APPENDIX G

Groundwater Studies

WorleyParsons Komex
resources & energy

GROUNDWATER STUDY: PROPOSED GRATON RANCHERIA CASINO AND HOTEL

ROHNERT PARK, CALIFORNIA

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EXECUTIVE SUMMARY

The Federated Indians of Graton Rancheria have proposed to establish a casino and hotel development northeast of the corner of Rohnert Park Expressway and Stony Point Road (the Site), on the northwestern edge of the City of Rohnert Park in Sonoma County, California (**Figure 1**).

It is proposed that the new development will be supplied with groundwater from two new production wells. WorleyParsons Komex has carried out a study to assess how the use of groundwater to supply the new development will affect local groundwater levels.

Two alternative sites are being considered in Rohnert Park as well as several development alternatives that together comprise Alternatives A, B, C, D, E and H. Alternative F is located at another site in southern Sonoma county and is evaluated in a separated report. Alternative G is the no project alternative. Alternatives A and H would utilize the "Wilfred Site, " which comprises a 253-acre site northeast of the intersection of Rohnert Park Expressway and Stony Point Drive. Alternatives B through E involve development of the 363-acre "Stony Point Site" that overlaps with the Wilfred Site and includes land that is contiguous to the north. An area of 180 acres is common to both the Wilfred and Stony Point Sites, and the water supply wells for the project are proposed to be located on the east side of this parcel under each of the alternatives that are being considered in Rohnert Park. (For the purposes of this report, the Wilfred and Stony Point Sites are collectively termed "the Site," unless they must be distinguished for clarity.) The Site comprises unincorporated, relatively level land that is presently used for agriculture and as pasture. A stream crosses the Site from north-northeast to south-southwest and flows into the Laguna de Santa Rosa off-Site to the southwest.

The locations and water demands for the various Alternatives are summarized in the table below.

Alternative	Development and Site Location	Total Water Demand (gpm)	Water Recycling (gpm)	Net Water Demand (gpm)
A	Hotel and Casino, Wilfred	250	50	200
B	Hotel and Casino, Stony Point	250	50	200
C	Hotel and Casino, Stony Point	250	50	200
D	Hotel and Casino – Reduced Intensity, Stony Point	150	25	125
E	Business Park, Stony Point	65	15	50
F	Hotel and Casino, Lakeville	250	50	200
G	No Project	95 [1]	0	95 [1]
H	Hotel and Casino – Reduced Intensity, Wilfred	150	25	125

1. According to HydroScience (2007), the water demand for the project area would be approximately 95 gpm if the site is developed as planned under Northwest Area Specific Plan. Approximately 50 gpm of this amount would be derived from groundwater.

The Site lies in the southern portion of the Santa Rosa Plain groundwater sub-basin, at the southern end of the Santa Rosa Valley groundwater basin. Geologic strata in the Site area include, in increasing order of age, basin deposits (poorly sorted, organic-rich clays and silty clays up to 250 feet thick), alluvium and alluvial fan deposits (mixed sand, silt, clay and gravel deposits between 300 and 450 feet thick), and the Wilson Grove Formation (formerly known as the Merced Formation; sandstone with thin clay and silty clay interbeds, about 500 feet thick). The alluvial deposits and Wilson Grove Formation are most important in terms of water supply in the area.

In the early 1950's, prior to the commencement of extensive pumping in the region for agricultural and municipal use, groundwater was present at between 5 and 20 feet below ground surface (bgs), equivalent to between about 70 and 85 feet above mean sea level (msl) in the Site area. During the 1970's, several studies were undertaken that specifically addressed groundwater resources for public supply in Sonoma County. The resultant reports highlighted the adverse effects of increased pumping on groundwater elevations, particularly in the Rohnert Park area, where groundwater elevations were depressed to as much as 40 feet below msl (Department of Water Resources (DWR), 1979, 1982 and 1987). Recent studies have shown that groundwater levels in the Rohnert Park area declined as a result of increased City pumping between the early 1970's and mid-1980's, after which a new equilibrium of relatively constant groundwater elevations was reached. Recently there have been indications that groundwater levels may have risen slightly.

The City of Rohnert Park relies on the use of surface water (supplied by Sonoma County Water Agency (SCWA)) and groundwater for its water supply. Legal action has obligated the City to reduce its reliance on groundwater. However, other legal action has restricted the quantity of surface water available from SCWA. Rohnert Park has reduced its groundwater pumping in the last seven years. This reduction brought groundwater usage down to levels that were envisaged in the City's 2000 General Plan in 2004.

The proposed pumping rate of the groundwater supply wells for development Alternatives A, B and C will equal about 7 percent of the City of Rohnert Park's average pumping rate in the period 2000 through 2002 (however, the proposed wells would not be part of the City's water-supply system). The proposed wells will represent approximately 0.8 to 1 percent of all current and 1 to 1.7 percent of future pumping in the Santa Rosa Valley basin, and about 4.5 percent of all current and future pumping in the southern Santa Rosa Plain. Under the reduced-intensity alternatives (D and H), pumping from the proposed wells would represent approximately 0.5 to 0.6 percent of current groundwater pumping and 0.6 to 1.1 percent of future groundwater pumping in the Santa Rosa Valley groundwater basin, and a 2.9 percent increase in current and projected future pumping in the southern Santa Rosa Plain. Under Alternative E, the business park alternative, the increased groundwater demand would be about half that of the reduced intensity alternatives. Under each of these scenarios, the basin-wide and local increase in groundwater demand is relatively modest.

Groundwater elevations in the Site vicinity have shown trends that appear to be strongly influenced by pumping in the southern Santa Rosa Plain groundwater sub-basin, of which the City of Rohnert Park's well field has been the largest component. This includes a strong seasonal variation, with spring and fall groundwater levels typically differing by more than 10 feet. A shallow well near the Site shows a smaller response to pumping from the City's well field, as compared to a nearby deeper well. The electric borehole log for City of Rohnert Park well 24, located adjacent to the southeastern Site boundary, reflects this observation. On this log, intervals having higher electrical resistivity (interpreted to be sand and gravel) alternate with intervals having lower electrical resistivity (interpreted to be silt and clay) over the well's multiple-screened interval between 258 and 582 feet bgs. Thus, fine-grained layers between the shallow and deeper zones may account for the shallow well having a smaller response to pumping from the City's well field.

An analytical drawdown model was developed for predicting water-level impacts due to proposed pumping at the Site under the assumption of constant sustained pumping rates for Alternatives A, B and C (200 gpm) and the reduced-intensity alternatives (D and H – 125 gpm). For Alternatives A, B and C, the predicted drawdown at the Site boundary in the deeper screened zone is 23.0 feet. In addition, the predicted drawdown in the deeper screened zone attenuates to about 1 foot at a distance of 17,000 feet from the proposed wells. For the reduced-intensity alternatives (D and H) the predicted drawdown in the deeper screened zone at the property boundary is 14.3 feet and attenuates to about 1 foot at a distance of 14,000 feet. These drawdown predictions are about 40 percent less than for Alternatives A, B and C. The predicted drawdown for the Alternative E (50 gpm) would be about half of that predicted for Alternatives D and H, based on a proportional response. Analysis has shown that off-Site pumping causes greater drawdown in deeper wells (more than 200 feet deep) than in a shallower wells (less than 200 feet deep). We expect that pumping from the proposed wells at the Site would produce a similar type of effect, causing more drawdown in the deeper screened zone versus the shallower screened zone. However, data are not available to allow the actual drawdown in the shallower screened zone to be predicted. Therefore the assumption has been made that shallower wells will experience the same amount of drawdown as deeper wells, when pumping occurs in the deeper zone.

Water levels in the shallower wells near the Site are about 50 feet above msl and water levels in the deeper wells are about 40 feet below msl. This corresponds to depths to water at the Site of about 40 feet for the shallower wells (completed at depths up to 200 feet bgs) and 130 feet for the deeper wells (completed at depths greater than 200 feet). Most recently, water levels have continued to recover above this elevation, as pumping the City of Rohnert Park has decreased. Records obtained from the DWR indicate there are at least 193 shallower wells and 61 deeper wells located within approximately 1.5 miles of the geographic center of the Site. It is not known how many of these wells are still being actively used, or whether there are other wells for which records were not available. All of these wells are predicted to experience some drawdown impacts (interference drawdown) and a resulting proportional decrease in well yield or efficiency, pumping cost and pump life. In the absence of well-specific data regarding transmissivity, use, condition and efficiency, these impacts may be assumed to be generally proportional to the amount of interference drawdown and the remaining saturated thickness of the well after interference drawdown.

The most serious impact that could be experienced by a nearby groundwater user would be having their well go dry or rendered unusable because the remaining saturated thickness after drawdown is too small to support pumping at the required rate. The wells most at potential risk for this impact are expected to be primarily shallow domestic wells near the site. For perspective, we have grouped the wells reported in the vicinity of the Site into several categories based upon the saturated thickness after interference drawdown. Shallow wells with a saturated thicknesses of less than 20 feet during the proposed pumping are considered at greatest risk for going dry or being rendered unusable by having insufficient available drawdown to support normal pumping. Eight such wells were identified under both Alternatives A, B and C and the reduced-intensity alternatives (D and H). Wells with saturated thicknesses between 20 and 40 feet during the proposed pumping may have a smaller but still potentially significant risk of experiencing these impacts. There were 32 such wells under Alternatives A, B and C and 29 under the reduced intensity alternatives. Wells with saturated thicknesses over 40 feet during the proposed pumping are at much lower risk of being dewatered or rendered unusable. All of the deeper wells fall into this category.

In some wells, if water levels fall to a point where the well is in danger of going dry or becoming unusable, the pump intakes can be lowered to extend the life of the well. Without more specific information regarding well construction and pump depth, it is not possible to estimate how many wells may be at risk of experiencing this impact. However, pump intakes for shallow or domestic wells are generally set near the well bottoms and cannot be lowered; whereas, pump intakes for deeper municipal, industrial or agricultural wells are sometimes set at a relatively shallow depth and could require lowering if the wells are located near the Site.

Interference drawdown will cause an increase in the electrical cost to pump a unit volume of groundwater from a well. This cost increase is not expected to be significant for domestic wells because of the relatively low volume of groundwater pumped by a typical household, but could be significant (ranging from several hundred to several thousand dollars) for higher capacity agricultural, industrial or municipal wells near the Site. However, the increased costs for higher capacity users represent a relatively low percentage of their overall pumping costs (2 to 5 percent for the pumps modeled in this analysis). This analysis is applicable to Alternatives A, B and C as well as to the reduced-intensity alternatives, although the increase in pumping costs for off-site wells resulting from the reduced-intensity alternative is expected to be about 40 percent lower based on proportionately less drawdown.

The overall groundwater level trend since the 1970's fits the California Department of Water Resource's definition of "historical overdraft"; however, groundwater levels appear to be recovering slightly as groundwater pumping has decreased beginning in the late 1990's. The groundwater divide between the Santa Rosa Plain groundwater sub-basin and the Petaluma Valley groundwater basin to the south may have migrated southward during the historical overdraft period, resulting in capture of some groundwater from the adjacent basin. The proposed project pumping represents a small increase to the overall regional current and future groundwater pumping rate (approximately 0.5 to 1.7 percent in the Santa Rosa Valley groundwater basin and 2.9 to 4.5 percent in the southern Santa Rosa Plain, depending on the development alternative), and projected groundwater pumping rates in the southern Santa Rosa Plain sub-basin are expected to stay below the peak levels of the 1980s and 1990s. It is therefore unlikely that groundwater pumping for the project will cause a resumption of declining groundwater level trends or further migration of the groundwater divide, but that does not mean the project will have no regional hydrogeologic impacts. Project pumping would be expected to result in a small decrease in the rate of recovery from the historical overdraft condition (proportional to ratio of Site pumping to regional pumping). In addition, to the extent that water groundwater capture is occurring from the adjacent basin, pumping at the Site could contribute to a fraction of this capture.

