFY of the 80% F3 pumping schedule, for a combined pumping of 14,000 AFY during a normal or above normal streamflow year. The additional pumping was expanded over time to match the growth that would be expected from the Fallbrook PUD released water during that same period of time. The median ground-water production is presented here instead of average pumping because it is more representative of a typical year of operation.

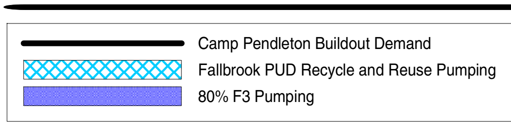
The 20-year period (water years 1980 through 1999) chosen for calibrating the ground-water model contains both drying (1987-1991) and wetting hydrologic (1980-1983) cycles. This same 20-year hydrologic cycle was projected forward to simulate future impacts from changes to the basin. The dry year management condition occurred during 3 consecutive years within the 20-year period chosen (May 1988 through April 1991) for this study. Ground-water production within the Upper Ysidora, Chappo, and Lower Ysidora sub-basins is shown in Figure 8-4. Table 8-6 summarizes the median annual pumping volumes and number of wells for the pumping schedules studied under Alternatives 1a and 9.

FIGURE 8-3









Alternative 9 Pumping Schedule
with Dry Year Management Plan

Ground-Water Production using Alternative 9 Pumping Schedule

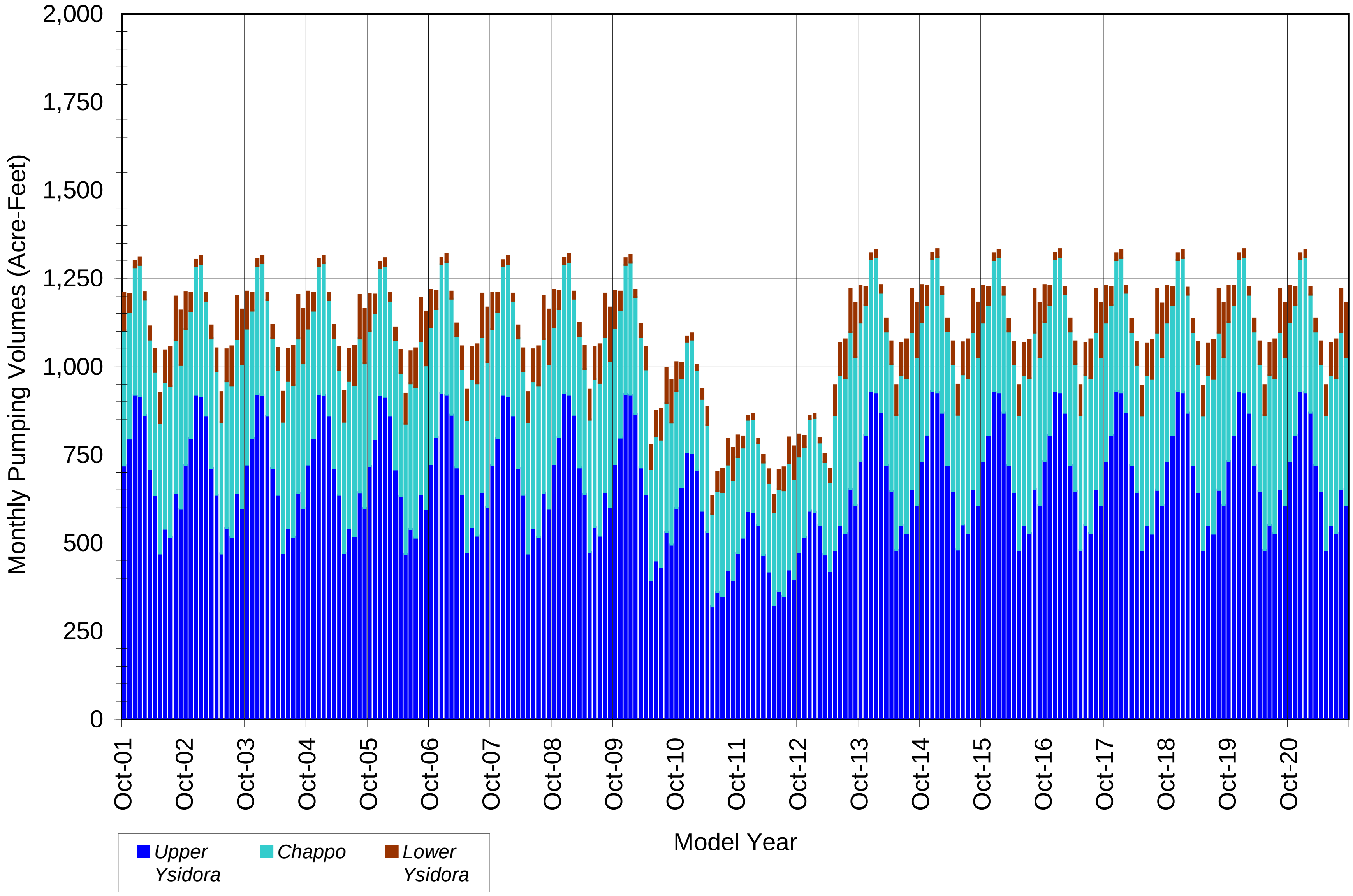


FIGURE 8-4

TABLE 8-6

Alternative 9 Well Production Summary during Normal or Above Normal Streamflow Year

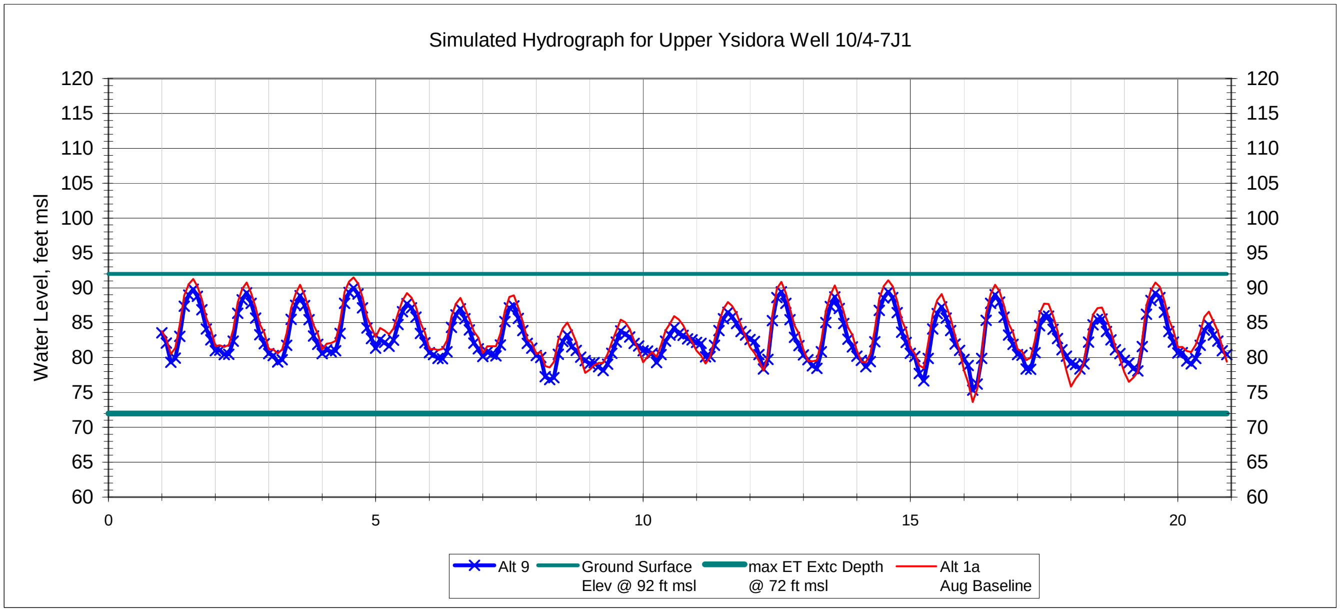
SUB-BASIN	NUMBER OF PRODUCTION WELLS	SUB-BASIN TOTAL PUMPING (AF/WY)	(%)	SUB-BASIN PUMPING TOTAL (AF/WY)	(%)
UPPER YSIDORA	5 Existing 3 Proposed 1 FPUD	4,150 2,340 1,070	55% 31% 14%	7,560	54%
CHAPPO	9 Existing 1 Proposed 1 FPUD	3,620 860 1,080	65% 16% 19%	5,560	40%
LOWER YSIDORA	2 Existing	880	100%	880	6%
TOTAL:	16 Existing 4 Proposed 2 FPUD	14,000		14,000	100%

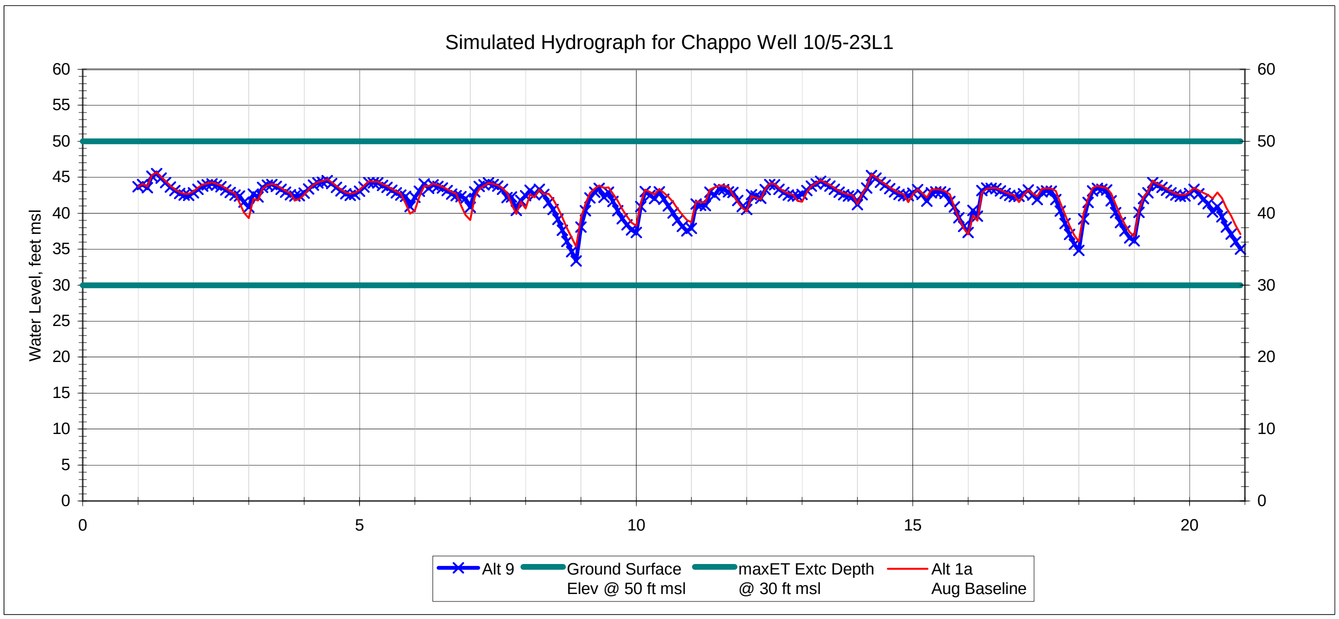
8.2.2.3 Ground-Water Model Results

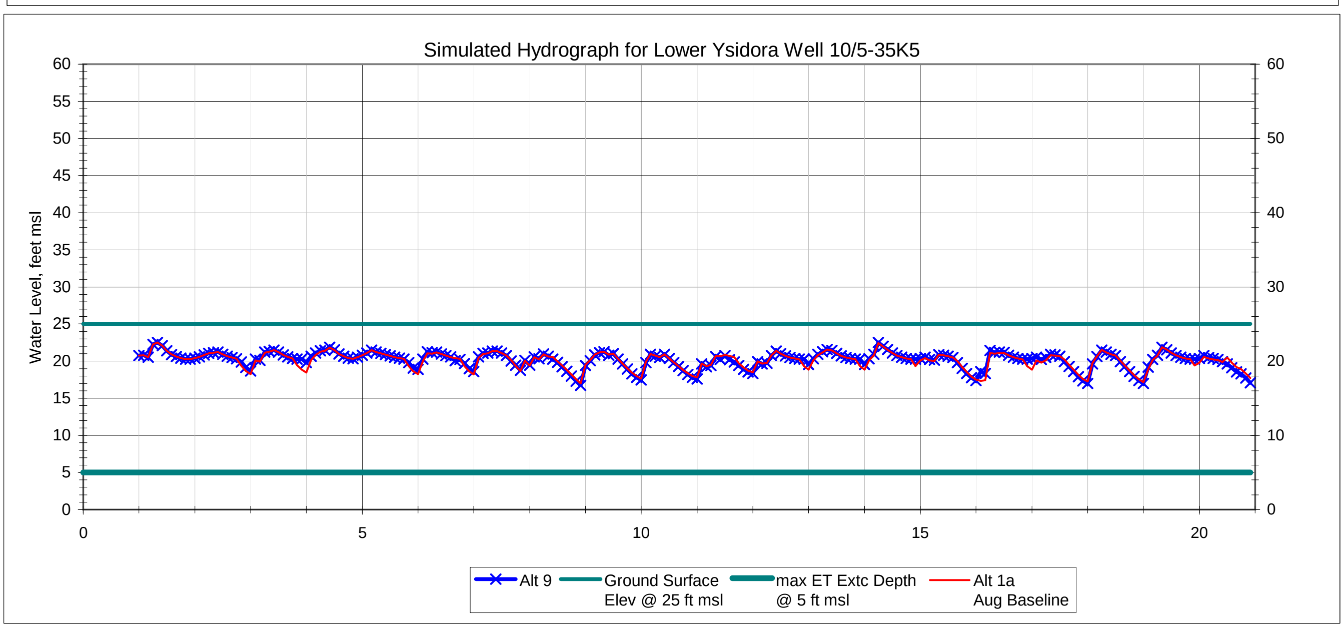
The lowest water level observed in the three simulated monitoring wells during the Alternative 9 model run occurred during Dec, model year (MY) 16 (corresponding to historic December 1994 climatic conditions) in the Upper Ysidora sub-basin with water level dropping to 75.3 feet, msl. Though this water level is close to the estimated ET extinction depth of 72 feet, msl, it occurs only once during a month where most riparian vegetation is less stressed. This Upper Ysidora observation well also occurs a distance of 600 feet from the Santa Margarita River in a grass field. Water levels are expected to be higher near the river where more riparian vegetation grows. The highest water level occurred during May, MY4 (corresponding to historic May 83 climate conditions) during late season precipitation events. Figure 8-5 shows baseline ground-water level data compared to model simulated results for Alternative 9 for all three sub-basins. Water level changes under Alternative 9 from baseline conditions are minimal in the Chappo (well 10/5-23L1) and do not appear to effect ground-water levels in the Lower Ysidora (well 10/5-35K5). The lack of response at the Lower Ysidora monitoring well is considered a good indicator that there will be no ill effects on the estuary or salt-water intrusion into the ground-water basin from implementation of Alternative 9.

