4.0 GROUND-WATER MODEL

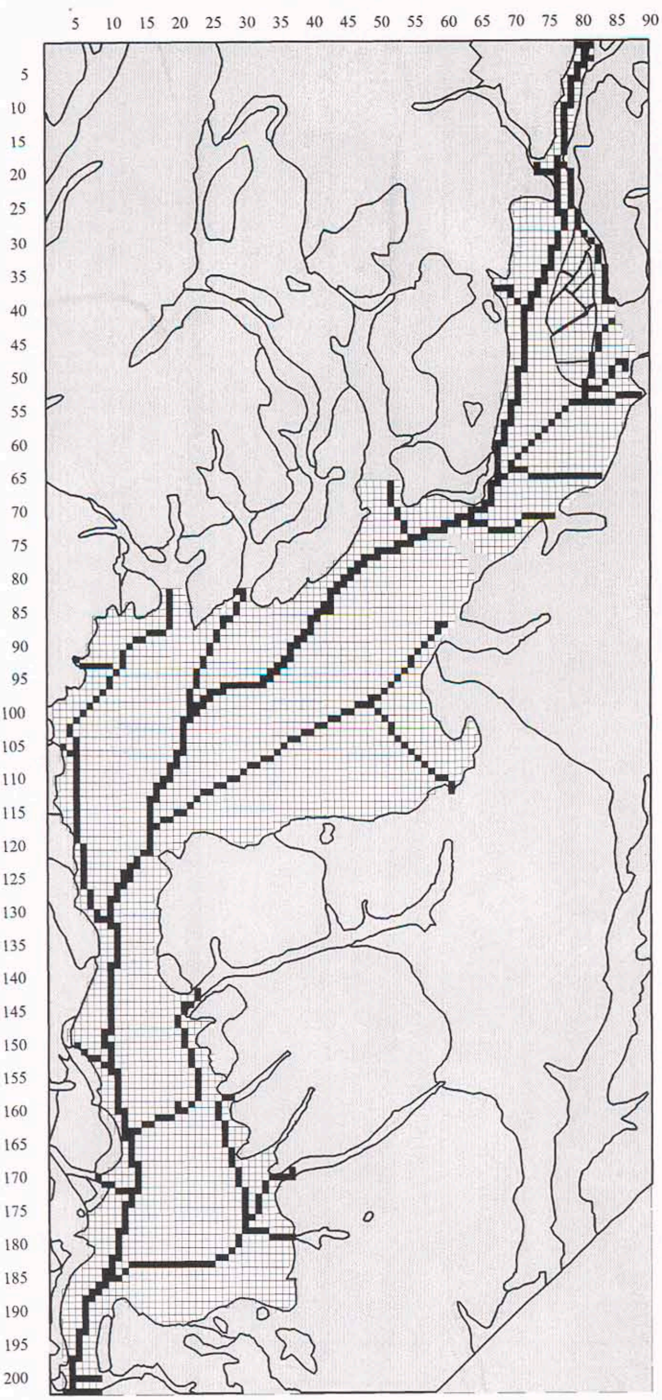
4.1 GENERAL

A ground-water flow model (Model) was developed to simulate the impacts to the ground-water basin due to historical hydrology and water management practices that affect the hydrologic condition of the Upper Ysidora, Chappo, and Lower Ysidora sub-basins. The Model also provides the necessary tool to measure the changes in ground-water conditions and the potential affect to riparian vegetation and streamflow in the study area, as various stresses are applied in relationship with development of Permit 15000. Changes in ground-water pumping, streamflow, diversions, and wastewater production are simulated so that each of these stresses can be reviewed to estimate their potential impact to the condition and health of the Santa Margarita River and the sub-basins. The impacts of these stresses were measured as changes in the overall water budget, changes in ground-water levels, and changes in evapotranspiration (ET) demands.

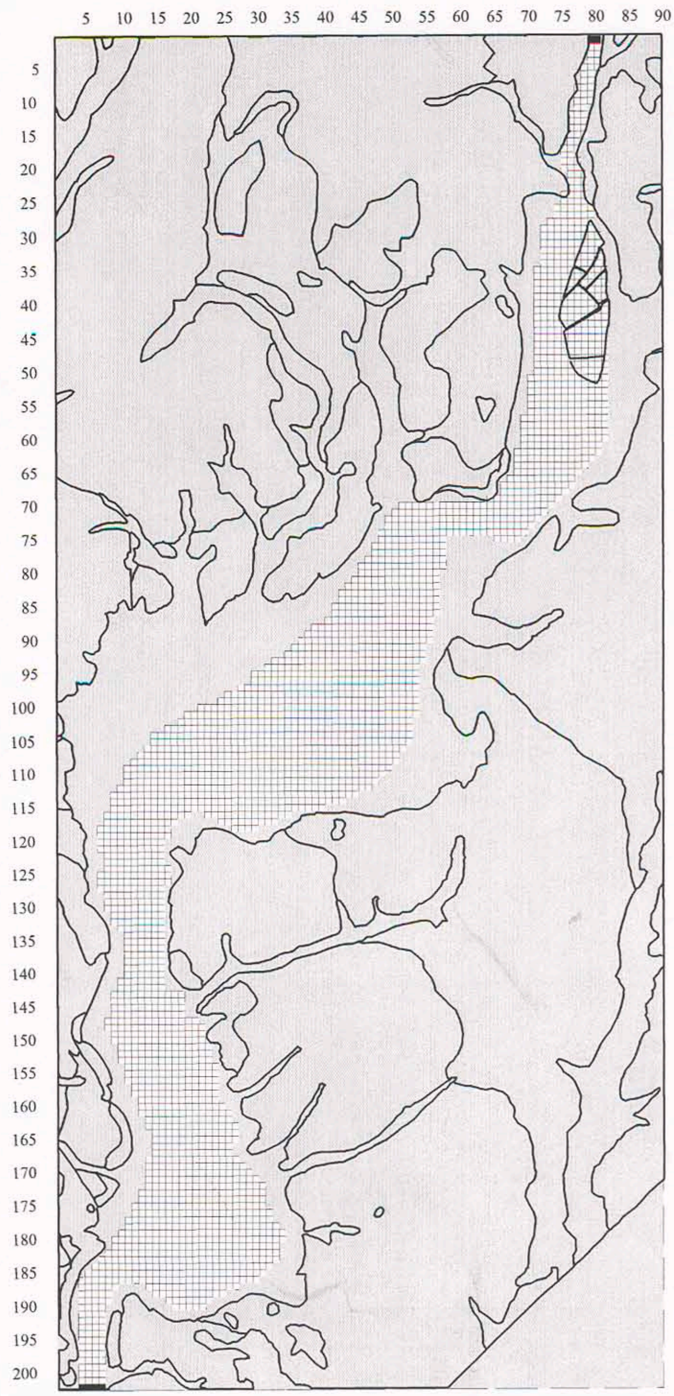
The Model described in this report is used in Chapter 7 to estimate the impact of each of four different project alternatives that could be constructed to perfect Permit 15000 and expand the Base's diversion of water from the Santa Margarita River. Equally important, the Model described in this report may also be used in the future as a management tool to determine the best location for ground-water pumping, effects of adding or removing sources of water from the basin, and use in negotiations with local, state and federal regulators. A particle tracking or contaminant transport package may also be added to the Model to estimate the impacts of pumping and hydrologic conditions on the transport and movement of organic and inorganic compounds in each of the three sub-basins. The Model is the compilation of all-environmental, wastewater, and water supply data on the Base and should be managed and maintained into the future in order to maximize water supply and minimize impact to the environment.

The Model consists of 2 layers, 202 rows, 90 columns, and 7,390 active cells (Figure 4-1). A 20-year calibration period from water year (WY) 1980 through 1999 was established to simulate extended wet and dry periods. Monthly stress periods were simulated to capture the seasonal variations observed in the existing water level and stream gage data. The Santa Margarita River was simulated to have the flexibility to be a gaining, losing, or dry stream at different stream reaches or with different seasonal variations.

A surface water model and reservoir operations model was also developed to estimate the surface flow at the Model's boundary and the potential water available for diversions. In addition to these flows, the surface water model also estimated tributary inflow to the Model area from smaller streams located below the confluence of the Santa Margarita River and De Luz



Ground-Water Model
Layer 1



Ground-Water Model
Layer 2

STETSON
ENGINEERS INC.

- ■ STREAM CELL
- □ NO FLOW CELL
- □ ACTIVE CELL
- ~ OUTLINE OF GEOLOGIC UNITS
- ■ GENERAL HEAD BOUNDARY CELL
- # FINITE DIFFERENCE GRID (200 ft. x 200ft.)

GROUND- WATER MODEL
BOUNDARY CONDITIONS AND
ACTIVE CELLS

Creek. All estimates of streamflow, available water for diversion, and tributary inflow were calculated on a daily basis using hourly precipitation available from the Oceanside gaging station.

4.2 PREVIOUS STUDIES

Two previous modeling studies were considered for compilation of the Model used to address concerns for Permit 15000's impact to ground water. The original base data for the Chappo and Upper Ysidora ground-water model were constructed from LAW/Crandall's work for the Department of the Navy, Southwest Division (1995). A ground-water model was later developed by IT Corporation to simulate the movement of volatile organic compounds (VOC) in the Chappo sub-basin (IT Corporation, 1996). In September 2000, Stetson Engineers extended the boundary of the original LAW/Crandall ground-water model to include the Lower Ysidora sub-basin and all contributions made by wastewater discharge to the Lower Santa Margarita River Basin (The Environmental Company, 2000).

Both LAW/Crandall and IT Corporation conducted aquifer-pumping tests to obtain hydraulic properties of the sub-basins, which were summarized in their reports and used to develop their respective models. IT Corporation's contaminant modeling work was used to verify hydrogeologic conditions within the Chappo sub-basin and placement of proposed production wells.

The ground-water model constructed for Camp Pendleton by Law/Crandall, Inc. (1995) was used to evaluate the potential effect of production wells on contaminant migration within the Chappo sub-basin. A MODFLOW™ flow model was coupled with MODPATH™, a particle-tracking model, to simulate flow within the drinking water supply basins. The MODFLOW™ river package was used to simulate recharge from the river to the ground-water aquifer, and the river was simulated as a losing stream throughout the model domain. The model was based on annual time-steps and assumed a continuous, steady source of water in the river. Hydraulic properties obtained from aquifer pumping tests were used in the model and summarized in their report. Their study was based upon average monthly pumping at the Upper Ysidora and Chappo production wells, and considered the effects of four proposed production wells. LAW/Crandall's study concluded that construction of a new well in the Lower Chappo might increase the potential for contaminants to be drawn into existing wells, and proposed three new production wells to be located in the Upper Ysidora.

A ground-water flow and contaminant transport model was used to study migration of VOC (volatile organic compounds) impacted ground water in the Chappo sub-basin as part of the draft Remedial Investigation and Feasibility Study for Operable Unit 2 (IT Corporation, 1996). The model was constructed to evaluate different remedial alternatives with respect to the VOCs located in the 22/23 Area of Camp Pendleton. The options included no action, pump and treat, and pumping/injecting scenarios. Given the highly porous media of the Chappo and the effects of dilution and dispersion, it was estimated that the impacted ground water would return to background conditions by natural attenuation within 10 years, and therefore no further action was recommended.

The two models described in this section represent the numerical ground-water modeling efforts previously performed on the Lower Santa Margarita Basin. In addition to these numerical models, development of analytical and spreadsheet models that account for the interaction between surface and ground water have been conducted by The Environmental Company (September 2000), Fallbrook Public Utility District (Fallbrook PUD, 1994) and Camp Pendleton (Leedshill, 1988).

4.3 GROUND-WATER MODEL CONSTRUCTION

The selected numerical model, MODFLOW™ (McDonald and Harbaugh, 1988) is a three-dimensional ground-water flow model developed by the USGS. MODFLOW™ uses mathematical expressions to represent the ground-water flow system, including boundary conditions, hydrogeologic attributes of the aquifer, and simplifying assumptions to capture the heterogeneities of the subsurface.

The model area extends from the bedrock narrows just north of the naval hospital to the narrows just south of the Lower Ysidora. The Model was constructed with two layers representing the two Quaternary alluvial units described in Chapter 3. The upper layer was assigned properties of an unconfined layer to capture the water table aquifer characteristics of the upper alluvium. The bottom layer of the Model was assigned an aquifer type of an unconfined unit with variable transmissivity, allowing for variability in the saturated thickness of the lower alluvium. Two layers were chosen to represent the alluvial aquifer in all three sub-basins. Well logs and cross sections of the Lower Santa Margarita River ground-water basin Worts and Boss, 1954; Shleman, 1978) show a coarser (cobbles, gravel and sand) lower alluvium beneath a finer (gravel, sand, silt, and clay) upper alluvium. Though the ground-water basin is considered to be one aquifer, the two layers fallow for the simulation of variable materials. Each layer is discretized into rows and columns with 200-foot by 200-foot spacing. There are 202 rows and 90 columns.

The top of the Model was assigned elevations based on the Army Corps of Engineers 5-foot interval topographical survey (MCB-CP, 1999). Well logs and geologic cross sections were used to determine the elevations of the interface of the upper and lower alluvium and the depth to bedrock (Worts and Boss, 1954). There is a general downward slope of the interface between the two layers from the northeast edge (south of the De Luz confluence) of the model domain toward the southwest edge (Lower Ysidora narrows). The finite-difference grid was constructed to account for the changes in elevations and downward slope of the surface and contacts from north to south.

The steady-state Model was constructed with monthly stress periods. During each stress period, streamflow, recharge, evapotranspiration, pumping rates, etc. remained constant. Average values for each month were used as input into the Model for each of these parameters, such that the Model simulates average constant conditions throughout each month. The average monthly values accounted for variation in the seasonal natural system with the highest stream flows and precipitation during the winter season and a dry climate during the summer and autumn.

4.4 GROUND-WATER FLOW MODEL PROPERTIES

The ground-water flow model parameters were developed based on the conceptual model described above. A numerical model inherently requires simplifying assumptions when defining a problem domain. Each volume element (a block defined by a row, a column, and a layer in the grid) is assigned a unique set of hydraulic parameters influencing the calculations depicting flow of ground water at the center of that particular block. Hydraulic properties incorporated into the Model include hydraulic conductivity (horizontal and vertical), effective porosity, specific yield, storativity, recharge, and evapotranspiration. Aquifer transmissivity was obtained by multiplying hydraulic conductivity by the thickness of the layer at that grid block.

Aquifer hydraulic characteristics were assigned based on aquifer pumping tests conducted by IT Corporation and previous model results (LAW/Crandall, 1995). Horizontal conductivities ranged from 0.8 and 37 ft/day in the silts and silty sands of the Chappo and Lower Ysidora sub-basins to approximately 300 to 490 ft/day in the gravels and sands of the lower alluvium in the Chappo and Upper Ysidora (LAW, 1995). Specific yield ranged from 0.05 in silts to 0.2 in sands and gravels (LAW, 1995). Storativity was estimated at 0.00002 to 0.00005 depending on soil type. Effective porosity was assigned values from 0.22 for sand and gravel units to 0.40 for silt/clay units.

Recoverable water by runoff and infiltration from rainfall was considered to be approximately 17% of measured precipitation (Crippen, 1965) typical of a Southern California coastal climate. This recoverable water was assigned to the upper model layer as recharge and side tributary runoff. The median annual precipitation from water years 1980 through 1999 was 12.0 in/yr, ranging from 3.6 in/yr in WY 1987 to 25.9 in/yr in WY 1980.

It was estimated that 10% of water stored in Oxidation Ponds 3, 8, and 13 (minus evaporation, plus rainfall) was recharged into the ground-water aquifer and included in the

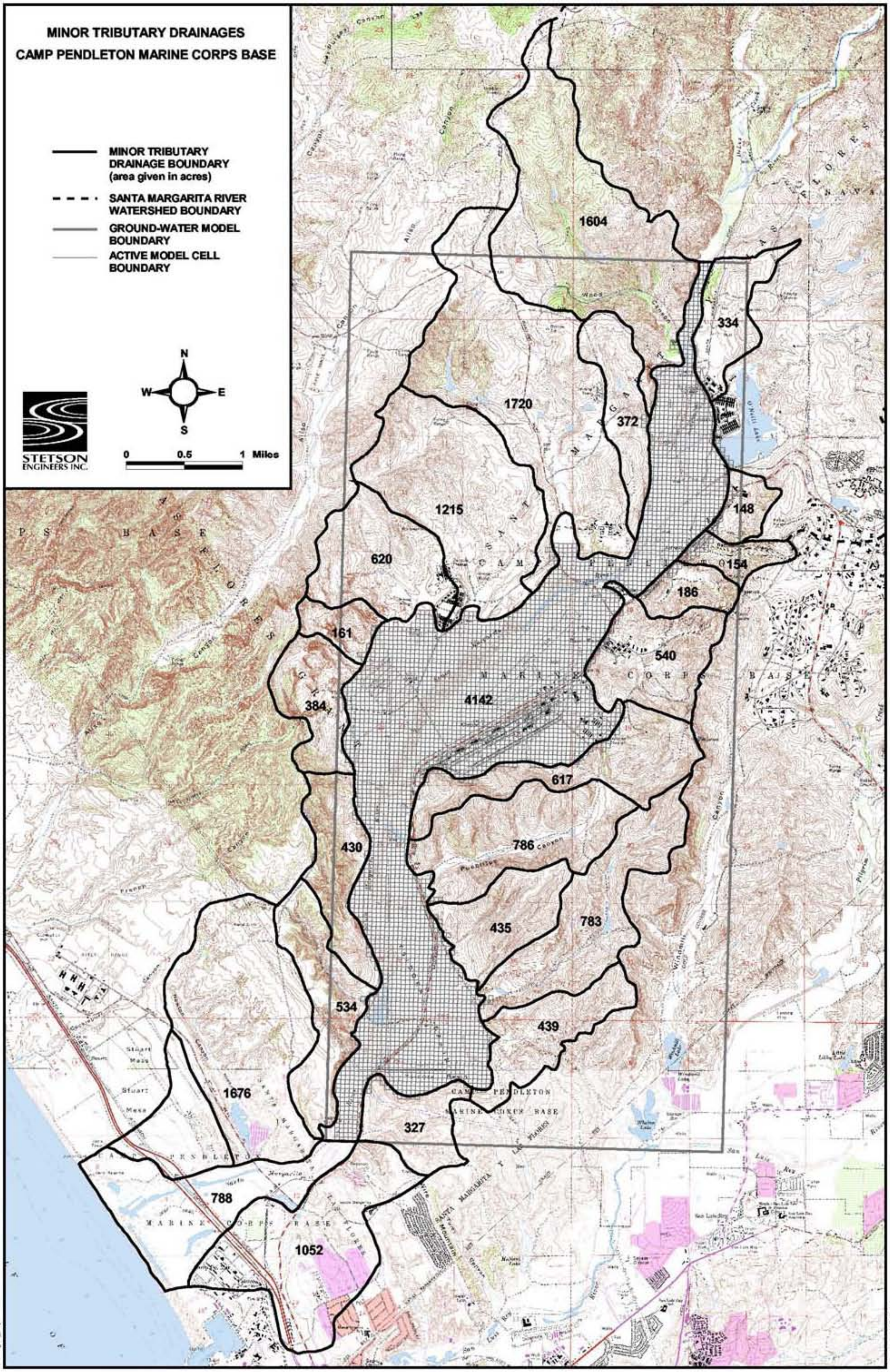
Model for the appropriate years of operation (Carlson, 2000). Using the historical diversion data (Malloy, 2000), infiltration rates at the Upper Ysidora recharge basins were calibrated with the ground-water model. The ground-water recharge pond infiltration rates were modeled with a seasonal variation ranging from 0.2 ft/day to 1.8 feet/day to account for percolation of the water diverted from the Santa Margarita River.

Phreatophyte location and density of coverage was estimated from infrared and aerial photos taken in 1980, 1982, 1989, 1993 and 1997 and a riparian vegetation survey conducted in 1997 (MCB-CP, 2000) to determine ground-water consumption by evapotranspiration. Dense cottonwood and willow riparian trees were assigned an ET rate of approximately 60 in/yr and an extinction depth of 20 feet. Dense wetland plants were assigned an ET rate of approximately 45 in/yr with an extinction depth of 8 feet. Different densities of phreatophytes were assigned values proportional to these values.

4.5 GROUND-WATER FLOW MODEL BOUNDARY CONDITIONS

The MODFLOW™ streamflow package was used to simulate the flow of the Santa Margarita River, including minor tributary drainages, historical oxidation pond discharges, diversions, Lake O'Neill spills and releases, and the river system's interaction with the alluvial aquifer. The streamflow package is able to account for flow in the river and whether a river reach is gaining water from or losing water to the aquifer. The USGS developed the Streamflow Package to account for intermittent rivers typical in the southwestern United States, like the Santa Margarita River. It permits rivers to go dry and then re-wet if ground water becomes available further downstream. The major inflows to the river that were simulated are: surface flow into the top of the Model domain, ground-water discharge into the river, wastewater discharge from Oxidation Ponds 1, 2, 3, 8, and 13 (after evaporation and infiltration to ground water), recoverable runoff from minor side tributary drainages (Figure 4-2), and spills and releases from Lake O’Neill. The major outflows from the river that were simulated include surface flow leaving the southern end of the model domain, infiltration to ground water, and diversions to the recharge ponds and Lake O’Neill.

General head boundaries were established at the upgradient (northeast) and downgradient (southwest) cells to simulate subsurface underflow. The bedrock units to the east and west of the river’s alluvial sub-basins were simulated as no-flow boundaries and considered as inactive cells without contributing to ground-water flow. Although there is some subsurface flow though the bedrock, it is generally considered to be non-water-bearing due to very low permeability.



drainages_c.apr 12/08/04

FIGURE 4 - 2

4.6 WELL INVENTORY AND WATER LEVEL DATA

During the model calibration period of WY 1980 through WY 1999, Camp Pendleton operated five production wells in the Upper Ysidora, nine production wells in the Chappo, and three irrigation wells in the Ysidora Narrows and Lower Ysidora. Figure 4-3 summarizes the ground-water production in the three sub-basins, showing the effects of the seasonal summer demand, extra demand of ground-water resources following drier than normal winters, and the 1995 base expansion. Table 4-1 lists the production wells, screen intervals, period of operation during the model calibration period, and average annual pumping volumes during the pumping period. Figure 4-4 shows the location of modeled production wells.

Historical water levels from two monitoring wells in the Upper Ysidora, four monitoring wells in the Chappo, and two monitoring wells in the Lower Ysidora were used for model calibration because of the continuity of the recorded data at these wells. Figure 4-4 shows the location of these wells (marked target wells) and Table 4-2 shows the annual average water level at these wells. Three monitoring wells, 10/4-7J1, 10/5-23L1, and 10/5-35K5, located near the South central part of the Upper Ysidora, Chappo, and Lower Ysidora, respectively, were used as indicator wells for changes to the aquifer. The historical calibration of the Model, as well as impacts from future model runs, use these three "target" wells to identify potential impacts to the streamflow and ground-water sub-basins.

4.7 SURFACE WATER ANALYSIS

The scope of the surface water analysis was to address the hydrologic possibilities for diverting water from the Santa Margarita River for use on Camp Pendleton and to construct streamflow quantities for use by the ground-water model. Streamflow quantities were estimated at a point below the confluence of the Santa Margarita River and De Luz Creek for the purpose of determining the amount of available water in the model area and on the Base. Results from the surface water analysis were used by the Model to simulate the amount of water that enters the Model's northeast boundary, the amount of water available for diversion, the amount of water that by-passes the diversion point, and the amount of water that reaches Lake O'Neill. In addition to these calculations, the surface water analysis also estimated the amount of streamflow that flows into the modeled area from side tributaries within the lower Santa Margarita River basin. A detailed discussion of the surface water analysis is provided in Appendix E.