Mitigation measures that are being considered as part of this project include implementation of a pumping test and groundwater level monitoring program to inform the mitigation process, production well design based on the pumping test results to minimize shallow zone impacts, implementation of on-Site BMPs and wastewater disposal that will enhance recharge, consideration of off-Site mitigation including in lieu recharge and sponsorship of water conservation measures, and reimbursement of affected nearby well owners.

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LIST OF ACRONYMS AND ABBREVIATIONS

AES	Analytical Environmental Services
AFY	acre-feet per year
Agency	Sonoma County Water Agency
ASR	aquifer storage and recovery
bgs	below ground surface
BMPs	Best Management Practices
City	City of Rohnert Park
County	County of Sonoma
DAU	Detailed Analysis Unit
DEIR	Draft Environmental Impact Report
DWR	California Department of Water Resources
EIR	Environmental Impact Report
EIS	Environmental Impact Statement
FTES	Full-time equivalent students
gpm	gallons per minute
HydroScience	HydroScience Engineers, Inc.
MCL	maximum contaminant level
mgd	million gallons per day
mg/l	milligrams per liter
MOU	Memorandum of Understanding
msl	mean sea level
Parsons	Parsons Engineering Science, Inc.
Penngrove	Penngrove Water Company
SCWA	Sonoma County Water Agency
TDS	total dissolved solids
USGS	United States Geological Survey
WAC	Water Advisory Committee
WSA	Water Supply Assessment
WSTSP	Water Supply and Transmission System Project

1 INTRODUCTION

1.1 PROJECT BACKGROUND

The Federated Indians of Graton Rancheria have proposed to establish a casino and hotel development on unincorporated land to the northwest of the City of Rohnert Park in Sonoma County, California (**Figure 1**). An Environmental Impact Statement (EIS) for the proposed development is to be prepared by Analytical Environmental Services (AES) as required by the National Environmental Policy Act.

Two alternative site configurations in Rohnert Park are being considered for the proposed development. A 253-acre site is proposed for development under Alternative A (the Wilfred Site), which is the primary alternative that is being considered. Alternatives B through E involve development of a 363-acre alternative site (the Stony Point Site) that overlaps with the primary site and includes land contiguous to the north. An area of 180 acres is common to both the primary and alternative sites (**Figure 2**). In this report, these two sites are referenced collectively as "the Site" for the purposes of general discussion. Where differentiation is required, the sites are referenced as "the Wilfred Site" and "the Stony Point Site," respectively. A third site in the vicinity of Lakeville in southern Sonoma County is also being considered, and a hydrogeologic study of that site is being presented in a separate report.

The proposed development (Alternative A) has a projected water demand of 250 gallons per minute (gpm). Of the total water demand, approximately 200 gpm is expected to be met from new water supplied to the development and 50 gpm is expected to be met by recycling highly treated wastewater from the development's wastewater treatment plant (HydroScience Engineers, Inc. (HydroScience), 2007). Alternatives B and C have the same water demand and water supply requirements as Alternative A. Alternatives D is a scaled-back hotel and casino development with a reduced water demand of 150 gpm, of which 25 gpm will be met by water recycling (the Reduced Intensity Alternative. Alternative E is a business park development with a water demand of 65 gpm, of which 15 gpm will be met by water recycling. Alternative F (development at an alternative site in southern Sonoma County) is discussed in a separate report. Alternative G is the no project alternative, with an estimated water demand of 95 gpm (including approximately 50 gpm of groundwater) if development proceeds as currently anticipated under Rohnert Park's Northwest Area Specific Plan (HydroScience, 2007). Alternative H is the Reduced Intensity Alternative on the Wilfred Site. A summary of the alternatives considered and their respective water demands is presented in the table below.

Alternative	Development and Site Location	Total Water Demand (gpm)	Water Recycling (gpm)	Net Water Demand (gpm)
A	Hotel and Casino, Wilfred	250	50	200
B	Hotel and Casino, Stony Point	250	50	200
C	Hotel and Casino, Stony Point	250	50	200
D	Hotel and Casino – Reduced Intensity, Stony Point	150	25	125
E	Business Park, Stony Point	65	15	50
F	Hotel and Casino, Lakeville	250	50	200
G	No Project	95	0	95
H	Hotel and Casino – Reduced Intensity, Wilfred	150	25	125

It has been recommended that the water for the development of Alternatives A through E and Alternative H be supplied by two new production wells drilled to approximately 600 feet below ground surface (bgs), and screened below 200 feet bgs (HydroScience, 2007). The water supply wells for each of the development alternatives in Rohnert Park (Alternatives A through E and Alternative H) are proposed to be located on the east side of the southern 180-acre parcel that is common to the development alternatives, in the vicinity of Rohnert Park well 24 (**Figure 2**).

1.2 PROJECT SCOPE

WorleyParsons Komex was contracted by AES to undertake a study relating to the potential impacts resulting from proposed withdrawal of groundwater to supply the new development, as set out in proposals of 10 June 2004 and 8 July 2005. Additional studies to address comments on the Preliminary Draft DEIS received from cooperating agencies were outlined in a contract amendment dated 12 October 2006. Further updates to this report were made in response to public comments on the DEIS. The objective of the study was to assess the how the use of groundwater to supply the new development will potentially affect water resources near the Site.

The study was restricted to the review and interpretation of existing data. The following key issues were identified for discussion:

- Geology and hydrogeology of the southern part of the Santa Rosa Plain groundwater sub-basin;
- Current and historical use of groundwater for water supply;
- Historical groundwater levels and trends;
- Historical groundwater pumping volumes and trends;
- Water balance issues;
- Estimated effects of groundwater pumping for the new development on groundwater levels near the Site;
- Estimated effects of groundwater pumping for the new development on nearby wells; and
- Cumulative and regional impacts of groundwater pumping for the new development.

1.3 REPORT ORGANIZATION

This report is subdivided as follows:

Section 1: Introduction.

Section 2: Regional Hydrogeologic Setting. The topography, drainage, climate, geology and hydrogeology of the region with particular reference to the Site area.

Section 3: Previous Groundwater Use Studies. A review of relevant groundwater studies undertaken between the early 1950's and the present, with particular emphasis on measured groundwater elevations and trends.

Section 4: Groundwater Use in the Area. An appraisal of the past, present and potential future use of groundwater in the vicinity of the Site.

Section 5: Site Evaluation. A Site-specific summary of geologic and hydrogeologic conditions, with discussion of historical groundwater levels.

Section 6: Potential Impacts of using Groundwater to Supply the Proposed Development Alternatives A, B and C. An evaluation of the likely effects of groundwater pumping to supply the proposed development on groundwater levels and wells in the vicinity, and the groundwater basin in which the Site is located.

Section 7: Potential Impacts of using Groundwater to Supply the Reduced Intensity Development (Alternatives D and H). An evaluation of the likely effects of groundwater pumping to supply the reduced intensity alternatives on groundwater levels and wells in the vicinity, and the groundwater basin in which the Site is located.

Section 8: Mitigation Measures. A summary of the potential mitigation measures proposed or considered for the project.

Section 9: Conclusions.

Section 10: Closure/Limitations.

Section 11: References.

2 REGIONAL HYDROGEOLOGIC SETTING

2.1 TOPOGRAPHY

The Site lies near the southern end of both the Santa Rosa Plain and Santa Rosa Valley (**Figure 1**). The Santa Rosa Valley occupies a northwest-trending structural depression in the southern part of the Coast Ranges of northern California. The Santa Rosa Plain is approximately 22 miles long, and about 6 miles wide in the Site vicinity. The ground surface elevation is approximately 90 feet above mean sea level (msl) along the western edge of the plain, rising to approximately 200 feet above msl along its eastern edge. The Santa Rosa Plain is bounded to the northwest by the Russian River Plain, to the west by the Mendocino Mountain Range, and to the east by the Sonoma and Mayacamas Mountains. A series of low hills marks the southern edge of the plain; these hills form a drainage divide between the Santa Rosa and Petaluma Valleys (California Department of Water Resources (DWR), 2004a).

2.2 DRAINAGE

The Santa Rosa Plain is drained principally by the Santa Rosa and Mark West Creeks, which flow westwards to the Laguna de Santa Rosa, which flows in a northwesterly direction near the western edge of the plain (**Figures 1 and 2**). The Laguna de Santa Rosa discharges into the Russian River north of the Santa Rosa Plain.

2.3 CLIMATE

The southern Santa Rosa Plain is characterized by an inland valley Mediterranean climate with cool, rainy winters and warm, dry summers. The climate is marked by variable rainfall, with multi-year periods of high rainfall and of drought (Todd Engineers, 2004). The average annual rainfall recorded at Santa Rosa, based on data collected between January 1931 and March 2004, was 30.30 inches. Over the same period, the average annual minimum temperature was 44.4 degrees Fahrenheit and the average annual maximum temperature was 71.1 degrees Fahrenheit (Western Regional Climate Center, 2004).

2.4 GEOLOGY AND HYDROGEOLOGY OF THE SANTA ROSA PLAIN

2.4.1 HYDROSTRATIGRAPHY

The Site lies within the Santa Rosa Plain sub-basin, which, together with the Healdsburg Area and Rincon Valley sub-basins, makes up the Santa Rosa Valley basin (**Figure 1**). The following are brief descriptions of the main hydrostratigraphic units present in the southern part of the Santa Rosa Plain sub-basin, listed from youngest to oldest:

Basin deposits: These Pleistocene and Holocene age, poorly sorted, organic-rich clays and silty clays generally produce little groundwater (the specific yield is between 3 and 7 percent) and may impede infiltration and downward percolation of water (DWR, 1982). However, they can be locally important in terms of water supply – City of Rohnert Park Well 6 is screened entirely in basin deposits and had an initial yield of 315 gpm (DWR, 1979). The basin deposits are up to 250 feet thick in the Rohnert Park area (DWR, 1979). The presence of these deposits in the Rohnert Park area is described in several DWR reports (DWR, 1979, 1982 and 1987), but they are not differentiated from alluvial deposits on the latest Santa Rosa quadrangle geologic map (California Department of Conservation, 1982).

Alluvial fan deposits: Late Pleistocene to Holocene age alluvial fan deposits cover most of the Santa Rosa Valley and comprise poorly sorted coarse grained sand and gravel, and moderately sorted fine grained sand, silt and clay, with a specific yield of between 8 and 17 percent (DWR, 1982). In the Rohnert Park area, these deposits range between 300 and 450 feet in thickness (DWR, 1979). DWR (1979 and 1982) stated that many of these deposits were previously described as the Glen Ellen Formation. Municipal well yields from alluvial deposits have been recorded up to 800 gpm, with an average of 300 gpm (DWR, 1979). Many of the City of Rohnert Park's municipal wells are completed within these units (Dyett & Bhatia, 2000a).

Glen Ellen Formation: The Glen Ellen Formation is of Pliocene to Pleistocene age and consists of partially cemented gravel, sand, silt and clay, representing older alluvial fan deposits (DWR, 1982). The extent and importance of the Glen Ellen Formation has been interpreted differently by different authors. DWR (2004a) relies on the interpretation of Cardwell (1958), describing the formation as cropping extensively in the center of the Santa Rosa Plain and ranging in thickness from 3,000 feet to less than 1,500 feet on the west side of the plain. However, DWR (1979 and 1982) cautioned that many outcrops formerly described as the Glen Ellen Formation were more recently assigned as alluvial fan and basin deposits. DWR (1979 and 1982) did not assign the alluvial and basin deposits in the Rohnert Park area to the Glen Ellen Formation.