Simulated and baseline monthly streamflows, observed at the Ysidora gage near Basilone Road and the southwest boundary in the Lower Ysidora sub-basin, are shown in Figure 8-6. The model predicts that Alternative 9 will have minimal impact on streamflow at these areas.

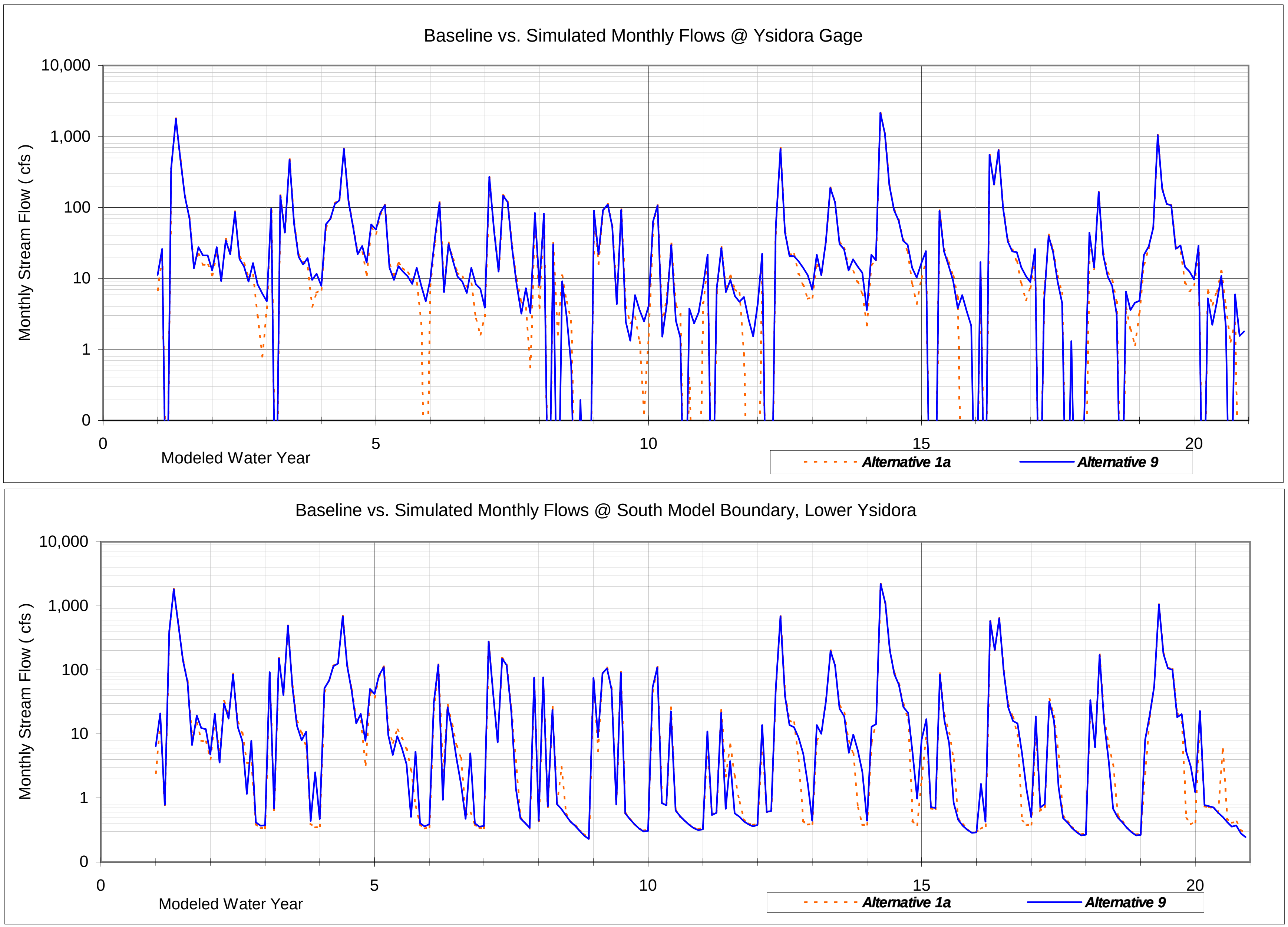
FIGURE 8-5







Alternative 9, Scenario 2 Ground-Water Model Simulated Water Levels



Alternative 9 Simmulated Stream Flow

FIGURE 8-6

The Alternative 9 model run is summarized in the water budget presented in Table 8-7. The Model provides calculated numbers for underflow, stream flow out of the model area, and evapotranspiration. Measured and estimated model input data provides water volumes for streamflow into the model domain, diversion to and release/spill from Lake O'Neill, ground-water pumping, and recoverable water from precipitation.

TABLE 8-7

Alternative 9 -- Average Annual Water Budget for MY 1 - 20 (af/wy)

		Alt 1a –Comparison Baseline		Alt 9 - Additional Streamflow and Pumping	
		Average	*Median*	*Average*	*Median*
Inflow:	Subsurface Underflow	1,290	1,310	1,280	1,300
	Santa Margarita River Inflow	55,860	30,740	58,030	32,860
	Lake O'Neill Spill and Release	2,060	2,150	2,060	2,150
	Fallbrook Creek Bypass	1,930	1,370	1,930	1,370
	Minor Tributary Drainages	2,120	1,720	2,120	1,720
	Waste Water Discharge	0	0	0	0
	Direct Precipitation	710	500	710	500
	Total Inflow:	63,970	37,790	66,130	39,900
Outflow:	Subsurface Underflow	230	220	230	230
	Santa Margarita River Outflow	47,940	21,120	47,980	20,940
	Ground-Water Pumping	11,240	11,850	13,330	14,000
	Evapotranspiration / Evaporation	2,680	2,710	2,580	2,680
	Diversions to Lake O'Neill	2,530	2,610	2,530	2,610
	Total Outflow:	64,620	38,510	66,650	40,460
Net change in GW and SW Storage:		650	720	520	560
Water Exchange within Model Domain					
Net Infiltration from Recharge Ponds		7,720	7,920	7,720	7,920
Net Stream Recharge to GW		4,470	3,940	6,550	5,980

8.2.3 Expected Yield for Alternative 9

The annual ground-water yield and surface diversion expected from the implementation of Alternative 9 are listed below in Table 8-8. The maximum annual surface diversion required to provide a median annual ground-water yield of 14,000 AFY is 13,920 AF. The median annual ground-water yield available for the Fallbrook PUD, would be 5,200 AFY, after the Base's build-out demand is met. The location of the point of diversion for the diversion and recharge component of the conjunctive use project would be at the identical location of the existing point of diversion.

TABLE 8-8

Alternative 9 – Annual Ground-Water Yield and Surface Diversion

WATER RIGHT	**ALTERNATIVE 1A (AFY)**	**ALTERNATIVE 9 (AFY)**
Base's Build-out Demand	8,800	8,800
Minimum Additional Ground-Water Yield (AFY)	3,050	5,200
Total Annual Project Yield	11,850	14,000
Maximum Additional Surface Water Diversion (AFY)	8,420	8,420

8.3 ALTERNATIVE 10

Alternative 10 includes the wetland pipeline, treatment wetlands, storage reservoir, reservoir discharge pipeline, and conveyance facilities described in detail in Chapter 7. In addition to these components of the conjunctive use program, all recommended facilities from Alternative 3 have also been included. The purpose of Alternative 10 is to estimate the maximum yield of the lower ground-water basin with release of tertiary treated wastewater from the Fallbrook PUD WWTP to the Santa Margarita River. The increased capacity of the diversion facilities outlined in Alternative 3 allow for the optimal diversion of winter flows from the Santa Margarita River. The additional release of treated wastewater to a treatment wetland and storage reservoir, before discharge to the Santa Margarita River, will provide riparian habitat maintenance flows during the dry summer and fall months. The release of these waters allow pumping levels to be maintained at higher levels during the dry season, providing a total yield of 16,200 AFY, 2,150 AFY more than compared to Alternative 3.

This section presents the results of a combination of increased diversion capacity coupled with a recycle and reuse program sized for the annual release and reuse of 2,500 AFY of tertiary treated wastewater (Scenario 2). The Appendix provides the modeling results for a recycling and reuse program sized for 1,500 AFY and 3,500 AFY, Scenarios 1 and 2, respectively.

8.3.1 Surface Water Analysis for Alternative 10

The surface water model for Alternative #10 integrated the Reclaimed Water Reservoir Operations Model (RWROM) with an existing Reservoir Operation Model for the Camp Pendleton Facilities (CPROM). The output from the RWROM adds to flows of the Santa Margarita River to serve as the input to the CPROM. The output from the CPROM provides the surface water input for the MODFLOW ground-water model.

8.3.1.1 Reclaimed Water Reservoir Operations Model (RWROM)

A Reclaimed Water Reservoir Operations Model (RWROM) was developed to simulate all surface water influencing the water reuse system, noting losses, gains, storage, and releases. The three scenarios for the RWROM used in Alternative 10 are the same as those used in Alternative 9. A summary of the results of the RWROM tracking tertiary treated wastewater from the Fallbrook PUD WWTP to the Santa Margarita River is provided in Table 6-6.

8.3.1.2 Camp Pendleton Reservoir Operations Model (CPROM)

The Camp Pendleton facilities modeled in Alternative 10 differ significantly than those used in Alternative 9. The additional facilities include an Obermeyer Dam diversion structure on the Santa Margarita River, improved diversion channel capacity, two new recharge ponds, and additional ground-water wells and pumping facilities. A detailed description of the CPROM model and the results can be found in Chapter 7 (Alternative 3) of the Permit 15000 study. A brief description the model parameters and Camp Pendleton facilities that are relevant to this study, is provided within this section.

Streamflow at Model Boundary

The simulated flow in the Santa Margarita River remains the same as in Alternative 9. See Table 8-2 for the results of the streamflow at the model boundary.

Lake O'Neill

Lake O'Neill remains unchanged from the Alternative 9 description.

Recharge Ponds

Alternative 10 has a total of seven recharge ponds located off the diversion channel from the Santa Margarita River. The additional ponds allow more water to recharge the ground-water system. The reservoir operations model calculates the daily flow of water into the recharge ponds, the net effect of precipitation and evaporation, the volume of water infiltrating into the ground, and finally the volume of water, which spills out of the last pond. These calculations provide input for the ground-water model MODFLOW™, which then simulates the path of the recharged water once it infiltrates below the surface.