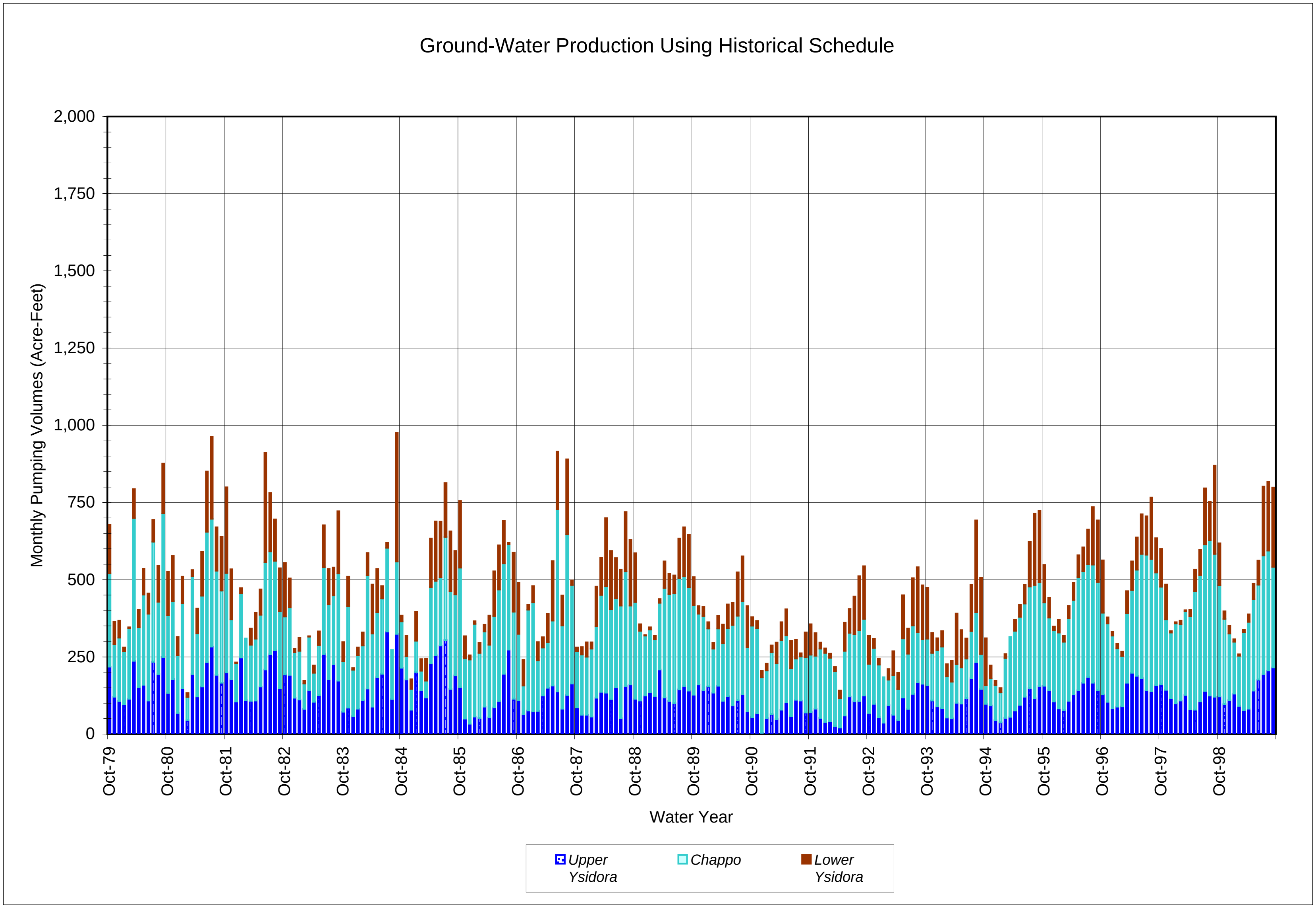
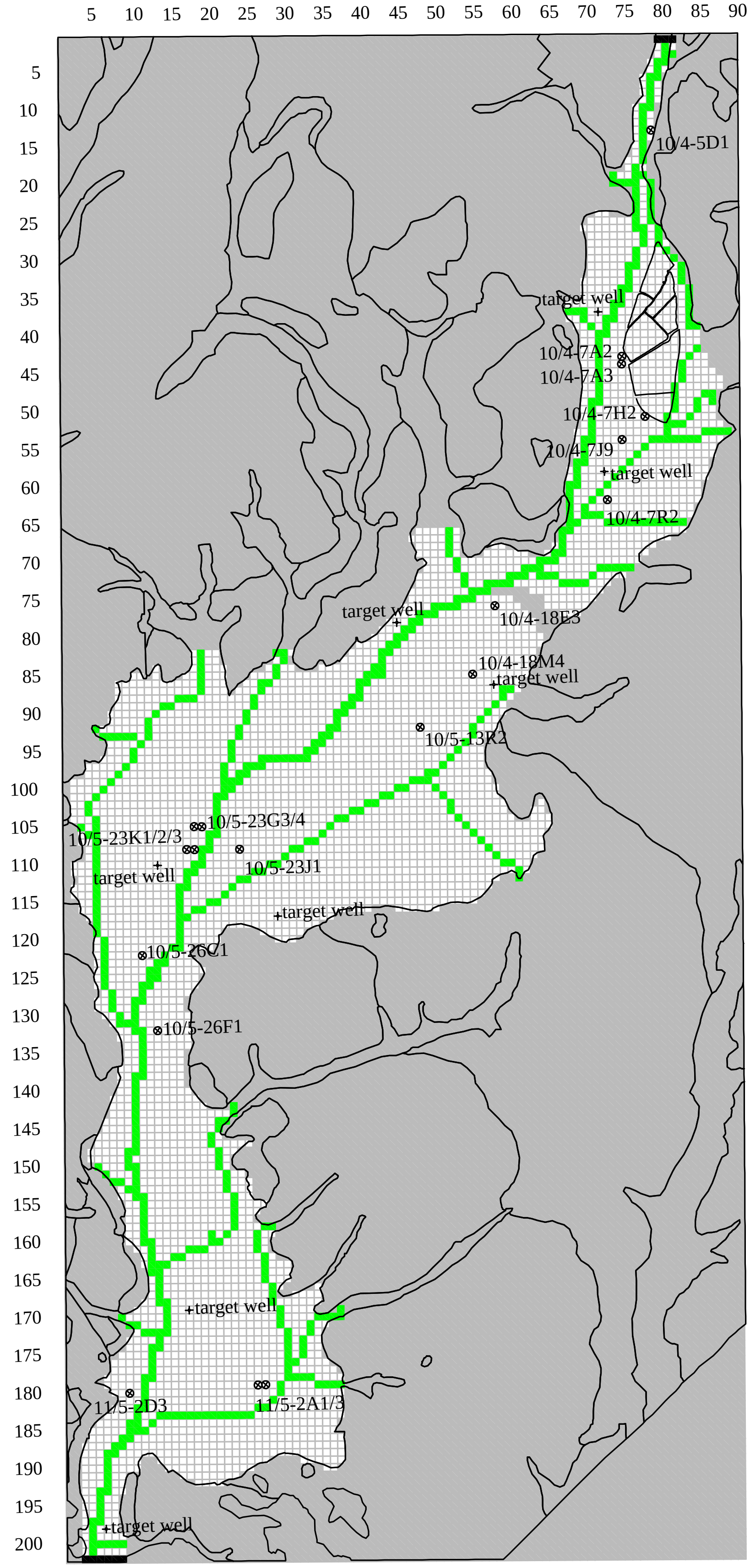

Ground-Water Production Using Historical Schedule
Monthly Pumping Volumes (Acre-Feet)
2,000
1,750
1,500
1,250
1,000
750
500
250
0
Oct-79
Oct-80
Oct-81
Oct-82
Oct-83
Oct-84
Oct-85
Oct-86
Oct-87
Oct-88
Oct-89
Oct-90
Oct-91
Oct-92
Oct-93
Oct-94
Oct-95
Oct-96
Oct-97
Oct-98
Water Year
Upper Ysidora
Chappo
Lower Ysidora

Figure 4 - 3



⊗ Production Well
+ Target Water Level Well

STETSON ENGINEERS INC.

Well Locations within the Model Area

F:\DATA\1828\REPORT\GWMODEL\DFX\Well_Loc.DWG

TABLE 4-1
PRODUCTION WELL INVENTORY

Well ID	Bldg No.	Drilled	Operation	Average AF/WY	Screen Interval (feet, bgs)	Ground Surface (feet, msl)
Upper Ysidora Sub-basin						
10/4-5D1	27911	1943	1981-1987	380	28-70	110
10/4-7A2	2673	1956	1980-1999	630	n/a	(7A1) 103
10/4-7A3	n/a	1999	1999	240	n/a	(7A1) 103
10/4-7H2	2671	1956	1980-1999	290	n/a	(7H1) 98
10/4-7R2	2603	1955	1980-1999	460	n/a	(7R1) 90
Chappo Sub-basin						
10/4-18E3	2393	1965	1981-1999	470	89-109	78
10/4-18M4	2373	1960	1980-1999	640	84-224	76
10/5-13R2	2363	1956	1980-1982	740	68-132	66
			1990-1999	450		
10/5-23J1	2301	1950	1980-1999	520	107-137	52
10/5-23G3	33926	1976	7 years	130	17-118	54
10/5-23G4	n/a	n/a	1999	440	n/a	n/a
10/5-23K2	33924	n/a	11 years	240	n/a	50
10/5-23K3	n/a	n/a	1999	460	n/a	n/a
10/5-26C1	2201	1959	1980-1999	810	96-162	44
Ysidora Narrows and Lower Ysidora Sub-basin (irrigation wells)						
10/5-26F1	2200	n/a	1980-1999	950	88-170	39
11/5-2D3	n/a	n/a	1986-1999	140	n/a	n/a
11/5-2A3/1	19122	n/a	1980-1989	90	n/a	n/a

Note: n/a indicates unknown or unavailable data; bgs is 'below ground surface'; msl is 'mean sea level'

TABLE 4-2
MONITORING WELL WATER LEVEL DATA

Well ID	Period of Record (WY)	Average Annual Water Level (ft, msl)	Measuring Point Elevation (ft, msl)
Upper Ysidora Sub-Basin			
10/4-6R1	1983-1995	93	105
10/4-7J1	1980-1999	86	92
Chappo Sub-Basin			
10/4-18L1	1980-1999	65	74
10/5-13G1	1996-1999	66	124
10/5-24N1	1980-1999	48	57
10/5-23L1	1985-1995	41	50
Lower Ysidora Sub-Basin			
10/5-35K5	1980-1993	22	25
11/5-2N4	1980-1993	12	16

4.7.1 STREAMFLOW AT MODEL BOUNDARY

The first step was to make a composite record of the historic streamflow data for water years 1925 to 1999. Daily historical mean streamflow data from the USGS gages were used to develop a streamflow hydrograph for the 75-year period of record. A schematic drawing of available USGS gages is shown in Figure 4-5. Since there is no USGS gage at either the model boundary or the existing diversion structure, the development of a complete hydrograph at this point on the river required combining the flow data from different gages. Missing data for periods of broken record were calculated, simulated, and calibrated to coincide with the data requirements of the ground-water model.

A spreadsheet model was used to reconstruct the surface 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 DeLuz 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 FPUD sump gage, Sandia Creek, and De Luz Creek were added together to approximate the flow at the Model boundary.

4.7.1.1 Water Years 1925 to 1980

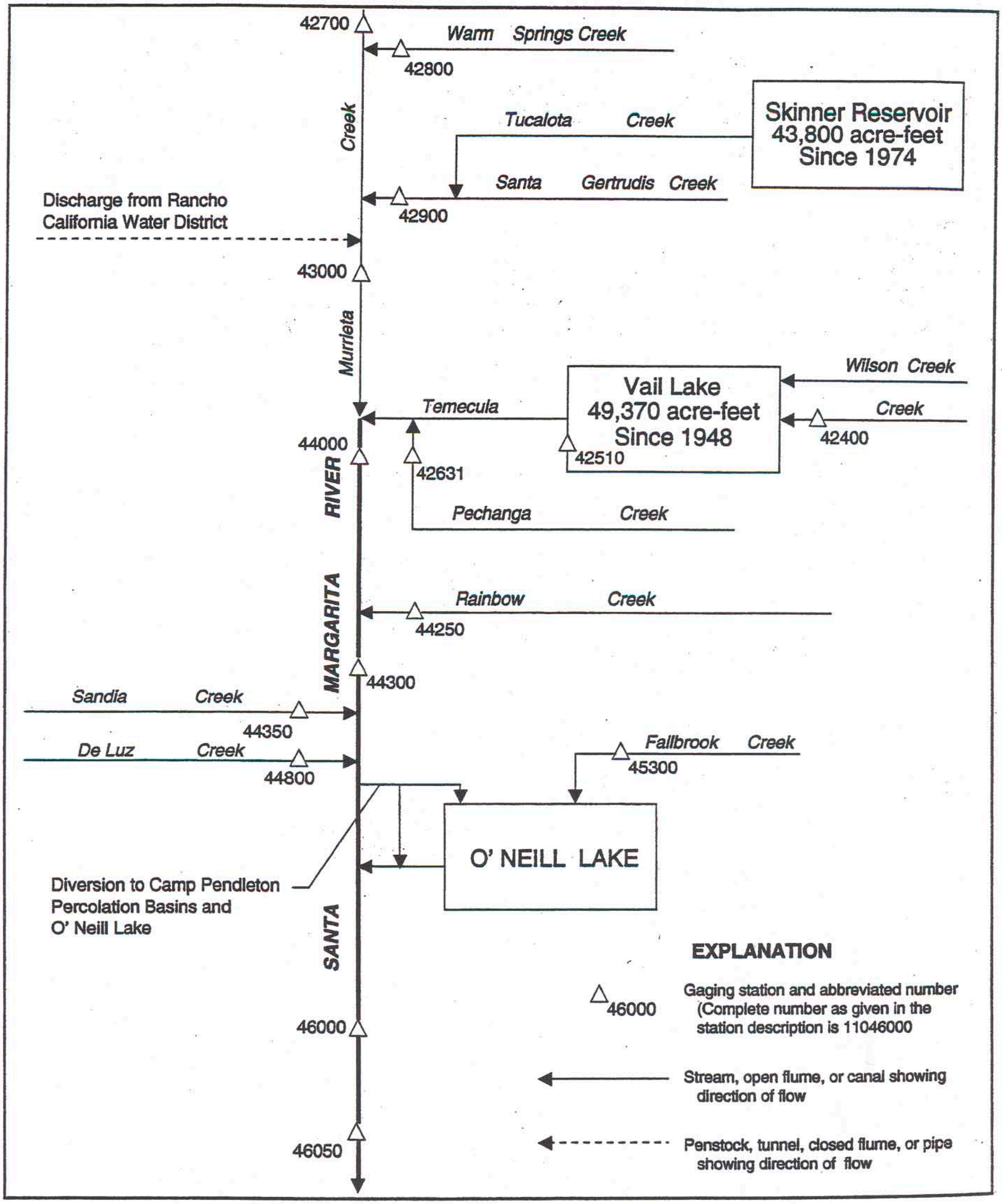
For this early period of record, the Fallbrook gage (44500) historical streamflow data set was complete, representing the mean daily streamflow at a point below the confluence of Sandia Creek and the Santa Margarita River. The contribution from De Luz Creek to the Santa Margarita River was simulated using a proportionality constant based on drainage areas. The total streamflow at the Model boundary was calculated based on adding simulated streamflow from De Luz Creek with observed streamflow from the Fallbrook gage.

4.7.1.2 Water Years 1981 to 1989

The only historical streamflow data set available for this period of record was from the Gorge gage (44000). The flow at this point is highly controlled by urban development in the Upper Basin, and is not necessarily representative of the factors that dictate the hydrology in the lower part of the watershed. Multiple methods of model simulation and calibration were explored to model the streamflow below the Gorge during this period.

FIGURE 4-5

Diversions and Storage in Santa Margarita River Basin



Hayes, P.D., Agajanian, J., and Rockwell, G.L., "Water Resources Data – California Water Year 1998, Volume 1, Southern Great Basin from Mexican Border to Mono Lake Basin, and Pacific Slope Basins from Tijuana River to Santa Maria River." U.S. Geological Survey, Water Resources Division, California District. 1998.

The Soil Conservation Service (SCS, 1972) developed a method for computing surface run-off from storm rainfall [Chow et. al. 1988]. The basic equation for computing the depth of excess rainfall or direct runoff from a storm by the SCS method is:

$$P_e = \frac{(P - 0.2S)^2}{P - 0.8S}$$

The variables in the SCS method include Pe= rainfall excess (direct runoff), P= total rainfall, and S= potential maximum water retention. To standardize this equation for different watersheds, a dimensionless curve number (CN) is defined, such that for impervious water surfaces CN=100, and for natural surfaces CN<100. Table 4-3 lists the curve numbers chosen for the streamflow model. The curve number and S are related by the equation S = 1000/CN - 10.

TABLE 4-3
CURVE NUMBERS FOR SMR WATERSHED

CN	S	
87	1.49	**Normal**
93.9	0.65	**Wet**
73.8	3.56	**Dry**

The SCS method was used to approximate flows after peak precipitation events for this period. Hourly and daily data from the Oceanside rainfall gage in Southern California was used to calculate precipitation runoff. Data sets were obtained from the Desert Research Institute (Appendix E, Attachment E-1).

During the 1980 flood, the Fallbrook gage (44500) was washed out, and in 1989 a new gage was installed at the FPUD Sump on the Santa Margarita River (44300) upstream of the confluence with Sandia Creek. The streamflow contribution between the Gorge and the future Fallbrook gage (drainage area = 32 mi^2) during a peak event was calculated from the SCS Curve Number Method. When there was not a precipitation event, the baseflow was simulated using the natural flow at the Gorge as modeled by HSPF.

This method was applied to a period of observed flow, water years 1989 to 1996, to determine if the predicted streamflow provided a reasonable calibration to the observed data. The modeled hydrograph for this period of comparison used Oceanside precipitation data (1989 to 1996) for the SCS method of calculating runoff during rain events and the HSPF model to simulate baseflows. The constructed hydrograph at the diversion point for this period was calculated as the sum of the observed data from the FPUD Sump gage (44300), Sandia Creek gage (44350), and the De Luz Creek gage (44800). The modeled data for Sandia Creek, De Luz Creek, and at the location of FPUD Sump were compared to the observed data at these USGS gages. The modeled data at the diversion point was compared to the constructed data at the diversion point. Based on the calibration to the observed data to water years 1989 to 1996, the proportionality constants based on drainage areas used in the surface water model were modified to present a more accurate replication of the streamflow during the 1980 to 1989 period of unknown flow (Appendix E).

Two final steps were performed on the simulated data set to refine the calibration. The MODFLOW model output showed that the simulated historical flows from water years 1980 to 1989 were underestimating spring baseflows and overestimating summer baseflows. The same simulation method was applied to the 1990 to 1996 period of observed streamflow. A plot of the simulated and observed daily streamflow hydrographs confirmed that the surface water model was underestimating the high spring baseflow and overestimating the low summer baseflow contributions. Due to the size and capacitance of the ground-water aquifer in the upper basin, baseflows are overestimated in the summer and underestimated in the winter. The base flow in De Luz and Sandia Creeks would proportionally be less during the summer due to the very low storage volume of the thin channel alluvium. Similarly, the baseflows in the winter would be proportionally greater due to the minimal ground-water storage available to capture rainfall-run off events. A series of monthly constants, shown in Table 4-4, were multiplied by the HSPF natural flow contribution used to more accurately calibrate the surface water model and the ground-water model to the observed streamflow.

TABLE 4-4
MULTIPLIERS USED TO RECALIBRATED BASEFLOWS

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2.0	2.0	2.0	2.0	2.0	1.75	1.25	0.75	0.75	1.25	1.75	2.0

Specific monthly corrections to historical flow were made to assure that there was enough water in the Santa Margarita River to satisfy historical diversions. Table 4-5 shows four months (out of 240 months) where the surface water analysis underestimated the flow at the diversion point and water was added to the analysis to meet historical diversions. This is most likely due to the low precipitation values from the Oceanside data set, or a function of the conservative baseflow approximation.

TABLE 4-5
SPECIFIC MONTHLY ADDITIONS

Date	SMR Flow Increase
May-86	3 [cfs/day]
Mar-87	4 [cfs/day]
Jan-89	3 [cfs/day]
Feb-89	6 [cfs/day]

To summarize, the surface water analysis estimated surface flow during water years 1981 to 1989 as follows. The peak flows, during precipitation events, were determined by the SCS method. The baseflow was determined by adding the observed Gorge flow to the calculated Sandia Creek and De Luz Creek streamflow (proportional to the natural flow at the Gorge modeled by HSPF) and the contribution between the Gorge and the Fallbrook PUD SUMP gage (also based on the HSPF natural flow). A monthly multiplier was used to recalibrate the model to account for the underestimation of spring flows and the overestimation of summer flows. A final refining step added 3 to 6 cfs of baseflow to four particular months where the modeled flow in the Santa Margarita River was insufficient to satisfy historically diverted quantities.

4.7.1.3 Water Years 1989 - 1999

For the most recent period of record there is an extensive set of historical streamflow data for the FPUD Sump gage (44300), Sandia Creek (44350), and De Luz Creek (44800). The streamflow from these three gages was added together to approximate the flow at the Model boundary. The only missing data were for De Luz Creek from 10/1/89 to 10/1/92. During this period, the contribution from De Luz Creek was simulated using a proportionality constant based on drainage areas. It was assumed that the De Luz watershed has similar runoff characteristics to the Sandia Creek watershed. A proportionality constant (33 mi^2 /21.1 mi^2) was multiplied by the streamflow at Sandia Creek to give a reasonable estimate of the flow from De Luz Creek.

4.7.2 DIVERSION CAPACITY

The diversion capacity is defined by the amount of water that can be directed into the O'Neill ditch based on the available streamflow in the Santa Margarita River, defined bypass flow, and diversion capacity. The available streamflow is the calculated or simulated values at the diversion structure for the period of record minus the bypass flow of 3 cfs. No water may be diverted from the Santa Margarita River if the flow in less than 3 cfs, and for all other flows at least 3 cfs must be bypassed through the sluice gates to maintain a clear and clean flow at the headgate. There are two existing permits that allow the Base to divert water from the Santa Margarita River.

1) The pre-1914 Water Right allows for 1,100 AF of storage, which includes 100 AF of dead storage, and 400 AF of evaporation and seepage from Lake O'Neill. The total diversion right is for 1,100 AFY, which for convenience is estimated at 400 AFY, at a maximum diversion rate of 20 cfs. The diversion period is between April 1st and October 31st, although the Base and FPUD have an agreement that allows for diversion from November 1st through March 31st.

2) Permit 15000 License 21471 A allows for 4,000 AFY to be collected in the underground storage reservoir by way of the percolation ponds and the natural channel of the river. The existing system can divert a maximum of 60 cfs, but by improving the existing constrictions, this capacity could be greatly increased. The permitted diversion period is between October 1st and June 30th.

Since one of the goals of the study is to find the optimal scenario for diverting water from the Santa Margarita River, year around, a range of capacities for the diversion channel were explored. At present, the bottleneck in the O'Neill ditch system is the road crossing located 1,045 feet downstream of the diversion weir. It is feasible to redesign the entrance to the diversion channel and the road crossing to handle a higher flow capacity. Ten scenarios were investigated to simulate the quantity of water that could be diverted based on channel capacities equal to 25, 60, 100, 150, 200, 250, 350, 450, and 600 cfs. Based on the median of a 75-yr potential annual diversion analysis for the range of diversion capacities, it was found that 200 cfs represented the optimal diversion capacity.

4.7.3 EVAPORATION

Evaporation removes water from the surface area of an open body of water. The water surface evaporation rates used for this analysis are provided in Appendix E. These monthly values were applied on a daily basis to the surface area of Lake O'Neill and the recharge ponds. The surface area of Lake O'Neill changes with the daily depth of water. Actual data from Public Works Survey Department at Camp Pendleton (1978), following a 1977 Dredging Survey, were used to construct a graph of volume vs. surface area. A trendline for this graph was used to calculate the volume of loss from evaporation each day, based on the daily changes in the volume of Lake O'Neill. The ponds were assumed to be rectangular in shape, such that for any depth of water, the surface area is constant. The evaporative loss for the ponds was also calculated on a daily basis based on the availability of water in the ponds and the precipitation falling on that day.