Sonoma Volcanics: These deposits only locally constitute a good water producer (DWR, 1979). They are of highly variable thickness in the Santa Rosa area. In Rohnert Park well 14, 430 feet of volcanic rocks were encountered below a depth of 900 feet (DWR, 1979). However, the formation may be absent beneath western Rohnert Park. The specific yield of the Sonoma Volcanics is highly variable, ranging from 0 to 15 percent, and well yields generally range from 0 to 50 gpm – although City of Rohnert Park well 14 had a yield of 3,500 gpm (DWR, 1979).

Wilson Grove Formation: The Pliocene-age Wilson Grove Formation was formerly known as the Merced Formation. It ranges in thickness from 300 feet to greater than 1,500 feet, and crops along the western side of the plain. It is a marine deposit of fine grained sand and sandstone with thin beds of clay and silty clay and gravel lenses. DWR (2004a) describes the Wilson Grove Formation as the main water-bearing unit of the Santa Rosa Plain sub-basin. Aquifer continuity and water quality are generally very good, with well yields from 100 to 1,500 gpm and specific yields from 10 to 20 percent. The yield of a large water supply well completed in the Wilson Grove Formation can be up to 2,000 gpm (DWR, 1979). Clay lenses can create local confining conditions. Many of the City of Rohnert Park's municipal wells are completed within the Wilson Grove Formation (Dyett & Bhatia, 2000a). It should be noted that DWR (2004a) still refers to the Wilson Grove Formation by its former name: the Merced Formation. The revised name is used on the latest geologic map of the Santa Rosa quadrangle (California Department of Conservation, 1982) and is more commonly used locally (Parsons Engineering Science, Inc. (Parsons), 1995; Todd Engineers, 2004).

Bedrock Formations: The strata described above are underlain by a succession of generally non-water-bearing deposits, including the mid- to late-Pliocene Petaluma Formation, the Miocene to early-Pliocene Tolay Volcanics, and the Jurassic-Cretaceous Franciscan Formation (DWR, 1982).

The DWR (1987) described the aquifers making up the Santa Rosa Plain sub-basin as a "random mixture of unconfined, semiconfined, and confined aquifers".

2.4.2 FAULTING

The Sebastopol fault, trending northwest to southeast and with a downthrow to the northeast, subcrops immediately to the southwest of the Site and underlies the southern part of Rohnert Park (**Figure 2**). This fault may act as a groundwater flow barrier. DWR (1982) reported that a pump test carried out at the Todd Road emergency well (located about 2.5 miles northwest of the Site; **Figure 2**) indicated that the fault impeded groundwater flow. Later testing of the same well (DWR, 1987) could not determine whether the fault acts as a groundwater flow barrier, apparently due to the lack of a suitable observation well on the opposite side of the fault. It should be noted that groundwater elevation contour maps generated by DWR show different interpretations of the Sebastopol fault in terms of its effect on groundwater flow. A selection of these maps is presented as **Figures 3 through 6** of this report. In **Figures 3 and 6**, groundwater elevation contours are indicated to continue across the fault trace, indicating the fault is not interpreted as a hydrogeologic barrier. In **Figure 5**, the contour lines to the southwest of the fault are shown perpendicular to the fault line, indicating the fault is interpreted as a hydrogeologic barrier.

Since the Sebastopol fault does not offset young alluvial fan deposits, it is assumed to be inactive (Parsons, 1995).

Northeast of Rohnert Park, the North College fault (**Figure 2**), which again trends northwest to southeast, but is downthrown to the southwest, accounts for an increased thickness of alluvial deposits beneath the City. This fault does not appear to directly influence groundwater flow (DWR, 1987).

2.4.3 GROUNDWATER FLOW

Groundwater level monitoring undertaken between 1949 and 1952, prior to extensive pumping in the region for agricultural and municipal use, indicated that groundwater was present between 5 and 20 feet bgs and flowed in a general northwesterly direction across the basin, towards the Laguna de Santa Rosa. In the southern part of the Santa Rosa Plain sub-basin, a relatively low hydraulic gradient reflected flat topography and a relative increase in the water-transmitting capacity of the subsurface deposits in the area (Dyett & Bhatia, 2000a after Cardwell, 1958).

Groundwater flow directions have changed in recent history, mainly as a result of increased groundwater pumping. In particular, groundwater extraction in the City of Rohnert Park has resulted in a depression of the groundwater surface, which in turn has caused a local reversal of the northwesterly groundwater flow direction (Todd Engineers, 2004).

Discussion of groundwater levels in the Santa Rosa Plain sub-basin in general, and beneath the City of Rohnert Park and the Site in particular, is included in **Section 3**, which presents a history of groundwater studies in the area.

2.4.4 GROUNDWATER STORAGE

Groundwater within the principal water-bearing deposits is generally present under unconfined conditions, except locally where clay or silt horizons occur and where conditions may be semi-confined (Dyett & Bhatia, 2000a). Locally confined conditions have been described in the Rohnert Park area (DWR, 1979). Cardwell (1958) noted that local artesian conditions once occurred in wells tapping the Wilson Grove Formation in the vicinity of Cotati Auxiliary Landing Field, which was formerly located adjacent to the southeastern Site boundary.

DWR (2004a) cites two calculations of the groundwater storage capacity of the Santa Rosa Plain sub-basin. In 1958, the United States Geological Survey (USGS) estimated the gross groundwater storage capacity as 948,000 acre-feet, based on an average specific yield of 7.8 percent for aquifer materials at depths of 10 to 200 feet. In 1982, the DWR carried out a computer-assisted calculation and derived a storage capacity of 4,313,000 acre-feet. The volume of water in storage in 1980 was calculated to be 3,910,000 acre-feet.

2.4.5 GROUNDWATER RECHARGE

In this report, the term "recharge" denotes groundwater inflows to the Santa Rosa Plain groundwater sub-basin from a variety of sources, including infiltration of precipitation and applied irrigation, stream bed leakage, and groundwater underflow from adjacent basins. The DWR (1982) calculated a water budget for the 15-year period from the 1960-61 to 1974-75 water years. The average annual natural recharge for this period was estimated to be about 29,300 acre-feet, whereas average annual pumping was estimated to be approximately 29,700 acre-feet (DWR, 1982).

The latest DWR information (DWR 2004a, modified 27 February 2004) cites 1982 data reporting that the Santa Rosa Plain groundwater sub-basin as a whole was about in balance, with increased groundwater levels in the northeast contrasting with decreased groundwater levels in the south.

According to Dyett & Bhatia (2000a), recharge to groundwater is derived mainly from surface infiltration around the basin boundaries, and by stream infiltration. In the immediate vicinity of Rohnert Park, recharge is limited by fine-grained deposits present above depths of 240 feet (Dyett & Bhatia, 2000a).

2.4.6 GROUNDWATER QUALITY

The overall quality of groundwater in the Santa Rosa Plain sub-basin is described by DWR (2004a) as "good". A DWR (1982) study found that few wells tested for water quality contained constituents over the recommended concentration for drinking water. However, many wells produced water with aesthetic problems, including high levels of iron and manganese, and elevated hardness. Private well owners also complained about the color and/or taste of the water.

DWR (1982) described the various water types encountered in different parts of the Santa Rosa Plain sub-basin. In a large part of the southern area, "shallow" (0 to 100 feet bgs) zone groundwater was characterized as magnesium and calcium bicarbonate, whilst below 150 feet bgs sodium became the dominant cation. Beneath the eastern part of Rohnert Park, "deep" groundwater (below 200 feet bgs) was characterized as calcium and magnesium bicarbonate, whereas to the west sodium was the dominant cation. These differences in water quality were suggested to be due to "compartmentalization" of aquifers or ion exchange between deep clay layers and groundwater (which would increase sodium concentrations) (DWR, 1982).

DWR (1979) reported total dissolved solids (TDS) values for 14 City of Rohnert Park municipal wells. The TDS concentrations ranged from 135 to 321 milligrams per liter (mg/l). This compares to the California Secondary Maximum Contaminant Level (MCL) of 500 mg/l.

3 PREVIOUS GROUNDWATER USE STUDIES

3.1 USGS STUDY, 1958

Early investigations related to groundwater resources in Sonoma County were generally limited to data collection in connection with specific water resource problems (Cardwell, 1958). According to Ford (1975), the first comprehensive study of the geology of Sonoma County was conducted by Weaver (1949). Subsequently, three water-supply papers were published by the USGS, dealing with geology in various parts of the County. The first of these, authored by Cardwell (1958), covered the Santa Rosa and Petaluma Valley areas.

Cardwell (1958) provided a brief history of water use in the area. Settlement began shortly before 1850, and the earliest drilled wells date from around 1875. By 1951, about 10,000 wells were in use, about 95 percent of them for domestic purposes. At that time, Sebastopol and Cotati relied solely on groundwater for their water supply, whereas Santa Rosa and Petaluma used both surface water and groundwater. The City of Rohnert Park did not yet exist. It was estimated that in 1949, the total volume of groundwater pumped in the Santa Rosa Valley area was 13,300 acre-feet, with approximately 3,100 acre-feet used for public supply. The gross groundwater storage capacity in the Santa Rosa Valley area was estimated to be about 1,000,000 acre-feet, but the calculation was undertaken only for strata within 200 feet of ground surface.

Cardwell (1958) provided the following general description of groundwater conditions in the Santa Rosa Valley in the early 1950's:

> "In Santa Rosa Valley water levels normally are 5-20 feet below the land surface in the spring. Before pumping is begun in the spring, high water levels in deep wells generally approximate those in nearby shallow wells. ...The autumn water levels in deep wells range from 15 to 35 feet below the land surface; in shallow wells, from 10 to 20 feet. Thus, during 1950 and 1951 the average seasonal decline of water levels in shallow wells was less than 10 feet; in deep wells, less than 15 feet. ...The hydrographs for the period of record [1949 to 1954] show no significant or widespread water-level trends other than those attributable to the amount of yearly rainfall. Except for some wells in areas where ground-water draft has increased, the range in seasonal fluctuation has been small, and the rise of water levels during the spring has, in general, been proportional to the amount of rainfall. ...Even in areas of concentrated pumping no net decline in water levels during the spring has been observed."

Plate 2 of Cardwell (1958) shows groundwater elevation contours in the Santa Rosa and Petaluma Valley areas (**Figure 3**). The contours were based on water level measurements taken in Spring 1951, before the City of Rohnert Park was established. The contours indicate a general groundwater flow direction towards the west in the vicinity of the Site; the flow is from the Sonoma Mountains in the east, towards the Laguna de Santa Rosa west of the Site. The groundwater elevation in the Site vicinity is indicated to be between 80 and 90 feet above msl, which equates to 10 feet or less bgs. A depression in the groundwater surface, to an elevation below 60 feet above msl, is centered on an area approximately 1 mile to the northwest of the Site. This is suggested by Cardwell (1958) to be a result of pumping of local irrigation wells.

Cardwell (1958) describes the environs of the Laguna de Santa Rosa as a "swampy area", with groundwater levels at about the same level as the water level in Laguna.

Cardwell (1958) provides the following conclusion on aquifer conditions:

> "...the fluctuations for the period 1949-53 show that the principal ground-water body has been essentially fully recharged in the spring of each year, which in turn indicates that there is no overdraft in Santa Rosa Valley."

3.2 SONOMA COUNTY STUDY, 1971

In 1971, the Sonoma County Planning Department, recognizing that groundwater use had been increasing over the previous 25 years and would continue to be of importance in the County's growth, requested that the DWR undertake a cooperative study of the County's groundwater resource. The aim of the study was to provide the County with guidelines regarding groundwater resources and their use, upon which to base future planning. The first published result of the study was a description of geology and hydrogeology (Ford, 1975) which, along with the work of Cardwell (1958), formed the basis for the current understanding of the area, as outlined in **Section 2.4.1**. This was followed in 1982 by an evaluation specific to the Santa Rosa Plain (DWR, 1982; see **Section 3.4**).