8.3.1.3 Alternative 3 Reservoir Operation Model

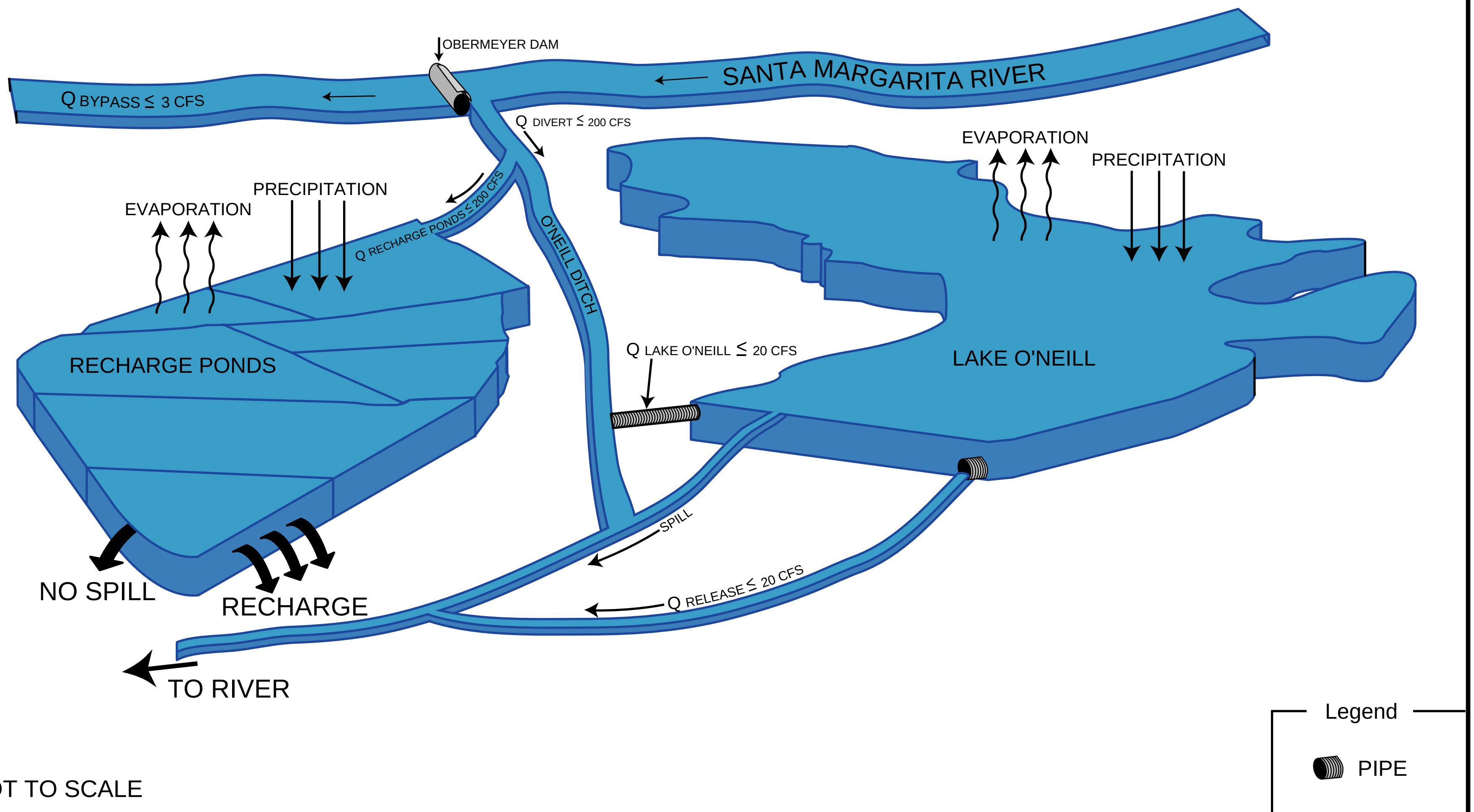
The reservoir operations model used for Alternative 3 estimated the rate of diversion from the Santa Margarita River to both the recharge ponds and Lake O'Neill. Limitations to the diversion rate from the Santa Margarita River accounted for in the reservoir operations model included not only the available water supply and physical limitations of the diversion facilities, but also such factors as available water rights, recharge pond infiltration rates, rainfall, evaporation, and spill from both the ponds and the lake. The Alternative 3 reservoir operations model also accounted for augmented surface flows and increased diversion efficiencies due to project improvements recommended in Chapter 6 of the Permit 15000 Study. Results from the model analysis were used by the ground-water model to estimate recharge at the ponds, streamflow past the diversion point, and releases from Lake O'Neill.

When combined with the RWROM, the CPROM portion of the Alternative 3 does not change. Thus, the Alternative 10 CPROM gives the same results as the Alternative 3 CPROM with the exception of the streamflow at the model boundary, which will increase by the amount of reclaimed flow released from the reservoir.

8.3.1.4 Alternative 10 Reservoir Operation Model

A schematic diagram of the reservoir operations model for Alternative 10 is shown in Figure 8-7. The Alternative 10 Diversion Schedule to Lake O'Neill and the Recharge Ponds is shown in Table 8-9. During periods of diversion, three cfs remains in the Santa Margarita River while the remaining surface flow may be diverted to either Lake O'Neill or to the recharge ponds. The simulated diversion to Lake O'Neill is limited to 20 cfs or less, while the maximum simulated diversion to the recharge ponds with the improved facilities has increased to 200 cfs.

1922/FigureD(LakeGraph-Alt10).ai



NOT TO SCALE

STETSON ENGINEERS INC.

Surface Water Analysis
Resevoir Operations Model

Alternative 10

New Diversion Dam Expanded
Headgate Improved Canal
New Recharge Ponds

October 15, 2001

FIGURE 8-7

TABLE 8-9

Alternative 10 Diversion Schedule to the Recharge Ponds and Lake O'Neill

Month	Activity	Rate	Limit	Water Right
Diversions to Lake O'Neill				
Nov	Drain	$Q_{release}$ <= 20 cfs	Min Volume = 100 AF	Pre-1914 Water Right
Dec to Jan	Fill	$Q_{lake\ O'Neill}$ <= 20 cfs	Max Volume = 1,200 AF	Permit 15000
Feb to May	Precip & Evap	Q_{spill} = f(precip & evap)	N/A	N/A
June to Oct	Fill	$Q_{lake\ O'Neill}$ <= 20 cfs	No spill of Pre-1914 water	Pre-1914 Water Right
Diversions to Recharge Ponds				
Nov	Fill w/ 100% Q_{divert}	$Q_{recharge\ ponds}$ <= 200 cfs	No Spill	Permit 15000
Dec to Jan	Fill w/ $Q_{divert} - Q_{lake\ O'Neill}$	$Q_{recharge\ ponds}$ <= 200 cfs	No Spill	Permit 15000
Feb to May	Fill w/ 100% Q_{divert}	$Q_{recharge\ ponds}$ <= 200 cfs	No Spill	Permit 15000
Jun	Fill w/ $Q_{divert} - Q_{lake\ O'Neill}$	$Q_{recharge\ ponds}$ <= 200 cfs	No Spill	Permit 15000
Jul to Sept	No Diversion	$Q_{recharge\ ponds}$ = 0 cfs	N/A	N/A
Oct	Fill w/ $Q_{divert} - Q_{lake\ O'Neill}$	$Q_{recharge\ ponds}$ <= 200 cfs	No Spill	Permit 15000

** Note: The first 4,000 AFY is attributed to permit 15000, license 10494 while the remaining diversion to the recharge ponds would be developed under permit 15000, Application 21471B.*

The simulated annual diversion to Lake O'Neill and the recharge ponds, under the Pre-1914 and Permit 15000 Water Rights is shown in Table 8-10

TABLE 8-10

ALTERNATIVE 10 - DIVERSIONS TO THE RECHARGE PONDS AND LAKE O'NEILL (AFY)

Model Year	Pre-1914 Water Diverted to Lake O'Neill from Dec 1st-Mar 31st (AFY)	Permit 15000 Water Diverted to Lake O'Neill from Jan 1st – Dec 31st (AFY)	Alternative 10 Diversions to Recharge Ponds (AFY)	Total Diversions from the Santa Margarita River (AFY)
Total	**28,200**	**22,300**	**219,400**	**269,900**
Average	1,400	1,100	11,000	13,500
Median	1,500	1,100	10,700	13,300
Min	900	1,100	4,500	6,500
Max	1,500	1,100	19,200	21,800

Table 8-11 summarizes the results of the Alternative 9 reservoir operations model. These values serve as the input to the MODFLOW ground-water model

TABLE 8-11

Alternative 10 – Scenario 2
Obermeyer Dam, New Headgate, and Improved Channel

20-year Simulated Period	Augmented Flow Santa Margarita River (AF)	Scenario 2 Reclaimed Water in SMR (AF)	Total Diversion Max 200 cfs (AF)	Total Diversion to Lake O'Neill (AF)	Diversion to Recharge Ponds (AF)	Ground Water at Recharge Ponds (AF)
20-yr Total	1,117,110	43,501	269,918	50,520	219,397	218,718
Average	55,860	2,150	13,496	2,530	10,970	10,936
Median	30,740	2,214	13,273	2,610	10,654	10,591
Min	10,730	2,025	6,540	2,000	4,540	4,544
Max	226,230	2,252	21,839	2,640	19,222	19,193

8.3.2 Ground-Water Analysis for Alternative 10

The ground-water model analysis for Alternative 10 compares the simulated results from the release of reservoir water and the addition of two new production wells to Alternative 3 conditions from the Permit 15000 Feasibility Study.

- Alternative 3 includes:
  - maintenance and repair of the existing diversion and recharge system on the Base,
  - augmented streamflow from the recent settlement with the RCWD,
  - 200 cfs capacity in the diversion system (headgate and canals),
  - optimized water management which yields
    - average annual diversion of 2,530 AFY to Lake O'Neill,
    - addition of two new recharge ponds
    - average annual diversion of 10,970 AFY to seven recharge ponds,
    - production of 14,050 AFY ground water during normal or above normal streamflow years.
- Alternative 10 incorporates
  - Alternative 3 as a baseline condition,
  - increased streamflow in the Santa Margarita River from the 5 month release of reservoir water between July and November, generated from Scenario 2 of Fallbrook PUD reclaimed water,
  - the addition of two new ground-water production wells yielding 2,150 AFY.

Implementation of Alternative 10 is estimated to yield a total of 16,200 AFY from the ground-water basin. As in all of the alternatives considered for this study and the Permit 15000 Study, water from Fallbrook Creek was modeled as passing through Lake O'Neill and discharging into the Lake O'Neill release canal to account for the separate water rights.

8.3.2.1 Modeled Stream with Reservoir Release

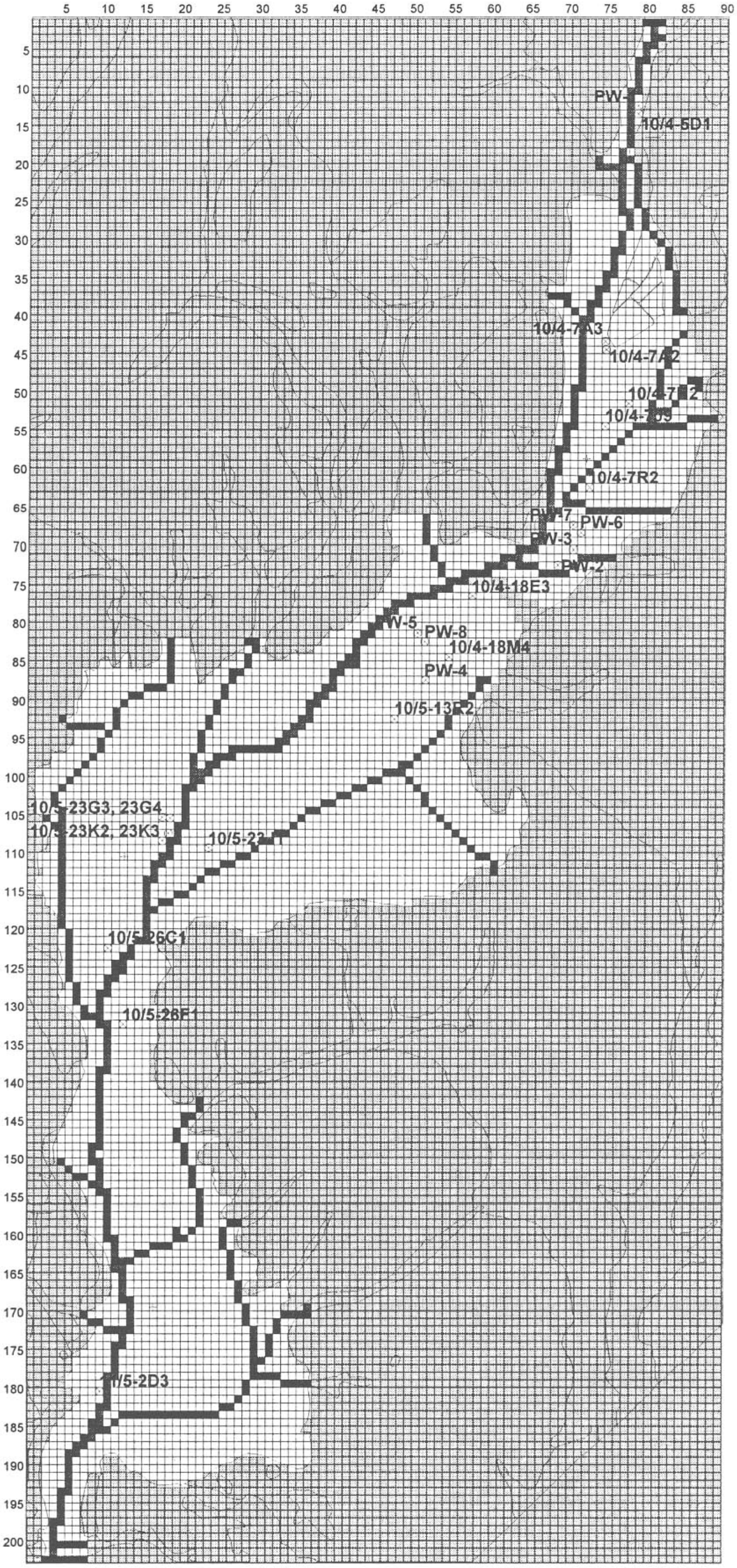
For the purpose of comparison with Alternative 9 and assessing the influence of the Alternative 10 scenario, the Fallbrook PUD reservoir releases were timed to occur from July 1 through November 30 each year (Section 8.2.2.1). In practice, actual day-to-day management would probably differ, depending on ground-water levels in the basin, projected water demand, and pattern of precipitation events during winter months. For purposes of this study, the release time was set as constant so that other varying influences could be monitored. Using the July through November simulated release of water from the reservoir, the model predicted similar results between Alternatives 3 and 10 with no additional diversions to the recharge ponds. When the simulated water released from the reservoir was diverted to the recharge ponds, the water levels in the Upper Ysidora rose during the late summer months. There was minimal difference in the Chappo Sub-Basin between the simulated effects of pond recharge versus stream recharge of the water released from the reservoir.

8.3.2.2 Ground-Water Production

Ground-water pumping under Alternative 10 combines the 95% F3 pumping of Alternative 3 with two additional wells pumping an amount equivalent to the reservoir release, adjusted for evaporative losses. Figure 8-8 shows the location of the six proposed wells (PW-1 through PW-6) for Alternative 3, 95% F3 pumping, and the two additional wells (PW-7 and PW-8) for Alternative 10.