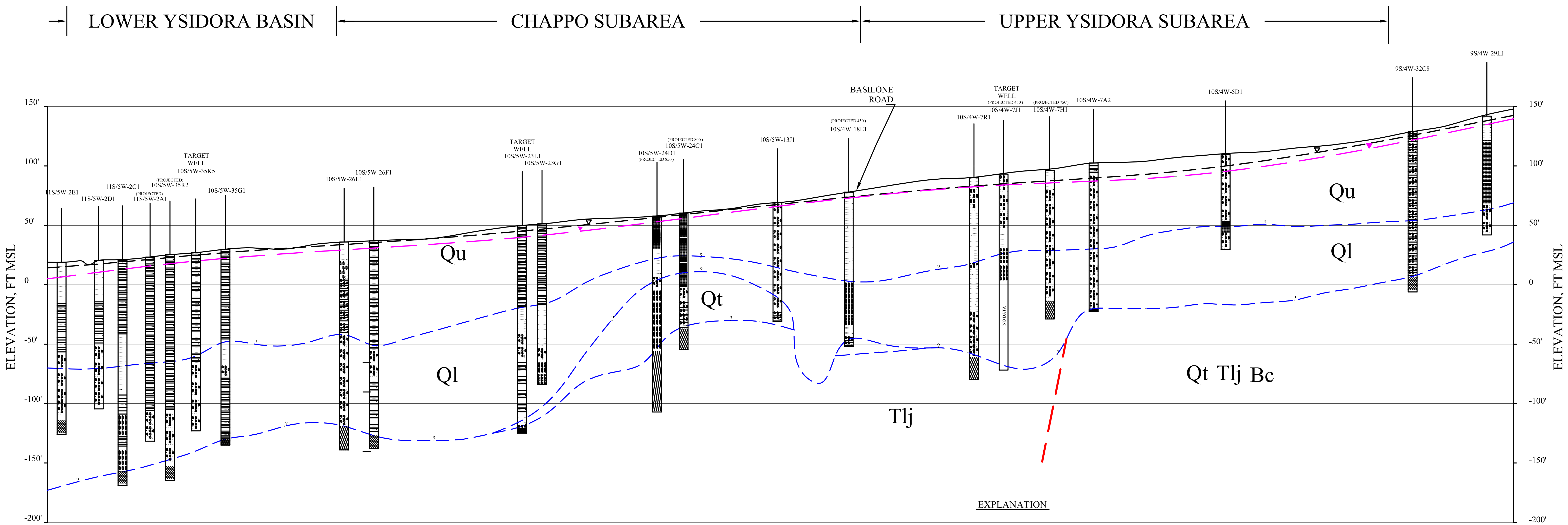
4.7.4 INFILTRATION RATES

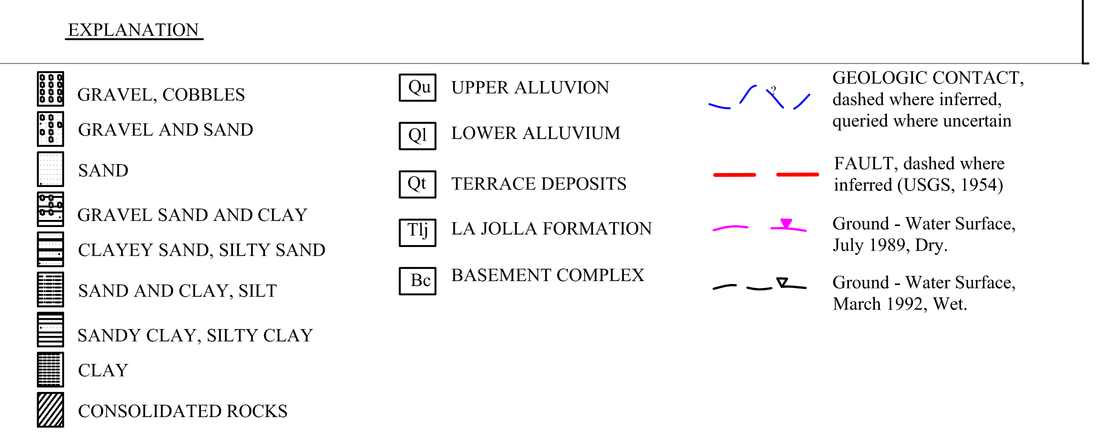
The most important attribute of the recharge ponds is the ability of water to infiltrate below the surface to recharge the ground-water aquifer. The reservoir operation model used infiltration rates ranging seasonally from 0.2 to 1.8 feet/day. The infiltration rates for January and June for ponds 1 and 2 are based on the results of an infiltration study conducted by Stetson Engineers (Chapter 5.3). The rates were interpolated between January and June to reflect the decrease in infiltration rates on spreading systems during periods of continual wetting (Bianchi 1970). After June, no water is diverted to the recharge ponds; thus, the simulated rates remain constant until maintenance in the fall rejuvenates the original infiltration rates. Ponds 3, 4, and 5 were assumed to have slightly higher infiltration rates than ponds 1 and 2, because most of the fine sediment settles out in the first two ponds, reducing the potential of clogging in the later ponds. The ground-water model simulated the conditions below the ground surface, to estimate if there would be enough room to store the percolating water. A reservoir operations model was constructed to supply input for the ground-water model, while also providing a balanced water budget for Lake O'Neill and the recharge ponds. The surface water analysis is discussed in Chapter 7 for future conditions.

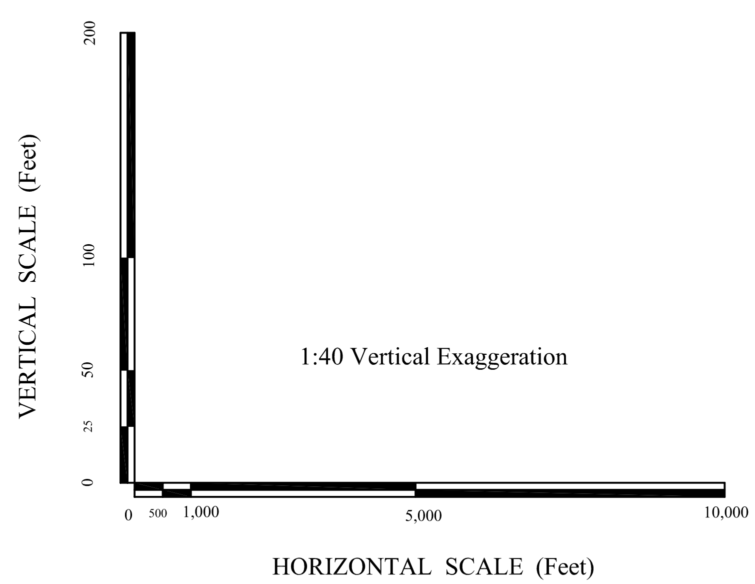
4.8 MODEL CALIBRATION

Data for streamflow, precipitation, and various diversions and releases were compiled from the Base's records for the 20-year period from water year 1980 through water year 1999. This calibration period included the wastewater contributions from sewage treatment plants (STP) 1, 2, 3, 8, and 13. Average input parameters were first used to establish a steady state model, followed by an annual average 20-year transient model calibration period. The final calibration was completed on a 20-year period of monthly time steps. The Ysidora stream gage, which has been monitored by the USGS at its current location near Basilone Road since December 1980, was used as a calibration point for the Santa Margarita River in the ground-water flow model. Ground-water levels from eight monitoring wells were used for calibration of water contour intervals. The model was calibrated to historical streamflow and ground-water level data for the period 1980 through 1999, resulting in the expected hydrologic response between wet and dry years.

The lowest water level during the calibration period occurred during July 1989, and the highest water level occurred during March 1992. Figure 4-6 shows a cross section through the sub-basins and the range in water levels observed during the calibration period. The available data shows that water levels appear to mound North of the narrows near Basilone Road. Each of the three target monitoring wells for the three sub-basins are also shown on this cross section. Figure 4-7 shows observed water level data compared to modeled simulated results for all three sub-basins.







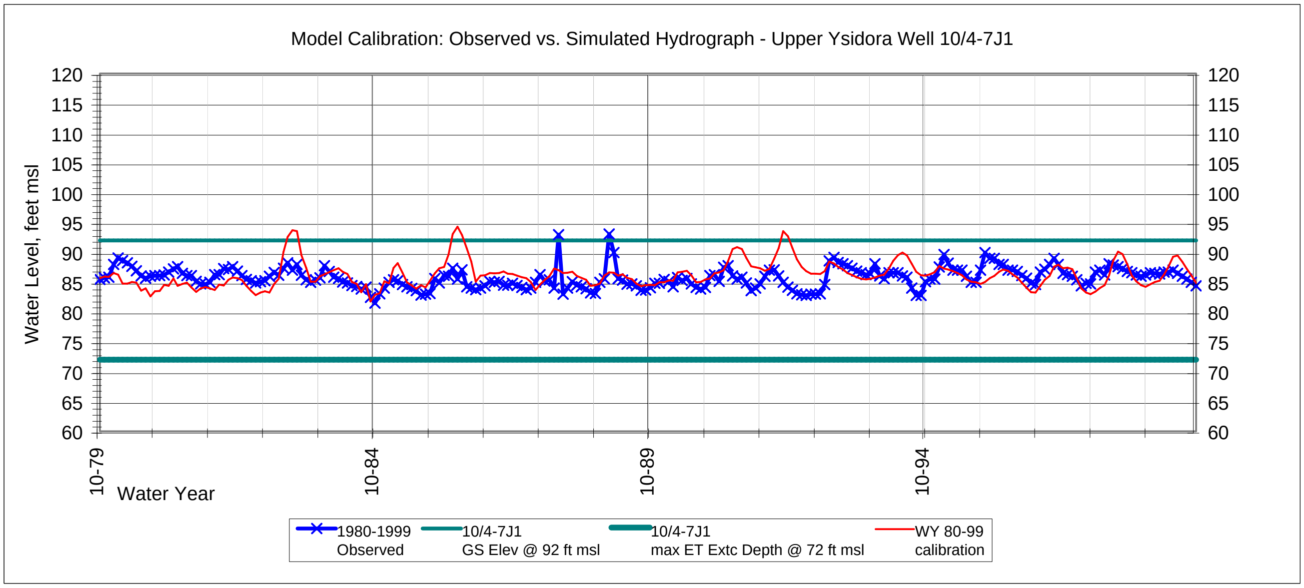
July 1989 and March 1992 Historic Water Levels on
Santa Margarita River Basin Cross Section
Marine Corps Base, Camp Penolleton

SOURCES: LAW/CRANDALL, INC. 1995
WORTS & BOSS, 1954

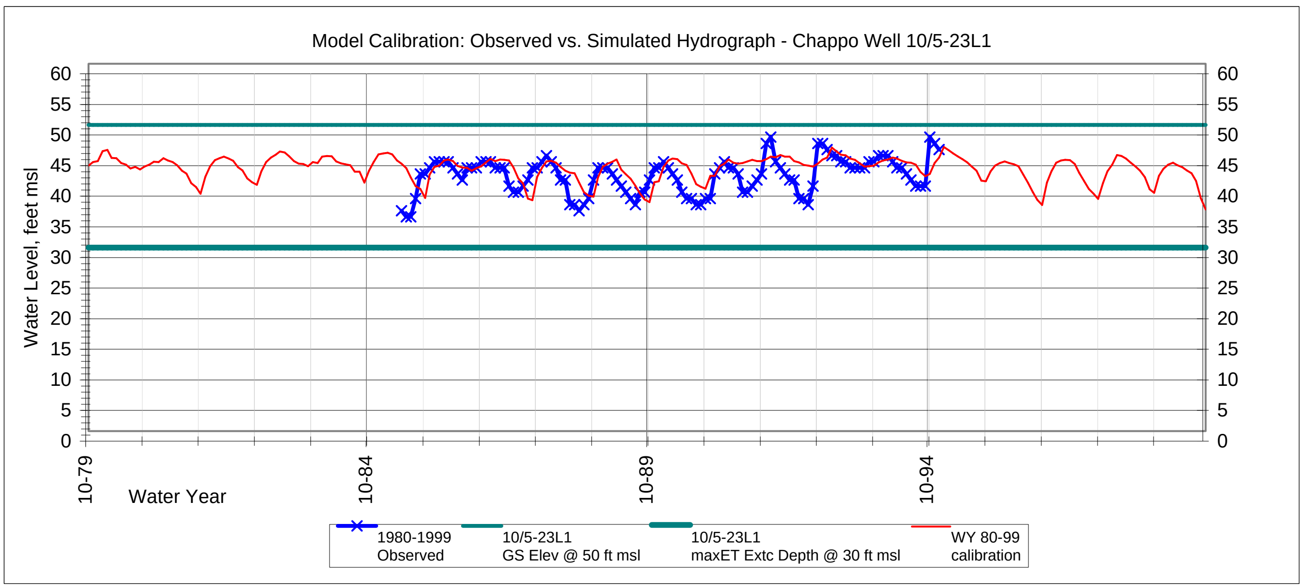
STETSON ENGINEERS INC.

F:\Data\1828\Gus\Law_crandall.dwg

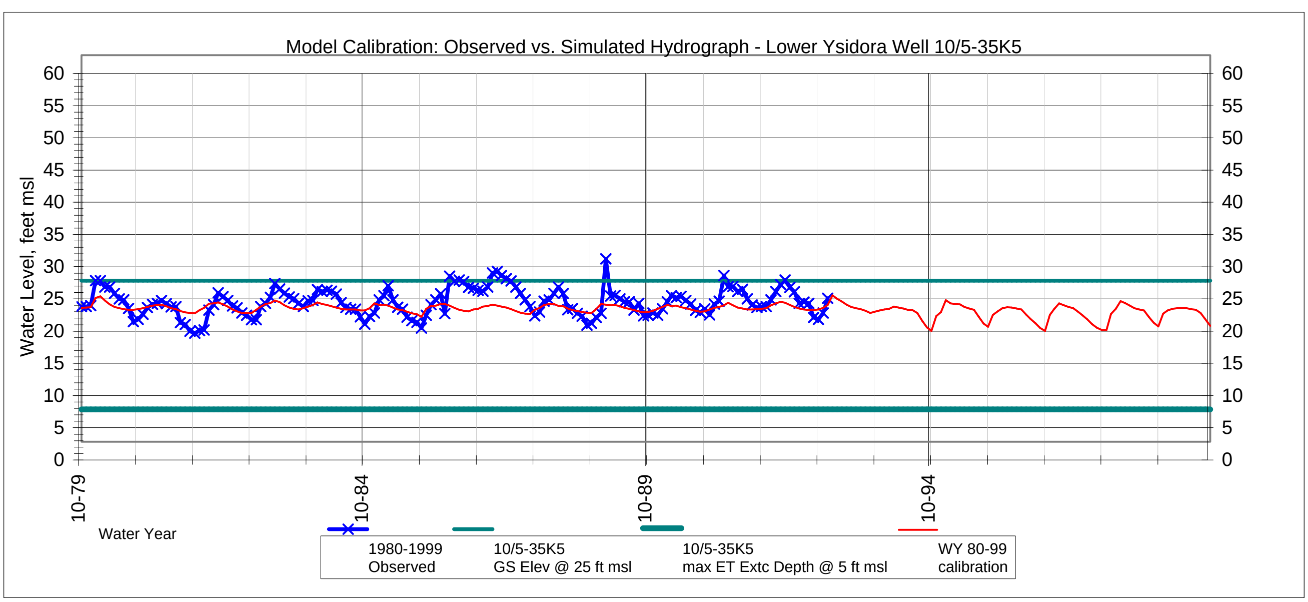
Figure 4-7



Model Calibration: Observed vs. Simulated Hydrograph - Upper Ysidora Well 10/4-7J1
Water Level, feet msl
120
115
110
105
100
95
90
85
80
75
70
65
60
10-79
10-84
10-89
10-94
Water Year
1980-1999 Observed
10/4-7J1 GS Elev @ 92 ft msl
10/4-7J1 max ET Extc Depth @ 72 ft msl
WY 80-99 calibration



Model Calibration: Observed vs. Simulated Hydrograph - Chappo Well 10/5-23L1
Water Level, feet msl
60
55
50
45
40
35
30
25
20
15
10
5
0
10-79
10-84
10-89
10-94
Water Year
1980-1999 Observed
10/5-23L1 GS Elev @ 50 ft msl
10/5-23L1 maxET Extc Depth @ 30 ft msl
WY 80-99 calibration



Model Calibration: Observed vs. Simulated Hydrograph - Lower Ysidora Well 10/5-35K5
Water Level, feet msl
60
55
50
45
40
35
30
25
20
15
10
5
0
10-79
10-84
10-89
10-94
Water Year
1980-1999 Observed
10/5-35K5 GS Elev @ 25 ft msl
10/5-35K5 max ET Extc Depth @ 5 ft msl
WY 80-99 calibration

Monthly flows observed at the Ysidora gage are shown in Figure 4-8, along with simulated flows at the Ysidora gage and surface flow out of the Model boundary in the Lower Ysidora sub-basin. The Model simulates a wetter river during winter months of the dry period (WY 1987 - 1989), which could be an effect of averaging the river flow over the whole month instead of the daily peaks. The simulated river flow does go to 0 cfs during the observed summer months of these same dry years.

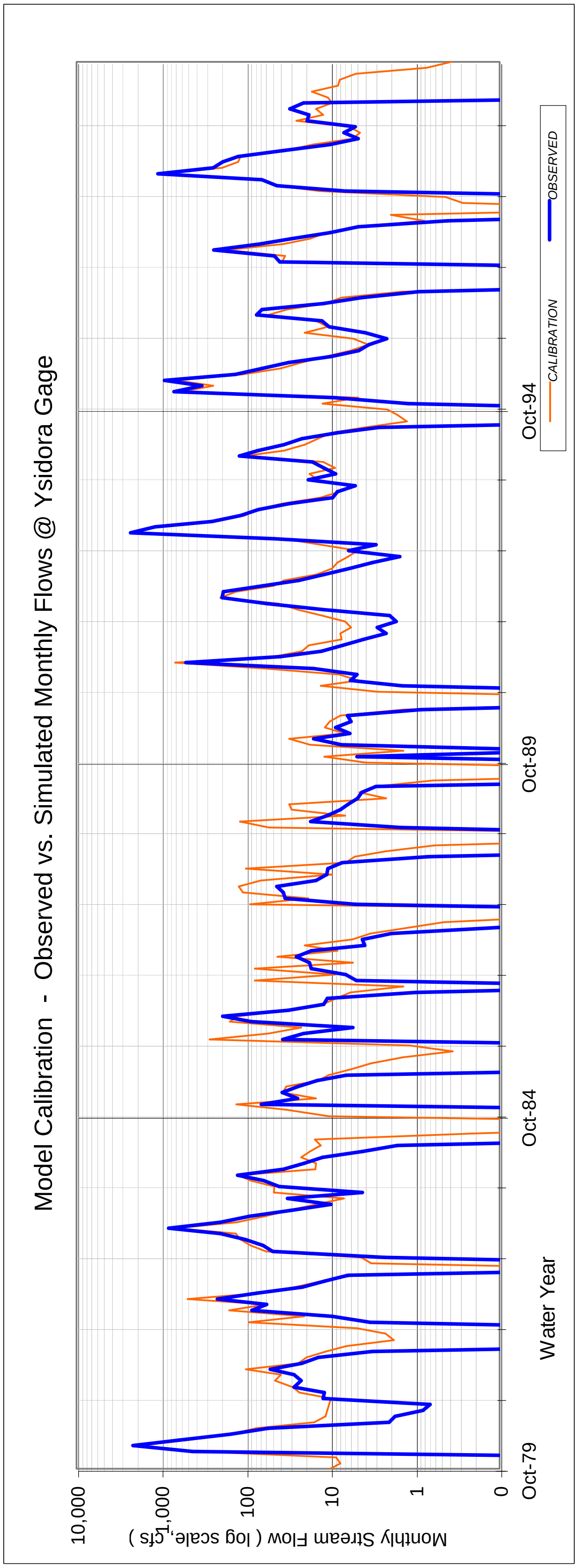
Each water level graph shows the ground surface elevation and the maximum estimated ET extinction depth for riparian vegetation (20 feet). Water levels near the ground surface are an indication of mounding, especially in the Upper Ysidora sub-basin near the recharge ponds. Water levels near the maximum ET extinction depth are considered critical during the summer months, but less critical during the winter months as long as there is no prolonged period of low water levels. Water levels below the maximum ET extinction depth are considered potentially harmful to riparian vegetation. The location of the three target wells has been chosen from available monitoring well data that best represents the overall water level in each sub-basin. In general, the target wells have been located near the center of each sub-basin away from the river so that streambed recharge does not greatly impact water levels in each well.

4.9 WATER BUDGET

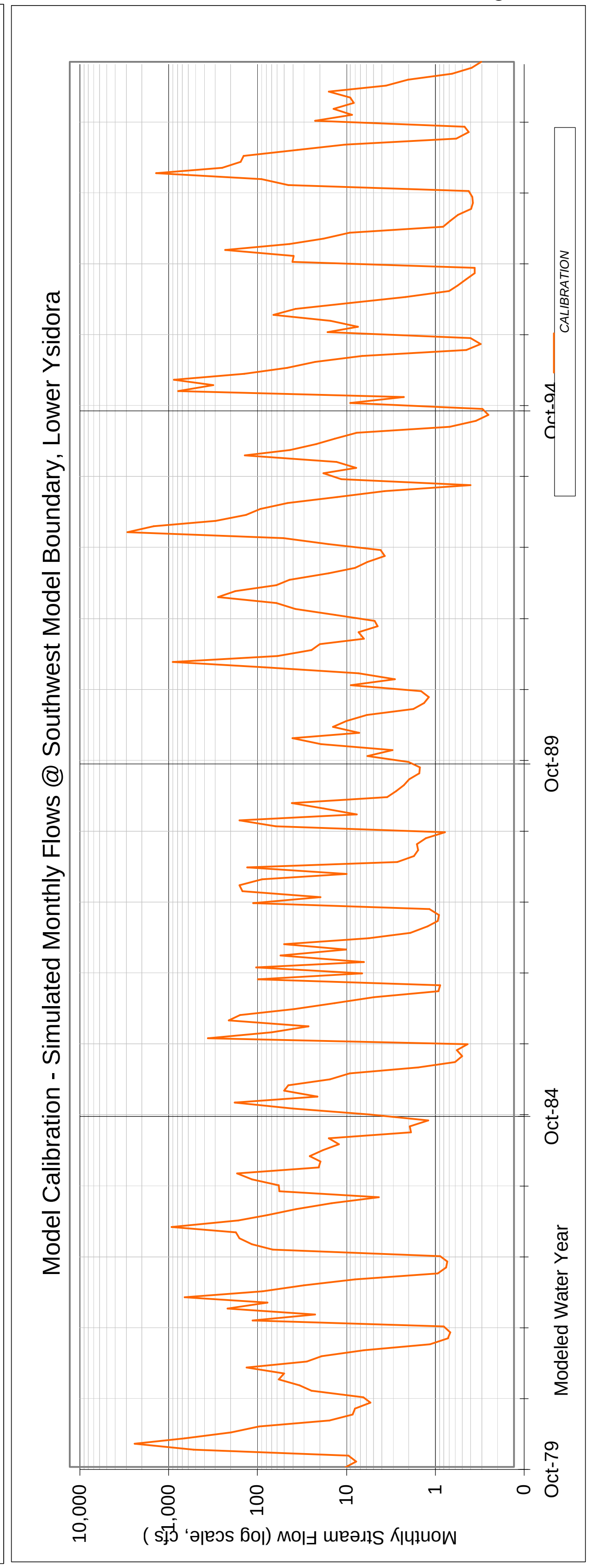
The major influence on the ground-water budget is the Santa Margarita River, which provides approximately 60- 65% of the total recharge to the ground-water basins. Of the major outflows from the ground-water aquifer, pumping of ground-water production wells account for approximately 50% and ET removes an additional 30%. Other influences on the ground-water budget include recharge from precipitation, percolation/oxidation ponds, and side tributary runoff. The Model was developed to account for the inflows and outflows of the river, and the impacts they have on the three ground-water sub-basins.

The calibrated model run is summarized in the water budget presented in Table 4-6. The Model boundary is the area for which the water budget is calculated. The ground-water model provides calculated numbers for underflow, stream recharge to ground water, streamflow 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, recoverable water from precipitation, and net infiltration from recharge ponds.