3.3 CITY OF ROHNERT PARK / DWR STUDY, 1979

In 1979, a study was undertaken by the City of Rohnert Park and the DWR "to determine the best management of the water resources of the basin to meet present and future water demands". The study was undertaken at a time when Rohnert Park was Sonoma County's fastest-growing city, with a population of 18,000 and an increasing water demand (670 million gallons in 1975; 1,080 million gallons in 1978 (DWR, 1979)). The 1979 study projected that, based on an expected maximum population that would be reached by 1990, the water demand would rise to 1,790 million gallons.

In 1979, most of Rohnert Park's water supply was derived from groundwater, via the City's municipal well system (which at the time comprised 16 wells, with two to be added during the summer of 1979). The City was entitled to an allocation of 1 million gallons per day (mgd) of water from the Sonoma County Water Authority (SCWA). However, water from this source was only used to meet peak demands, and in 1978, only 56 million gallons (equivalent to 5 percent of the total water demand) were imported from SCWA.

The 1979 report includes seven maps showing groundwater elevations in each spring and fall season between spring 1976 and spring 1979. Each map shows a pumping depression beneath Rohnert Park. The spring 1976 map (**Figure 5**) shows the depression lowering groundwater elevations to below sea level. Each map also shows a groundwater depression to the northwest of – and sometimes encroaching beneath – the Site. Such a depression was also indicted on the 1951 map and attributed to agricultural pumping (Cardwell, 1958; see **Section 3.1**). The maps show groundwater elevations beneath the Site varying from about 32 to 65 feet above msl; 25 years earlier, the groundwater elevation had been about 80 feet above msl (see **Section 3.1**).

As part of the study, theoretical cones of depression for each City well were determined in an attempt to examine the magnitude of a future composite pumping cone beneath the City, and in particular to evaluate the effects of the City's pumping on wells outside Rohnert Park. The study concluded that wells northwest of the City, in the vicinity of Wilfred, might be affected by the City's pumping, but that elsewhere, water levels had remained at "essentially the same level" as prior to the development of the City of Rohnert Park (DWR, 1979).

The study noted that there were insufficient available data to enable the long-term effects of groundwater pumping beneath the City to be evaluated.

At the time, existing City plans called for the intensified use of groundwater to meet future needs; for total buildout the City envisaged a system of 30 to 35 wells, with an average yield of 300 gpm per well and a total yield of up to 15 mgd. While the reasonably anticipated total well yield (accounting for well down-time for repairs) would meet the projected average daily demand (approximately 5 mgd), it would not accommodate the estimated peak daily demand of 12.25 mgd at total buildout, even accounting for the SCWA entitlement of 1 mgd. Therefore, DWR (1979) cautioned:

> "Problems may result if ground water is used as the sole water supply to meet both base and peak water demands of the City. Safe yield of the basin cannot be determined until hydrologic studies of the Santa Rosa Plain are completed. However, there is a large pumping depression underneath Rohnert Park at the present time, and additional pumping wells will increase the size of this depression. ...Greatly increasing the number of pumping wells may cause an overdraft situation; if future studies indicate an overdraft situation, a recharge program should be initiated."

It was anticipated that a larger water supply would be available to Rohnert Park from SCWA, following that agency's completion of its Warm Springs Dam project. It was noted hypothetically that importation of 100 percent of the City's water supply would reduce the demand on the groundwater basin and allow the pumping depression to slowly recover, though it was conceded that importation costs could be higher than pumping costs. However, it was also noted that groundwater extraction costs would increase if the pumping depression was allowed to deepen and well efficiency decreased as a result.

The 1979 study made the following recommendations:

- The optimum method for meeting future water demands of Rohnert Park is the conjunctive use of ground and surface water;
- The City should consider increasing surface water imports and reducing groundwater pumpage if future studies indicate an overdraft condition in the Southern Santa Rosa Plain;
- Imported water could be used to recharge the ground water basin, by way of recharge wells or surface infiltration; and,
- New municipal wells should be sited carefully, ensuring penetration of the maximum thickness of water-bearing materials.

3.4 DWR SANTA ROSA PLAIN GROUNDWATER MODEL, 1982

The DWR study reported in 1982 included an evaluation of geologic and hydrogeologic characteristics of the Santa Rosa Plain groundwater sub-basin, an estimate of the volume of groundwater contained in the sub-basin, and considerations of recharge and the effects of pumping. A mathematical model was developed to assist the evaluation.

In DWR's (1982) model, the Sebastopol fault was assumed to be a no-flow boundary. However, the available data used to support this were inconclusive (see **Section 2.4.2**).

DWR (1982) reported that as of spring 1980, the total quantity of groundwater present in water-yielding materials beneath the Santa Rosa Plain study area was 3,910,000 acre-feet. The average annual volume of groundwater pumped between 1960 and 1975 was estimated to be 29,700 acre-feet. This compared with an annual average recharge estimated at 29,300 acre-feet. The estimated total storage capacity of the Santa Rosa Plain sub-basin was 4,313,000 acre-feet.

The 1982 report includes groundwater elevation maps for fall 1960, fall 1975, and spring 1980. The fall 1960 map (**Figure 4**) shows the groundwater elevation beneath the Site at approximately 70 feet above msl, 10 feet or more lower than in 1951 (see **Section 3.1**). The 1975 and 1980 maps show depressions in the groundwater surface beneath Rohnert Park, down to about 50 feet above msl. These maps do not depict the pumping depression shown to the northwest of the Site on the earlier DWR (1979) maps. It appears that the DWR (1982) maps, which cover the entire Santa Rosa Plain, were constructed using fewer data points than the earlier maps and thus do not show local detail.

DWR (1982) noted that between 1960 and 1975, groundwater levels beneath the southern Santa Rosa Plain declined as much as 40 feet (2.7 feet per year), with a pumping depression developing beneath Rohnert Park. At the time, Rohnert Park was the largest single user of groundwater in Sonoma County. Between July 1980 and June 1981, the City pumped 4,000 acre-feet of groundwater. The study noted that of 18 municipal wells present in Rohnert Park at the time, 12 showed declines in successive spring water level measurements between 1977 and 1981, whereas six showed water level rises. The measured declines ranged from 30 to 50 feet, and the rises ranged from 10 to 29 feet. The average change in spring measurements was a decline of 5 feet. According to DWR (1982), such rates of change "warrant monitoring by ground water users in the area".

In spring 1982, water levels in 14 of the wells were between 1 and 43 feet higher than the levels of the previous spring, because of the high winter rainfall of that year.

In 1977, piezometers were installed in a Rohnert Park well, at depths of 214, 312 and 640 feet. Water levels in the two shallower piezometers were similar, and about 20 feet above the water level in the deepest piezometer. Subsequent measurements showed that water levels declined in the shallower piezometers about 5.5 feet per year between 1977 and 1981. In the same period, the water level in the deepest piezometer declined about 12.5 feet per year; the difference in water levels between the shallower and deepest piezometers increased to almost 60 feet. The deepest piezometer was installed within the depth interval from which most Rohnert Park municipal wells were extracting water. The differences in water levels between the piezometers indicate the presence of restrictive layers above the depths at which the City extracts most of its groundwater (DWR, 1982).

Among DWR's (1982) conclusions were:

> "The Santa Rosa plain ground water basin as a whole is about in balance, with increased ground water levels in the northeast contrasting with decreased ground water levels in the south."

Nevertheless, it was recommended that a basin management plan and groundwater management program should be adopted for the area, and that artificial recharge techniques should be evaluated specifically for the Rohnert Park area (DWR, 1982).

3.5 DWR SANTA ROSA PLAIN GROUNDWATER MODEL, 1987

In 1983, SCWA and the City of Rohnert Park entered into a contract with the DWR to calibrate and verify a new mathematical model of the Santa Rosa Plain groundwater sub-basin. Due to data limitations, the study focused on the area of the Santa Rosa Plain groundwater sub-basin to the east of the Sebastopol fault line (it was not known whether this fault line constitutes a true no-flow boundary; see **Section 2.4.2**). Within the study area, which included Rohnert Park and the Site, the model was able to replicate historical measurements of groundwater elevations for the period 1978-1983 (DWR, 1987).

The 1987 report includes maps showing groundwater elevations in fall (October) and spring (April) between fall 1977 and spring 1984. Each map shows a pumping depression beneath Rohnert Park, and a second, shallower depression to the northwest of the Site (although in some cases the two depressions merge). The Rohnert Park depression is shown to decline continuously between fall 1977 and spring 1984 (**Figure 6**), reaching a depth of about 40 feet below msl in the fall of 1983. The groundwater elevation beneath the Site tended to be around 50 feet above msl in the spring and 40 feet above msl in the fall, with an overall decline during the period of between 5 and 10 feet.

The 1987 report describes aquifer tests undertaken on five wells in the Santa Rosa Plain. One of these, the Todd Road well, is located about 2.5 miles northwest of the Site. The lithology at this well includes predominantly clayey basin and alluvial fan deposits overlying sands and gravels of the Wilson Grove Formation. The aquifer was described as being 160 feet thick, and the Todd Road well was screened between 650 and 800 feet bgs (DWR, 1987).

During the aquifer test, groundwater levels were observed in a cluster of three observation wells, which were 80, 257 and 570 feet deep. After the Todd Road well was pumped for 24 hours at 1,500 gpm, the two shallowest observation wells remained unaffected, suggesting the existence of a hydraulic discontinuity between shallow and deep wells. DWR estimated that the depth of the discontinuity was about 400 to 500 feet. The lithologic log for the Todd Road well shows a clay layer from about 220 to 250 feet bgs. The groundwater level in the deepest observation well was drawn down 50 feet. Data interpretation using different methods derived transmissivity values of 10,100 and 15,200 gallons per day per foot, hydraulic conductivity values of 8.5 and 12.7 feet per day, and storativity values of 0.0015 and 0.001 (DWR, 1987).

3.6 SCWA STUDY, 1995

In 1995, Parsons reported the results of a hydrogeologic assessment undertaken for SCWA (Parsons, 1995). The study was undertaken as part of SCWA's evaluation of a Water Supply and Transmission System Project (WSTSP), designed to meet future water supply needs in SCWA's supply area. SCWA had determined that the WSTSP would need to supply an additional 30,000 acre-feet per year (AFY) of water. Three solutions involving groundwater were under consideration:

1. The additional water would be supplied by conventional production wells;
2. An aquifer storage and recovery (ASR) project would be undertaken to divert Russian River water during the wet season and inject it into the aquifer beneath the project area for storage and later extraction during the summer; or,
3. ASR would provide only an emergency water supply.

The area under consideration for application of these alternatives was located northwest of Rohnert Park; the southern boundary of the area was near the Todd Road well as indicated on **Figure 2**, approximately 2.5 miles northwest of the Site. The area was chosen in part because an earlier study had suggested this area to have the highest potential for supporting high-capacity municipal wells (Parsons, 1995).

Parsons carried out two-dimensional computer modeling to assess the effects on the aquifer of the three groundwater supply scenarios. For the first scenario, it was calculated that 16 extraction wells would be required, each pumping at 1,200 gpm, and that these could result in maximum groundwater surface drawdowns of between 200 and 520 feet, with an average of 360 feet, in the central portion of the study area after 20 years of pumping. Lesser effects were modeled for alternatives 2 and 3.

Parsons (1995) noted that because 90 percent of the approximately 222 domestic wells in the study area were screened above 200 feet depth, the adoption of groundwater pumping under alternative 1 could eliminate the water supply of many domestic users. The final design of the WSTSP relied upon surface water rather than groundwater (see **Section 4.5.2**).