The 95% F3 pumping schedule considered in Alternative 3 was established under the Permit 15000 Study to minimize impacts of ground-water level drawdown on riparian vegetation. This conjunctive use pumping schedule has been designed to lower the ground-water levels in the aquifer during the dormant season in order to create storage capacity in the aquifer to capture wintertime flow events and minimize mounding at the recharge basins. Based on this schedule, pumping rates are greatest during the winter and curtailed during the summer to help protect the riparian habitat. The 95% F3 pumping schedule also incorporates a dry year management plan that reduces pumping during consecutive dry years to minimize the impact at the time of drought conditions. Ground-water production is reduced by 2,850 AFY (to 80% of full pumping) commencing with the summer months following the second below normal winter/spring streamflow. If the below normal streamflow continues through a third consecutive

FIGURE 8-8



Lower Santa Margarita River

Alternative 10

Existing and Proposed Well Locations within the Model Area winter/spring, ground-water production will be curtailed by a total of 5,700 AFY (59% of full pumping) until normal or above normal streamflow conditions return. Figure 8-9 compares the 95% F3, Fallbrook PUD release water, and Base full build-out monthly pumping schedules during different conditions in Alternative 10.

Alternative 10, scenario 2 incorporates a median increase of 2,150 AFY of pumping in addition to the median 14,050 AFY of the 95% F3 pumping schedule, for a combined pumping of 16,200 AFY during a typical year. The additional pumping was expanded over time to match the growth that would be expected from the Fallbrook PUD released water (see Section 6) during that same period of time. Median pumping is representative of a typical year of operation during normal or above normal streamflow. The dry year management condition occurred during three consecutive years within the 20 year simulated period. Projected ground-water production within the Upper Ysidora, Chappo, and Lower Ysidora sub-basins is shown in Figure 8-10. Table 8-12 summarizes the median annual pumping volumes and number of wells for the pumping schedules studied under Alternatives 3 and 10.

TABLE 8-12

ALTERNATIVE 10 WELL PRODUCTION SUMMARY DURING NORMAL OR ABOVE NORMAL STREAMFLOW YEAR

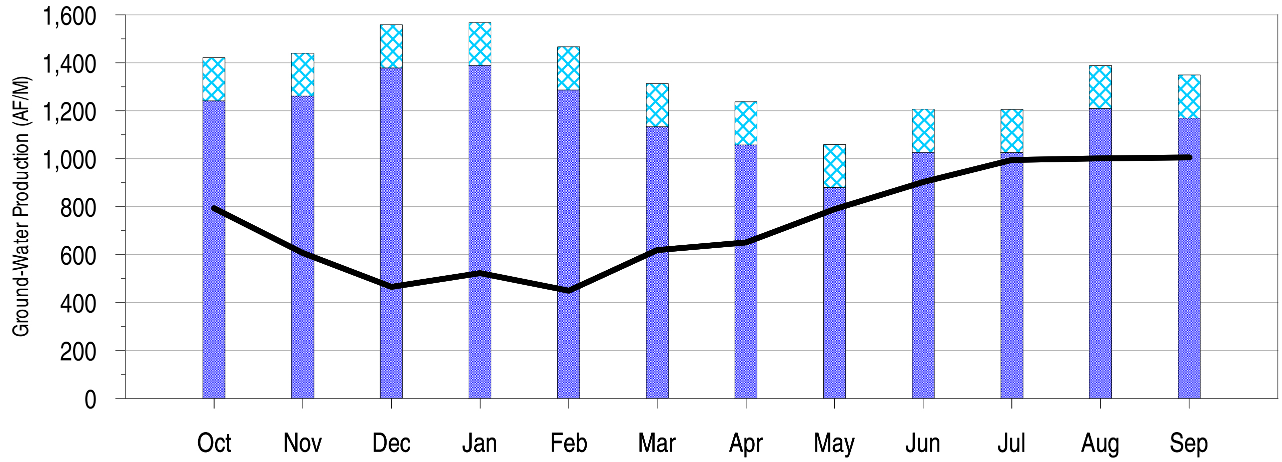
SUB-BASIN	Number of Production Wells	Sub-Basin Total Pumping (af/wy)	Sub-Basin Total Pumping (%)	Sub-Basin Pumping Total (af/wy)	Sub-Basin Pumping Total (%)
UPPER YSIDORA	5 Existing	6,230	63%		
	4 Proposed	2,510	26%		
	1 FPUD	1,070	11%	9,810	60%
CHAPPO	9 Existing	2,620	65%		
	2 Proposed	1,670	16%		
	1 FPUD	1,070	19%	5,360	33%
LOWER YSIDORA	2 Existing	1,030	100%	1,030	6%
TOTAL:	16 Existing six Proposed 2 FPUD	16,200		16,200	100%

8.3.2.3 Ground-Water Model Results

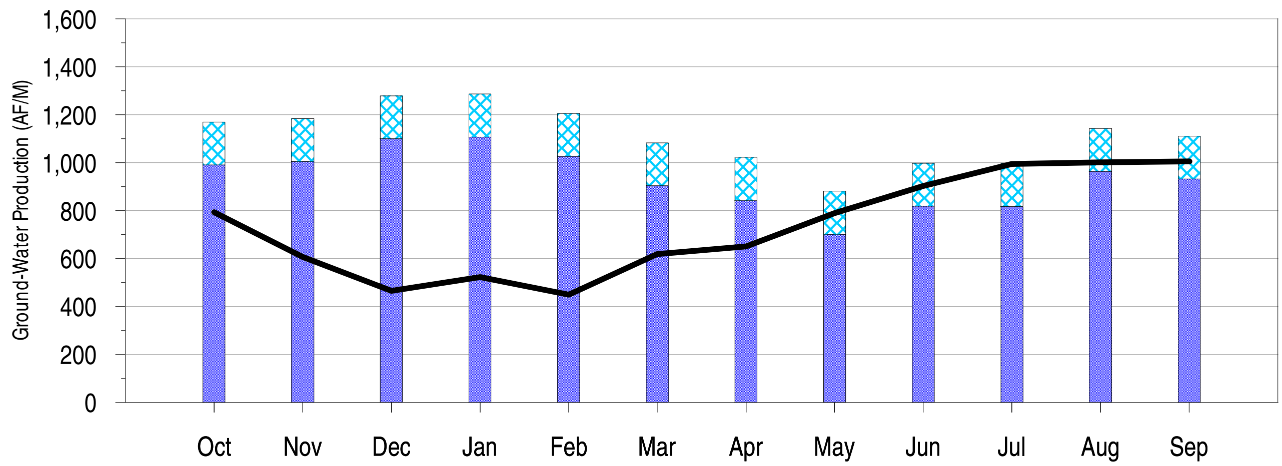
The pumping schedule proposed for Alternative 10 produces about 15% more water than Alternative 3, and almost 27% more water than Alternative 1a, on an average annual basis. The

FIGURE 8-9

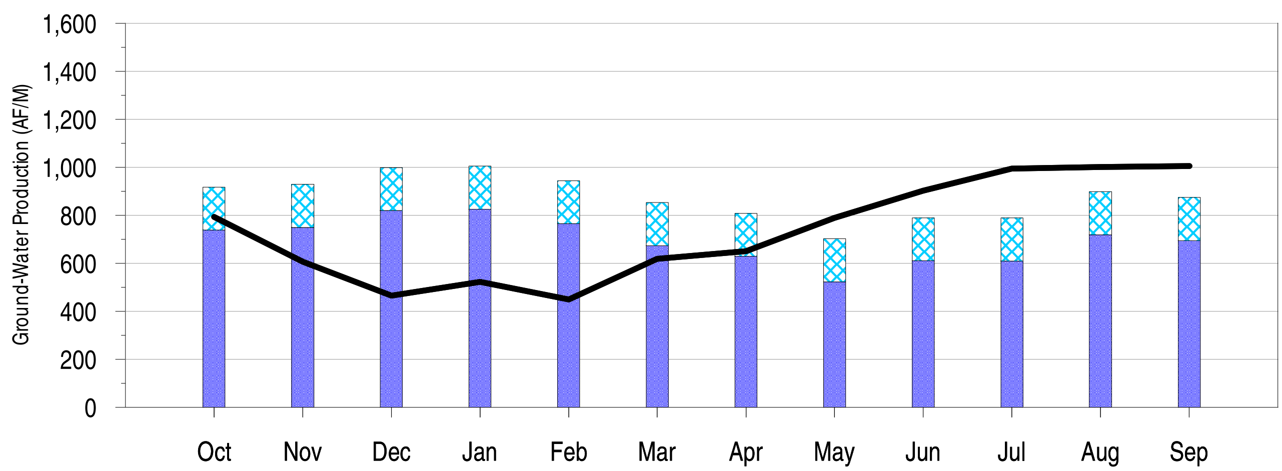
Alternative 10 Pumping During Normal Streamflow (af/m)
Annual Totals: 8,800 afy production for Base buildout; 16,200 afy production total

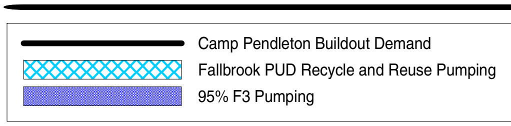


Alternative 10 Pumping Following the Second Below Normal Winter Streamflow (af/m)
Annual Totals: 8,800 afy production for Base buildout; 13,400 afy production total



Alternative 10 Pumping Following the Third Below Normal Winter Streamflow (af/m)
Annual Totals: 8,800 afy production for Base buildout; 10,500 afy production total





Alternative 10 Pumping Schedule
with Dry Year Management Plan

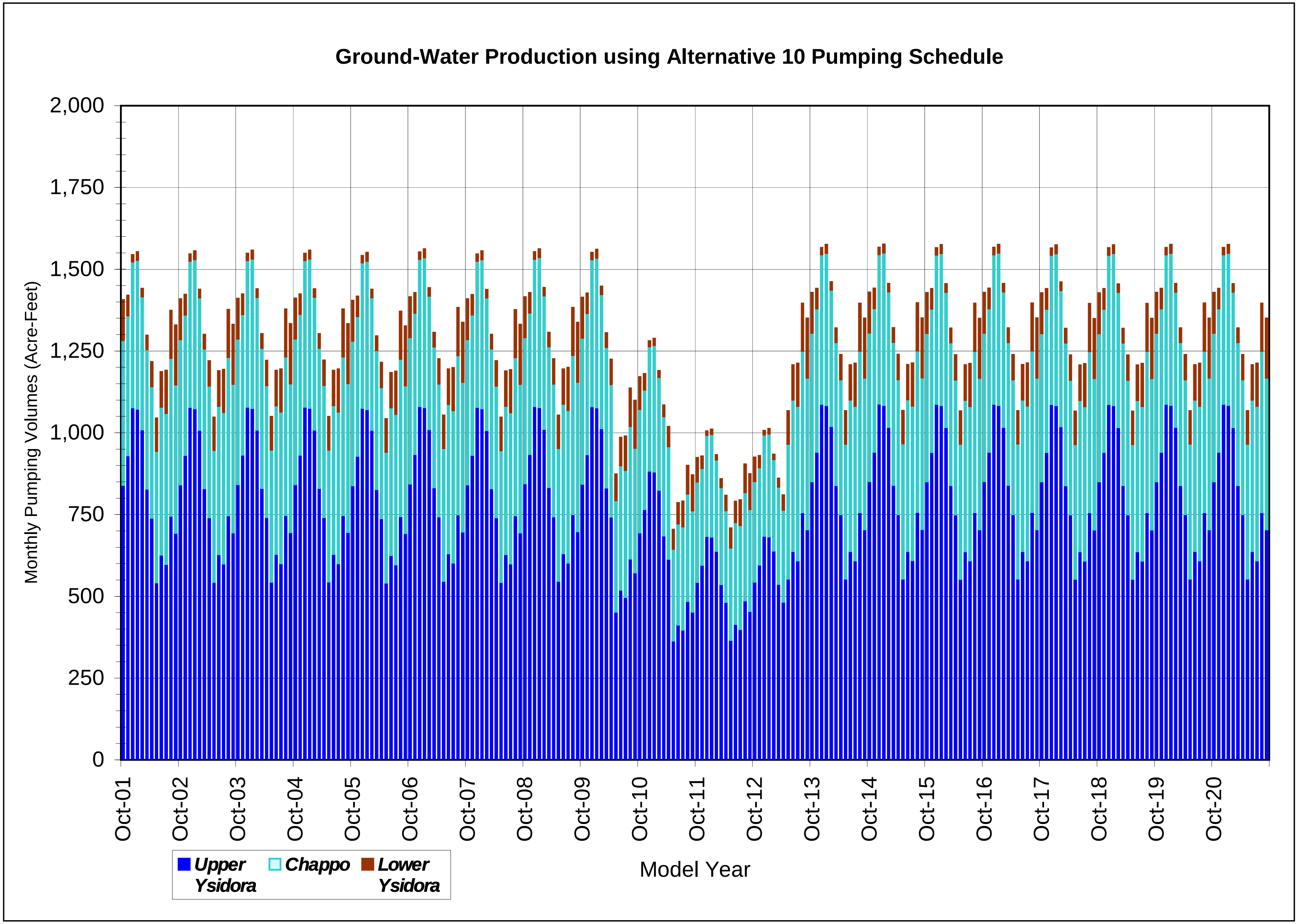

Ground-Water Production using Alternative 10 Pumping Schedule
2,000
1,750
1,500
1,250
1,000
750
500
250
0
Monthly Pumping Volumes (Acre-Feet)
Oct-01
Oct-02
Oct-03
Oct-04
Oct-05
Oct-06
Oct-07
Oct-08
Oct-09
Oct-10
Oct-11
Oct-12
Oct-13
Oct-14
Oct-15
Oct-16
Oct-17
Oct-18
Oct-19
Oct-20
Upper Ysidora
Chappo
Lower Ysidora
Model Year