Figure 4-8



Model Calibration - Observed vs. Simulated Monthly Flows @ Ysidora Gage
Monthly Stream Flow (log scale, cfs)
10,000
1,000
100
10
1
0
Oct-79
Oct-84
Oct-89
Oct-94
Water Year
CALIBRATION
OBSERVED



Model Calibration - Simulated Monthly Flows @ Southwest Model Boundary, Lower Ysidora
Monthly Stream Flow (log scale, cfs)
10,000
1,000
100
10
1
0
Oct-79
Oct-84
Oct-89
Oct-94
Modeled Water Year
CALIBRATION

TABLE 4-6
MODEL CALIBRATION -- AVERAGE ANNUAL WATER BUDGET FOR 1980-1999
(AFY)

		Average Annual	Median Annual
Inflow:	Subsurface Underflow	850	820
	Santa Margarita River Inflow	53,340	27,690
	Lake O'Neill Spill and Release	1,990	1,780
	Minor Tributary Drainages	2,120	1,720
	Waste Water Discharge	2,030	2,260
	Direct Precipitation	690	500
	Total Inflow:	61,020	34,770
Outflow:	Subsurface Underflow	240	240
	Santa Margarita River Outflow	52,380	25,460
	Ground-Water Pumping	5,5560	5,870
	Evapotranspiration	2,880	2,830
	Diversions to Lake O'Neill	490	430
	Total Outflow:	61,570	34,830
	Net change in Storage:	790	160
Exchange of Water within Model Domain			
	Net infiltration from Recharge Ponds	2,850	2,480
	Stream Recharge to Ground Water	4,280	4,700

4.10 MODEL SCENARIOS OF ANTICIPATED BASIN CHANGES

The calibrated Model was used as a predictive tool to ascertain the potential effect of various stresses and changes to the ground-water system that are expected to occur in the future. These anticipated changes include: removal of wastewater from the Santa Margarita River basin, augmentation to streamflow due to an agreement with the RCWD, and increase in ground-water pumping. Table 4-7 below summarizes the model runs that were performed to estimate the impact of these future changes to the ground-water system on the Base.

TABLE 4-7
SUMMARY OF MODEL SCENARIOS FOR ANTICIPATED BASIN CHANGES

Run #	SMR Flow	Ground-Water Pumping	Wastewater Release	Comment
1	H	H	Yes	Calibration Run
2	H	H	No	Effect of no Wastewater Release
3	A	H	Yes	Effect of Augmented Flows
4	A	F1	No	Effect of F1 Pumping
5	A	F2	No	Effect of F2 Pumping
6	A	F3	No	Effect of F3 Pumping
7	H	F3	No	Effect of F3 Pumping with no Augmentation or Wastewater Flows

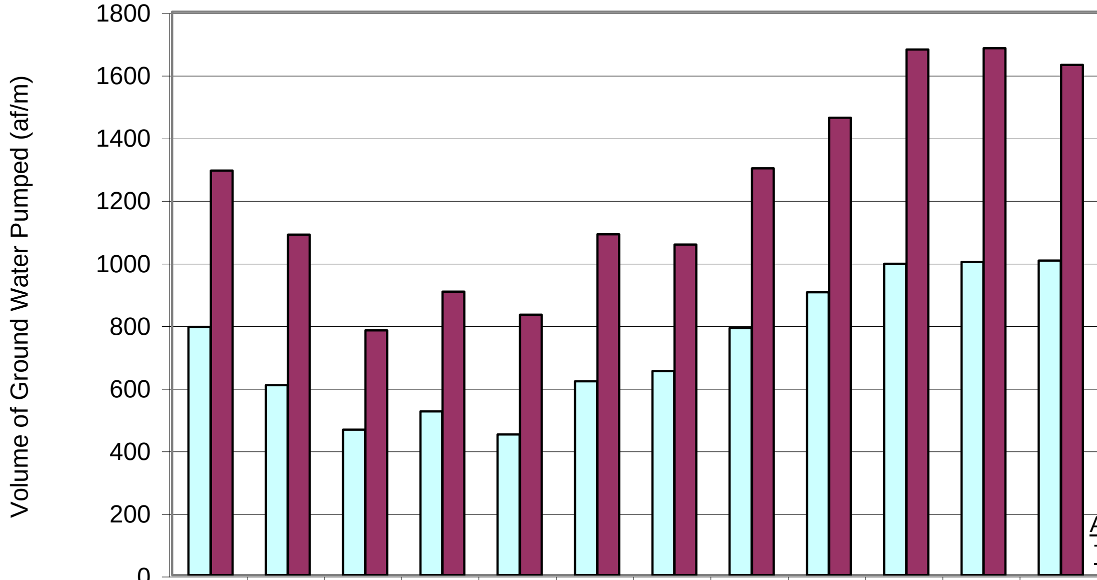
Notes: H indicates 1980 to 1999 historical value
A indicates Augmented streamflow due to the RCWD Agreement
F1 indicates 14,800 AFY ground-water pumping
F2 indicates 14,800 AFY conjunctive use ground-water pumping
F3 indicates 14,050 AFY conjunctive use ground-water pumping

Different pumping scenarios were analyzed to determine optimal ground-water pumping management practices during seasonal changes as well as extended dry periods. Camp Pendleton's historical maximum use of water from the lower Santa Margarita River basin is 8,300 AFY with a build-out demand estimate at 8,800 AFY (MCB-CP, 2001). The existing average annual ground-water well production rate for WY 1980 through 1999 is 5,555 AFY, ranging from 3,724 AF in WY 1991 to 6,705 AF in WY 1981. The F1 pumping schedule was developed from the average historical (WY 1980-1999) monthly distribution of production with historical maximum production occurring in July and August of each year and minimum production occurring during the winter months. This pumping schedule (Figure 4-9) includes 6 new production wells and increases the average annual production to 14,800 AFY in a direct proportion to the historical demand, independent of management for drought or wet years. Model locations for the 6 proposed wells (designated by "PW-") are shown in Figure 4-10.

The F2 (Figure 4-9) pumping schedule also maximizes annual ground-water production of 14,800 AFY, but shifts the maximum production rates to occur in the winter months. Figure 4-9 compares the average potential build-out pumping with average monthly pumping under the F1 and F2 ground-water production schedules. Monthly pumping rates for a potential build-out demand of 8,800 AFY were based proportionally to 1980-1999 historical average monthly pumping. The F3 pumping schedule is similar to F2 with the maximum production in winter months, but includes management practices that reduce ground-water production by 3,000 AFY starting during 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 3,000 AFY until normal or above normal streamflow conditions return. Figure 4-11 compares the different monthly F3 pumping schedules during these different conditions. Reduced percentages of F3 pumping were also

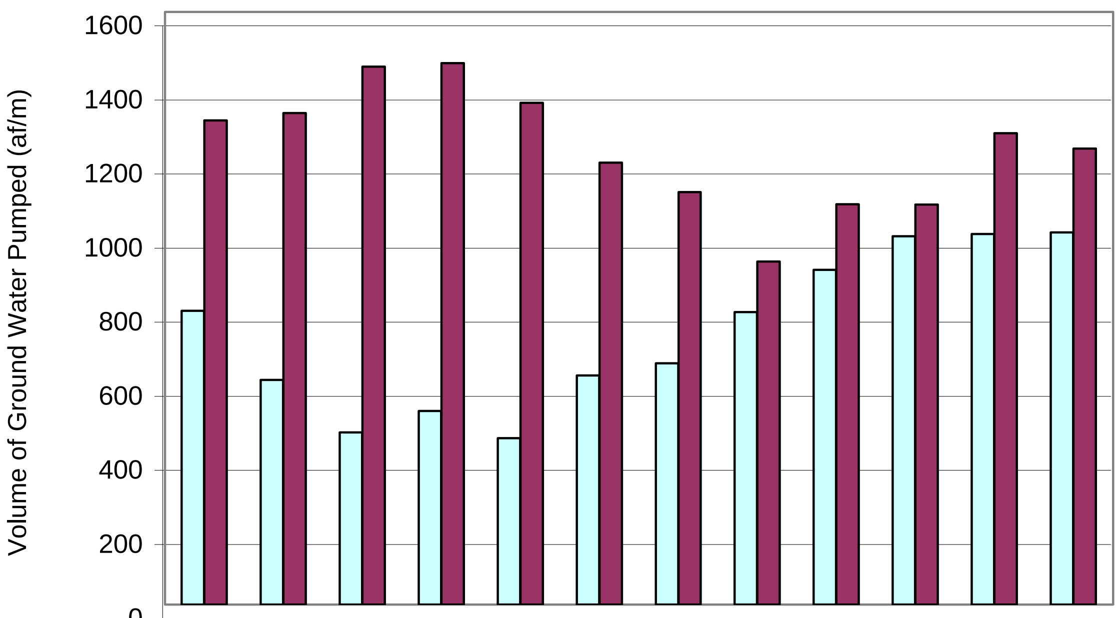
Figure 4-9

F1 Pumping Schedule



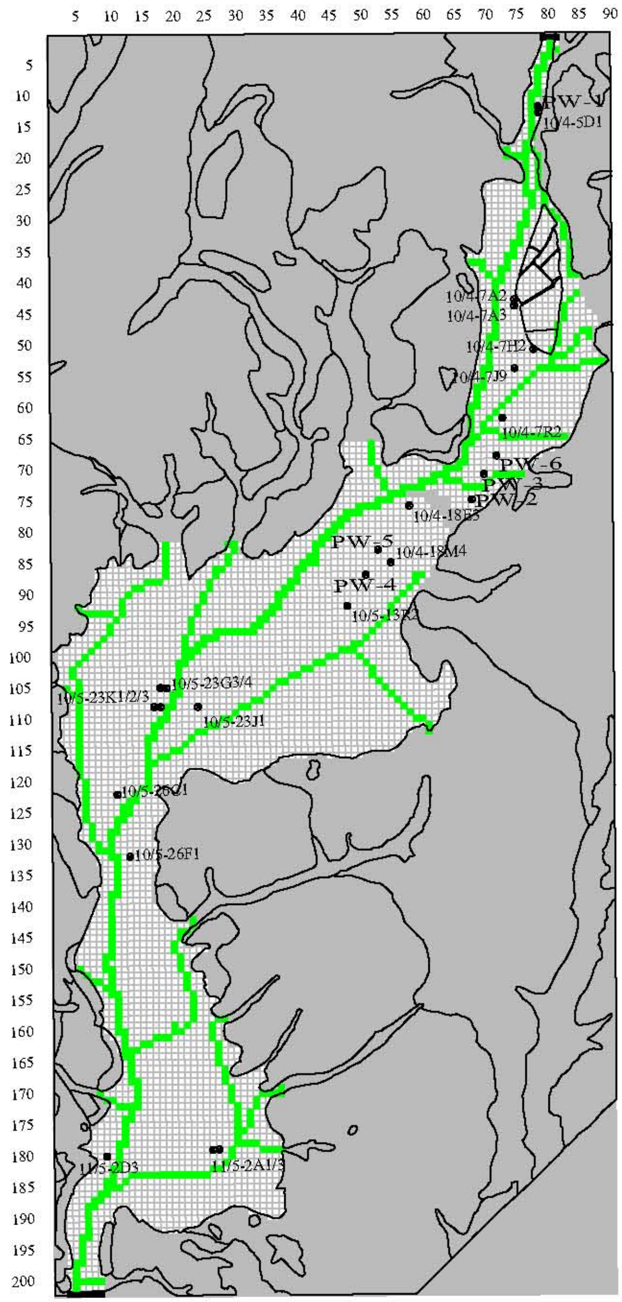
	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual Totals
Buildout	793	607	465	523	449	619	651	789	903	994	1,001	1,005	8,800
F1	1,293	1,088	782	905	832	1,089	1,056	1,300	1,462	1,680	1,683	1,630	14,800

F2 Pumping Schedule



	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual Totals
Buildout	793	607	465	523	449	619	651	789	903	994	1,001	1,005	8,800
F2	1,307	1,327	1,452	1,462	1,355	1,193	1,114	926	1,081	1,080	1,273	1,231	14,800

FIGURE 4-10

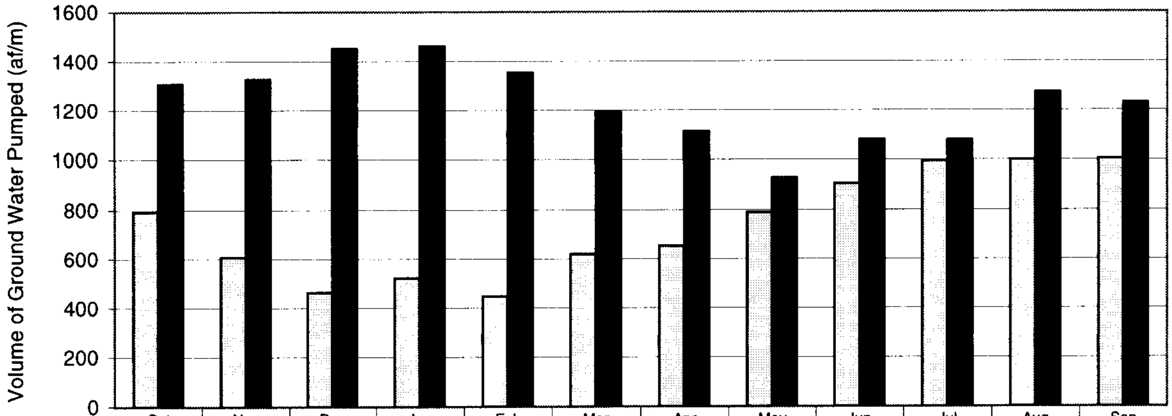


STETSON ENGINEERS INC.

⊗ Production Well Location
PW- Proposed Well Location

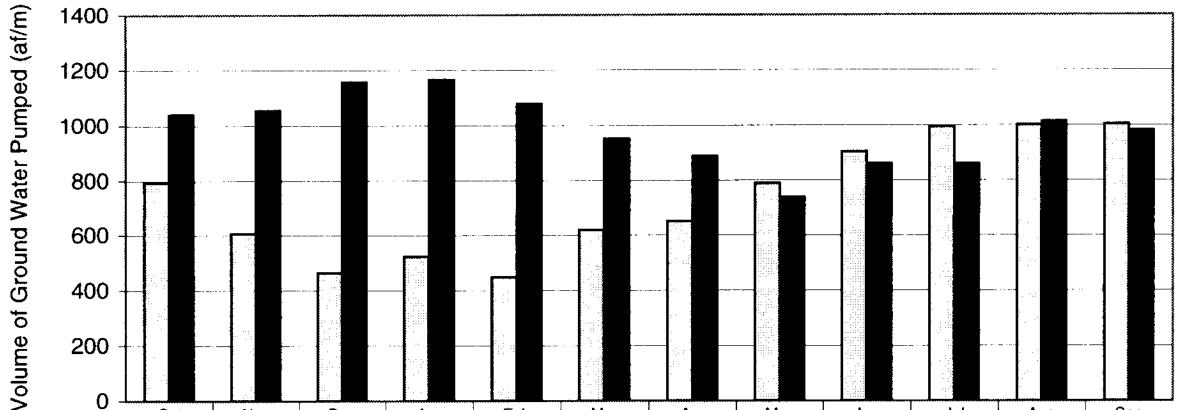
Existing and Proposed Well Locations within the Model Area

F3 Pumping during Normal Streamflow (af/m)



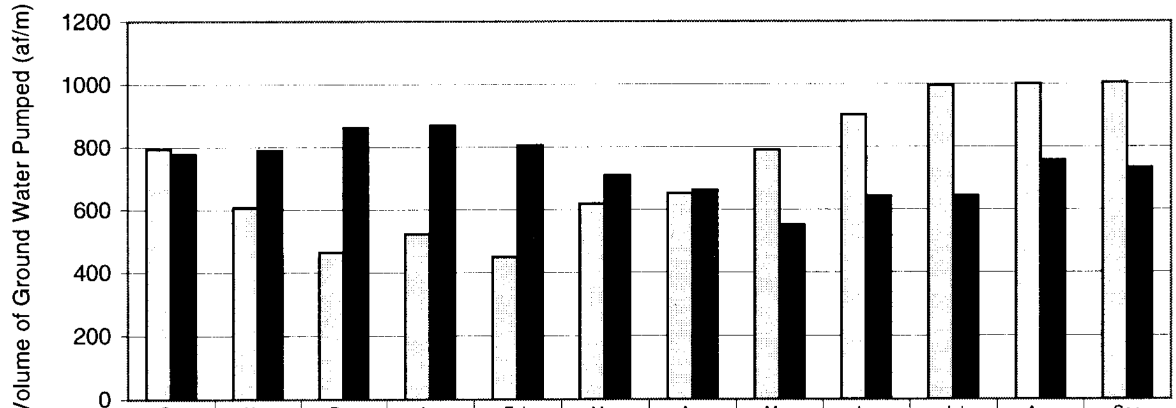
	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual Total
□ Buildout	793	607	465	523	449	619	651	789	903	994	1,001	1,005	8,800
■ F3	1,307	1,327	1,452	1,462	1,355	1,193	1,114	926	1,081	1,080	1,273	1,231	14,800

F3 Pumping during Second Below Normal Winter (af/m)



	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual Total
□ Buildout	793	607	465	523	449	619	651	789	903	994	1,001	1,005	8,800
■ F3-3K	1,042	1,058	1,158	1,165	1,080	951	888	739	862	861	1,015	981	11,800

F3 Pumping Following Third Below Normal Winter



	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual Total
□ Buildout	793	607	465	523	449	619	651	789	903	994	1,001	1,005	8,800
■ F3-6K	777	789	863	869	805	709	662	551	643	642	757	732	8,800

considered to minimize impacts to riparian habitat during dry years and increase diversions from the river. These reduced F3 production schedules will be discussed under different Alternatives in Chapter 7. The following table summarizes the average annual pumping volumes and number of wells for the pumping schedules studied.

TABLE 4-8
SUMMARY OF GROUND-WATER PRODUCTION SCHEDULES

Pumping Schedule	Average Annual Ground-Water Production (AFY)	# of Proposed Wells (pw)	Comment
F1	14,800	6	Increase proportional to historical monthly pumping; maximum production in summer months.
F2	14,800	6	Increase proportional to historical annual pumping; maximum production in winter months.
F3	14,050	6	Identical to F2 pumping with dry year management reduction of 3,000 AFY during second dry summer; reduction of 6,000 AFY during third dry summer until next year that normal stream flow occurs.
80% F3	11,240	4	80% of F3 production, installing 3 proposed wells in the Upper Ysidora and 1 proposed well in the Chappo.
90% F3	12,640	5	90% of F3 production, installing 3 proposed wells in the Upper Ysidora and 2 proposed well in the Chappo.
95 % F3	13,350	6	95% of F3 production, installing 4 proposed wells in the Upper Ysidora and 2 proposed well in the Chappo.

The distribution of ground-water pumping in the Upper Ysidora sub-basin has been established to maximize the yield from the Lower Santa Margarita River basin. Among other recommendation, the 1987 Basewide Study suggests increasing ground-water pumping to safe yield, purchasing imported water supplies to meet excess demand, interconnecting the north and south water systems, and protecting the Base's rights to waters of the Santa Margarita River. The pumping schedule outlined herein meets the recommendations of the earlier study. The excess water produced under the F2 and F3 pumping scenarios may be used to offset the purchase of imported water supplies through the establishment of a conjunctive use project with cooperating local water agencies. Or, the excess water produced during the winter period may be used throughout the entire Base following construction of a "cross-base" pipeline. In either situation, the F2 and F3 pumping scenarios maximize the safe-yield of the Lower Santa Margarita River basin while at the same time protecting all of the Base's valuable water rights.

Elimination of wastewater discharge to the river and oxidation pond infiltration shows decreases in evapotranspiration and streamflow out of the Lower Ysidora. As would be expected from the conceptual model, the Model predicts the impact to be greater during consecutive years of below normal streamflow and precipitation. The modeled effects of augmented flows under historical conditions of pumping and wastewater discharge shows an increase in stream leakance (water flow through the streambed recharging the ground-water aquifer) and an increase in streamflow out of the model area. The Model showed reduced evapotranspiration with F1's maximum pumping in summer months compared with F2's maximum pumping in the winter months, indicating less ground water available for riparian vegetation. By adding the management plan of reduced pumping during continued dry years with F3 pumping, this impact was further reduced. Table 4-9 quantifies and compares the results from these anticipated basin changes with the calibrated Model run (Run #1).

TABLE 4-9
ANTICIPATED BASIN CHANGES -- AVERAGE ANNUAL WATER BUDGET (AFY)

	Run # 1	Run # 2	Run # 3	Run # 5	Run # 6
SMR Streamflow:	H	H	A	A	A
Ground-Water Pumping:	H	H	H	F2	F3
Wastewater Release:	yes	no	yes	no	no
Inflow:					
Subsurface Underflow	850	840	840	1,460	1,420
Santa Margarita River Inflow	53,340	53,340	55,860	55,860	55,860
Lake O'Neill Spill and Release	1,990	1,990	1,990	1,990	1,990
Minor Tributary Drainages	2,120	2,120	2,120	2,120	2,120
Waste Water Discharge	2,030	0	2,030	0	0
Direct Precipitation	690	710	690	710	710
Total Inflow:	61,020	59,000	63,530	62,140	62,100
Outflow:					
Subsurface Underflow	240	230	240	200	220
Santa Margarita River Outflow	52,380	50,290	54,660	45,590	46,090
Ground-Water Pumping	5,560	5,560	5,560	14,800	14,050
Evapotranspiration	2,900	2,810	2,960	1,950	2,120
Diversions to Lake O'Neill	490	490	490	490	490
Total Outflow:	61,570	59,380	63,910	63,030	62,970
Net change in Storage:	790	380	380	890	870
Water Exchange within Model Domain					
Net Infiltration from Recharge Ponds	2,850	2,850	2,850	2,850	2,850
Net Stream Recharge to GW	4,280	4,330	4,370	11,910	11,380

4.11 MODEL RESULTS

Comparison of the Model's transient calibration to the observed data in the Lower Santa Margarita Basin between water years 1980 to 1999 shows that the Model is an excellent tool for simulating both surface and ground-water conditions on the Upper Ysidora, Chappo, and Lower Ysidora sub-basins. Due to the monthly time steps, the Model matches the seasonal variation in water levels throughout all three sub-basins. The Model is also able to closely match streamflow at the Ysidora gage, especially during low and medium flows during the last ten years of records. The minor discrepancies in the difference between observed and simulated streamflow at the Ysidora gage between 1980 and 1989 are likely due to the use of simulated streamflow during this period.

The model budget, accounting for the inflows and outflows to the ground-water aquifer, had an average annual percent discrepancy of ±0.02% (10 AFY), ranging from ±0.00% to ±0.05% (26 AFY). The Model was able to calculate a solution under highly variable streamflow, recharge rates, and pumping schedules. The calibrated model run shows the dominating influence of the Santa Margarita River on the ground-water basin. Stream leakance into the ground-water aquifer historically accounts for approximately 63% of the ground-water budget, and infiltration from recharge and oxidation ponds accounts for approximately 24% of the ground-water budget.

The high degree of calibration between observed and simulated data suggest that the Model is an excellent tool for defining the impacts to the ground-water basin due to future changed conditions. The Model's ability to account for changes in surface flow, wastewater influences, ground-water pumping, and other fluxes that affect the surface water and ground water in the three sub-basins suggest that the Model can be used for estimating any potential impacts due to future increases in diversions and ground-water pumping. Increased surface diversions from the Santa Margarita River and increased ground-water pumping from the ground-water basins, above baseline conditions, are identified in three out of four project alternatives in Chapter 7 for perfecting Permit 15000. The Model is used as a tool in each of these alternatives to define the impact of the project on the surface water, ground water, and riparian resources in each of the three sub-basins.

5.0 INVENTORY AND PERFORMANCE OF EXISTING FACILITIES

5.1 OVERVIEW OF EXISTING SYSTEM

The existing water diversion and production facilities located in the Santa Margarita River basin serve domestic, military, and agricultural water to the southern portion of Camp Pendleton. Some of the developed areas in the southern portion of the Base include the military headquarters, the United States Naval Hospital, the Marine Corps Air Station, and military family residential areas. The source of water supply serving these developments is ground water that is pumped from the Upper Ysidora, Chappo, and Lower Ysidora ground-water basins. An off-channel surface water spreading system, in operation since 1955, replenishes water pumped from the ground-water basins. The existing off-channel surface water spreading system, located west of the Naval Hospital, consists of a steel sheet pile diversion weir constructed across the Santa Margarita River and an earthen channel to convey river surface diversions to a series of five interconnected ground-water recharge ponds and Lake O'Neill. Details regarding the size, capacity and performance of the surface water diversion and ground-water recharge facilities are described in the following sections.

Historical operations of the diversion ditch, ground-water recharge ponds, and Lake O'Neill indicate that the diversion facilities are operated between October 1st and June 30th of each year. A surface water diversion license allows Camp Pendleton to divert water to the recharge ponds from October 1st of each year through June 30th while a pre-1914 vested water rights allows Camp Pendleton to divert surface water to Lake O'Neill between April 1st and October 31st of each year (see Chapter 2). An agreement between the Base and the FPUD, stipulated in Interlocutory Judgement 24, allows for diversions to Lake O'Neill between November 1st and March 31st of each year. Lake O'Neill is emptied generally during the month of November and subsequently filled from the first stream flow events of the winter season (personal communication Malloy, 1999). This changed operation of Lake O'Neill has been performed at the request and with the approval of the court-appointed watermaster.