3.7 DYETT & BHATIA STUDY, 2000

A study reported in the Environmental Impact Report (EIR) for the City of Rohnert Park's 2000 General Plan found that groundwater levels in the vicinity of the City had been declining for many years, and continued to decline until 1988, at which time they tended to stabilize. However, wells on the eastern side of the City, where the primary water-bearing deposits are relatively thin, continued to exhibit a declining trend in water levels through 1999 (Dyett & Bhatia, 2000a).

Between 1970 and 1999, groundwater levels declined from approximately 35 feet at the northwest perimeter of the City of Rohnert Park's proposed Urban Growth Boundary, to "approximately 100 to 150 feet" along the eastern boundary (Dyett & Bhatia, 2000a). The decline of groundwater levels was a consequence of extraction exceeding recharge. The EIR includes a graph showing the average annual pumping rate from the City well field and the estimated annual aquifer recharge rate, for the years 1970 to 1999. The pumping rate is shown to increase markedly and in an almost linear fashion from 1970 to 1980, and then develop a more gradual trend. From the mid-1990's, the City began to rely less on groundwater and more on supply from SCWA, for economic reasons. Nevertheless, since 1975 the pumping rate exceeded the estimated recharge rate, by between 0.15 mgd (1975) and 3.2 mgd (1996). In 1999, the difference between the pumping and estimated recharge rates was approximately 2.6 mgd (Dyett & Bhatia, 2000a).

3.8 KLEINFELDER STUDY, 2003

The Kleinfelder study was undertaken on behalf of Sonoma County to provide a better understanding of groundwater conditions in water-scarce areas of the County. Three areas were selected for pilot study:

- Joy Road, approximately 13 miles west of the Site;

- Mark West Springs, about 12 miles north of the Site; and,

- Bennett Valley, about 4 miles northeast of the Site.

Though the results of the Kleinfelder study are not specific to the Site, they do provide an indication of groundwater issues that are affecting Sonoma County as a whole.

The study found that the mean depth to water in new wells drilled in all three areas was increasing over time. It was determined that the change was not attributable to decreasing rainfall. Kleinfelder stated that the increasing depth to water in new wells, plus reports of seasonal well failures and dropping water levels in other wells, suggested that an overdraft condition existed in Sonoma County (Kleinfelder, 2003).

3.9 SONOMA COUNTY GRAND JURY STUDY, 2003-2004

In 2003, the Sonoma County Grand Jury began an investigation into "how the county, its cities and public agencies monitor their groundwater resources in order to assure adequate water supply availability for today and sustainability for future generations". The study was initiated due to "overwhelming citizen interest and concern regarding the availability and sustainability of Sonoma County's groundwater" (Sonoma County Grand Jury, 2004).

Among the findings of the study were the following:

- The volume of groundwater being extracted in the County is not monitored and is not definitively known (there are approximately 40,000 domestic wells in the County, but some of these are not functioning);

- Sonoma County's proposed General Plan Update to the year 2020 includes a "Water Resources Element", designed to ensure the County's water resources are sustained and protected;

- Although many cities in the County refer to water supply in their general development plans, they do not always consider the present availability and/or future sustainability regarding other water users outside their own city limits;

- There is currently no regional governing board to monitor and coordinate countywide water issues; and

- In 1992 the California State Legislature adopted the Groundwater Management Act, which provides a framework for groundwater management plans that can be adopted by any agency, city or county that provides water service. Sonoma County has no such plan.

Among the report's recommendations were that the County and each of its cities should adopt a sustainable water element as part of their general plan, together with a comprehensive groundwater management plan.

3.10 TODD ENGINEERS STUDY, 2004

In 2004, Todd Engineers carried out a groundwater study as part of the draft EIR for a proposed project located in Canon Manor West, an unincorporated area of Sonoma County located adjacent to Rohnert Park, about 3.5 miles southeast of the Site. The proposed project includes the provision of a new groundwater supply to Canon Manor West. Thus, the groundwater study and the EIR, were completed to address similar issues to those that are of concern to the Graton Rancheria project.

The proposed new groundwater supply would be provided by Penngrove Water Company (Penngrove). Initially, it was proposed that the increased production could be provided by an existing well. Among the objectives of the groundwater study were to evaluate the hydrogeologic setting of the existing well, to assess potential impacts to local wells on increased pumping of the Penngrove well, and to assess cumulative effects on groundwater resources (Todd Engineers, 2004).

The study included an analysis of groundwater level trends in the study area, and found that increased groundwater pumping by Rohnert Park in the 1970's and 1980's was a major factor in local groundwater level decline. However, the study also noted that city pumpage stabilized after 1985, and that groundwater levels also stabilized, with the local system establishing a new equilibrium between recharge and pumpage (Todd Engineers, 2004). This implies that recharge increased to meet the increased pumping.

Todd Engineers carried out a water balance analysis for the study area, which was defined as the watershed above the USGS gage station on Laguna de Santa Rosa at Stony Point Road, and thus encompasses the Site. The study was based on water years 1986-87 through 2000-01. As part of the analysis, Todd Engineers estimated groundwater pumping by municipal, commercial and domestic users as 6,927 AFY, which equates to an average of about 6.2 mgd. The water balance analysis indicated a close balance of inflows (77,511 AFY) and outflows

(76,716 AFY), with a computed change in storage of +795 AFY. This small but positive change in storage was considered "consistent with the observed groundwater level trends that have leveled off and even increased slightly since 1987…" (Todd Engineers, 2004).

In evaluating the long-term effects of the proposed project on groundwater levels, a major factor and area of uncertainty was determined to be future pumping by the City of Rohnert Park. The study noted that should the City decrease its reliance on groundwater pumping, in line with the City's General Plan (see **Section 4.1.1**), then this would compensate entirely for not only the water demand of the Canon Manor West project, but also for all other future increased groundwater demands predicted by Todd Engineers (including the proposed Graton Rancheria development). The converse was that if Rohnert Park did not decrease its pumping from current levels, then the predicted groundwater demands could lead to another period of groundwater level decline.

3.11 ROHNERT PARK WATER SUPPLY ASSESSMENT, 2005

Rohnert Park's Water Supply Assessment (WSA) was published in January 2005 (Winzler & Kelly, 2005). The WSA was prepared in compliance with Senate Bill 610 to assist the City in making decisions related to land use and water supply through the year 2025.

The report provides background on the City's water supply, together with a detailed study of groundwater conditions in the upper Laguna de Santa Rosa watershed (similar to the area studied by Todd Engineers – see **Section 3.10**). For the purposes of the study, the subsurface was divided into a shallow zone (ground level to 200 feet bgs), an intermediate zone (200 to 600 feet bgs), a deep zone (600 to 800 feet bgs) and a lower zone (deeper than 800 feet bgs). After study of hydrographs for wells in the area, the following conclusions were made:

- Most shallow zone wells located on the periphery of the City's well field were interpreted to exhibit relatively stable long-term groundwater levels, with little response to changes in (City) pumping or climatic conditions.

- Intermediate zone wells were interpreted to exhibit changes in groundwater elevations in response to changes in the City's pumping. Most of the City's wells extract water from the intermediate zone. In central Rohnert Park wells, Spring groundwater elevations were generally stable from 1977 to 1981, declined from 1982 to 1990 when City pumping increased, and were stable to slightly increasing from 1990 to 1997 when total pumping was relatively stable. Groundwater levels remained stable until 2003, and then showed a marked recovery as pumping was reduced.

The WSA states that the groundwater level rises (in intermediate zone wells) since 1990 indicate that the groundwater level declines of the 1980's were not the result of overdraft conditions. Furthermore, the report states there is no indication of generally declining groundwater levels elsewhere in the sub-basin; that is, there is no indication of overdraft on a sub-basin scale.

The WSA discusses water budgets for the area, including review of previous recharge estimates made by PES (see **Section 3.7** of this report) and Todd Engineers (**Section 3.10** of this report). The WSA concludes that PES's estimate of annual average recharge (1.6 mgd) pertains to a limited geographic area that does not include the areas of highest recharge to the northeast of Rohnert Park. The WSA also presents calculations that derive a recharge estimate of 8,300 AFY (7.4 mgd) from data previously reported by Todd Engineers (Todd, 2004).

The WSA provides an analysis of future groundwater supply sufficiency, noting that the City has recently shifted its primary source of water supply from groundwater to surface water supplied by SCWA. The report estimates projected future pumping in the study area through 2025, and notes that the projected total of 7,350 AFY (6.6 mgd) is less than the total pumping in the 1990 to 1997 time period of 8,700 AFY (7.8 mgd) during which groundwater elevations were stable or rising slightly. The conclusion reached by the WSA is that projected groundwater pumping through 2025 falls in the range of historically sustainable pumping and would not result in overdraft conditions in the study area.

The O.W.L. Foundation (a local citizen's group) filed suit in Superior Court challenging adoption of the WSA in 2005. On May 31, 2006 the court issued a decision invalidating the WSA. The reason cited for this action was the fact that the WSA's evaluation of water supply sufficiency was based on considering existing groundwater demand for a study area that encompassed only the upper Laguna de Santa Rosa Watershed, and not the entire groundwater basin or sub-basin in which the project is located. The court also found that the WSA used the DWR's definition of "critical overdraft" rather than the DWR's definition of "overdraft" when discussing the adequacy of the groundwater supply. The court specified that it was not ruling as to the sufficiency of the water supply, but only the method used to support the sufficiency determination. It should be noted that the court's decision to invalidate the WSA is currently being appealed by the City of Rohnert Park.

3.12 JOINT USGS-SCWA STUDY

Starting in October 2005, USGS and SCWA undertook a 5-year cooperative study of the Santa Rosa Valley groundwater basin. USGS staff indicated the work will include data compilation and conceptual model development, preparation of a numerical groundwater flow model of the basin, and reporting. Among other items, the final report will address the question of whether the basin is in overdraft. In fiscal year 2006, USGS completed a preliminary analysis of the hydrostratigraphy of the basin. In fiscal year 2007, this analysis will be used to help develop a conceptual model, which in turn will be used to build the numerical model. The project is still in its preliminary stages and there are not yet any findings to report.

4 GROUNDWATER USE IN THE SANTA ROSA VALLEY BASIN

4.1 MUNICIPAL AND INDUSTRIAL GROUNDWATER USE

DWR estimated the average municipal and industrial groundwater pumping in the Santa Rosa Plain sub-basin (the largest of the three sub-basins in the Santa Rosa Valley basin) from 1974 through 1984 as approximately 5,200 AFY and 1,300 AFY, respectively (DWR, 1987).

The California Water Code requires the DWR to publish an update of the California Water Plan every five years. This is accomplished through the Bulletin 160 series, which includes an evaluation of agricultural, urban and environmental water uses throughout the state during the study period. In addition, through 1998, Bulletin 160 included projected future water use. The most recent update to the California Water Plan was issued in Bulletin 160 Update 2005. WorleyParsons Komex contacted DWR staff to obtain spreadsheets summarizing recent water use data used in this update (DWR, 2006). In addition, spreadsheets of forecast water use compiled for the 1998 update were obtained. The spreadsheets included data for the Santa Rosa and Dry Creek (Sonoma County) Detailed Analysis Units (DAUs), which include much of the Santa Rosa Plain but not the entire Santa Rosa Valley groundwater basin. Based on data compiled by the DWR in these spreadsheets, the municipal and industrial groundwater pumping in these DAUs in 1999 was 18,400 acre-feet. Annual municipal and industrial groundwater demand for these detailed analysis units is projected to decrease slightly to 17,000 AFY by the year 2020. Because the urban/municipal areas of the Santa Rosa Valley basin are included in these DAUs, these are considered reasonable estimates for the basin. Municipal groundwater pumping data are summarized in greater detail below.