FIGURE 8-10

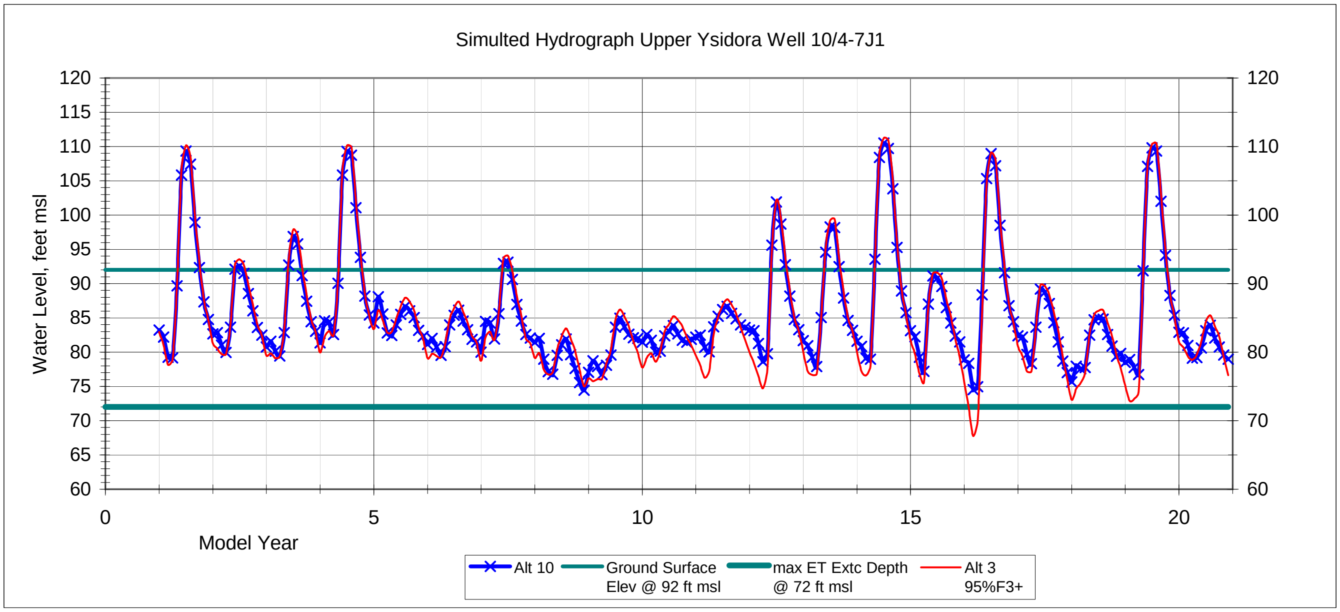
water table is drawn down in the wintertime by the seasonal pumping thereby creating more aquifer storage capacity in winter months when water is available for diversion. Combining the additional pumping with expanded ground-water recharge capacity by approximately doubling the existing recharge ponds, is predicted to yield an average diversion of 10,970 AFY. This is 40% more diversion to the ponds than under Alternatives 1a and 9 (7,760 AFY)

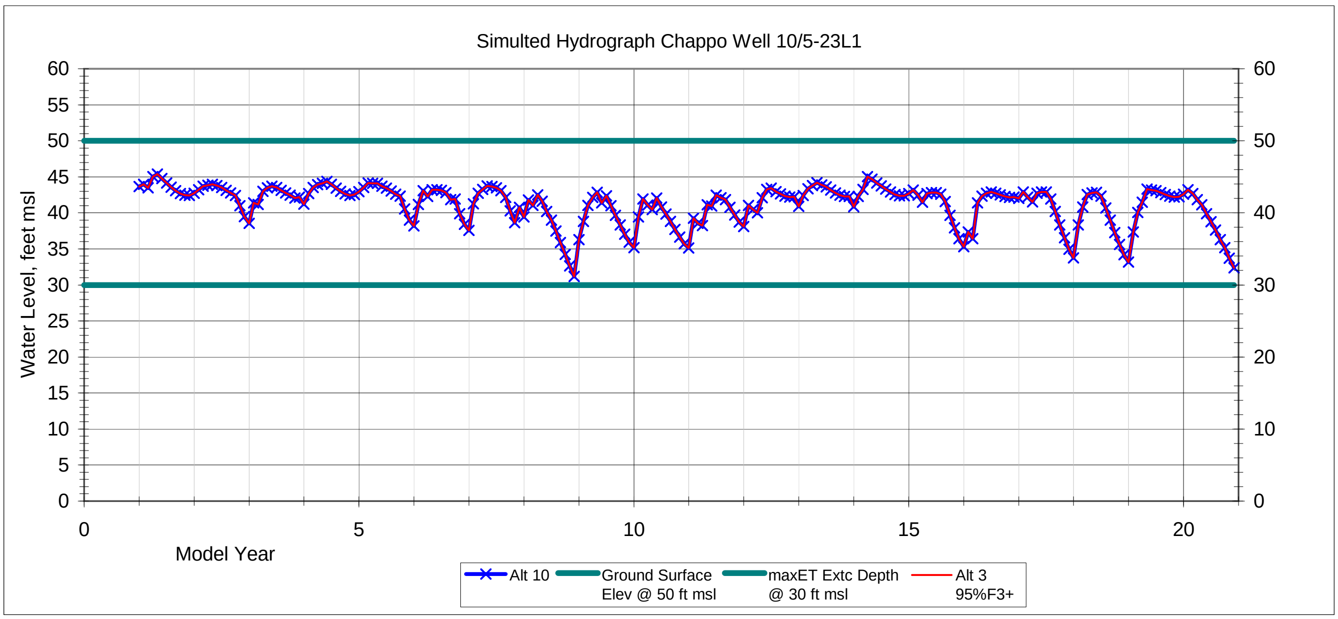
The large fluctuation in water levels simulated for the Upper Ysidora (Figure 8-11) is related to the greater volumes of water that can be diverted to the expanded recharge ponds with the 200 cfs diversion structure and increased canal capacity. The ground-water model simulates water levels, with respect to porosity of the unconsolidated alluvium, approximately 4.5 times higher above land surface than would occur. The occurrences of water levels above land surface indicate water ponding on the surface in the Upper Ysidora during the wet season. The Model does not account for the sheet runoff that would occur under these circumstances. The existing Upper Ysidora observation well, 10/4-7J1, is located approximately 3,000 feet down gradient of the proposed recharge ponds and 200 feet from the Lake O'Neill spill and release ditch. Diversions to Lake O'Neill have been maximized for these simulations, with all of Fallbrook Creek passing through. Higher simulated spills and releases from Lake O'Neill are probably the cause of this simulated response of water levels above ground surface. Under day-to-day management practices, this flooding and ponding would probably not occur, and storm events would pass through the system more rapidly, reducing the amount of ponding that would occur due to high ground-water levels. Field measurements of the canal efficiency of the Lake O'Neill ditch, and effects of mounding on the recharge at the percolation basins would need to be analyzed to develop the best management practice for these conditions.

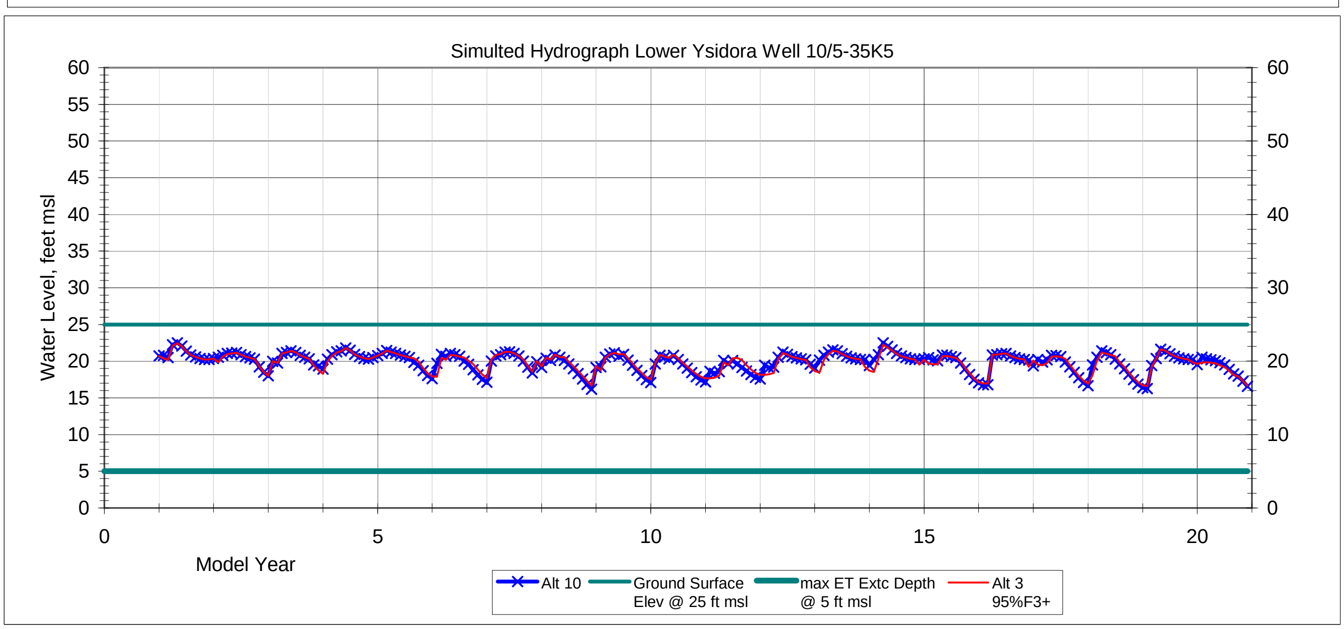
The lowest water level observed in the three simulated monitoring wells during the Alternative 10 model run occurred during Dec, MY 16 (corresponding to historic December 1994 climatic conditions) in the Upper Ysidora sub-basin with water level dropping to 72.36 feet, msl. December MY 16 corresponds to the second lowest precipitation year with 4.4 inches of rainfall. The average precipitation for this 20-year period is 12.01 inches. Though this water level is close to the ET extinction depth, it occurs only once during a month where most riparian vegetation is less stressed. This well also occurs a distance of 600 feet from the Santa Margarita River in a grass field. Water levels are expected to be higher near the river where more riparian vegetation grows. Figure 8-11 shows Alternative 3 baseline ground-water level data compared to model simulated results of Alternative 10 for all three sub-basins.

Water level changes under Alternative 10 from baseline conditions in Alternative 3 are minimal in the Chappo (well 10/5-23L1) and do not appear to effect ground-water levels in the Lower Ysidora (well 10/5-35K5). The lack of response at the Lower Ysidora monitoring well is considered a good indicator that there will be no ill effects on the estuary or salt-water intrusion into the ground-water basin from implementation of Alternative 10.

FIGURE 8-11







Alternative 10, Scenario 2 Ground-Water Model Simulated Water Levels

Simulated and baseline monthly streamflows observed at the Ysidora gage near Basilone Road and the southwest boundary in the Lower Ysidora sub-basin are shown in Figure 8-12. The model predicts that Alternative 10 will have minimal impact on streamflow at these areas.

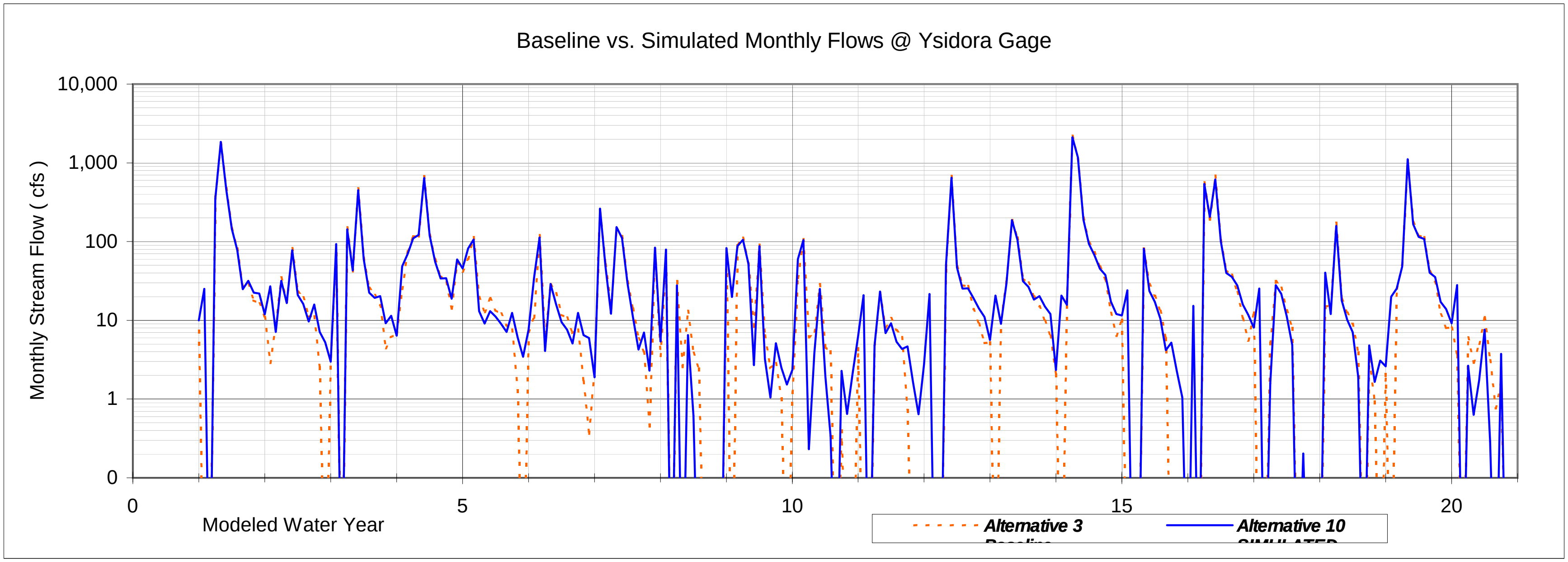
The Alternative 10 model run is summarized in the water budget presented in Table 8-13. The Model provides calculated numbers for underflow, stream flow out of the model area, and evapotranspiration. Measured and estimated model input data provides water volumes for streamflow into the model domain, diversion to and release/spill from Lake O'Neill, ground-water pumping, and recoverable water from precipitation.