Other factors that control the timing and rate of diversion throughout the year include inefficiencies due to sedimentation and clogging behind the diversion weir and limited surface flows available for diversion. Review of the data and analysis presented below show that the poor design and placement of the existing headgate and headwall have drastically reduced the amount of water that was diverted into either the ground-water recharge ponds or Lake O'Neill.

5.2 SIZE AND CAPACITY OF EXISTING FACILITIES

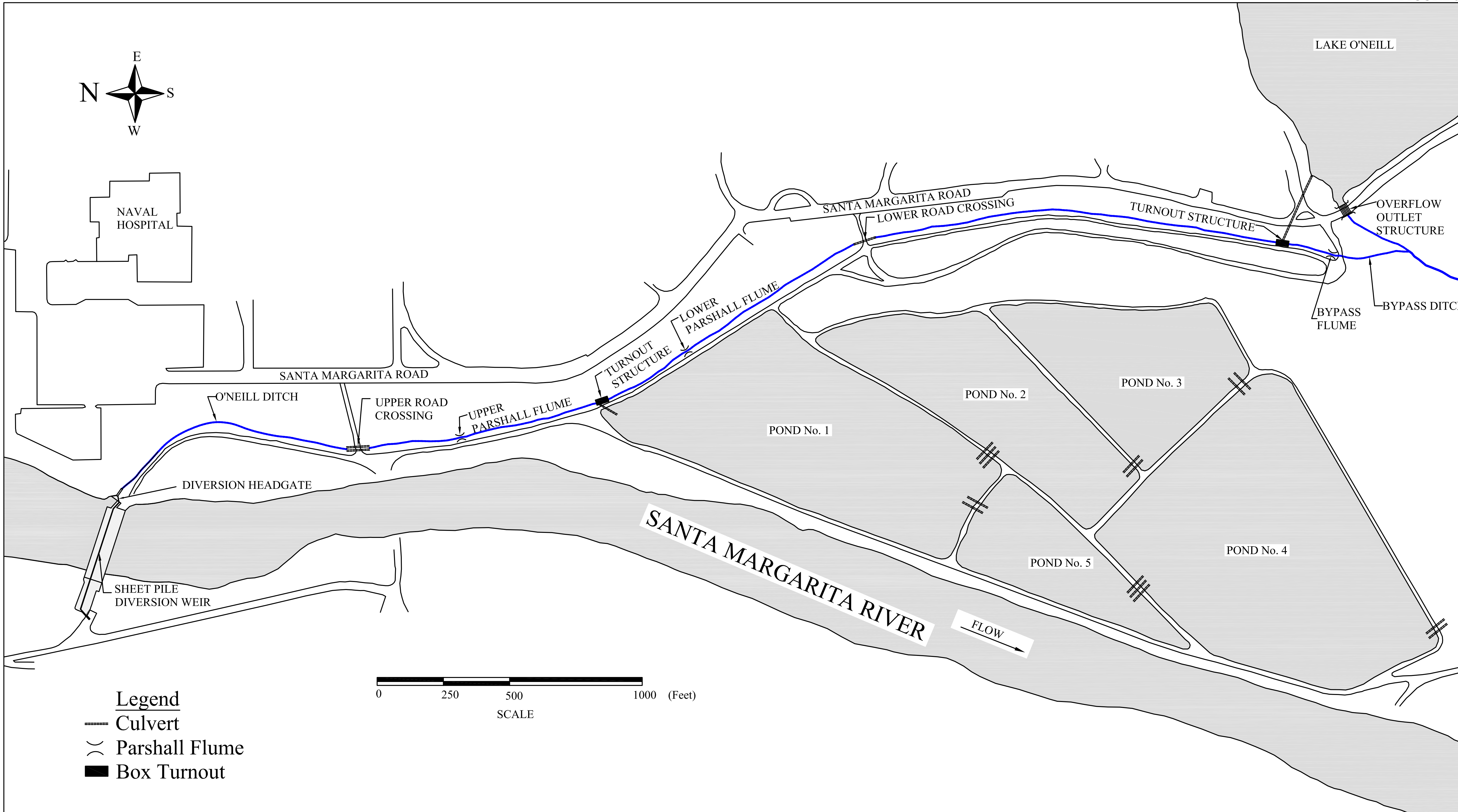
Information regarding the size and capacity of Camp Pendleton's surface water diversion and ground-water recharge facilities was obtained from Camp Pendleton's Office of Water Resources, field investigations conducted by Stetson Engineers, and previous studies and reports prepared by others. The general location of the existing diversion weir, ditch, and ground-water recharge facilities is shown on Figure 5-1. The diversion weir diverts surface flow of the Santa Margarita River into the O'Neill Ditch, which carries flows to both the ground-water recharge ponds and Lake O'Neill.

A sheet pile weir located in the Santa Margarita River channel allows water to be collected and diverted into the O'Neill ditch through an existing headgate and diversion structure located on the eastern bank of the river. The O'Neill Ditch is then able to divert water either to the five ground-water recharge ponds or Lake O'Neill, depending on the time of year, available supply, and required demand. During the diversion season, a series of control structures and measuring devices allows Base personnel to manage, control and measure the diversion to each of the different facilities. The operation of each of these facilities is discussed in the following sections of this report

5.2.1 SANTA MARGARITA RIVER DIVERSION STRUCTURE

The existing Santa Margarita River diversion structure was constructed in 1982 and consists of a steel sheet pile weir approximately 280 feet long. The sheet pile weir was constructed as a more permanent structure to replace previous rock weir designs that washed out during large flood events. According to the 1982 construction drawings, the sheet piles are 30 feet in length and were driven to a depth that fixed the weir crest elevation at 115.5 feet.

Water impounded behind the sheet pile weir may be diverted through a 60-inch by 48-inch (span by rise) slide gate mounted on a concrete headwall on the eastern bank of the river. The existing slide gate was constructed as a result of the Department of Public Work's 1970 plans to repair the flood damaged diversion system. The slide gate is manually operated to pass river diversions through a 45-foot long section of arch corrugated metal pipe (CMP) having dimensions of 65-inches by 40-inches. The invert elevation of the arch CMP at the entrance of the diversion is 112.1 feet according to the 1982 construction drawings. The capacity of the arch CMP diversion pipe is estimated to be 75 cubic feet per second (cfs) with a water surface elevation 3.4 feet (115.5 feet - 112.1 feet) above the pipe inlet. A generalized diagram of the existing Santa Margarita River diversion structure shown in plan view is represented in Figure 5-2.

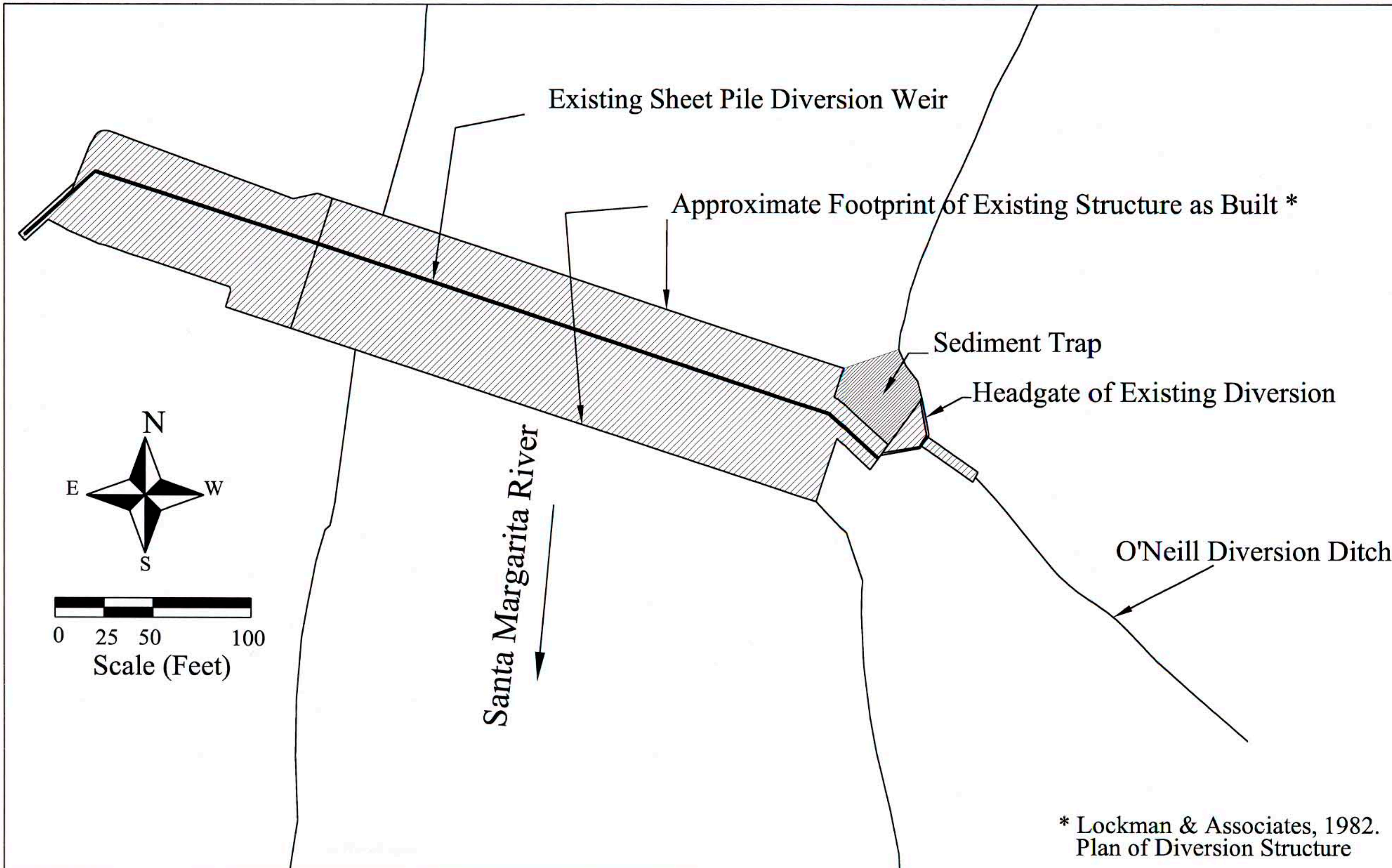


F:\DATA\1828\GUS\SITE_PLAN.DWG

STETSON ENGINEERS INC.

Diversion and Ground-Water Recharge System - Existing Facilities

Santa Margarita River



F:\Data\1828\Gus\divplan.dwg

STETSON ENGINEERS INC.

Footprint of Existing Diversion Structure

Santa Margarita River

FIGURE 5-2

5.2.2 DIVERSION CHANNEL (O'NEILL DITCH)

Water is diverted from the Santa Margarita River into an earthen channel (O'Neill Ditch) and delivered to either Lake O'Neill or to a series of five interconnected ground-water recharge ponds. The total length of O'Neill Ditch is approximately 5,100 feet from the head of the channel to Lake O'Neill. The channel consists of an upper reach which extends approximately 2,000 feet from the head of the channel to a concrete turnout structure that delivers water to the recharge ponds; and a lower reach which extends approximately 3,100 feet from the recharge pond turnout structure to the lake. The upper reach of the O'Neill Ditch has a base width of approximately 14 feet and a channel capacity ranging from 60 cfs to 230 cfs. The lower reach of O'Neill Ditch has a base width of 8 feet and a channel capacity ranging from 20 cfs to 124 cfs.

Two road crossings, two concrete Parshall flumes, and two turnout structures are located along O'Neill Ditch between the head of the diversion channel and the lake. A double culvert at the first road crossing in the upper reach of the ditch limits the diversion channel capacity to 60 cfs. The lake turnout structure at the end of the ditch is limited to 20 cfs. Surface flow reaching the end of the channel in excess of 20 cfs bypasses the lake turnout structure and is returned to the river below the ground-water recharge pond area. The structures located along O'Neill Ditch and their estimated capacities are described below.

5.2.2.1 Upper Road Crossing (Double Culvert)

Water diverted from the Santa Margarita River into the O'Neill Ditch passes through a double road culvert approximately 1,045 feet downstream from the head of the ditch. The double road culvert consists of one 36-inch CMP pipe and one 36-inch reinforced concrete pipe. The culverts are 78 feet in length and have a total estimated capacity of 60 cfs. The total capacity of the double culverts (60 cfs) represents the limiting capacity of the conveyance ditch.

As discussed above, the Upper Road Crossing limits the maximum diversion from the Santa Margarita River to 60 cfs, 15 cfs less than the design capacity of the weir and head gate. The location of this road crossing has presented operation and maintenance challenges in the past due to debris in the ditch clogging the culverts.

5.2.2.2 Upper Parshall Flume (5-foot)

The upper Parshall Flume is located approximately 1,523 feet downstream from the head of the ditch. The width of the flume is 5 feet across at the throat. The 5-foot flume has an estimated capacity of 139 cfs and is used to measure the total flow of Santa Margarita River diversions into O'Neill Ditch. Due to the slope of the canal, the maximum rate of flow through the upper flume is limited to approximately 60 cfs before backwater effects require water level measurements downstream of the flume (Malloy, 2000). The flume was originally constructed with concrete blocks and subsequently partially covered with a concrete mortar. In 1982, the original flume design was modified to provide a more accurate flow measurement. Modifications to the original design included raising the floor of the flume and relocating the stilling well pipes.

5.2.2.3 Turnout Structure to Ground-Water Recharge Ponds

A concrete turnout structure exists in the O'Neill Ditch approximately 2,000 feet downstream from the head of the ditch. The turnout structure consists of a 48-inch by 48-inch slide gate installed perpendicular to the channel flow for diverting water into the recharge pond system. A second 48-inch by 48-inch slide gate controls the flow into the pond system through a 79-inch by 49-inch arch CMP pipe. The estimated capacity of the turnout structure is 106 cfs.

5.2.2.4 Lower Parshall Flume (3-foot)

The lower Parshall Flume is located approximately 350 feet downstream from the turnout to the recharge ponds. The throat width of the lower Parshall flume is three feet. The three-foot flume has an estimated capacity of 82 cfs and is used to measure the total flow of Santa Margarita River diversions that bypass the recharge ponds and fill Lake O'Neill. The Water Resources Office subtracts the flow measured by the lower Parshall flume from the flow measured by the upper Parshall flume, to calculate diversions from O'Neill Ditch into the recharge pond system.

5.2.2.5 Lower Road Crossing

A second road crossing is located approximately 800 feet downstream from the lower Parshall flume. The road crossing consists of one 42-inch CMP pipe, 54-feet in length, with an estimated capacity of 39 cfs.

5.2.2.6 Turnout Structure to Lake O'Neill

A concrete turnout structure exists at the end of O'Neill Ditch that is used to deliver water to the lake at a limiting capacity of 20 cfs. Surplus water in O'Neill Ditch bypasses the lake and is returned to the Santa Margarita River below the ground-water recharge pond area. A Parshall Flume has recently been installed on the bypass ditch, downstream from the Lake O'Neill turnout, in order to measure the diversions that return directly to the river.

5.2.3 GROUND-WATER RECHARGE PONDS

The Santa Margarita River diversion system conveys water to either Lake O'Neill or to ground-water recharge ponds consisting of five interconnected ponds. The ground-water recharge pond system was constructed between 1955 and 1962 and Santa Margarita River diversions to the recharge ponds were first recorded in October 1960. The total surface area of the five-pond system is approximately 49 acres and the capacity of the ponds is estimated to be approximately 260 AF. Table 5-1 summarizes the five existing ground-water recharge ponds.

TABLE 5-1
CAPACITY OF EXISTING GROUND-WATER RECHARGE PONDS
CAMP PENDLETON MARINE CORPS BASE

POND NUMBER	SURFACE AREA (ACRES)	AVERAGE WATER DEPTH* (FEET)	VOLUME (AF)
1	13.9	3.2	44.5
2	7.0	6.1	42.7
3	7.0	8.4	58.8
4	16.5	5.4	89.1
5	4.7	5.1	24.0
Total	**49.1**		**259.1**

** Approximate average depth of existing ponds based on 1962 survey map.*

The capacity of each pond shown in the table above is based on estimating the average pond depths from information provided on a 1962 survey map, and multiplying the estimated average pond depths by their respective pond areas. The actual bottom elevation of each pond will likely vary from the 1962 survey map due to the maintenance practice of scraping and disking the soil in the ponds as well as sediment accumulating in the ponds due to sediment laden surface water. The exact dates that maintenance has been performed on the ponds have not been recorded and were not available for review.

The recharge ponds are formed by sand levees approximately 10-feet in height and are interconnected by buried non-gated CMP pipes that pass flow, uncontrolled, between recharge ponds. The locations of the CMP pipes that are currently available to pass water between recharge ponds were previously shown in Figure 5-1. The flow rate through each CMP varies depending on water levels in each pond and diversion rate from the Santa Margarita River. Specific flow rates between each pond were not identified since they do not represent a restriction in the capacity of the system.

Under the current recharge pond operations, water is diverted from O'Neill Ditch into the recharge pond system through a single 79-inch by 49-inch CMP pipe at the head of Pond No. 1. When the water level in Pond No. 1 rises to the pond's outlet pipe invert elevations, flow passes ("spills") from Pond No. 1 into either Pond Nos. 2 or 5. The pipe invert elevations from Pond No. 1 to Pond No. 2 are slightly lower (3-4 inches) than the pipe invert elevations from Pond No. 1 to Pond No. 5, therefore, water first spills from Pond No. 1 into Pond No 2 before spilling into Pond No. 5.

Water filling above the invert elevation of the outlet pipes from Pond No. 2 spills into Pond No. 3 and water filling above the outlet pipes from Pond No. 3 spills into Pond No. 4. Similarly, water filling above the invert elevation of the outlet pipes from Pond No. 5 spills into Pond No. 4. At the lower end of Pond No. 4 (the last pond in the system), two 30-inch CMP pipes exist to return spills from Pond No. 4 to the river. Based on the recollection of the Office of Water Resources staff, Pond No. 4 only spilled in March of 1983 and has only filled twice since that time (Malloy, 2000).

5.2.4 LAKE O'NEILL

Lake O'Neill is a man-made reservoir formed by an earthen levee located on Fallbrook Creek, a tributary to the Santa Margarita River. The lake is filled primarily from Santa Margarita River diversions conveyed to the lake through the O'Neill Ditch. The capacity of the Lake is approximately 1,200 AF. The levee that impounds water in Lake O'Neill and the diversion canal from the river to the lake were constructed in 1883 as part of a farm system (Leedshill-Herkenhoff, 1988). The water rights associated with Lake O'Neill carry a priority date of 1883 and stipulate a maximum diversion rate to the lake at 20 cfs, not to exceed 1,500 AF (including evaporation losses) annually.

Diversions from O'Neill Ditch to the lake are made through a concrete turnout structure and a 24-inch reinforced concrete pipe located at the lower end of O'Neill Ditch. Adjacent to the 24-inch pipe that fills the lake, is a concrete overflow outlet structure with four 60-inch reinforced concrete pipes (RCP). The overflow outlet structure returns reservoir spills to a ditch that eventually drains back to the river. Lake water can also be returned to the river through an outlet pipe located in the southern corner of the lake. Table 5-2 summarizes the exiting facilities of the Santa Margarita River diversion and ground-water recharge system.

5.3 OVERALL PERFORMANCE OF EXISTING SYSTEM

The overall performance of the existing Santa Margarita River diversion and ground-water recharge system is reduced by the following system components:

1) River Diversion Weir (functional inefficiencies)
2) River Diversion Inlet Headwall (location inefficiency)
3) O'Neill Ditch (conveyance capacity restrictions)
4) Recharge Ponds (lack of water level control and flow measurement)
5) Recharge Ponds (cleaning and maintenance inefficiencies)

Each of these inefficiencies and restrictions that limits surface diversions from the Santa Margarita River is discussed in detail below. Maintenance and repair projects that solve the engineering flaws associated with the originally constructed diversion headwall and recharge pond operations are presented in Chapter 6. As explicitly stated in this chapter and the following Maintenance and Repair chapter, these components of the system must be replaced in order to achieve the original design capacity of the facilities.

5.3.1 RIVER DIVERSION WEIR (FUNCTIONAL INEFFICIENCIES)

The existing sheet pile weir design results in large volumes of sediment accumulation behind the weir, particularly during high flow events. Sediment accumulation behind the weir reduces diversion capacity from the river and increases maintenance costs. Figure 5-3 is a photograph of the existing sheet pile diversion weir taken during June 2000. As seen in this figure, sediment accumulates behind the weir up to the level of the weir crest. In 1994, an estimated 33,000 cubic yards of sediment was removed behind the river diversion weir. Designs for replacing the inefficient diversion weir are discussed in Chapter 7.

1828-324Photo4.ai

STETSON ENGINEERS INC.

Santa Margarita River Sheet Pile Diversion Weir
Viewed West
June 26, 2000

FIGURE 5-3

TABLE 5-2
SUMMARY OF EXISTING FACILITIES
SANTA MARGARITA RIVER DIVERSION AND GROUND-WATER RECHARGE SYSTEM

FAC ILITY	DESCRIPTION	CAPACITY
CONVEYANCE FACILITY		
River Diversion Dam	Steel sheet pile weir, 283 feet in length	----
River Diversion Inlet	60-inch × 48-inch slide gate mounted on concrete headwall 65-inch × 40-inch × 45-feet arch corrugated metal pipe	75 cfs
O'NEILL DITCH		
Earthen Channel	Unlined earth ditch approximately 5,100 feet in length	73-174 cfs
RoadCrossing (double culvert)	36-inch corrugated metal pipe and 36-inch reinforced concrete pipe	60 cfs
Upper Flume	5-foot Parshall flume; concrete block and concrete lined	105 cfs
Recharge Pond Turnout Structure	Concrete turnout structure with two 48-inch slide gates	82 cfs
Lower Flume	3-foot Parshall flume; concrete block and concrete lined	62 cfs
Road Crossing (single culvert)	42-inch corrugated metal pipe	39 cfs
Lake O'Neill Turnout Structure	Concrete turnout structure with 24-inch slide gate	20 cfs
STORAGE FACILITIES		
Ground-Water Recharge Ponds	5 ground-water recharge ponds totaling 49 acres	260 AF
Lake O’Neill	Lake formed by earthen levee	1,200 AF

*Note: Capacity of conveyance facilities calculated based on river water levels equal to crest height of the sheet pile weir.

Field inspection of the dam site indicates that sediment accumulates behind the weir following years of high flow events. Various field trips by Mr. Stephen Reich of Stetson Engineers between 1995 and 2000 indicated that sediment remained trapped behind the weir throughout the entire five-year period. During the same period, annual maintenance by the Facility and Maintenance Department at Camp Pendleton has included the removal of sediment above the weir and from in front of the headgate. This maintenance has been performed under 404 Permit that allows for the removal of sediment 50 feet upstream of the weir over the entire width of the river. Historical records from the period 1980 to 1999 indicate an average annual diversion of 2.150 AFY, well before the intended design capacity of the system.

5.3.2 RIVER DIVERSION INLET HEADWALL (LOCATION INEFFICIENCY)

The slide gate headwall of the river diversion structure was constructed approximately 40 feet from the low flow channel of the Santa Margarita River and is recessed into the topography at the site, contributing to sediment accumulations in front of the diversion pipe. Sediment accumulation in front of the diversion pipe reduces diversion capacity from the river, reduces capacity in the diversion channel, and increases operation and maintenance costs. Notches were installed in the sheet pile diversion dam near the diversion headwall during the mid-1990's in an effort to reduce sediment accumulations behind the weir. To some extent, the weir notches were effective, however, only in the immediate vicinity of the notches. Figure 5-4 is a photograph showing the river diversion inlet headwall located on the east bank of the river, the portion of the sheet pier diversion weir that was previously notched, and the sediment accumulation problem. Designs for replacing the poorly positioned headwall are discussed in the Maintenance and Repair Chapter.

Similar to the design of the sheet pile weir, the location and design of the headwall and headgate contribute to the inefficiency of Camp Pendleton's diversion facilities. Figure 5-5 is a graph showing the relationship between theoretical diversions, based on a 100 cfs maximum diversion rate, and actual diversions from 1960 to 1999. The figure indicates that diversions were most efficient during low flow years (early 1960s) and years following initial construction (1960 and 1982), but were grossly inefficient during high flow years such as 1993, 1995, and 1998. The large discrepancy between theoretical diversions and historical diversions can be attributed to the functional inefficiency of both the sheet pile weir and headwall location.