The primary municipal groundwater pumpers in the Santa Rosa Valley basin are the Cities of Rohnert Park, Cotati, Sebastapol,[1] Windsor (including Shiloh Estates and Sonoma County Airport) and (in the future) Santa Rosa.[2] Additional municipal groundwater pumpers relying on the basin include SCWA, Penngrove Water Company and Sonoma State University. The

[1] The City of Sebastapol operates several wells that are reported by DWR to draw water from the Santa Rosa Plain groundwater sub-basin; however, a Water Supply Assessment for the City's Northeast Area Specific Plan (PES, 2007) indicates that the wells pump water from the Wilson Grove Formation Highlands groundwater basin, with some possible contribution from the Santa Rosa Plain groundwater sub-basin.

[2] The City of Healdsburg also pumps municipal water from several wellfields; however, these wells are located adjacent to Russian River and Dry Creek and function as Ranney collectors to pump surface water from the rivers under water rights held by the City and SCWA.

largest municipal groundwater pumper in the basin historically has been the City of Rohnert Park, which is discussed in additional detail in **Section 4.1.1**. Information regarding the other municipal groundwater pumpers in the basin is summarized below.

The City of Cotati obtains Russian River water from SCWA and operates three groundwater production wells with a capacity of 2.3 mgd and yields ranging from 310 to 670 gpm, and has pumped, on average, about 380 AFY (0.3 mgd) since 1986 (Todd Engineers, 2004; SCWA, 2006; Winzler & Kelly, 2006). The Urban Water Management Plan for Cotati indicates that the City pumped an average of 403 AFY between 1988 and 2005 (Winzler & Kelly, 2006), that pumping decreased to 49 AFY in 2005, and that groundwater demand is projected to be 243 AFY in 2015, 172 AFY in 2020 and 133 AFY in 2025. The UWMP notes that projected groundwater pumping volumes are the amount of pumping that is assumed to be needed to meet the forecast demand given assumed surface water delivery volumes from SCWA. Ludorf & Scalmanini, in a document prepared in support of the City of Rohnert Park Water Supply Assessment, indicated that the City of Cotati groundwater demand would remain constant (Ludorf & Scalmanini, 2004b). Additional information regarding projected future groundwater use was requested from the City of Cotati but has not been provided as of the date of this report.

DWR reported that the City of Sebastapol pumped approximately 494 million gallons (1,520 acre-feet) from a system of four wells in 2001 (DWR, 2006). DWR projects that groundwater pumping by the year 2020 will be 2,400 AFY. A Water Supply Assessment prepared for the City of Sebastapol Northeast Area Specific Plan (PES, 2007), indicates that the City pumped an average of 1,400 AFY between 2001 and 2006. The Water Supply Assessment estimates future groundwater pumping to grow to 1,617 AFY over the next 20 years based on planned projects and growth projections from the Association of Bay Area Governments. It should be noted that although DWR indicates the City's wells are located in the Santa Rosa Plain sub-basin, the Water Supply Assessment indicates they are actually located in the adjacent Wilson Grove Formation Highlands groundwater basin. The Water Supply Assessment states that the planned cooperative USGS-SCWA study will shed light on relative percentage of water the City derives from each basin. For the purposes of this report, we have assumed the City derives its groundwater supply from the Santa Rosa Plain groundwater sub-basin.

The City of Windor (including Shiloh Estates and Sonoma County Airport) pumped approximately 1,300 million gallons (4,000 acre-feet) in 2001. By 2020, DWR projects that the City of Windsor will pump approximately 2,400 AFY of groundwater for its municipal supply.

The City of Santa Rosa currently derives all of its municipal water supply from water deliveries by SCWA and has not used any groundwater in the last 25 years; however, the City has several wells it can use on a standby emergency basis and has taken steps to make well water available as part of its regular water supply in the near future (City of Santa Rosa, 2006). The City's water demand is projected to increase by almost 10,000 AFY between 2005 and 2030, exceeding its current allotment under the Restructured Agreement for Water Supply with SCWA (SCWA, 2006). The additional water is anticipated to be derived from recycled water, additional surface water deliveries that may become available, and/or groundwater. The City's projected groundwater pumping is expected to reach 900 AFY by 2010, 1,800 AFY by 2015, and 2,300 AFY from 2020 through 2030. Groundwater is not expected to be used as a water supply for the City's planned Southwest Projects (City of Santa Rosa, 2004).

Sonoma State University operates two private wells that have supplied, on average, 79 AFY over the last 10 years. Groundwater use has been increasing as student numbers increase; in 2003 the total pumped was 106 AFY (Todd Engineers, 2004). The university reportedly expects its groundwater demand to increase in the future as enrollement increases from its present value of approximately 7,000 full-time equivalent students (FTES) to 10,000 FTES by 2010 (http://www.sonoma.edu/facilities/proposal.htm). The projected groundwater use by 2025 is reported to be 186 AFY (Sonoma State University, 2006)

Penngrove Water Company serves a small area to the east of Rohnert Park; its average annual groundwater pumping over the 12 years prior to 2003 was 37 AFY (Todd Engineers, 2004). Information provided by the Penngrove Water Company indicates they currently operate three water systems. Pumping for the Canon Manor system alone averaged 34.5 AFY for 2004 and 2005. Pumping is expected to increase in the near future with the planned addition of approximately 200 new customers. Groundwater pumping by Penngrove Water Company is projected to increase to 178 AFY by the year 2025 (Ludorff & Scalmanini, 2004).

SCWA's Todd Road well (located about 2.5 miles northwest of the Site) pumped an average of 1.4 mgd (1,600 AFY) over the four water years from 1999 to 2002. The total average production from SCWA's three wells on the Santa Rosa Plain over the same period was 2.8 mgd (3,100 AFY) (Todd, 2004). The Draft 2005 Sonoma County Urban Water Management Plan (SCWA, 2006) indicates that the amount of groundwater pumped by SCWA from its three wells in the Santa Rosa Plain sub-basin was 2,363 acre-feet in 2000 and increased steadily to 5,906 acre-feet in 2005. The projected future pumping from these wells is 3,870 AFY through 2030.

As discussed in greater detail in **Section 4.5**, the City of Rohnert Park operates a system of 42 groundwater wells that has pumped between 850 and 4,600 AFY between 2000 and 2005. By 2010, the City plans to use groundwater only as a backup/emergency water source. The City adopted a Water Supply Resolution in 2004 that caps future groundwater pumping at a rate of 2,577 AFY. An Urban Water Management Plan prepared for the City of Rohnert Park in 2007 projects future groundwater pumping at the limits adopted under the City's Water Supply Resolution through the year 2030. Therefore, for the purposes of this report, we have assumed that future groundwater pumping by the City of Rohnert Park will be 2,577 AFY (Winzler & Kelly, 2007).

4.2 AGRICULTURAL GROUNDWATER USE

The Site lies in an agricultural area, where groundwater pumping for irrigation purposes is significant. DWR (1987) estimated that, based on data from water years 1974 through 1984, agricultural pumping in the Santa Rosa Plain sub-basin accounted for approximately 14,200 AFY. This represented approximately 54 percent of the total pumping (which included rural, municipal and industrial uses).

Data compiled by the DWR for the pending Bulletin 160 update indicates that 12,200 acre-feet of groundwater were used in 1999 for agricultural supply that year in the Dry Creek and Santa Rosa DAUs, which represents approximately 40 percent of the total groundwater pumped for those areas. Projections for future agricultural groundwater use were not available, but with the current influences of urbanization and increasing reliance upon recycled water in the area, it is reasonable to assume that agricultural groundwater demand during average years will remain similar to, or decrease somewhat from, the current demand.

In order to estimate agricultural groundwater demand in the Santa Rosa Valley Basin, agricultural land use (cropping) data were downloaded from the DWR website (http://www.landwateruse.water.ca.gov/), overlayed on the boundaries of the Santa Rosa Valley Basin (downloaded from the California Spatial Information Library (CaSIL) and geospatially analyzed. Land use data downloaded form DWR included spatial data regarding agricultural land use types in the basin, whether the land is irrigated, and the mode of irrigation (e.g., surface water, groundwater, reclaimed water, etc) for the most recent year available, which was 1999. Information regarding agronomic water application rates (in acre-feet/acre) for the various crops identified were also downloaded from DWR. These rates were calculated by

DWR based on local 2002 meteorological conditions in the Santa Rosa Detailed Analysis Unit.[3] As summarized below, 22,716 acres of irrigated agricultural land were identified in the Santa Rosa Valley Basin in 1999. The distribution of irrigated agricultural land and the mode of irrigation are shown in **Figure 7.** Of the irrigated area, 14,972 acres or 65.9% were reported to be irrigated using groundwater.

Source of Agricultural Irrigation Water	Irrigated Area (acres)	Percent of Total Irrigated Area
Surface Water	24	0.1
Groundwater	14,972	65.9
Reclaimed Water	1,620	7.1
Unspecified or Unknown	6,100	26.9
Total	22,716	100

The table below summarizes the land use (cropping) of the irrigated agricultural areas in the Santa Rosa Valley basin (shown graphically in **Figure 8**) and the amount of area devoted to each use. Also shown are the irrigation water application rates associated with each crop type and the calculated total amount of groundwater applied for irrigation purposes.

[3] Precipitation in 2002 in the Santa Rosa Plain was near average, so these data represent reasonable irrigation rates for average years.

Crop Type	Groundwater Irrigated Area (acres)	Irrigation Rate (acre-feet/acre)	Amount of Groundwater Applied (acre-feet)
Grain and hay crops	42	0.8	34
Pasture	3,292	4.69	15,440
Truck, nursery and berry crops	193	1.84	354
Deciduous fruits and nuts	144	2.7	388
Vineyards	10,970	0.96	10,531
Field crops	281	2.207	621
Idle land, new land being prepared for production	49	1.52	75
Total	**14,972**		**27,444**

Based on the above analysis, the amount of groundwater pumped for agricultural use from the Santa Rosa Valley Basin in 1999 is 27,444 acre-feet. In similar fashion, the amount of applied irrigation water from other sources is estimated as follows:

- Surface water: 185 acre-feet;
- Reclaimed Water: 6,411 acre-feet; and
- Unkown or unspecified sources: 7,865 acre-feet.

In data compiled for the Bulletin 160 update (DWR, 2006) DWR reports that the amount of surface water used for agricultural irrigation in the Dry Creek and Santa Rosa DAUs is 13,400 acre-feet, and the amount of groundwater is 12,200 acre-feet. This discrepancy was discussed with DWR staff, who indicated that most of the reported surface water is actually groundwater pumped from wells located close to the Russian River or Dry Creek, and therefore essentially induced infiltration of surface water and not water pumped from storage in the groundwater basin. Therefore, the GIS-derived estimate of pumped groundwater was adjusted by subtracting 13,400 acre-feet to generate a lower-bound estimate of agricultural groundwater use of 14,044 acre-feet, or approximately 14,000 acre-feet. This is similar to the average agricultural groundwater use estimated by DWR between 1974 and 1984 (DWR, 1987).

The difference between adjusted (lower bound) GIS-derived estimate and DWRs reported agricultural groundwater use in the Santa Rosa and Dry Creek DAUs is approximately 1,800 acre-feet. This is a relatively close correlation for this type of estimate. It is also similar to the difference between the GIS-derived estimate of total agricultural water use in the Santa Rosa Valley basin (41,900 acre-feet) and the total agricultural water use reported by DWR for the Santa Rosa and Dry Creek DAUs (39,800 acre-feet), which is 2,100 acre-feet. This suggests that the difference in the boundaries between the DAUs and the Santa Rosa Valley basin could account for at least some of the difference in the estimated agricultural groundwater use.