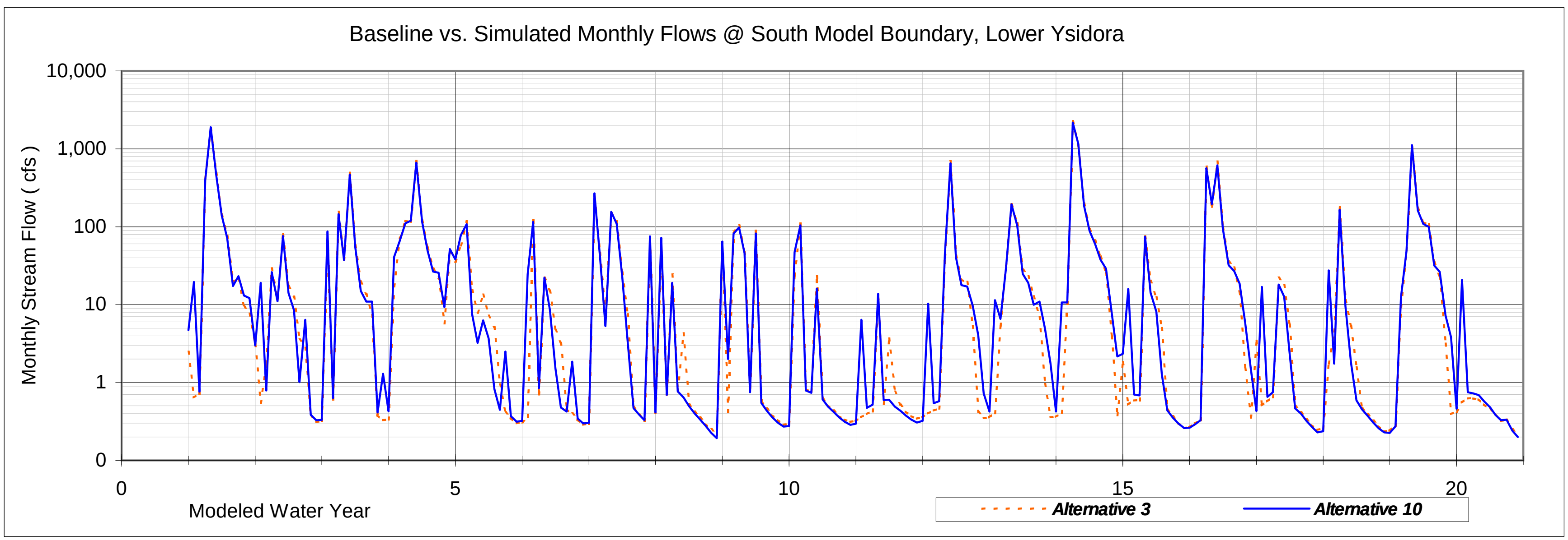
TABLE 8-13

Alternative 10 -- Average Annual Water Budget for MY 1 - 20 (af/wy)

		Alt 3 -Comparison Baseline		Alt 10 - Additional Streamflow and Pumping	
		Average	*Median*	*Average*	*Median*
Inflow:	Subsurface Underflow	1,320	1,340	1,310	1,330
	Santa Margarita River Inflow	55,860	30,740	58,030	32,860
	Lake O'Neill Spill and Release	2,060	2,150	2,060	2,150
	Fallbrook Creek Bypass	1,930	1,370	1,930	1,370
	Minor Tributary Drainages	2,120	1,720	2,120	1,720
	Waste Water Discharge	0	0	0	0
	Direct Precipitation	710	500	710	500
	Total Inflow:	64,000	37,820	66,520	39,930
Outflow:	Subsurface Underflow	220	220	230	220
	Santa Margarita River Outflow	47,480	19,740	47,220	18,850
	Ground-Water Pumping	13,350	14,050	15,370	16,200
	Evapotranspiration / Evaporation	2,580	2,420	2,510	2,690
	Diversions to Lake O'Neill	2,530	2,610	2,530	2,610
	Total Outflow:	66,160	39,040	67,860	40,570
Net change in GW and SW Storage:		2,160	1,220	1,700	640
Water Exchange within Model Domain					
	Net Infiltration from Recharge Ponds	10,940	10,590	10,940	10,590
	Net Stream Recharge to GW	2,780	4,150	4,800	5,250

The net stream recharge to ground water is higher in Alternative 10 compared with Alternative 3. The simulated average annual seepage from all reaches of the stream to the





Alternative 10 Simulated Stream Flow

FIGURE 8-12

ground-water aquifer is 10,320 AFY for Alternative 10, compared with 8,430 AFY under Alternative 3. During this same Model run, the simulated average annual water gaining to all reaches of the stream from the ground-water aquifer is 5,520 AFY for Alternative 10, compared with 5,650 AFY under Alternative 3. There are more gaining sections of the stream during summer months under Alternative 10 compared with the baseline, due to the reservoir release of water from July through November.

Evapotranspiration from vegetation averages 75 AFY less on an annual basis for all three sub-basins under Alternative 10 compared with Alternative 3. This reduced ET appears to occur in winter months when the vegetation is either dormant or less stressed. It may be necessary to curtail pumping during observed critical months, though simulated water levels do not indicate any prolonged low ground-water level conditions.

8.3.3 Expected Additional Yield for Alternative 10

The annual ground-water yield and maximum surface diversion expected from the construction of Alternative 10 facilities are listed below in Table 8-14. The maximum annual surface diversion required to provide a median annual ground-water yield of 16,200 AFY is 16,300 AF. The median annual ground-water yield available, after the Base's build-out demand is met, for the conjunctive use project would be 7,400 AFY. The location of the point of diversion for this project would be at the identical location of the existing point of diversion.

TABLE 8-14

Alternative 10 – Annual Ground-Water Yield and Maximum Surface Diversion

WATER RIGHT	ALTERNATIVE 3 (AFY)	ALTERNATIVE 10 (AFY)
Base's Build-out Demand	8,800	8,800
Minimum Additional Ground-Water Yield (AFY)	5,250	7,400
Total Annual Project Yield	**14,050**	**16,200**
Maximum Additional Surface Water Diversion (AFY)	16,300	16,300

9.0 PROJECT ECONOMICS

9.1 OVERVIEW

This chapter presents a description of the methods used and assumptions made in estimating the project costs for a joint Fallbrook PUD/Camp Pendleton conjunctive use project. Specifically, this chapter presents project cost estimates for the engineering facilities designed for Alternatives 9 and 10, Scenario 2. Detailed project cost estimates for scenarios 1 and 3 are presented in the Appendix. The cost estimates for Alternatives 1A and 3, developed as part of the Permit 15000 Feasibility Study, are incorporated into Alternatives 9 and 10, respectively. A summary of the project facilities associated with each alternative is presented in Table 9-1 below.

TABLE 9-1

Summary of Facilities associated with the Recycle and Reuse Alternatives

Item	Alternative 1A	Alternative 3	Alternative 9	Alternative 10
Ground-Water Wells	●	●	●	●
Obermeyer Diversion Dam		●		●
O'Neill Ditch Enlargements		●		●
Recharge Ponds 1-5 (Flow Structures)		●		●
New Recharge Pond Nos. 6 and 7		●		●
Wetland Pipeline			●	●
Treatment Wetland			●	●
Dam and Storage Reservoir			●	●
Reservoir Discharge Pipeline			●	●

The yield used in developing unit costs of water is based on the total project yield of each alternative. This methodology differs slightly from the Permit 15000 Study that used incremental increase in yield to calculate unit costs. A comparison of unit costs for Alternatives 1 through 4, using the new methodology, is included at the end of this chapter.

The total estimated capital cost of Alternative 10, Scenario 2 is $19.3 million, which yields an annualized unit cost of water of $140 per acre-foot, based on a yield of 16,200 AFY. A complete summary of unit cost per project water yield for Alternatives 1A, 3, 9, and 10 is presented in Table 9-2.

TABLE 9-2

Summary of Project Cost Estimates

Project Alternative & Scenario	Estimated Capital Cost (Million $)	Estimated Annual Cost[1] (Million $)	Annual Project Yield[2] (AF)	Unit Cost[3] ($/AF)
Alternative 1A	**2.0**	**0.2**	**11,850**	**15**
Alternative 3	**5.5**	**0.6**	**14,050**	**40**
Alternative 9	**15.8**	**1.8**	**14,000**	**129**
Alternative 10	**19.3**	**2.2**	**16,200**	**136**

1) Annual costs are based on capital costs amortized over 30 years at 8 percent interest plus power and labor to maintain and operate the facilities.
2) Project yield for Alternative 9 scenario 2 is base on Fallbrook Supplemental Feasibility Study. Project yield for Alternative 10 scenario 2 are base on Alternative 9 yields plus additional yields from Camp Pendleton's Recharge and Recovery Enhancement Program.
3) Unit Costs are based on annual project yields.

9.2 METHODOLOGY

The methodology used to develop cost estimates for the conjunctive use project facilities is based on annualizing the total capital cost of the project facilities, and dividing the annualized cost by the water yield produced by each project alternative. The annualized costs include estimates for capital facilities, contingencies, engineering, planning, design, and operation and maintenance costs. The methodology used to develop cost estimates for the recycle and reuse facilities is summarized below.

1. The total capital cost of each project alternative was estimated by summing the individual capital costs for each component of the alternative. Contingencies were estimated at 25% of the capital cost, with planning, engineering, and design estimated at 15% of the capital cost. An additional 10% was added to the capital cost to cover project management and administration.

2. The total estimated capital cost was then annualized over a 30-year period using an interest rate of 8.0%.

3. Annual operation and maintenance costs were calculated based on staffing requirements to operate and maintain the facilities, power requirements, and environmental sampling.

4. The annualized capital cost of each project alternative was added to the cost estimate for annual operations and maintenance, resulting in a total annual project cost.

5. The total annual project cost was divided by the total annual project water yield to achieve a unit project cost in dollars per acre-foot ($/AF). The unit project cost provides the basis for comparing alternatives.

The cost estimates prepared for this study reflect the level of accuracy that allows for an evaluation and comparison of proposed project alternatives. Verification of certain assumptions would be required to refine the cost estimates to the pre-design level.

9.3 CAPITAL COSTS

The description of capital costs focuses on the Alternative 10, Scenario 2 consumptive use project, including the recycle and reuse facilities. The total capital cost of the Alternative 10, Scenario 2 facilities is estimated to be $19.3 million, including $5.5 million in facilities associated with Alternative 3.

The project costs were estimated by applying unit costs to design quantities for most of the project facilities. The costs of some facilities were estimated as lump sum values. Unit cost estimates were based primarily on actual costs for similar projects completed or under construction, actual materials costs obtained from suppliers, and published unit cost data for heavy construction. As necessary, unit costs were indexed to 2001 dollars using *Engineering News Record* cost indices.

9.3.1 Wetland Pipeline

The estimated capital cost for the proposed 9,000 foot,12-inch diameter HDPE pipeline from the ocean outfall to the treatment wetland is $521,000. The cost for the pipeline is based on a unit cost of $45/foot, including the pipe, excavation, and backfill. Other costs associated with the wetland pipeline include isolation, flow control, air/vacum, and pressure reducing valves, flow meters, and two concrete outlet structures at the discharge end of the pipeline, located upstream of the treatment wetland. The pipeline, valve, and meter costs are based on budget level estimates provided by the manufacturer.

9.3.2 Treatment Wetland

The estimated capital cost of the 18-acre treatment wetland associated with Alternative 10, Scenario 2 is $268,000. The primary cost for the treatment wetland relates to excavation, grading, and berm construction. The cost for this earthwork is estimated at $200,000 based on a unit cost of $4.00 per cubic yard applied to 50,000 cubic yards of material to be excavated and graded. Other costs associated with the treatment wetland include construction of two concrete weir box outlet structures and associated outlet pipelines to convey water from the treatment wetland to the storage reservoir located downstream from the treatment wetlands. Planting the treatment wetland cells in cattails and bulrush is also included in the cost estimate for the treatment wetland.

9.3.3 Dam and Storage Reservoir

The estimated capital cost to construct the dam and storage reservoir is $6.8 million. The cost to construct the dam is based on the application of appropriate unit costs to the quantities and the appurtenant facilities associated with the dam design. The primary sources of information used in the cost estimate were bid abstracts and design drawings for previously constructed earthen dams in California, including the Fallbrook PUD's Red Mountain Dam and Reservoir. Table 9.3 presents the details of the cost estimate for constructing the dam and storage reservoir. Brief descriptions of the major components of the construction cost for the dam are provided in the following sections.

9.3.3.1 Excavation and Clearing

The total cost for all excavation and clearing activities is estimated at $715,000. Excavation and clearing costs account for approximately 10 percent of the total cost to construct the dam and reservoir. Excavation was divided into four categories: foundation (rock and common), key trench, outlet trench, and spillway excavation. Foundation excavation accounts for $330,000. In addition to excavation, the reservoir and dam site must be cleared of all plants and trees. It is estimated that 53 acres need to be cleared and grubbed at a total cost of $185,500.