1828-324Photo7.ai

STETSON ENGINEERS INC.

Santa Margarita River Diversion Weir and Headgate
East Bank of River Viewed Downstream
June 26, 2000

FIGURE 5-4

Total Diversions from the Santa Margarita River
Historical Versus Calculated (1961 to 1999)

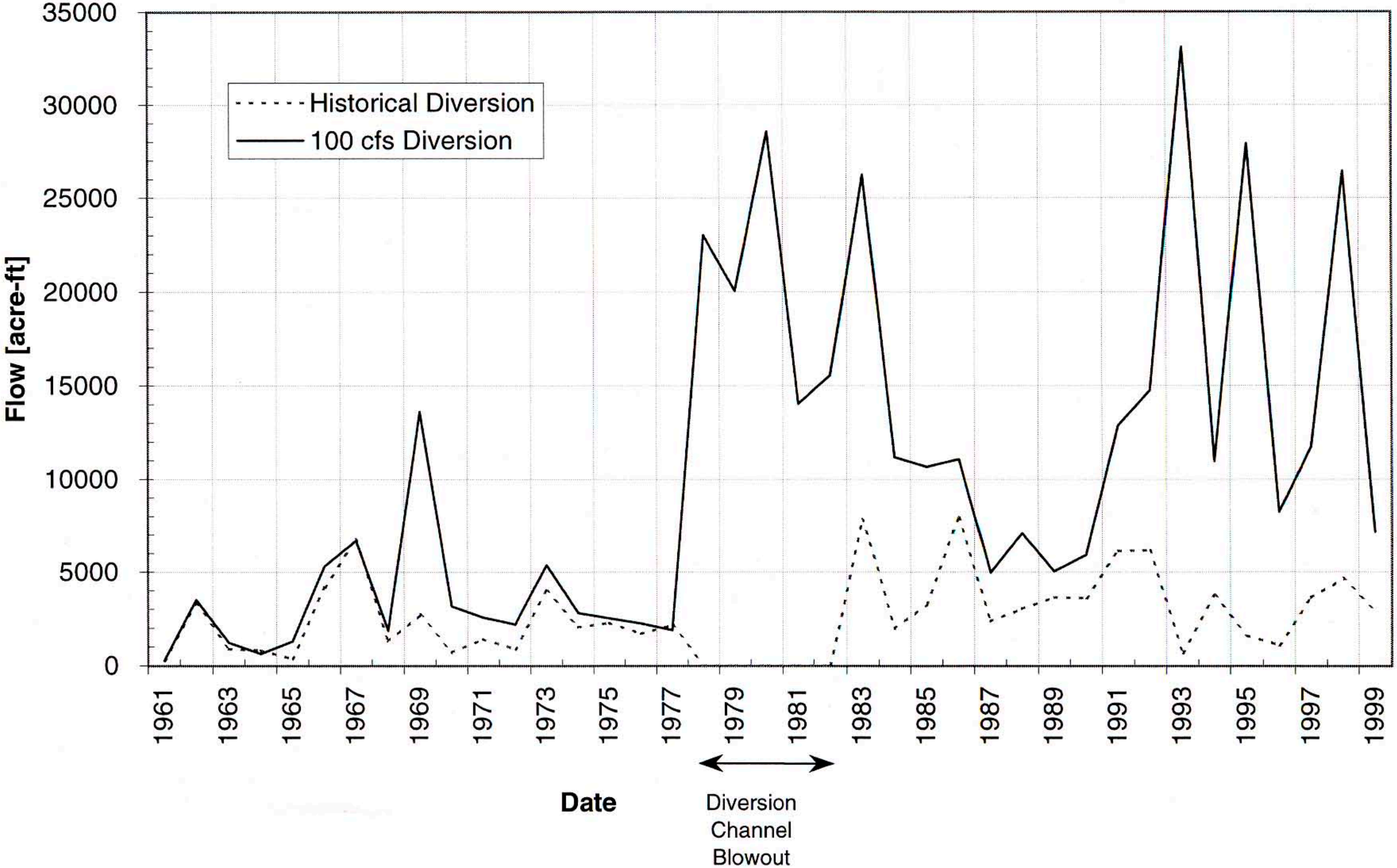


FIGURE 5-5

5.3.3 O'NEILL DITCH (CONVEYANCE CAPACITY RESTRICTIONS)

The existing capacity of O'Neill ditch is restricted to an estimated flow rate of 60 cfs at the location of the first road crossing in the upper reach of the ditch. Figure 5-6 shows the road crossing where flow in the diversion channel is restricted. Sediment accumulation, vegetative growth, and other debris also reduce channel capacity. In the portion of O'Neill Ditch below the turnout to the recharge ponds, the estimated ditch capacity ranges from 78 to 93 cfs. Ditch improvements are not necessary below the recharge pond turnout structure because the maximum allowable diversion rate to Lake O'Neill is 20 cfs. Improvements to the conveyance capacity of O'Neill Ditch are discussed in Chapter 7.

5.3.4 RECHARGE PONDS (LACK OF WATER LEVEL CONTROL AND FLOW MEASUREMENT)

The overall performance of the ground-water recharge system is reduced by operational inefficiencies related to lack of pond water level control and the inability to measure flow between ponds. Under the current conditions, water level control is limited to manual placement of plywood boards across outlet pipes of the recharge ponds. Without the plywood boards, the water level in the ponds is fixed by the invert elevations of each pond's set of outlet culverts. Culvert flow between ponds often occurs under submerged or partially submerged conditions at both the inlet and outlet ends, which causes backwater effects between the ponds. Backwater effects also occur between Pond No. 1 and the turnout from O'Neill Ditch.

According to the 1962 survey drawing of the recharge ponds, the invert elevation of the culverts draining Pond No. 1 is only 1.15 feet below the floor elevation of the recharge pond turnout structure in O'Neill Ditch. This finding indicates that whenever the water level in Pond No. 1 rises higher than 1.15 feet above the invert elevation of the pond's outlet pipes, water will back up into O'Neill ditch, reducing conveyance capacity and potentially submerging or partially submerging the outlet of the upper Parshall flume. Repairs associated with constructing facilities to control recharge pond water levels and measure flow between ponds are discussed in Chapter 6 of this report.

1828-324Photo6.ai

STETSON ENGINEERS INC.

Upper Road Crossing on O'Neill Ditch
Viewed Downstream
February 2000

FIGURE 5-6

5.3.5 RECHARGE PONDS (CLEANING AND MAINTENANCE INEFFICIENCIES)

The overall performance of the ground-water recharge system is reduced by inefficiencies related to pond cleaning and maintenance. Common to most artificial ground-water recharge projects is the on-going problem of maintaining infiltration rates. Under Camp Pendleton's current recharge pond operation, algae growth, fine sediment, and vegetation growth within the wetted perimeter of the recharge ponds is disked into the bottom of the recharge ponds, effectively reducing infiltration capacity. Figure 5-7 is a photograph taken from the bank of recharge Pond No.1 towards the pond's outlet culverts to Pond No. 2. As seen in this photograph, a significant film of clogging algae forms along the banks of the recharge ponds. This clogging layer was also observed in the bottom of Pond Nos. 1 and 2 after they had drained in June 2000.

Infiltration rates within a recharge pond will decrease over time due to accumulations of sediment suspended in the water diverted into the pond. Algae and bacteria growth in the water and on the bottoms of a recharge pond can also contribute to clogging and reduced infiltration rates. Bauer (1988) suggests that clogging materials consisting of inorganic matter must be removed by scraping, raking, or other procedures. Disking the clogging material into the soil gives temporary improvements; however, the entire soil layer to the depth of disking may have to be removed because of the accumulated fines. If the clogging material is sludge, bacteria or algae, drying can improve infiltration rates and reduce cleaning cycles to once or twice a year.

Permeable soils typically have hydraulic conductivity rates in the range of three-feet per day (fine loamy sands) to 30-feet per day and higher (sands, and sandy gravels). Actual infiltration rates during flooding vary from about one foot per day to ten feet per day, due to clogging layers above the underlying soil materials (Bauer, 1988). Schmidt (1977) reported Camp Pendleton's coarse, sandy recharge basins as having infiltration rates up to five inches per hour (ten feet per day). Drilling and percolation tests conducted by Almgren & Koptionak (1990) in an area adjacent to the existing recharge ponds on Camp Pendleton produced percolation rates greater than 20 feet per day.

Stetson Engineers conducted an infiltration rate study of Camp Pendleton's recharge Pond Nos. 1 and 2 during calendar year 2000. Stetson Engineers' investigation of the recharge ponds determined infiltration rates as high as 1.4 feet per day at the beginning of the diversion season (February) and as low as 0.2 feet per day at the end of the diversion season (June). Improved infiltration rates in Camp Pendleton's ground-water recharge ponds may be achieved and maintained by establishing suitable operations, monitoring, cleaning, and maintenance programs for the ponds (Chapter 6).

1828-324Photo1.ai

STETSON ENGINEERS INC.

Ground-water Recharge Pond No. 1
Viewed Southeast Towards Outlet Culverts to Pond No. 2
June 26, 2000

FIGURE 5-7

5.3.5.1 Infiltration Rate Study Conducted by Stetson Engineers

On February 15 and 16, 2000, Stetson Engineers installed water level recording equipment in Pond Nos. 1 and 2. The equipment was installed to obtain a continuous record of pond water levels that could be used to calculate infiltration rates. The water level recording equipment was only installed in Pond Nos. 1 and 2. Additionally, it was assumed that the infiltration rate of Pond No. 2 would provide a preliminary and reasonable estimate for the infiltration rates in Pond Nos. 3, 4, and 5.

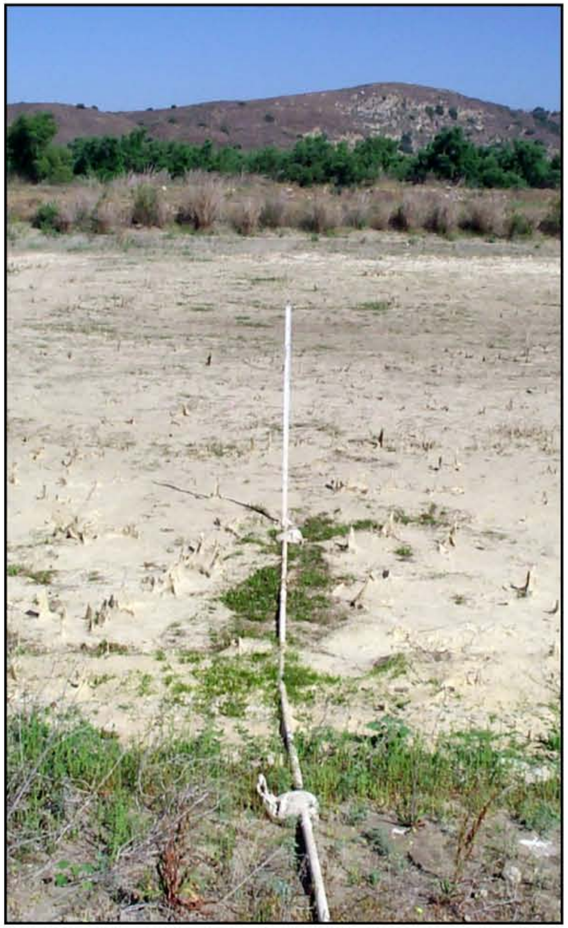
The equipment installed in Pond No. 1 consisted of a Global Water submersible pressure transducer with temperature compensation, connected as a single unit by cable to a battery powered data logger. The pressure transducer was fixed to a flat plate and installed in Pond No. 1 near the outlet pipes to Pond Nos. 2 and 5. A locking concrete box was placed in the pond levee to protect the data logger. The cable from the data logger to the submersible pressure transducer was inserted in 2-inch diameter PVC pipe for protection.

The equipment installed in Pond No. 2 was identical to that installed in Pond No. 1 with the exception that a water level staff gage was also installed in Pond No. 2 to provide a comparison with the water levels recorded by the pressure sensor. The Pond No. 2 water level recording equipment was also installed near the pond outlet where the expected water levels were deepest and where flow from Pond No. 2 passes into Pond No. 3. Figure 5-8 shows a photograph of recharge Pond No. 2 when Stetson Engineers installed the water level recording equipment (February 15, 2000) and a photograph at the same location at the end of the diversion season after the pond had drained (June 27, 2000). This photograph shows an accumulated layer of algae, vegetation, and other clogging material on the pond bottom left behind after the pond had drained.

The data loggers were programmed to record pond water levels in 15-minute intervals. Stetson Engineers utilized and laptop computer to periodically download data from the water level recording equipment and returned the data to the office for processing. A plot of the pond water level data that was collected for the percolation rate study is shown in Figure 5-9.

The data collected from the infiltration rate study was useful in calculating the infiltration rate of Pond Nos. 1 and 2 at the beginning of the season when the ponds were filling, and at the end of the season (June) when the ponds were receding. During these periods, infiltration rates could be calculated from the measured water levels because spill from one pond to another was not occurring. During the middle of the season, Pond No. 1 was spilling to both Pond Nos. 2 and 5 and flow passing uncontrolled between ponds was not measured. Similarly, uncontrolled flow from Pond No. 2 to Pond No. 3 occurred during the middle of the season and was not measured.

Water Level Recording Equipment
and Staff Gage Installation
February 15, 2000

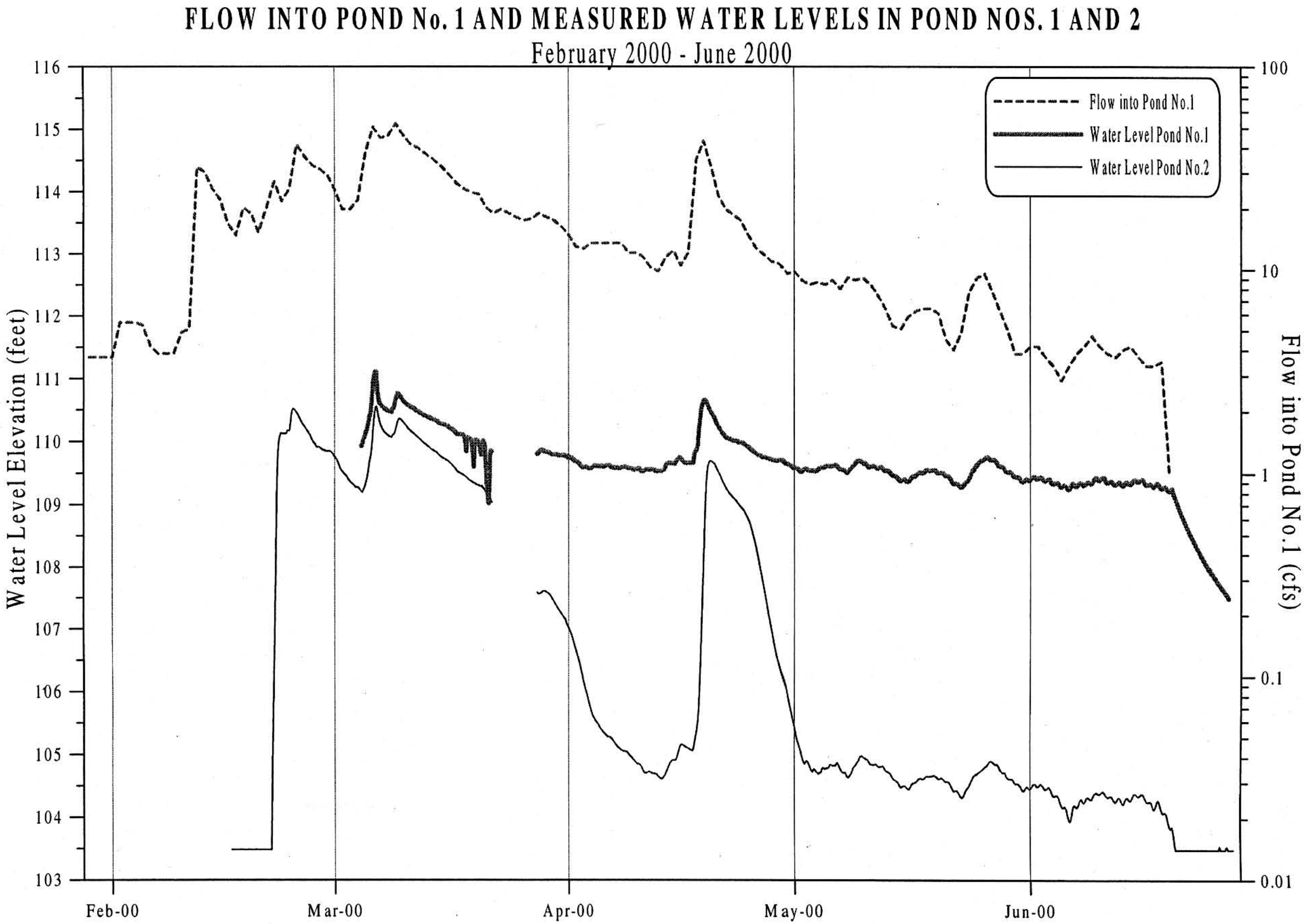


Algae and Other
Clogging Material
June 27, 2000

STETSON
ENGINEERS INC.

Ground-water Recharge Pond No. 2

FIGURE 5-8



F:\Data\1828\EngineeringFeasibilityStudy\RechargePonds\2000pondlevels1.grf 11/14/00

FIGURE 5-9

The data collected for this study indicate that the infiltration rate of Pond No. 1 was approximately 1.4 feet per day during February 2000 when the pond was filling, and diminished to approximately 0.2 feet per day by June 2000. The end-of-season infiltration rate for Pond No. 2 was calculated to be 0.9 feet per day.

The data collected from the fieldwork was used to calibrate the reservoir operations and ground-water models described in Chapter 4. As discussed later in this report, a monitoring program should be established to measure the flow from pond to pond and maximize the amount of water that may be infiltrated in the ground-water basin. Based on the current pond maintenance program ground-water infiltration rates have drastically been reduced from estimates made by other consultants at earlier dates. This reduction is likely due to the practice of disking and reworking organic and inorganic particulates in the soil and not following best management practices that enhance recharge infiltration rates.

6.0 MAINTENANCE AND REPAIR

Historical operation and maintenance tasks that have been completed behind the diversion weir, in front of the headwall and headgate, and in the recharge ponds suggest the facilities that exist today were not properly designed to meet the diversion goals of the original project. Existing water rights allow for 1,500 AF of water to be diverted to Lake O'Neill and 4,000 AF of water to be diverted to the ground-water recharge ponds annually. In order to address future alternatives that will allow for the development of the unused portion of Permit 15000, maintenance and repair of the existing facilities are required. The following chapter describes the maintenance and repair tasks that will provide increased efficiency of the diversion facilities, resulting in the ability to fully exercise the Base's existing water rights. The proposed maintenance and repair tasks suggested in the following sections service previously authorized facilities and allow for only minor deviations necessary to make repair to existing streamflow diversion and recharge facilities.

The performance review of the existing diversion facilities has shown that the system has performed below its original intended design capacity. This fact is emphasized by the large amount of sediment that accumulates behind the sheet pile weir and in front of the existing headwall and headgate, and by the performance of the recharge ponds. Based on actual records, diversions to the recharge ponds have averaged 2,870 AF over the last 20 years and 2,150 AF over the entire life of the project (1960 to 1999). The original license allows for a maximum of 4,000 AFY to be diverted through O'Neill Ditch, twice as much as has been historically diverted. Despite these diversion difficulties, Camp Pendleton has been able to fully protect its legal right to divert water under the license through water conservation and reclamation measures. The diversion facilities have often been in a state of disrepair or clogged with sediment, resulting in minimal diversions even during the wettest of years.

Beyond the ability to divert surface flow to Lake O'Neill, the original intent of the 1960 recharge project was to exercise Camp Pendleton's full appropriative water right of 4,000 AFY as allowed under license 2147A. Under this license, Camp Pendleton is allowed to divert surface water from the Santa Margarita River and store underground using ground-water recharge ponds located in the Upper Ysidora sub-basin. Historical records and actual diversions show that the diversion facilities built to meet the original project goals were inadequate or poorly designed. Improvements in technology and a better understanding of the sediment process on the Santa Margarita River provide a means to correct the original design flaws that prevent Camp Pendleton from diverting the maximum amount of water allowed under its license.

The pre-1914 water right that allows Camp Pendleton to divert up to 1,500 AFY (including evaporation) from the Santa Margarita River to Lake O'Neill has also been impeded due to the poor design and inefficiency of the existing diversion facilities. Due to sediment accumulations behind the diversion weir and in front of the headwall and headgate, Camp Pendleton has been prevented from diverting its full pre-1914 water right between April 1st and October 31st of each year. Field inspection of the diversion facilities in July 2000 revealed that the headgate invert elevation to the O'Neill Ditch was approximately two feet above the low flow channel of the Santa Margarita River, making diversion to Lake O'Neill impossible due to sediment accumulation.

The following section discusses the minimum maintenance and repair projects that are required to return the system back to the capacity it was originally designed for and allow Camp Pendleton to fully exercise its appropriative rights, including its pre-1914 water right. Three projects described below are required to return the diversion system to its original design capacity, including replacement of the headwall and headgate, scraping fine sediment from the existing recharge ponds, and installation of control and monitoring devices between ponds. Completion of the design and construction of these maintenance and repair projects is used as a baseline from which to compare to future alternative projects. Alternative projects have been designed to enhance and expand the diversion capabilities of the diversion system.

6.1 PERFORMANCE OF EXISTING FACILITIES

Chapter 5 discusses the inventory and performance of the existing diversion and recharge facilities located in the Upper Ysidora sub-basin, Camp Pendleton. In summary, surface diversion from the Santa Margarita River has been severely impeded, in part, due to a poorly positioned headwall and headgate structure. The present location of this structure allows for large amounts of sediment to accumulate in front of the headgate, restricting diversion capabilities to O'Neill Ditch. In order to improve the efficiency of the headgate, the headwall must be positioned near the low flow channel of the river. The overall performance of the existing diversion facilities is highlighted in Table 6-1.

TABLE 6-1
AVERAGE ANNUAL DIVERSIONS FROM THE SANTA MARGARITA RIVER

Period of Record (Water Year)	Diversion to Recharge Ponds (AF)	Diversion to Lake O'Neill (AF)	Total Diversion (AF)
1961 to 1979	1,460	400	1,860
1980 to 1999	2,870	520	3,390
Overall	2,150	460	2,620

Review of the table and raw data shows that the existing facilities were not able to fulfill the basic requirement of 4,000 AFY to the recharge ponds or 1,500 AFY to Lake O'Neill. In many cases, during the wettest of years including 1978, 1980, 1993, and 1995, the total annual diversion to the recharge ponds did not exceed 800 AF, and in many years was zero due sediment accumulation in the headgate. In addition to the accumulation of sediment between the headgate and the stream channel causing a restriction in diversion capabilities, high flood flow events have also washed-out the headgate structure resulting in zero diversions during some years.

Similar data suggests that the existing diversion facilities did not meet their design goals on a month-to-month basis, or in the time allotted under the existing pre-1914 water right. Table 6-2 outlines the historical amount of water diverted and the frequency that diversions occurred from the Santa Margarita River to Lake O'Neill, as permitted under the pre-1914 vested water right.

TABLE 6-2
WATER DIVERTED TO LAKE O'NEILL DURING THE PRE-1914 WATER RIGHT ALLOCATED PERIOD FROM APRIL 1ST TO OCTOBER 31ST
(1961 THROUGH 1999)

Month	Total Water Diverted (AF)	Number of Months Diversion = 0	Number of Months Diversion > 0
April	1,630	28	10
May	470	31	7
June	40	37	1
July	0	38	0
August	0	38	0
September	55	37	1
October	0	38	0
1961-1999 Total	2,195	247	19

Review of the table shows the design of the diversion structure prevents Camp Pendleton from diverting its full water right as permitted in the pre-1914 water right. Due to the pre-1914 water right having the most senior priority on the Santa Margarita River and the ability of Camp Pendleton to call for deliveries under this right, the total streamflow that should have been diverted under this water right would have amounted to 57,000 AF of water from the period 1961 through 1999. The records of diversions to the recharge ponds, as well as diversions to Lake O'Neill, clearly show that diversions are diminished because of sediment accumulation and build-up, which is caused by the distance between the low flow channel and the headgate.

The historical diversion data shows that the existing diversion facilities did not meet the basic diversion requirements intended in the original design the project due to inefficiencies in the facilities. Not included in the historical diversion data is recharge to the ground-water aquifer from both streambed infiltration and sewage effluent. Both the appropriative license and pre-1914 water right have been adequately exercised during the historical period when these additional sources are accounted for when determining total diversion under each water right.