The DWR (2006) estimated the amount of reclaimed water used for agricultural irrigation in the Santa Rosa and Dry Creek DAUs is 11,700 acre-feet, which is approximately 3,920 acre-feet more than was estimated from the available GIS data. According to DWR staff, this total consists mainly of reclaimed wastewater from the City of Santa Rosa reported that is applied for agricultural purposes. Two scenarios could possibly account for the difference in volume estimates. First, the water application rate for lands irrigated with reclaimed water could be higher than the typical rates estimated by DWR if sprayfield irrigation is essentially being used as a means of reclaimed water disposal. Secondly, it is possible that the additional reclaimed water is attributed in the GIS data to coming from an unknown or unspecified sources. The first scenario leaves approximately 7,870 acre-feet of irrigation water as coming from unknown or unspecified sources, whereas the second scenario leaves approximately 3,950 acre-feet. An upper bound estimate for agricultural groundwater use, based on the assumption that this water is actually groundwater, would range from approximately 18,000 to 21,900 acre-feet. Based on changes in agricultural landuse patterns (primarily vineyard development) and increasing urbanization between the 1980s and the present, it seems more reasonable to adopt 18,000 AFY as the upper bound number.

DWR (2006) projects that agricultural groundwater use in the Santa Rosa and Dry Creek DAUs will decrease to 500 AFY by the year 2020. This projection is based on projected urban growth in areas away from the Russian River and Dry Creek, and assumes that most of the remaining agricultural groundwater pumping will occur near these streams, where the groundwater is derived from induced infiltration of surface water. This represents the low bound of future projected agricultural groundwater use. For the purposes of this report, we propose an upper bound estimate of 14,000 AFY based on the assumption that agricultural groundwater use will decrease slightly or remain relatively stable.

4.3 RURAL DOMESTIC GROUNDWATER USE

DWR (1987) estimated the average annual rural domestic groundwater usage in the Santa Rosa Plain sub-basin from 1974 through 1984 as approximately 5,600 AFY. More recent or projected future data were not available from DWR. According to Dyett & Bhatia (2000b), many private wells are used for drinking water supply outside the City of Rohnert Park. Wells in the Canon Manor area (to the southeast of the City; see **Section 3.10**) are reported to be normally less than 200 feet deep. Parsons (1995) noted that 90 percent of the domestic wells in the WSTP study area (northwest of the Site) were screened above 200 feet bgs.

To estimate rural domestic water usage, US Government census block data and information regarding the areas in the Santa Rosa Plain Basin served by municipal water agencies were spatially analyzed. Information regarding the boundaries of municipal water agencies in the basin was obtained from SCWA, the California Spatial Information Library (CaSIL) and the City of Healdsburg. The areas served by these agencies were overlaid on the groundwater basin, and any households in the areas not served by municipal water agencies were assumed to be supplied by domestic wells. In order to estimate the number of households falling into this area, census block data for the year 2000 was downloaded from the CaSIL website and overlaid onto the map. The number of households in each census block served by domestic wells was assumed to be proportional to the percentage of the census block area inside the basin but outside established municipal water service areas. The locations of the municipal water service areas and census blocks relative to the Santa Rosa Plain groundwater basin are shown on **Figure 9**.

Based on our analysis, we estimate that in 2000, approximately 4,450 households were located in the basin but were not served by municipal water agencies. The American Water Works Association (AWWA) indicates that the average household water consumption in the United States is approximately 1/3 AFY. An alternative rule of thumb that is commonly used is that the average household water consumption is approximately ½ AFY. Based on this analysis, we estimate that the annual rural domestic household water demand in the Santa Rosa Plain Basin in 2000 was approximately 1,500 to 2,200 AFY.

In general terms, it seems reasonable to assume that the effect on future groundwater use of future population and development trends will tend to offset each other. That is, trends in conversion of rural agricultural to rural domestic development (which would result in an increase in rural domestic groundwater demand) will be offset by the projected future growth of urban boundaries (which will increase the number of homes serviced by municipal water connections. As such, we assume that the rural domestic groundwater demand will remain relatively stable.

4.4 TOTAL PRESENT AND PROJECTED GROUNDWATER PUMPING IN THE SANTA ROSA PLAIN BASIN

To provide perspective on the existing and potential future groundwater pumping in the basin, the estimates from the preceding sections are summarized and added below. It is important to note that these are rough estimates and not intended to be used to establish a water balance or estimate the safe yield for the basin; rather their purpose is to provide a context for understanding the significance of potential basin-wide impacts from groundwater pumping associated with the proposed hotel/casino project.

Groundwater Use	Approximate Recent Groundwater Demand (AFY)	Approximate Projected Future Groundwater Demand (AFY)
Municipal and Industrial	18,400	17,000
Agricultural	14,000 – 18,000	500 – 14,000
Rural Domestic	1,500 – 2,200	1,500 – 2,200
Total	**33,900 – 38,600**	**19,000 – 33,200**

The above estimates are based on the following assumptions.

- Except when more current data were available as noted in the text of this report, recent groundwater demand is based on data representative of conditions between 1999 and 2002.

- The recent and projected municipal and industrial groundwater demand is based on DWR provided estimates for the year 2020 for the Santa Rosa and Dry Creek DAUs, which do not coincide precisely with the boundaries of the Santa Rosa Plain groundwater basin.

- Estimates for agricultural and domestic groundwater use were derived using conventional methods from readily available data regarding land use, water application rates, municipal water agency boundaries, and census block data. The estimates are subject to uncertainties associated with the accuracy of the available data and the assumptions inherent in the approach discussed earlier.

- We have assumed that agricultural and rural domestic groundwater use in the basin will not change significantly in the next two decades.

4.5 CITY OF ROHNERT PARK

Historically, the City of Rohnert Park has been the largest single user of groundwater in the Site vicinity. As indicated by several of the historical studies described in **Section 3**, pumping by the City has been associated with a lowering of the groundwater surface beneath both the City and surrounding area – including the Site. Rohnert Park's WSA provides several estimates of historical, recent and future total pumping in that report's study area – the upper Laguna de Santa Rosa watershed (see **Section 3.11**). In 2003, the City's pumping was estimated to represent about half of the total groundwater pumping in the area (7,078 AFY or 6.31 mgd). The WSA estimates that by 2025, City pumping will account for only about 35% of the projected total area pumping (7,350 AFY or 6.56 mgd; note that this figure includes 100 AFY attributed to the Graton Rancheria hotel and casino project) (Winzler & Kelly, 2005).

4.5.1 ROHNERT PARK GENERAL PLAN, 2000

The current edition of the City of Rohnert Park General Plan (Dyett & Bhatia, 2000b) states that the City derives its drinking water supply from a well field comprising 42 municipal supply wells, 31 of which were active in 1999; and from eight active connections to the SCWA Petaluma Aqueduct. In 1999, groundwater supplied 4.19 mgd, or 61 percent, of the City's total supply of 6.87 mgd. According to HydroScience (2005), the rated capacity of the City's well field is 6.3 mgd.

The 11 wells inactive in 1999 comprised five that are used for water level measurement only, and six that were not used due to decreases in well production and/or maintenance issues (Dyett & Bhatia, 2000a). The five wells used only for monitoring are wells 9, 17, 24, 26 and 38 (Luhdorff & Scalmanini, 2004a). Well 24 (located approximately 300 feet east of the Site) is not used because of high levels of iron and manganese in the groundwater (HydroScience, 2007).

The City's municipal wells typically pump from aquifers between 200 and 1,200 feet bgs (Dyett & Bhatia, 2000a). City staff report that there is a significant difference in aquifer properties between the east and west halves of the City, with the eastern area characterized by shallow, low-yield aquifers and the western half having deeper, more productive aquifers (HydroScience, 2007).

In recent years, the City's water supply has been subjected to constraints arising from legal action. In 2000, a lawsuit was filed against the City of Rohnert Park by The South County Resource Preservation Committee. The lawsuit concerned potential effects of the City's groundwater pumping on wells outside the City limits. A court-sanctioned settlement agreement was negotiated. As a result, the City of Rohnert Park agreed to cut back groundwater usage from 4.2 mgd to 2.3 mgd, before additional growth could take place on yet-to-be-annexed county lands (Phillips & Gordon, unknown date). This 2.3 mgd limit is affirmed in the City's 2004 Water Policy Resolution (Winzler & Kelly, 2005).

However, the City's ability to cut back on groundwater production was limited by constraints on the supply from SCWA. In 1998, SCWA had proposed the WSTSP (**Section 3.6**), the construction of which aimed to increase the quantity of water that SCWA could supply in its service area. However, implementation of the project was delayed by litigation and regulatory constraints (see **Section 4.1.2**). As a result, the project was not expected to be completed until the year 2010. In the meantime, a Memorandum of Understanding (MOU) was established between the SCWA and the City (along with seven other cities), relating to water allocation leading up to final WSTSP implementation. Upon implementation in 2010, the City expected to receive all of its water from SCWA and eliminate groundwater as a source of drinking water. The following table shows the City's actual 1999 demand and expected water supply requirements and sources leading up to 2020.

Year	Supply Requirement (mgd)	SCWA (mgd)	Groundwater (mgd)
1999	6.87	2.68	4.19 (4,690 AFY)
2000	6.89	4.8	2.09 (2,340 AFY)
2001	6.97	4.8	2.17 (2,430 AFY)
2002	7.05	4.8	2.25 (2,520 AFY)
2003	7.12	5.2	1.92 (2,150 AFY)
2004	7.20	5.3	1.90 (2,130 AFY)
2005	7.28	5.3	1.98 (2,220 AFY)
2006	7.36	5.3	2.06 (2,310 AFY)
2007	7.44	5.3	2.14 (2,400 AFY)
2008	7.52	5.3	2.22 (2,490 AFY)
2009	7.60	5.3	2.30 (2,600 AFY)
2010	7.68	15.0	Backup / Emergency
2020	8.47	15.0	Backup / Emergency

Source: Dyett & Bhatia (2000a) Tables 4.10-1 and 4.10-3
mgd = millions of gallons per day

As can be seen in the table above, the City projected an immediate reduction in its reliance on groundwater supply, from 4.19 mgd in 1999 to 2.09 mgd in 2000.

The EIR (Dyett & Bhatia, 2000a) considered the effects of the projected City of Rohnert Park municipal wellfield requirements (tabulated above) on groundwater levels. Model-derived annual recharge rates ranging from 0.66 to 3.28 mgd were generated for 1970 through 1999. The average annual recharge rate was estimated to be 1.6 mgd. Comparing this average recharge to annual pumping projections in the table above, the EIR concluded that during 2004 through 2009, wellfield extraction rates would exceed recharge by between 0.3 and 0.7 mgd. The EIR's model-based analysis indicated that: (1) there was potential for short-term impacts on groundwater levels to occur before 2010 if annual recharge was less than 1.9 to 2.3 mgd, and (2) this condition of insufficient recharge would result should annual rainfall be less than 36 to 44 inches (Dyett & Bhatia, 2000b).

Dyett & Bhatia (2000a) expected recovery of groundwater levels within the City's proposed Urban Growth Boundary to begin during the year 2000, due to an increase in SCWA water allocation and corresponding decrease in groundwater pumping. Further, the impact on groundwater conditions would be "eliminated or substantially reduced" beginning in the year 2010, when groundwater was expected to be used for backup and emergency supply only.

Two of the City's stated goals in the General Plan were that (1) reliance on groundwater should be reduced, and (2) groundwater withdrawal should not exceed safe yield (Dyett & Bhatia, 2000b). A stated policy was the monitoring of the municipal wellfield to ensure that production would not exceed recharge rates and result in a "substantial lowering of groundwater levels in the vicinity of the Urban Growth Boundary." Furthermore, it was intended that should any development outside the 1 July 2000 City limits be deemed likely to result in such a condition, the development would be disapproved or denied.