9.3.3.2 Embankment and Blanketing

The cost to construct the dam embankment and place the gravel drain and slope protection materials is estimated at $2.6 million. The cost for constructing the dam embankment and placing the drain and slope protection materials accounts for approximately 40% of the total cost to construct the dam and reservoir. A unit cost of $4.00 per cubic yard was used for the excavation and placement of the dam embankment material. The unit cost assumes the dam embankment materials, including the impervious core material, would be available at local borrow sites within the area to be inundated by the reservoir, or within a one-mile radius of the dam site.

9.3.3.3 Major Structures

The total cost for major structures associated with the dam and storage reservoir was estimated at $2.7 million, roughly equivalent to the costs of the dam embankment. The costs for major structures include foundation treatment, grout curtain, intake/outlet works, spillway, drain recovery system, and other site drainage work. The cost of the intake/outlet works was estimated to be $1 million. This estimate is a lump sum cost conservatively based on 10% of the total cost of the dam and reservoir. The costs for site drainage work and drain recovery system were also estimated as lump sums. Site drainage is estimated at $500,000, while the drain recovery system is estimated to cost $100,000. The costs for all remaining structures were estimated using design quantities and unit costs. The foundation treatment and the gout curtain are estimated to cost $250,000 and $300,000, respectively. The spillway, including ogee crest, canal, and stilling basin, was estimated to cost approximately $600,000.

9.3.3.4 Appurtenant Costs

Additional costs for the dam and reservoir include the construction of roads and utilities to the dam site, bridge construction, dam instrumentation, erosion control, and fencing. The cost of the roads and utilities was estimated at $210,000, while the cost of the remaining items was estimated at $260,000.

Table 9-3 shows the complete details of the cost estimate for constructing the proposed dam and storage reservoir.

TABLE 9-3

Cost Estimate for Construction of the Dam and Storage Reservoir

Item	Quantity	Units	Unit Cost	Total Cost
Mobilization and Preparatory Work	1	lump sum		**$330,000**
EXCAVATION AND CLEARING				
Foundation Excavation				
- common	11,000	cubic yards	$3	**33,000**
- rock	33,000	cubic yards	9	**297,000**
Key Trench Excavation	12,500	cubic yards	9	**112,500**
Outlet Trench Excavation	1,700	cubic yards	15	**25,500**
Spillway Excavation	6,000	cubic yards	9	**54,000**
Stripping	2,500	cubic yards	3	**7,500**
Clearing & Grubbing	53	acres	3,500	**185,500**
Subtotal – Excavation and Clearing				**$715,000**
EMBANKMENT AND BLANKETING				
Zone 1 (Embankment Core)	143,400	cubic yards	4	**573,600**
Zone 2 (Embankment Fill)	144,000	cubic yards	4	**576,000**
Sand and Gravel Drain	12,000	cubic yards	45	**540,000**
Riprap	12,000	tons	70	**840,000**
Blanketing	2,800	cubic yards	35	**98,000**
Subtotal - Embankment and Blanketing				**$2,627,600**
MAJOR STRUCTURES				
Foundation				
- Foundation Treatment	16,700	square yards	6	**100,200**
- Dental Concrete	1,900	cubic yards	80	**152,000**
Grout Curtain				
- Drill Setups for Grout Holes	90	setups	50	**4,500**
- Drilling Grout Holes	4,500	lineal feet	30	**135,000**
- Pressure Grouting Foundation	1,100	sacks	30	**33,000**
- Misc. work associated with grouting	1	lump sum		**86,000**
Intake/Outlet Works	1	lump sum		**1,000,000**
Spillway				
- Backfill	7,200	cubic yards	20	**144,000**
- Concrete: Canal, Ogee, and Stilling Basin	900	cubic yards	500	**450,000**
Misc. Drainage Work	1	lump sum		**500,000**
Drain Recovery System	1	lump sum		**100,000**

Subtotal - Major Structures				**$2,704,700**
ROAD AND UTILITY CONSTRUCTION				
Road Construction	1.0	miles	150,000	**150,000**
Utility Construction	1.0	miles	60,000	**60,000**
Subtotal - Road and Utility Construction				**$210,000**
MISCELLANEOUS				
Concrete Bridge Deck and Misc. Metal				**100,000**
Dam Instrumentation				**80,000**
Erosion Control				**60,000**
Fencing				**20,000**
Subtotal – Miscellaneous				**$260,000**
TOTAL COST				**$6,800,000**

9.3.4 Reservoir Discharge Pipeline

The estimated capital cost for the proposed 5,800 foot, 20-inch HDPE pipeline from the storage reservoir to the Santa Margarita River is $628,000. The cost of the pipeline was estimated at $406,000 based on a unit cost of $70/foot, including the pipe, excavation, and backfill. Some clearing and grubbing costs will be associated with the construction of the pipeline due to the proposed alignment. The valve and meter costs are higher for this pipeline, compared to the wetland pipeline, due its larger pipe diameter. Other costs associated with the reservoir discharge pipeline include isolation, flow control, air/vacum, and pressure reducing valves, flow meters, and a single concrete outlet structure at the discharge to the Santa Margarita River. The valve and meter costs include delivery and installation concrete vaults. The pipeline, valve, and meter costs are based on budget level estimates provided by the manufacturers. The outlet structures costs are based on quantity and unit cost estimates for concrete.

9.4 OPERATION AND MAINTENANCE

The costs for operation and maintenance were estimated for both the Diversion and Recharge component and the Recycle and Reuse component. The operation and maintenance cost for both of these components reflect the manpower, testing, and ongoing maintenance for each of the facilities described in both components.

The Alternative 9, Scenario 2 annual O&M cost for all facilities was estimated at $400,000. The Alternative10, Scenario 2 annual O&M cost for all facilities was estimated at $480,000. The O&M costs associated with the remaining capital projects includes salaries for two full time personnel, annual wetland repair and maintenance, and water quality testing.

9.5 ALTERNATIVE 10 CONJUNCTIVE USE PROJECT COSTS

Alternative 10 conjunctive use project costs are calculated based on the total cost of the recycle and reuse facilities and the Alternative 3 (Obermeyer diversion dam, O'Neill ditch enlargements, recharge ponds Nos. 1-5 flow structures, construction of new recharge ponds Nos. 6 and 7, and six ground-water wells) facilities. The yield of the project that was applied to deter-

TABLE 9-4

Summary of Alternative 10, Scenario 2 Project Costs

Item	**Cost**
Recycle and Reuse Facilities	
Wetland Pipeline	$521,000
Treatment Wetland	268,000
Dam & Storage Reservoir	6,800,000
Reservoir Discharge Pipeline	628,000
Alternative 3 Facilities	
Obermeyer Diversion Dam	621,000
O'Neill Ditch Enlargements	108,000
Recharge Pond Nos. 1-5 (flow structures)	200,000
New Recharge Pond Nos. 6 and 7	673,000
Subtotal (all items above)	**$9,819,000**
Contingencies and Unlisted Items @ 25%	2,455,000
Subtotal	**$12,274,000**
Planning, Engineering, and Design @ 15%	1,841,000
Project Management and Administration @ 10%	1,227,000
Alternative 3 Ground-Water Wells	3,000,000
Additional Alternative 10 Ground-Water Wells	1,000,000
Total Estimated Capital Cost	**$19,342,000**
Amortized Capital Cost [1.]	1,718,000
Annual O&M Cost	480,300
Total Estimated Annual Cost	**$2,198,000**
Unit Cost per acre-foot [2.]	**$136**

1. *Capital costs amortized over 30 years at 8 percent interest.*
2. *Unit cost based on project yield of (AFY):* 16,200

mine a unit cost is based on Alternative 10 median project yield (16,200 AFY). The estimated capital and unit costs of Alternative 10 are shown in Table 9-4 above.

9.6 PROJECT COST COMPARISON OF ALTERNATIVES

The two alternatives were compared on the basis of the total estimated annual cost, unit cost per acre-foot of water, and project yield. Project yields for Alternatives 9 and 10 are based on the total ground-water yield of each project. Table 9-5, below, presents a summary of the total estimated capital cost, annual cost, unit cost and annual project yield for all three scenarios of Alternative 9 and 10.

TABLE 9-5

Summary of Project Cost Estimates

Project Alternative & Scenario	Estimated Capital Cost (Million $)	Estimated Annual Cost[1] (Million $)	Annual Project Yield[2] (AF)	Unit Cost[3] ($/AF)
Alternative 9				
Scenario 1	15.1	1.7	13,100	133
Scenario 2	15.8	1.8	14,000	139
Scenario 3	17.7	2.0	14,600	135
Alternative 10				
Scenario 1	18.6	2.1	15,400	139
Scenario 2	19.3	2.2	16,200	136
Scenario 3	21.2	2.4	16,900	136

1) *Annual costs are based on capital costs amortized over 30 years at 8 percent interest plus power and labor to maintain and operate the facilities.*

2 *Project yields for Alternative 9 scenarios are base on Fallbrook Supplemental Feasibility Study. Project yields for Alternative 10 scenarios are base on Alternative 9 yields plus additional yields from Camp Pendleton's Recharge and Recovery Enhancement Program.*

3) *Unit Costs are based on annual project yields.*

The total estimated capital cost of Alternative 9, Scenario 2 was $15.8 million, including the cost of Alternative 1A ground-water wells. The total estimated capital costs of Alternative

10, Scenario 2 was $19.3 million, including Alternative 3 facilities. The estimated yield of Scenario 2 (2,500 acre-foot release) was 14,000 acre-feet for Alternative 9 and 16,200 acre-feet for Alternative 10. The unit costs per water yield for Scenario 2 were $129/AF and $136/AF for Alternative 9 and 10, respectively.

As these tables indicate, the capital costs and annual costs increase as the project yield increases, while the unit cost of the project water decreases. The tables also show that Alternative 10, Scenario 2 provides a 45% increase in project yield over Alternative 9, Scenario 2 while decreasing the unit cost by approximately 5%. When the same comparison was applied to capital and annual costs, it was found that approximately a 42% increase in capital cost and annual cost is required to gain the additional 45% project yield.

9.7 OPPORTUNITIES FOR FUNDING

The following summary of potential funding sources is provided in order to introduce those sources that are likely to supply some funding to this proposed water recycling and reuse project. This is not intended to represent a complete list of potential sources. Should this project proceed to the next phase, it is likely that such sources will be examined in a more exhaustive manner after a more detailed design and cost analysis of project facilities has been completed. This summary does indicate that there is a favorable market for funding projects such as this.

U.S. Bureau of Reclamation

- ***Title XVI – Reclamation Wastewater and Groundwater Study and Facilities Act –*** Provides Federal funding of up to 25% of the capital cost of water recycling projects.

State of California

- ***State Revolving Fund & Water Reclamation Loan Program –*** Provides low interest loans for design and construction of water recycling and ground-water development projects. The interest rate is defined as one half the State's general obligation bond interest rate.
- ***Proposition 13 – Safe Drinking Water, Clean Water, Watershed Protection, and Flood Protection Act (Water Bond 2000) -*** Authorizes the State to sell general obligation bonds to fund various studies, programs, and projects related to improved management of California water Resources.
    - Construction loans up to $5 million for projects under the Ground-Water Recharge Program.
    - Feasibility Study and Pilot Study grants of up to $500,000 for Ground-Water Storage Programs.
    - Ground-Water Storage construction grants of up to $50 million per project.

- ***Proposition 82 – Local Water Supply Project Feasibility Studies and Construction Loans –*** Provides feasibility study loans of up to $500,000 per project and construction loans of up to $5 million per project.
- ***Local Groundwater Assistance Fund Grant Program –*** Provides grants of up to $250,000 for ground-water studies or implementation of monitoring and management programs.

San Diego County Water Authority

- ***Financial Assistance Program –*** Provides 50:50 cost sharing for planning, feasibility, and research studies in water recycling and ground-water development projects with a cap of $150,000.
- ***Reclaimed Water Development Fund –*** Provides an incentive of up to $100/acre-ft, for up to 25 years, for development of water recycling projects that reduce the demand on the SDCWA .

Metropolitan Water District

- ***Local Resources Program –*** Provides an incentive of up to $250/acre-ft for up to 25 years for recycled water and ground-water projects that reduce the demand for imported water.

Other Agencies currently supporting Water Recycling Projects

- U.S. Environmental Protection Agency
- WateReuse Association of California
- Regional Water Quality Control Board
- California Department of Health Services
- California Department of Water Resources
- CALFED

10.0 CONCLUSIONS AND RECOMMENDATIONS

10.1 CONCLUSIONS

The feasibility of developing a conjunctive use project between the Fallbrook PUD and the Base is predicated on the implementation of two components, Diversion and Recharge and Recycle and Reuse. The Diversion and Recharge component was developed in the Permit 15000 study and includes the diversion of naturally occurring streamflow from the Santa Margarita River for recharge to the aquifers in the lower ground-water basin. The Recycle and Reuse component developed in detail throughout this study includes the development of an alternative source of water supply from the Fallbrook PUD. The anticipated yield of each component has been quantified by applying hydrologic and hydrogeologic modeling techniques. Engineering design and economic analysis has further provided the unit cost of delivering these waters for use in a conjunctive use project between the two parties. Along with project yields and their related costs, elements of the project such as emergency supply and development of local ground-water basins that have not been economically quantified are discussed in greater detail below.

The Diversion and Recharge component of this project includes the construction of new facilities in the Upper Ysidora subbasin on Camp Pendleton. Specifically, these projects include the construction of a new diversion structure and recharge ponds, enhancement to the existing canal capacity, and installation of new ground-water wells. The anticipated median yield of this component is 14,050 AFY with associated capital costs of $5.5 million. The yield following a second below normal hydrologic yield drops 2,850 AFY to 11,200 AFY, and an additional 2,850 AFY to 8,350 AFY following a third below normal hydrologic year. The only method to curtail the mandatory reduction in ground-water production during dry cycles is to develop an alternative source of supply.