6.2 MANDATORY MAINTENANCE AND REPAIR PROJECTS

Maintenance and repair projects that should have historically been completed to allow Camp Pendleton to divert its full appropriative and vested water right include replacement of existing headwall and headgate, scraping fine sediment from the existing recharge ponds, and installation of control and monitoring devices between ponds. The design and construction of these facilities will allow the Base to divert its full allotment of Santa Margarita River water.

Review of the historical data suggests that facilities performed at their original design capacity during the years immediately following reconstruction or maintenance activities. In 1983 and 1994, following reconstruction and maintenance of the facilities, 7,600 AF and 3,800 AF were diverted to the ponds, respectively. The subsequent years following major maintenance repair work showed a decrease in the amount of water diverted to the ponds. The maintenance and repair projects outlined in the following section will allow the diversion facilities to operate at their original intended capacities.

6.2.1 REPLACEMENT OF EXISTING HEADWALL AND HEADGATE

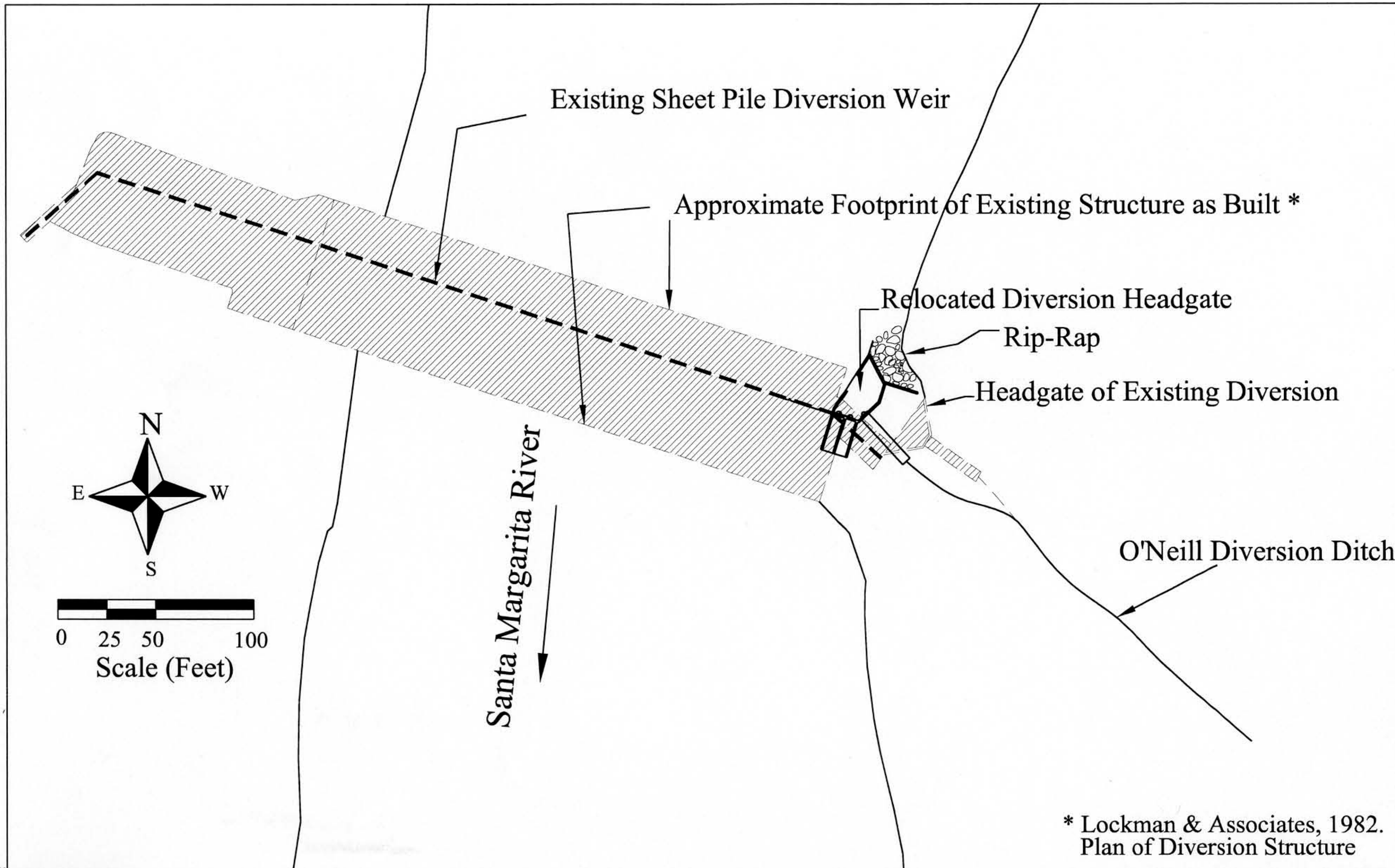
The purpose of this project is to relocate the existing headwall and headgate in order to improve the efficiency of the diversion structure. The replacement headwall and headgate will also include sluice gates located near the active low-flow channel of the Santa Margarita River. The approximate cost of replacing this facility is $171,000, not including contingencies and engineering. The impact of this project will minimize operation and maintenance expenses while meeting the requirements of the Base’s water rights.

The existing location of the headwall and headgate with respect to the diversion weir and the Santa Margarita River was previously shown in Figure 5-2. Water impounded behind the sheet pile weir is diverted through a 60-inch by 48-inch (span by rise) slide gate mounted on a concrete headwall on the eastern bank of the river. The slide gate is manually operated to pass river diversions through a 45-foot long section of arch corrugated metal pipe (CMP) having dimensions of 64-inches by 40-inches.

The existing slide gate headwall of the river diversion structure was constructed approximately 40 feet from the low flow channel of the Santa Margarita River and is recessed into the topography at the site, contributing to sediment accumulations in front of the diversion pipe. Sediment accumulation in front of the diversion pipe reduces diversion capacity from the river, reduces capacity in the diversion channel, and increases operation and maintenance costs.

The replacement project will relocate the existing headwall and headgate and install sluice gates at the side of the existing sheet piling diversion dam near the east abutment, as shown on Figure 6-1. Sluice gates located adjacent to the headgate will help prevent the sediments from accumulating in front of the headgate and restricting the diversions. The existing headgate will be relocated adjacent to the proposed sluice gates. Figure 6-2 provides a design of a sluice gate and headgate and replacement structure that will prevent large amounts of sediment from accumulating in front of the headgate.

The estimated capital cost for the sluice gate and headwall replacement project is $171,000 based on the cost of the foundation, headwall, headgate, riprap, and sluice gates that will be supplied by Waterman and installed by a local contractor. The construction of the foundation and headwall cost was based on 120 cubic yards of reinforced concrete at $500 per cubic yard. The removal of a section of existing sheet pile diversion dam was estimated at $10,000. Additional costs for contingencies and unlisted items ($40,000) planning, engineering and design ($30,000) and project management and administration ($20,000) are not included in the estimated cost to relocate the headwall ($171,000). A summary of the cost is shown on Table 6-3.



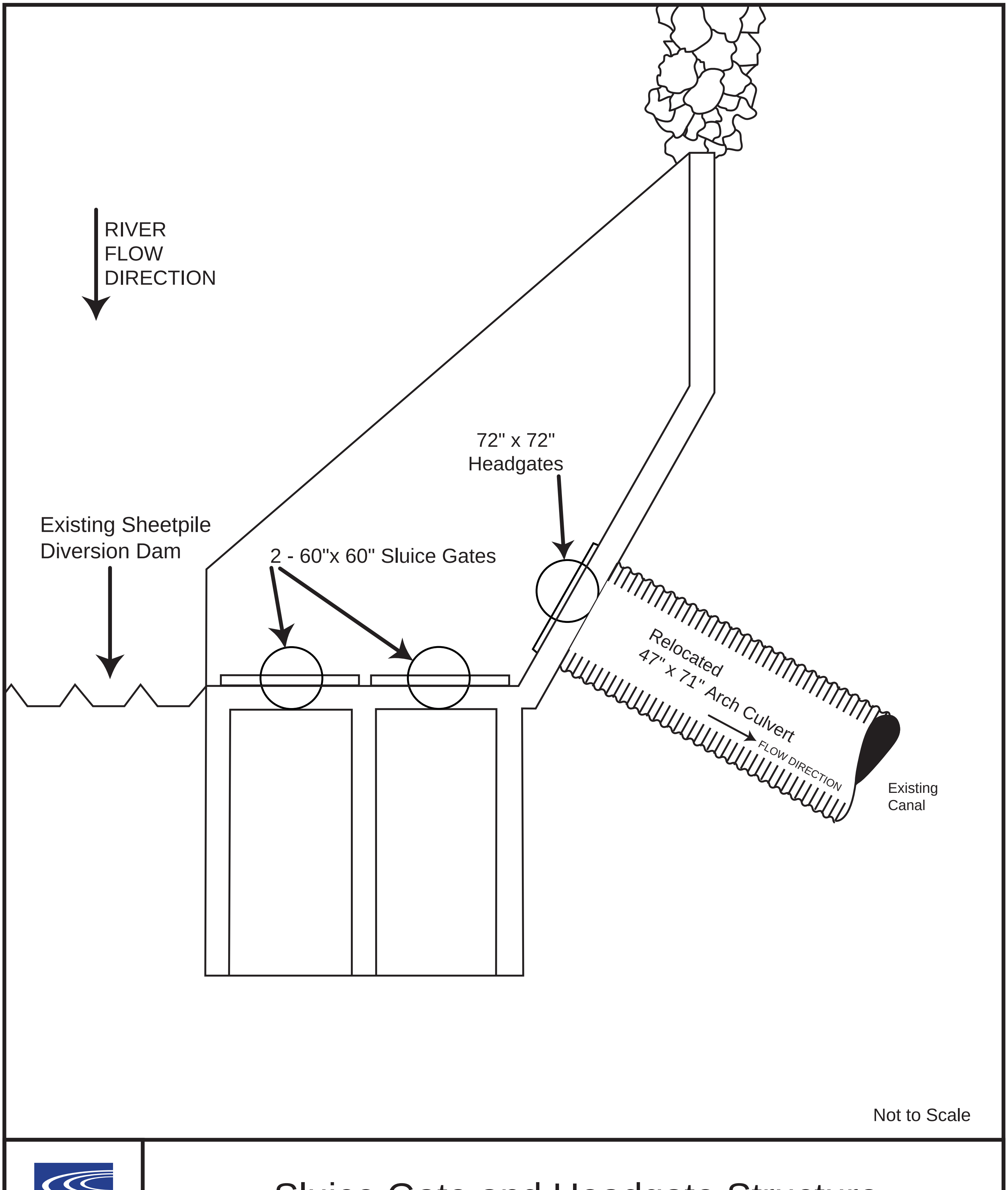
F:\Data\1828\Gus\Rec_divplan.dwg

STETSON ENGINEERS INC.

Relocated Diversion Headgate Structure

Santa Margarita River

FIGURE 6-1



STETSON ENGINEERS INC.

Sluice Gate and Headgate Structure
Maintenance and Repair Project

1828-324SluiceGate3.ai

TABLE 6-3
COST ESTIMATE FOR
MAINTENANCE AND REPAIR ITEMS

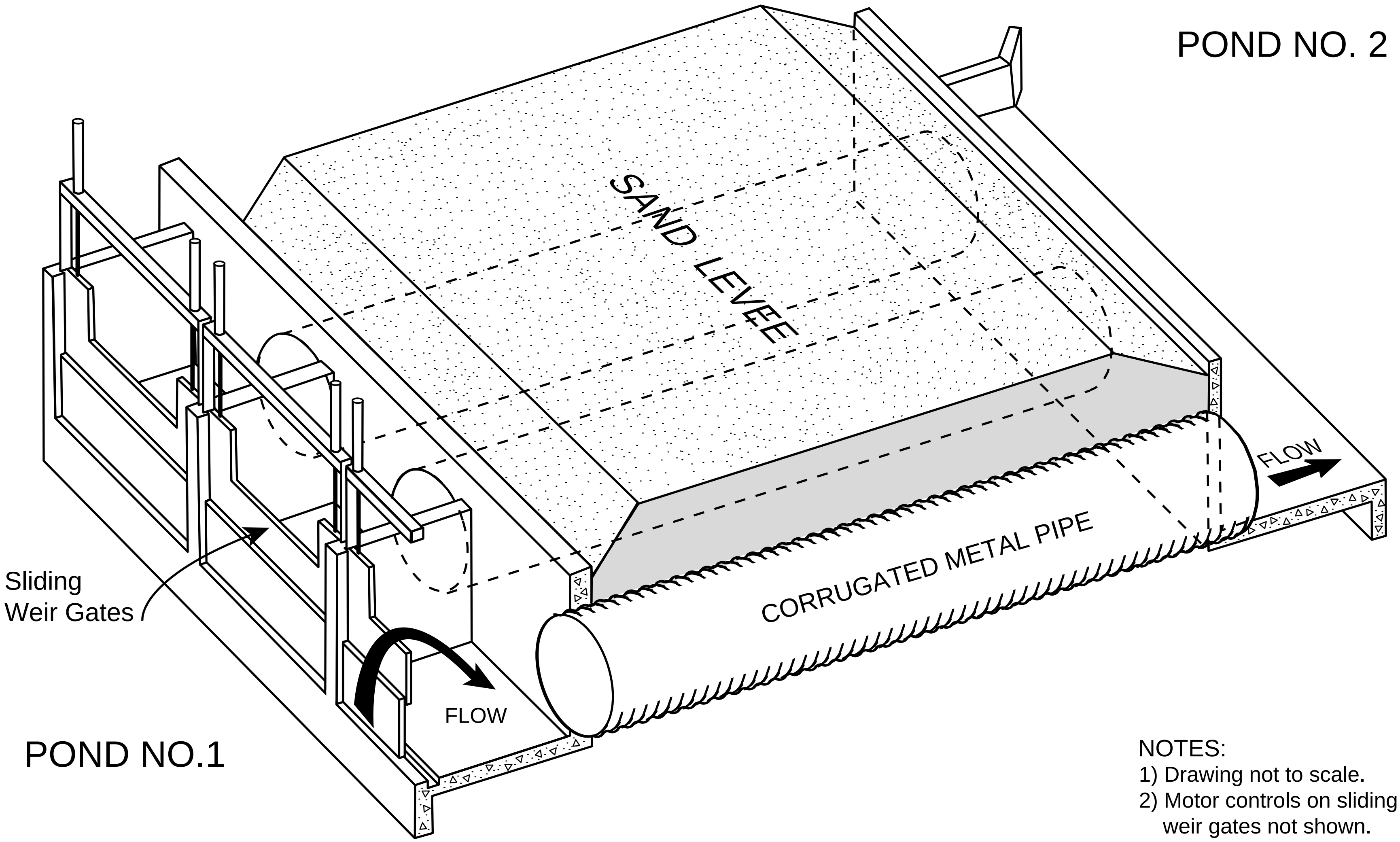
Item	**Cost**
Diversion Dam	
Sluice Gates & Headgate Relocation	$161,000
Remove Portion of Existing Dam	10,000
Subtotal (Diversion Dam)	**171,000**
Recharge Ponds	
Clear, Scrape, and Grade Existing Pond Nos. 1-3	310,000
Flow Control and Measurement Structures between Recharge Pond Nos. 1-5 (10 @ $20,000 each) for 100cfs	200,000
Subtotal (Recharge Ponds)	**510,000**
Ground-water Piezometers (2 wells and 4 loggers)	36,600
Field Survey of Diversion Dam and Recharge Pond Area	16,500
Subtotal (all items above)	**734,100**
Contingencies and Unlisted Items @ 25%	184,000
Subtotal	**$918,100**
Planning, Engineering, and Design @ 15%	137,700
Project Management and Administration @ 10%	91,800
Total Estimated Capital Cost	**$1,147,600**

6.2.2 WEIR GATE STRUCTURES FOR RECHARGE POND NOS. 1-5

The existing recharge pond system does not include structures or devices to control pond water levels or measure flow between individual recharge ponds. It is recommended that sliding weir gate structures be constructed in Pond Nos. 1-5 to allow for water level control and flow measurement. Flow control between recharge ponds will allow the Base to operate the recharge basins in the most efficient manner, allowing for the maximum infiltration rates possible. A conceptual drawing of the proposed sliding weir gate structure is shown in Figure 6-3.

Pond water level control is currently limited to manual placement of plywood boards across the openings of the culverts that drain each pond. When the plywood boards are removed, pond water levels are fixed by the invert elevation of the culverts that drain each pond. Under the current operating condition, uncontrolled culvert flow between ponds often occurs under submerged or partially submerged conditions causing backwater effects between the ponds.

1828-324aSlidingGate.ai



NOTES:
1) Drawing not to scale.
2) Motor controls on sliding weir gates not shown.

STETSON ENGINEERS INC.

SLIDING WEIR GATE STUCTURE FOR CONTROLLING AND MEASURING FLOW BETWEEN RECHARGE PONDS

FIGURE 6-3

Backwater effects also occur between Pond No. 1 and the turnout from O'Neill Ditch as a result of high pond water levels.

The sliding weir gates will be installed on cast-in-place concrete box structures and will be motor controlled for ease of operation. Water flowing over the sliding weirs will pass through the levees separating the recharge ponds, in buried corrugated metal pipes. Installing the weir structures between each pond will allow water level control such that water will cascade from one pond to another without backwater effects.

The flow rate of water passing over the weirs will be a function of the height of water passing over the crest of the weirs. Therefore, water level recording equipment will also be installed at the location of each weir structure enabling a continuous record of pond water level height, the height of water passing over the weir crests, and the ability to calculate the flow rate between each pond. The flow recording equipment will include submersible pressure transducers to sense water level heights and data loggers to allow for a continuous record of water level measurement. The equipment required to monitor pond water levels and measure flow between ponds will be installed at convenient and appropriate locations near the sliding weir gate structures. Nearby utility lines will need to be extended to each flow recording station to power the equipment.

In addition to the pond water level and flow measuring instrumentation, two new ground-water piezometers should be installed within the recharge pond system to provide the ability to monitor ground-water levels. Data collected from the new piezometers will supplement data collected from two existing piezometers. A total of four data loggers will also be required for the piezometers which can be powered from the same utility lines that will power the pond water level and flow measuring instrumentation. The cost estimate for the two new piezometers and four data loggers is $36,600.

To achieve cascading flow between ponds, without backwater effects between individual ponds and without backwater effects between Pond No. 1 and the turnout from O'Neill Ditch, it will be necessary to modify the existing pond operations such that maximum pond water levels are restricted to lower elevations than currently allowed. Operations with the sliding weir gate structures in place will require water levels in each pond be reduced by approximately 1-2 feet, depending on the individual pond.

Two 8-foot sliding weir gates will be required in order to pass a maximum of 100 cfs between individual ponds. A total of ten 8-foot sliding weir gates will be required under this maintenance item. The cost estimate for each 8-foot sliding weir gate mounted on a concrete box structure is $20,000 or $200,000 for ten gates. This estimate includes the cost for the continuous water level and flow recording equipment.

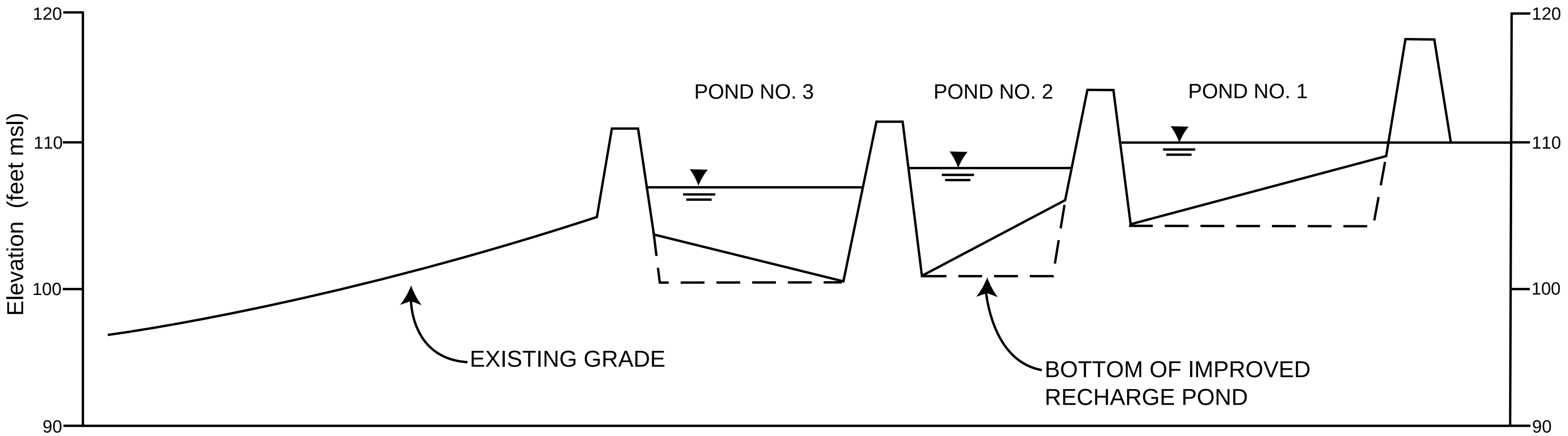
6.2.3 EXCAVATION AND REMOVAL OF FINE SEDIMENT FROM POND NOS. 1-3

Based on the survey data currently available for the ground-water recharge pond system (survey data from the 1962 map), it is recommended that an average of 2 feet of sediment be removed from the bottoms of recharge Pond Nos. 1-3 as a maintenance item. Acknowledging that all the surveyed elevations shown on the 1962 maps are probably not representative of the existing conditions, the amount of sediment proposed for removal from the bottoms of recharge Pond Nos. 1-3 (and the cost for sediment removal) under this maintenance item may vary. The excavation and removal of fines, as well as proper control and measurement of water between ponds, will allow the Base to meet the original design goals of the project and maximize the amount of water recharged to the ground-water aquifer. Prior to completing the design and construction of any of the maintenance items proposed in this chapter, it is recommended that a new land survey be conducted and elevation maps be prepared in connection with this work. The cost estimate for the land survey is $16,500. The maintenance improvements for recharge Pond Nos. 1, 2, and 3 are generalized in Figure 6-4. The reasoning for proposing this maintenance item is three-fold and is described in detail below.

First, the overall performance of the ground-water recharge system is reduced by inefficiencies related to pond cleaning and maintenance. The historical practice of recharge pond maintenance has involved mechanical ripping and disking of the pond bottoms after the ponds have drained and dried at the end of the diversion season. The practice of ripping and disking the pond bottoms at the end of the diversion season causes the accumulated fine sediments, dried algae growth, and vegetation to be mixed and held into the underlying soils. Disking these clogging materials into the soil reduces infiltration rates.

Based on previous infiltration rate studies in or near Camp Pendleton's recharge basins (Schmidt, 1977, and Almgren and Koptionak, 1990), and the results of Stetson Engineers' recent infiltration rate study, much higher infiltration rates are achievable in the recharge ponds. Higher infiltration rates could be achieved if a pond cleaning and maintenance program were implemented whereby clogging layers were periodically removed and not disked into the underlying soil.

1828-324A1PondProfile.ai



NOTES:
1) Pond water levels represent maximum levels
after improvements.
2) Horizontal distances not drawn to scale.
3) Existing and proposed grade lines are approximate.

STETSON ENGINEERS INC.

Maintenance Improvments for Recharge Pond Nos. 1, 2, and 3

FIGURE 6-4

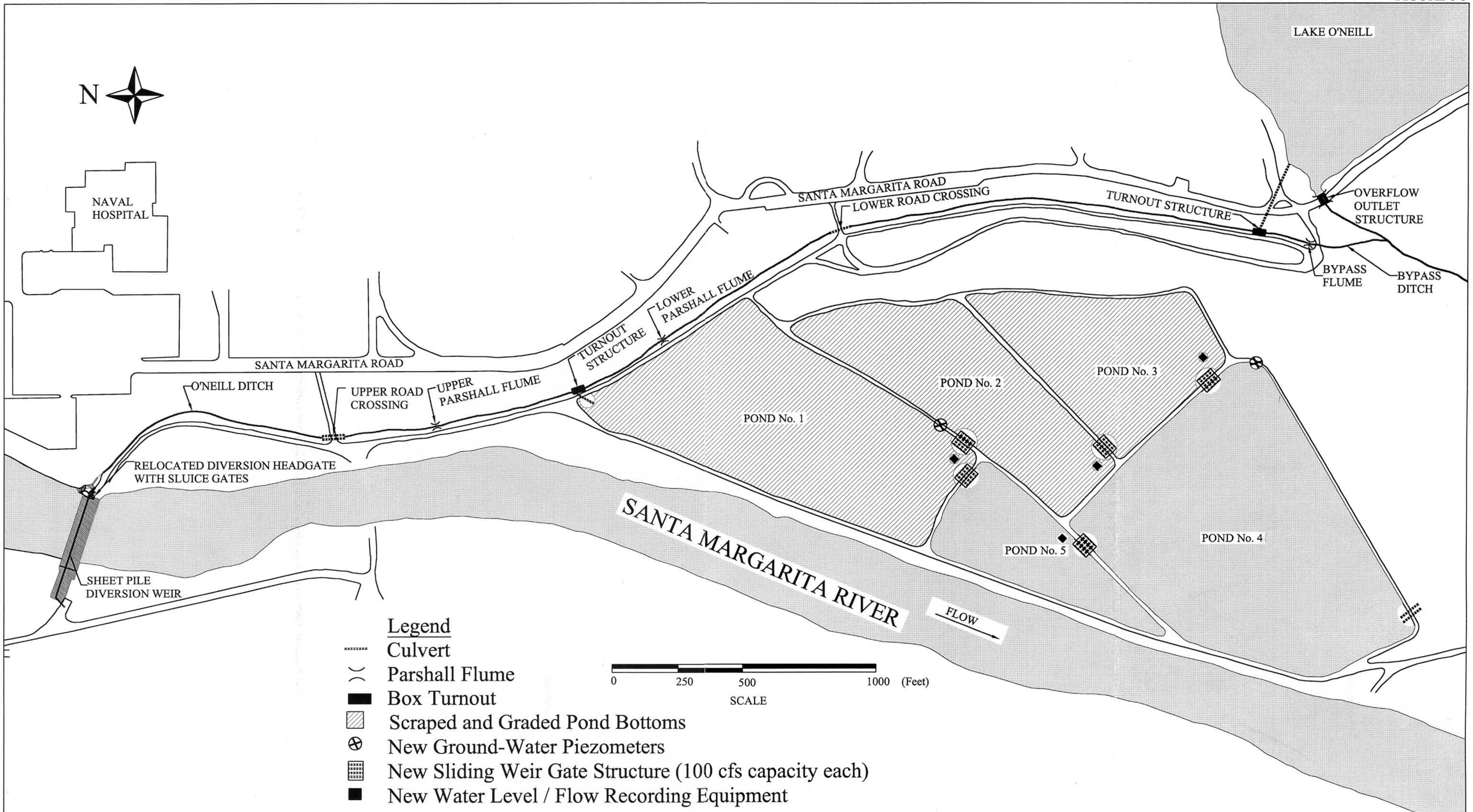
Higher infiltration rates might also be achieved if the ponds were operated to allow for wetting and drying periods. For example, the recharge pond system could be operated such that water first passes from Pond No. 1 into Pond No. 2, and then into Pond. No. 3. After filling, the system consisting of Pond Nos. 1, 2, and 3 could then be allowed to drain and dry while flow was transferred from Pond No. 1 into Pond No. 5, and then into Pond No. 4. Under the current recharge pond operations, water remains in one or two ponds for much of the diversion season. Operating the pond system to allow wetting and drying cycles could reduce biological activity in the water and on the pond bottoms, thereby reducing the required cleaning frequency.

The second reason for excavating the pond bottoms two feet lower than the elevations shown on the 1962 survey map is that installation of the sliding weir structures to control water levels will require the ponds be operated at lower surface water elevations than they are currently operated at. Excavating the pond bottoms will offset the loss of surface water storage capacity when the water levels are lowered for proper operation of the weir structures.

The third reason for excavating the pond bottoms relates primarily to Pond No. 1 and the estimated average depth of water in the pond. The average depth of water in Pond No. 1 was estimated to be only 3.2 feet based on the 1962 survey. The survey map shows that the bottom elevation of the pond in the area near the inlet from O'Neill Ditch is approximately the same elevation of the floor of the turnout structure in O'Neill Ditch. The surface water storage capacity in Pond No. 1 could be significantly increased and the potential for backwater effects could be reduced if 4-5 feet of sediment were excavated from the bottom of Pond No. 1 in the vicinity of where the pipe inlet from O'Neill Ditch enters the pond. The proposed maintenance and repair items are shown in Figure 6-5. Table 6-3 presents the cost estimate to design and construct the maintenance and repair facilities discussed above.

6.2.4 OPERATION AND MAINTENANCE COSTS

The maintenance and repair items summarized in Table 6-3 are expected to increase the current operation and maintenance costs by approximately $17,000 per year. The current annual operation and maintenance costs were estimated at $87,800 and included labor ($25,000), sediment removal from behind the existing weir structure ($150,000 every three years or $50,000 per year), and disking of the pond bottoms ($12,800). The estimated increase in the annual operation and maintenance cost ($17,000) represents an increase in labor costs to operate and maintain the new facilities ($10,000) and an increase in the annual pond maintenance cost for sediment removal ($7,000).



F:\DATA\1828\GIS\MANT&REPAIR_SITE_PLAN.DWG

STETSON ENGINEERS INC.

Diversion and Ground-Water Recharge System - Maintenance and Repair Items

Santa Margarita River

7.0 ALTERNATIVE EVALUATION

The project alternatives discussed below have been formulated based on maximizing surface diversions from the Santa Margarita River while minimizing the impact to the environment. Project alternative sites were investigated both on and off Camp Pendleton, including sites within the ground-water basin located beneath the cities of Murrieta and Temecula. The following chapter describes all alternatives that were considered, including a detailed discussion of four alternatives that were determined to be feasible.

The various alternatives described in this chapter were developed for the purpose of maximizing Application 21471B, the undeveloped portion of Permit 15000. Originally filed in 1963, this portion of Permit 15000 was to be used to divert a maximum of 165,000 AFY at the proposed De Luz dam site for military, municipal, domestic, and agricultural purposes. Identical to the original intent of the Permit filed in 1963, the project alternatives described in this chapter have also been designed to meet Camp Pendleton's existing and future military, municipal, domestic and agricultural demands. Due to the conclusion of previous studies that prevent the construction of the Santa Margarita Project, the alternatives described herein provide a means for Camp Pendleton to meet its future water demand without adverse impact to the environment. Each of the four alternatives described in detail discusses the proposed project alternative yield and place of diversion to be applied to Application 21471B, Permit 15000

Four alternatives, including a No Project alternative, were chosen for further evaluation following initial review of all possible alternatives. The feasibility of each of these four alternatives, including conceptual designs and cost estimates, is discussed in detail below. This study does not recommend a preferred alternative; rather, this study has been designed to show the most feasible project for each of the four alternatives. A summary of the four alternatives is provided in Table 7-1.

TABLE 7-1
SUMMARY OF ALTERNATIVES

Project	Alternative 1	Alternative 2	Alternative 3	Alternative 4
New Diversion Dam		✓	✓	✓
Improve Existing Ditch Capacity		✓	✓	✓
New Recharge Ponds			✓	✓
New Off-Stream Storage Reservoir				✓
Alternative Capital Cost ($ Mil)	0	3.5	5.5	47.7
Annual Cost Per Acre-Foot	N/A	$120	$100	$730

As shown in the Table 7-2, Alternative 1 is the "No Project" alternative that does not provide any additional ground-water yield or surface water diversion from the Lower Santa Margarita River basin. It is estimated that completion of the proposed maintenance and repair work will increase the amount of water diverted from the Santa Margarita River by 2,190 AFY, with no additional pumping. Alternative 2 through 4 present projects that increase both the ground-water yield from the basin and amount of water diverted from the Santa Margarita River. The expected median annual increase in ground-water yield, above the no project conditions, in Alternatives 2 through 4 ranges from 3,000 AFY to 6,000 AFY, respectively. Alternative 2 includes the construction of a new diversion ditch and improvement to the existing water conveyance facilities at a capital cost of $3.5 million. Alternative 3 includes all of Alternative 2 projects, as well as the construction of additional ground-water recharge ponds, at a total capital cost of $5.5 million. Finally, Alternative 4 includes all of the additions and improvements outlined in Alternative 3, as well as the construction of an off-stream storage reservoir, at a total capital cost of $47.7 million. The maximum additional surface water diversion indicates the future amount of additional water that would need to be diverted under application 21471B, Permit 15000.

TABLE 7-2
SUMMARY OF WATER RIGHTS AND PROJECT YIELD

Water Right	Alternative 1 (AFY)	Alternative 2 (AFY)	Alternative 3 (AFY)	Alternative 4 (AFY)
Maximum Existing License Yield	4,000	4,000	4,000	4,000
Maximum Pre-1914 Rights Yield	1,100	1,100	1,100	1,100
Maximum Altemative Riparian Water Right Yield	3,200	3,700	3,700	3,700
Minimum Additional Ground-Water Yield (AFY)[1]	N/A [2]	3,000	5,500	6,000
Total Annual Project Yield	**8,300**	**11,800**	**14,300**	**14,800**
Maximum Additional Surface Water Diversion (AFY)	N/A [2]	8,600	16,300	21,000

Note:

[1] - Minimum additional ground-water yield is based on the median project yield for each alternative as determined by the ground-water model.

[2] - N/A indicates not applicable.

Chapters 5 and 6 outlined the capacity of the existing system and the projects required to repair the diversion and recharge facilities to meet the original design capacity required to fully exercise the rights and licenses available to the Base to divert surface water. As each alternative is discussed below, it is assumed that the Base has implemented the proper maintenance and repair projects suggested in the previous chapter. Maintenance and repair projects, including the relocation of the headwall and headgate, the weir and control improvements between recharge ponds, and the scraping of the existing recharge ponds, will increase the efficiency of the existing facilities, allowing the Base to maximize its diversions under existing appropriative rights on an annual basis. The additional ground-water yield, surface diversions, and the cost per acre-foot of each alternative assumes existing conditions include the completion of the maintenance and repair projects.

The Base has proactively surveyed and documented sensitive species, cultural resource sites, hazardous material locations and ground and surface water contaminants as part of it's on-going Base management and stewardship activities. In addition, the Base has conducted focused and intensive survey and documentation programs in response to environmental review and compliance programs for specific projects. As a result, the Base has developed, and maintains, a relatively comprehensive database of information that is the basis for the constraints analysis in this Section.

To determine the potential for regulatory-based constraints, a Geographic Information System (GIS) product containing project features and various biological, hydrological, cultural resource and hazardous material coverage was created. The GIS was used to ascribe appropriate buffer zones of various widths to project features denoting the construction zone of these features that might affect sensitive resources. For biological resources, buffer widths were determined by researching life history attributes of sensitive species that may present a significant project implementation constraint (Focus Species) and, based on this information, estimating the distance beyond which construction would not affect breeding or foraging activities. If a sensitive resource was documented within a buffer zone, it was assumed the species could be affected by project actions and would be subject to applicable regulatory compliance and permitting requirements. The location of cultural resource features were not included in constraints figures due to the sensitive nature of these sites.

The GIS analysis process also revealed initial project implementation opportunities. Although the collective project alternatives present little opportunity for avoidance, alternative pipeline routing, reservoir configuration, or location of new percolation ponds are examples of features with some flexibility in location and design. Potential opportunities revealed through the GIS analysis process could be applied, along with other criteria, to determine precise routing or component placement. A more refined analysis could result in significantly reduced mitigation costs or permitting requirements.

The region of influence (ROI) for this project is defined as the area potentially affected by the four alternatives proposed in this feasibility analysis. This would include those areas of the Upper Ysidora and Chappo sub-basins of the Santa Margarita River ground-water basin supporting features, and those areas in the immediate vicinity which support sensitive resources, which may constrain implementation of the selected project. The ROI also includes similar areas in the southernmost portion of the U.S. Naval Weapons Station (Fallbrook Annex) as shown on Figure 1-2, *Lower Ground-Water Basin*.

7.1 DEVELOPMENT OF ALTERNATIVES

Stetson Engineers reviewed various project alternatives throughout the Santa Margarita River Basin. Many of these alternatives were discussed at the project kick-off meeting in October 1999 and investigated over the following 12 months. Factors that were considered when determining various alternative projects included, but were not limited to, quantity of water diverted from the Santa Margarita River, amount of water available for direct or indirect use, impact to local environment, and the ability to meet existing water rights provided by Application 21471B, Permit 15000. The choice of alternatives included the examination of both local and regional projects located within and outside the Santa Margarita River Basin.

The following section identifies each of the various project alternatives that were considered for review in this feasibility analysis. A brief description and a statement as to whether the alternative was given further consideration are provided for each project alternative. Alternatives 1 through 4 listed below were determined to be viable alternatives and are described in detail throughout the remainder of this chapter. Alternatives 5 through 8 were eliminated from further consideration due to environmental or physical constraints. The construction of in-stream reservoirs sites was eliminated from further consideration for the same reasons that the Santa Margarita River Project was determined to be infeasible in 1989.

7.1.1 ALTERNATIVE 1 – NO PROJECT

Alternative 1 is considered the "No Project" alternative and provides baseline conditions for comparison to other alternatives. The baseline condition provided in this alternative assumes that all projects recommended in Maintenance and Repair (Chapter 6) are properly designed and constructed. In addition, Alternative 1 also includes the addition of augmented flows to the Santa Margarita River provided by the 2000 agreement between Camp Pendleton and the Rancho California Water District. The No Project alternative also assumes that all wastewater is exported from the Santa Margarita basin to the Oceanside Outfall.

A ground-water model scenario was run to represent baseline conditions under the No Project conditions. Assumptions and conditions of this model included: augmented stream flow, no wastewater discharge to the basin, full diversions to the recharge ponds and Lake O'Neill under the existing license and water right, and historical ground-water pumping. The results of this model run are used to compare impacts from Alternatives 2 through 4 to baseline conditions. The disposition of the wastewater will not change until the completion of the P002 project currently being investigated by Camp Pendleton.

7.1.2 ALTERNATIVE 2 – DIVERSION WEIR AND DITCH IMPROVEMENTS

Alternative 2 includes the construction of a new diversion weir, improvements to the existing ditch capacity and expansion of the instantaneous capacity of the head-gate diversion from 60 cfs to 200 cfs. In addition to these improvements, augmented flow from the RCWD agreement is included in the streamflow analysis, and new ground-water wells have been added to increase the extractions from the ground-water basins. This alternative was considered for further investigation because it minimized the impact to the environment with an increase of 8,000 AFY diverted from the Santa Margarita River for ground-water recharge, and an additional 3,000 AFY from well production.

7.1.3 ALTERNATIVE 3 – DIVERSION WEIR, DITCH IMPROVEMENTS AND CONSTRUCTION OF NEW RECHARGE PONDS

Alternative 3 includes the construction of a new diversion weir, improvements to the existing ditch capacity, expansion of the instantaneous capacity of the headgate diversion from 60 cfs to 200 cfs, and construction of new recharge ponds. Similar to Alternative 2, augmented flow and new ground-water wells have been included in this alternative. This alternative was considered for further investigation because it minimized the impact to the environment, increased the annual diversions from the Santa Margarita River by 16,300 AFY, and increased average annual ground-water well production by 5,500 AFY.

7.1.4 ALTERNATIVE 4 – DIVERSION WEIR, DITCH IMPROVEMENTS, AND CONSTRUCTION OF NEW RECHARGE PONDS AND OFF-STREAM RESERVOIRS

Alternative 4 includes the construction of a new diversion weir, improvements to the existing ditch capacity, expansion of the instantaneous capacity of the head-gate diversion from 60 cfs to 200 cfs, construction of new recharge ponds, and construction of off-stream reservoir sites and related facilities. Similar to Alternatives 2 and 3, augmented flow and new ground-water wells have been included in this alternative. This alternative was considered for further investigation because it provided seasonal storage, increased the annual amount of water available for diversion by 21,000 AFY and provided water for drought relief during extended dry periods. This alternative is expected to increase the annual ground-water production by 6,000 AFY

7.1.5 ALTERNATIVE 5 – AQUIFER STORAGE AND RECOVERY WELLS

Alternative 5 included the construction of aquifer storage and recovery wells (ASR) for the purpose of injecting water directly into the aquifers. Under this alternative, surface water would be diverted from the stream, filtered, and then injected into the aquifer for recovery at a later date. This alternative was dropped from further consideration because of the high transmissivity of the ground-water basin and the shallow depth to ground water provided no advantage to ASR wells when compared to ground-water recharge ponds.

7.1.6 ALTERNATIVE 6 – RECHARGE AND RECOVERY OF STORM WATER IN THE UPPER BASIN

Alternative 6 investigated the feasibility of constructing ground-water recharge basins in the Upper Santa Margarita basin in the vicinity of the cities of Murrieta and Temecula. Floodwater from Murrieta Creek and its tributaries would be diverted to recharge ponds and recovered from the ground-water system at a later date. This alternative was dropped from further investigation because the geologic restrictions in the Upper Basin inhibited substantial quantities of ground water infiltrating into the aquifers (see Appendix H).

This alternative was originally considered because of the large amount of available storage in the Upper Santa Margarita Basin and the Army Corps of Engineers (ACOE) flood study of Murrieta Creek. The flood control project proposed by the ACOE did not provided for long-term (more than 1 day) storage of water for percolation purposes. In addition, the deep Pauba Aquifer that has a large potential for ground-water storage is isolated from the surface by a 30 to 60 foot clay layer, restricting recharge to the deeper aquifer. The only potential site for ground-water recharge and storage in the Upper Basin is located in Pauba Valley. This site was dropped from further consideration because of adverse environmental impacts, existing and future urban development and incompatibility with the ACOE's project.

7.1.7 ALTERNATIVE 7 – ENLARGEMENT OF LAKE O'NEILL

Alternative 7 considered the enlargement of Lake O'Neill for the purpose of storing high flow events for later release to the recharge ponds. Existing uses of Lake O'Neill include water supply and recreation. Recreation facilities at Lake O'Neill include miniature golf, picnic cabanas, playgrounds, volleyball, basketball, softball and horseshoe throwing areas. Several boating activities are available, including bumper boats, paddle boats, rowboats, and pontoon boats for picnics and fishing. The Lake O'Neill peninsula is used for large group activities such as promotions, retirements and wedding receptions. Additionally, the Lake O'Neill campgrounds offer campsites with water, electricity, sewer hookups and tent camping. Fishing is permitted year round. The lake is stocked and bait is sold at facility offices.

The Lake O'Neill enlargement alternative was dropped from further consideration due to environmental and physical considerations. Enlarging Lake O'Neill would require realignment of the Santa Margarita River road and recreation facilities. Diversions to the enlarged lake would require realignment of O'Neill ditch and/or installation of high volume – low lift pumps. Furthermore, the location of the nearby Naval Hospital would limit the amount of increased storage available from an enlarged lake. Review of the soils surrounding Lake O'Neill also suggested that some form of barrier would be required to minimize leakage into the adjacent permeable Visalia, Tujunga, Greenfield, and Cieneba Series soils.

7.1.8 ALTERNATIVE 8 – IN-STREAM RESERVOIR SITES

Alternative 8 considered the construction of on-stream reservoir sites for the purpose of diverting large flood events from the Santa Margarita River. Similar to the U.S. Bureau of Reclamation's Two-Dam project, this alternative would capture large flood events on the Santa Margarita River and release these flows at a controlled rate to recharge ground-water basins on Camp Pendleton. This alternative was dropped from further considerations due to the high environmental costs associated with this project.

Due to the physical possibilities and minimized environmental impact, the first four Alternatives described above were determined to be feasible and considered for further review. The last four alternatives were determined not to be feasible due to environmental and physical restrictions. In each of the eight alternatives, many related alternatives were considered and reviewed in order to minimize these restrictions. For instance, the construction of new recharge ponds, conveyance facilities, and diversion weirs apart from the existing diversion facilities were considered, but dropped from further consideration because of geologic and ground-water contamination restrictions. An additional diversion point on the Santa Margarita River to an off-stream reservoir was also considered, but eliminated from further consideration due to lack of benefit and environmental restrictions. Many of these alternative projects that are not addressed in detail below are discussed in the appendices. Alternatives 1 through 4 are described in detail below, including a discussion of the different alternatives that were addressed to minimize environmental restrictions and maximize ground-water yield to enhance the feasibility of the alternative.

7.2 CHOSEN ALTERNATIVES

The purpose of this study was to maximize the amount of water available for diversion from the Santa Margarita River while at the same time minimizing the potential adverse impact to the environment. In order to review the impact to the environment and determine the amount of water available for recovery, the ground-water model (Model) was used as a tool to quantify the impact of each of the alternatives. In addition to the project description and costs, a detailed presentation of the Model output and potential impact to the affected environment is discussed in detail for each of the alternatives. In all cases, the Model was used to measure and minimize the impact to ground-water levels and streamflow of the Santa Margarita River for each of the three sub-basins on Camp Pendleton.

Each of the alternatives discussed below present a description of the preferred project for that alternative and provides a summary of costs and water available for storage and recovery. A detailed surface water model was used to estimate the amount of water available to the ground-water model area, as well as determine the amount of water available for diversion to both Lake O'Neill and the ground-water recharge ponds (Appendix E). The surface water model provided the necessary data to both the Model and the engineering design team in order to maximize the available water for diversion and recovery. In order to accomplish this task, all surface water modeling was performed using daily precipitation data and accounted for evaporation losses from both Lake O'Neill and the recharge ponds.

The best available technology was also reviewed for each alternative. For example, a detailed discussion of various river diversion dam designs provides alternative technology that was considered, but eliminated from further review due to inefficiencies or cost. In each alternative listed below, the goal of maximizing the amount of water diverted from the Santa Margarita River was heavily considered when comparing project costs and yield. Each alternative discussed below presents the total project costs, as well as the cost per acre-foot of water based on amortizing the project over 30 years. Project costs and yields are compared and discussed in detail at the end of this chapter.

7.2.1 ALTERNATIVE 1 – NO PROJECT ALTERNATIVE

The No Project alternative was included as Alternative 1 in order to provide a baseline for measuring the impacts of Alternatives 2 through 4. Maintenance and repair projects recommended in Chapter 6 increase the efficiency of the existing system, returning the existing facilities to their original design capacity. Therefore, for the purposes of measuring both the cost and the yield of each alternative, it is assumed that all the maintenance and repair recommendations in Chapter 6 have been constructed and are in place under the No Project alternative. In addition to the replacement and repair of the existing facilities, augmented flow from the settlement agreement with the RCWD has also been included, and wastewater flows have been excluded, in the baseline conditions.

The maintenance and repair of the existing river diversion headgate and headwall structure increases the efficiency of the recharge and recovery facilities on Camp Pendleton. The amount of sediment accumulating in front of the headgate will be reduced, resulting in greater stream flow diversions into the O'Neill ditch and the recharge ponds. Results of the surface water analysis and available stream flow indicates that Camp Pendleton would have maximized its license to divert 4,000 AF of water per annum for recharge to the ground-water system, and its pre-1914 water right to divert 1,500 AFY to Lake O'Neill.

Augmentation to stream flow due to Camp Pendleton's agreement with the RCWD has also been included in the No Project baseline conditions. Based on the twenty previous years of hydrology and stream flow, an average of 2,500 AFY of water would have been augmented to the observed stream flow at the Gorge. The increase in daily stream flow ranges from 3.0 cfs to 11.5 cfs depending on the hydrologic condition and time of the year. The hydrologic index developed from the settlement agreement provides a means for determining Very Wet, Above Normal, Below Normal, and Extremely Dry hydrologic conditions in the basin. These hydrologic conditions are applied in Alternatives 2 through 4 for adjusting pumping during periods of extended below normal conditions.

Historical wastewater releases to the Santa Margarita River and related oxidation ponds have also been excluded in the No Project baseline condition. Although it has not yet been determined if the wastewater produced on Camp Pendleton will remain within the Santa Margarita basin or be exported to an adjacent basin or the ocean, Alternatives 2 through 4 assume that the historical wastewater is no longer available as a source of recharge to the three sub-basins. The discussion of the ground-water model presented in Chapter 4 addresses the impact to resources on Camp Pendleton due to the loss of this source of recharge.

The No Project alternative provides a means for measuring the impact of each of the "project" alternatives addressed in Alternatives 2 through 4. The cost and yield of each alternative have been estimated based on the assumption that Camp Pendleton will maintain and repair the facilities identified in Chapter 6. There are no capital costs associated with the No Project alternative. Similarly, there is no increase the amount of water diverted from the Santa Margarita River, nor the amount of water extracted from the ground-water basins.

7.2.1.1 Project Design and Operation

There is no project design in the No Project alternative.