The Recycle and Reuse component of the conjunctive use project develops an alternative source of water supply for beneficial use by both parties. This study has found that tertiary treated wastewater from the Fallbrook PUD can be beneficially used as an alternative source of water supply for the lower Santa Margarita basin. The facilities included in this component include the construction of a treatment wetland and storage reservoir, a delivery pipeline from the Fallbrook PUD's ocean outfall to the wetlands, and a pipeline from the reservoir to the Santa Margarita River. The yield of this component of the project ranges from 1,280 AFY to 2,800 AFY depending on the size of the reservoir that is constructed. Combined with the Diversion and Recharge component, the cost of developing an additional 2,150 AFY, providing a median project yield of 16,200 AFY, is estimated to be $19.3 million. Although similar reductions in ground-water pumping will be required during dry cycles, the annual ground-water yield will be 10,500 AFY following the third below normal hydrologic year, The cost of the facilities required to develop 16,200 AFY is not credited with the value of developing an emergency water supply and ocal ground-water supply for the Fallbrook PUD. The development of an emergency water supply provides insulation from future reductions of imported water supplies originating from the MWD. During periods of extended drought and forced cutbacks from the MWD, the Fallbrook PUD will be able to call upon the local ground-water supplies on the Base. Due to the development of the Recycle and Reuse component of the project, these ground-water supplies will be available even during years of extend drought in the Santa Margarita watershed. Because it is difficult to quantify the monetary value of the availability of an emergency supply, the Fallbrook PUD and its customers will need to determine if this component of the project is a requirement to meet their future goals.

The implementation of a conjunctive use project between the Fallbrook PUD and the Base benefits all water users on a regional level. Water supplied from a local ground-water basin reduces the demand on both the State Water Project and Colorado River imported supplies. The reduction in demand increases the efficiency of each of these systems, providing additional capacity that can be used for environmental or other needs. The state and federal governments, as well as MWD and the CWA, provide programs and fiscal incentives for water agencies to reduce their dependence on imported supplies to help meet future demands. The Recycle and Reuse component and the development of local ground-water basin inherent to the conjunctive use program allows Fallbrook to participate in these programs.

The development of a conjunctive use project also provides a physical solution to the ongoing legal dispute *U.S. v Fallbrook*. As agreed to in the 1968 MOU, the conjunctive use project provides a means to reach a physical solution to the division of waters of the Santa Margarita River. Failure of the two parties to reach a joint physical solution may result in costly litigation or an inferior project that may not provide either the Fallbrook PUD or the Base with a viable long-term solution to future water demands. While the Fallbrook PUD benefits from the development of a local ground-water supply, the Base will benefit from the direct connection to imported water supply. In both cases, the project will provide a means for both parties to ensure that future water demands, even during periods of extend drought, will be met by the development of the conjunctive use project.

10.2 RECOMMENDATIONS

The results of the Fallbrook PUD Supplemental Study show that it is possible to develop a conjunctive use program that contains a Diversion and Recharge and a Recycle and Reuse component. Although both Alternatives 9 and 10 were presented in this study, the latter alternative provides a more reasonable and viable option to be implemented as part of a conjunctive use program. Alternative 10 may also be addressed as a viable conjunctive use project without the Recycle and Reuse component, identical to Alternative 3, with the exception that it will not provide an alternative supply of water that proves crucial to a successful project during the dry summer months and periods of extend drought. The following recommendations have been provided to successfully implement a conjunctive use project between the Fallbrook PUD and the Base.

1) Adopt a conjunctive use policy that will utilize the safe yield of the ground-water basin on Camp Pendleton. The policy should specifically address, but not be limited to, the use of recycled water, surface and ground-water management of the Santa Margarita River, development of emergency supplies for the Fallbrook PUD, and improvement to potable and basin water quality.

2) Proceed with the NEPA/CEQA environmental analysis to determine the environmental feasibility of each alternative.

3) Investigate third party sources of tertiary treated wastewater. Scenario 3, which includes the 3,500 AFY reservoir scenario, indicates that the unit cost of developing the project is identical to the Scenario 2 unit cost. Additional yield from the ground-water basin could be realized from a larger Recycle and Reuse component.

4) Complete feasibility level design work for the conveyance pipeline from the Base to the Fallbrook PUD. Adjust capacity to account for the F3 pumping schedule. Investigate the best alignment for a dual purpose, multi-directional, pipeline to meet both the Fallbrook PUD's and the Bases future needs.

5) Continue to use the Model as a predictive, investigative, and design tool to study potential hydrogeologic and environmental impacts prior to management decisions. It is recommended that the Model be updated with future field data, thereby continually improving its reliability.

6) Expand the ground-water flow model with particle tracking and contaminant transport models to study issues specific to each sub-basin:

7) Improve the model with field data measurements of gaining and loosing stream reaches, and streambed conductance. This would help to better define the relationship between surface and ground water.

8) Develop a complete and up-to-date cross-division/cross-department ground-water management and monitoring plan. This could potentially reduce detrimental impacts of contaminated sites on drinking water wells, potential salt water intrusion, reduce

unnecessary or duplicate sampling and monitoring, and streamline the planning and development process.

9) Investigate the availability, and/or construction, of a brine line that could be used to discharge reject water from the proposed advanced treatment facilities to be located on the Base.

REFERENCES

Bicknell, et. al. "Hydrological simulation Program—FORTRAN User's Manual for Release 10." USEPA. 1993.

Bianchi, W. C., and Muckel, Dean C. "Ground-Water Recharge Hydrology." Agricultural Research Service. United States Department of Agriculture. 1970.

Black and Veatch. "Supplement to the Project Report for the Groundwater Replenishment System." November 1998.

Bouwer, Herman "Systems for Artificial Recharge of Ground Water." Paper presented at the International Symposium on Artificial Recharge of Groundwater. 1988.

Bouwer, Herman, and Back, Jennifer T., and Oliver, James M.. "Predicting Infiltration Rates and Groundwater Mounds for Artificial Recharge." 1998.

Bureau of Reclamation "Design of Small Dams." U.S. Department of the Interior. 1987. page 370.

California Department of Health Services. "Draft Regulations 4-23-01." April 2001. (www.dhs.cahwnet.gov)

California Department of Health Services. "California Health Laws Related to Recycled Water 'The Purple Book' ." June 2001. (www.dhs.cahwnet.gov)

California Department of Health Services. "California Safe Drinking Water Act and Related Laws 'The Blue Book' 7th ed.." January 2001. (www.dhs.cahwnet.gov)

California Regional Water Quality Control Board, San Diego Region. "Water Quality Control Plan for the San Diego Basin (9)." September 1994.

California Regional Water Quality Control Board, San Diego Region. "Order No. 2000-12, NPDES No. CA0108031, Waste Discharge Requirements for the Fallbrook Public Utility District Wastewater Treatment Plant No. 1." February 2000.

California Regional Water Quality Control Board, San Diego Region. "Addendum No. 1 to Order No.91-39." August 1995

California Regional Water Quality Control Board, San Diego Region. “Order No. 91-39, Waste Discharge Requirements for the Fallbrook Sanitary District Plants Nos. 1 and 2, Reclamation Projects, San Diego County.” May 1991..

Camp Pendleton Office of Water Resources, Public Works Survey Department Drawing #10601. 1978.

Camp Pendleton Office of Water Resources, Historical Diversions to Lake O’Neill and Recharge Ponds.

Carlson, Larry. Personal communication regarding waste water oxidation ponds. Camp Pendleton Office of Water Resources. September 2000.

Chow et. al., “Applied Hydrology”, McGraw-Hill International Editions, Civil Engineering Series. 1988. pp 147-152.

Crippen, John R., “Naturatl Water Loss and Recoverable Water in Mountain Basins of Southern California.” Geological Survey Professional Paper 417-E. USGS. 1965.

Desert Research Institute, Hourly and Daily Precipitation Data at Oceanside, CA.

The Environmental Company (TEC), "Draft, Surface Water and Groundwater Modeling Study, Lower Santa Margarita River Basin, Marine Corps Base Camp Pendleton". September 2000.

Fallbrook Public Utility District, 1994, “Conjunctive Use Study – Lower Santa Margarita River Basin.” June 1994.

Groundwater Replenishment System, Orange Count Water District (www.gwrsystem.com)

Hayes, P.D.,Agajanian, J., and Rockwell, G.L., “Water Resources Data – California, Water Year 1997, Volume 1, Southern Great Basin from Mexican Borde to Mono Lake Basin, and Pacific Slope Basins from Tijuana River to Santa Maria River.” U. S. Geological Survey, Water Resources Division, California District. 1997.

Holland, R.F. “Preliminary descriptions of the terrestrial natural communities of California.” Unpublished report for Nongame-Heritage Program, California Department of Fish and Game. Sacramento, CA. 1986.

International Technology Corporation, "Draft Final, Remedial Investigation and Feasibility Study for Operable Unit 2, Site 8 and 22/23 Area Sites," Volume 1, Appendix C: Groundwater Fate and Transport Model of 22/23 Area Sites in the Chappo Subbasin. September 1996.

Irvine Ranch Water District (www.irwd.com)

Johnson, A.I. "A Field Method for Measurement of Infiltration." U.S. Geological Survey Water Supply Paper 1544-F. 1963.

Los Angeles Department of Water and Power (www.ladwp.com)

Law/Crandall, Inc. "Groundwater Modeling for the Lower Santa Margarita Groundwater Basin." Prepared for the Department of the Navy Southwest Division, Naval Facilities Engineering Command, Contract No. N68711-92-D-4570. November 1995.

Leedshill-Herkenhoff, Inc. "Basewide Water Requirement/Availability Study, Marine Corps Base Camp Pendleton." Draft July 1987.

Leedshill-Herkenhoff, Inc. "Basewide Water Requirement/Availability Study, Marine Corps Base Camp Pendleton." September 1988.

McDonald, Michael G., and Harbaugh, Arlen W., "A Modular Three Dimensional Finite-Difference Ground-Water Flow Model." U.S. Department of the Interior and USGS. 1984.

McMahon, Leonard, "Heavy Construction Cost Guide." Annual Edition No. 7. Geraldine C. McMahon. 1995.

Malloy, Mike. Personal communication. Camp Pendleton Office of Water Resources. September 2000.

MCBCP, GIS coverages received from Facilities and Maintenance Department. August 1999.

MCB-CP, GIS coverages received from Environmental / Securities Department. July 2000.

Means, R. S., "Heavy Construction Cost Data." 14th Annual Edition. RSMeans Company, Inc. 2000.

NBS Lowery. "Conjunctive Use Study, Lower Santa Margarita River Basin." Volume I. June 1994.

Orange County Regional Water Reclamation Project. "Final Feasibility Study Report." March 1995.

Orange County Water District (www.ocwd.com)

Pettyjohn, Wayne A. "Introduction to Artificial Recharge." Robert S. Kerr Environmental Research Laboratory. Office of Research and Development. U.S. Environmental Protection Agency. 1981.

P & D Consultants, Inc. "Draft Program EIR/TIER 1EIS for the Groundwater Replenishment System." November 1998.

PRC Engineering, Inc. "Groundwater Basin Hydrologic and Water Quality Study for Marine Corps Base Camp Pendleton, California." October 1983.

San Diego County Water Authority (www.sdcwa.org)

Sawyer, J.O. and T. Keeler-Wolfe. "Manual of California Vegetation." California Native Plant Society. Sacramento, CA. 1995.

Stetson Engineers Inc., "Santa Margarita River Recharge and Recovery Enhancement Program – Permit 15000 Feasibility Study for Marine Corps Base Camp Pendleton" U. S. Bureau of Reclamation, March 23, 2001.

Stetson Engineers Inc., "Addendum to Draft Technical Memorandum Dated April 11, 1995." April 25, 1995.

Schiff, Leonard. "Water Spreading and Ground-Water Recharge." Proceedings Conference on Water Spreading for Groundwater Recharge. Davis, California. March 19, 1957.

Schiff, Leonard, and Dyer, Kenneth L. "Some Physical and Chemical Considerations in Artificial Recharge." Extract of Publication No. 64 of the I.A.S.H. Commission of Subterranean Waters, pp., 347-358.

Schmidt, Curtis J., and Clements, Ernest V. III. “Reuse of Municipal Wastewater for Groundwater Recharge.” Municipal Environmental Research Laboratory, Office of Research and Development, U.S. Environmental Protection Agency. September 1977.

Shlemon, Roy, J., “Late Quaternary Evolution of the Camp Pendleton-San Ononfre State Beach Coastal Area, Northwestern San Diego, CA.” January 1978.

Soil Conservation Service, “Section 4 Hydrology.” National Engineering Handbook. March 1985.

Troxell, H. C. and Walter Hofmann. "Surface-Water Resources," in "Geology and Ground-Water Resources of Camp Pendleton, California" by Worts and Boss. July 1954.

U.S. Bureau of Reclamation, Southern California Office (www.lc.usbr.gov/~scao)

Water Reclamation District. (www.wrd.org)

WateReuse Association, Potable Reuse Committee. “Use of Recycled Water to Augment Potable Supplies: An Economic Perspective.” September 1999.

WateReuse Association (www.watereuse.org)

Worts, G. F., and R. F. Boss. "Geology and Ground-Water Resources of Camp Pendleton, California." July 1954.

Zedler P., S. DeSimone, J. Giessow, D. Lawson, J. Else, and S. Bliss. “The Plant Communities of Camp Pendleton Marine Corp Base, California.” San Diego State University,Department of Biology, San Diego, CA 1997.