4.5.2 WATER SUPPLY AGREEMENT WITH SCWA

The following provides background on the City of Rohnert Park's water supply from SCWA. The SCWA water supply potential is of critical importance in determining the City's ability to cut back on groundwater pumping in the future.

Rohnert Park receives water from SCWA under the terms of an agreement that was first signed in October 1974, and has since been amended twelve times. The original agreement entitled Rohnert Park to receive an average of 1.0 mgd of water from SCWA. The Eleventh Amended Agreement for Water Supply (SCWA, 2004a), allocated a total entitlement of 133.4 mgd to the signatory water contractors and set Rohnert Park's entitlement to 15.0 mgd (based on the maximum average day demand of any month), with an annual cap of 7,500 acre-feet. However, in December 1999 the SCWA announced that actual production capacity was limited to 84 mgd, and that an interim impairment condition existed. John Olaf Nelson Water Resources Management (2001) explained this circumstance as being a consequence of the SCWA falling behind in building capacity ahead of need, due to "challenges in meeting new regulations from many sectors, particularly meeting the requirements of the California Environmental Quality Act and the Federal Endangered Species Act...". In response to the impairment condition, SCWA and the water contractors, including Rohnert Park, signed the "Memorandum of Understanding Regarding Water Transmission System Capacity Allocations During Temporary Impairment" in March 2001 (John Olaf Nelson Water Resources Management, 2001). The MOU (SCWA, 2004a) allocated the available SCWA supply among the signatories through the summer of 2005.

In 2003, SCWA suffered a significant setback in its proposed expansion plans, when the First District Court of Appeals ruled against SCWA in the case *Friends of the Eel River et al. v. SCWA and Pacific Gas and Electric Company.* The lawsuit had been filed by Friends of the Eel River in 1999, and challenged SCWA's WSTSP, which sought to increase SCWA's annual diversion of water from the Russian River by 33 percent (Friends of the Eel River, 2003a). The Sonoma County Superior Court ruled against Friends of the Eel River in August 2000, but the decision was reversed by the First District Court of Appeals in its 16 May 2003 ruling (*Friends of the Eel River v. Sonoma County Water Agency*, 108 Cal. App. 4th 859). The ruling concluded that the EIR submitted by the SCWA for the WSTSP was inadequate. The effect of the ruling was to postpone the project, while SCWA prepared a supplemental EIR (SCWA, 2004b). Recently, SCWA has announced that an entirely new EIR will be prepared for a modified project known as the Water Supply, Transmission, and Reliability Project (Winzler & Kelly, 2005).

In August 2003, following the Court of Appeal ruling, SCWA wrote to water contractors, customers and other parties, providing an update on SCWA's Russian River water supply (Friends of the Eel River, 2003b). SCWA stated:

> "The Agency's state water rights permits limit the Agency's Russian River diversions and rediversions to 75,000 acre-feet per year. The Agency's Water Supply and Transmission System Project ("WSTSP") had contemplated an increase in diversions and rediversions to 101,000 AFY. However, with the Court of Appeal decision in the FRIENDS OF THE EEL RIVER litigation, the Agency cannot implement the WSTSP at this time. Thus, it would be inappropriate for water suppliers relying on water diverted

under the Agency's water rights to anticipate water deliveries based upon diversions of 101,000 AFY, or to rely on the delivery estimates in the Agency's Urban Water Management Plan 2000 (which indicated that water supplies available to the Agency's water transmission customers would be adequate over the next 20 years.)"

The letter requested contractors relying entirely or in part on water diverted under the SCWA's water rights to evaluate expected future water demands for existing/approved development projects, as well as proposed but not yet approved projects. On 1 March 2004, the City provided the requested information to SCWA, stating that the City's total water demand would be 6,926 AFY (6.18 mgd), which would be provided by SCWA under the 7,500 AFY limit in the Eleventh Amended Agreement for Water Supply. This total demand represented a 4,280 AFY (3.82 mgd) increase in SCWA water-use compared to the 2003 water year, and included a requirement of 2,200 AFY (1.96 mgd) from SCWA to offset reduced groundwater pumping, as required by the Rohnert Park General Plan (City of Rohnert Park, 2004).

The current agreement, entitled the Restructured Agreement for Water Supply was approved in 2006 and reaffirmed Rohnert Park's entitlement to 15.0 mgd (based on the maximum average day demand of any month), with an annual cap of 7,500 acre-feet (SCWA, 2006). However the agreement states that SCWA will not be liable in the event it is not able to meet its entitlement due to drought, environmental laws or regulations, or other causes beyond SCWA's control, and prioritizes delivery of available water if the entitlements cannot be met.

4.5.3 CURRENT AND FUTURE GROUNDWATER USE BY ROHNERT PARK

The following table compares Rohnert Park's projected groundwater use, as given in the 2000 General Plan (Dyett & Bhatia, 2000b), with the actual use of recent years (obtained from the City, except for the 2004 and 2005 figures, which are from Winzler & Kelly [2005 and 2007]).

Year	Projected Requirement (mgd)	Actual Pumped Volume (mgd)
2000	2.09	4.11 (4,600 AFY)
2001	2.17	4.00 (4,480 AFY)
2002	2.25	3.82 (4,280 AFY)
2003	1.92	3.14 (3,520 AFY)
2004	1.90	1.36 (1,520 AFY)
2005	1.98	0.76 (850 AFY)
2006	2.06	0.30 (340 AFY) (through September 30)

These figures show that from 2000 to 2003, although the City was decreasing its groundwater usage, it did not achieve the targets outlined in the 2000 General Plan. However, beginning in 2004 the City has pumped significantly less groundwater than its projected requirements.

Although the City of Rohnert Park General Plan indicates that groundwater use by 2010 will limited to emergency and backup use only (Dyett & Bhatia, 2000a), the City's Water Supply Assessment and Urban Water Management Plan projects future groundwater pumping as 2,577 AFY (Winzler & Kelly, 2005 and 2007). This is consistent with the cap specified in by the City's 2004 Water Policy Resolution (2.3 mgd, or 2,577 AFY). This total appears to include any future groundwater demand associated with the City of Rohnert Park Northwest Specific Plan, Southeast Specific Plan and University District Specific Plan, because the plan documents and/or supporting environmental documents reference the City's Water Supply Assessment in discussions regarding the adequacy of the water supply (Parsons, 2004; Parsons, EIP, 2005; Jones & Stokes,). The Wilfred Dowdell Village Specific Plan (Parsons Harland Bartholemew Associates, 2007) does not include reference to the Water Supply Assessment or Urban Water Management Plan; however, the Environmental Impact Report for this area was adopted in 2000, so it is likely that the water demand for this area was included in the projections discussed in the Water Supply Assessment and Urban Water Management Plan. Similarly, the Northeast Area Specific Plan was prepared in 2003 and predates both the Water Supply Assessment and Urban Water Management Plan, and the water demand associated with this area would likely have been considered in drafting these documents.

4.6 GROUNDWATER WELLS IN THE SITE VICINITY

Well construction records were requested from the DWR for water supply wells completed in the vicinity of the Site. As indicated on **Figure 10** and **Figure 11**, 193 shallower (up to 200 feet deep) and 61 deeper (more than 200 feet deep) water supply wells were identified within approximately 1.5 mile from the proposed supply well location at the Site. The installation dates, purpose, depth and screened intervals of these shallower and deeper wells are summarized in **Table 1** and **Table 2**, respectively. Note that well completion records may not have been provided by DWR for every well in the vicinity of the Site, and that it is not known how many of the wells identified, especially the older wells, are still actively used.

The well completion reports provided by DWR indicate that most of the water supply wells in the vicinity of the Site were installed for domestic supply purposes and a smaller number serve irrigation, stock, municipal and industrial needs. Most of the domestic wells are less than 200 feet deep (and many are less than 100 feet deep); whereas, most of the municipal supply wells are over 200 feet deep. Pumping rates from these wells were not available in the obtained data.

5 SITE EVALUATION

5.1 TOPOGRAPHY AND LAND USE

The Site is approximately flat-lying, at an elevation of about 90 feet above msl (**Figure 2**). The Site is currently used as irrigated farmland, pasture, and for the disposal of recycled water from the nearby City of Santa Rosa Laguna Regional Wastewater Treatment Facility (HydroScience, 2007). The Site is bounded by commercial and residential areas of the City of Rohnert Park to the southeast, and by agricultural land and ranch dwellings on other sides. The Site area is identified as "community separator" land in the City of Rohnert Park's General Plan (Dyett & Bhatia, 2000a).

5.2 DRAINAGE

The Laguna de Santa Rosa flows from east to west beyond the southern boundary of the Site. A tributary to this channel flows across the southern portion of the Wilfred Site, and divides the Stony Point Site, from north-northeast to south-southwest. This tributary is identified as the North Fork to the Laguna de Santa Rosa (HydroScience, 2007) or the Bellevue-Wilfred Channel (Dyett & Bhatia, 2000a).

5.3 GEOLOGY

The reviewed information provides the following indications of the hydrostratigraphic sequence beneath the Site:

1. Maps presented by DWR (1979) suggest that the total thickness of water-bearing sediments (defined as alluvial deposits and the Wilson Grove Formation) in the immediate vicinity of the Site is approximately 900 feet, and that the thickness of low-permeability (*i.e.* basin) deposits overlying these varies from less than 40 feet in the northwest to about 160 feet in the southeast.

2. A cross-section presented as Figure 5E in DWR (1982) suggests that the following approximate stratigraphic succession is present beneath the Site (**Figure 12**):

Depth Range (feet)	Thickness (feet)	Name
0 to 180	180	Basin Deposits
180 to 540	360	Alluvial Fan Deposits
540 to 1,000	460	Wilson Grove Formation
1,000 to 2,200	1,200	Petaluma Formation
Below 2,200	-	Tolay Volcanics

3. The electric log for City of Rohnert Park well 24, located adjacent to the southeastern boundary of the Site, suggests that the clay-rich basin deposits may be only around 120 feet thick, and that these are underlain by interbedded sand, silt and clay deposits to a depth of at least 600 feet bgs.

Note that the interpretations described in (1) and (2) above are themselves derived from DWR interpretations of raw data that were not available for our review. The proximity of these raw data to the Site is not known.

The information suggests that the main water-bearing strata beneath the Site, consisting of alluvial fan deposits and the Wilson Grove Formation, may be between 820 and 900 feet thick, and overlain by between 40 and 180 feet of relatively low-permeability materials. The alluvial deposits and Wilson Grove Formation are anticipated to consist of sand/gravel-rich, water-yielding layers, interbedded with silt/clay-rich, comparatively non-water bearing layers.

5.4 HYDROGEOLOGY AND WATER RESOURCES

5.4.1 GROUNDWATER ELEVATION DATA FROM ON-SITE WELLS

HydroScience (2004a) identified three wells on the Stony Point Site and one well that is encompassed by both the Wilfred and Stony Point Sites. In addition, we obtained well completion reports for two wells that appear to be located on the Wilfred Site (shallow well number 58 and deep well number 38, shown on **Figure 10** and **Figure 11**, respectively -- note that exact location information for these wells was not available and the locations of these wells were not confirmed in the field). Two of the four wells identified by HydroScience have been "abandoned and sealed". The status of shallow well number 58 is not known. The two active wells identified by HydroScience include a recently constructed, small diameter feed for cattle watering troughs, and an older (believed to date from the 1950's), larger diameter (12-inch) agricultural well that is used to feed watering troughs and to supply barnyard wash water. Although we have not been able to establish a conclusive correlation, the latter well may be deep well number 46, shown on **Figure 11** and included in **Table 2**.

A video survey found that the active agricultural well is at least 610 feet deep. The casing is perforated from top to bottom, though many of the slots are plugged by corrosion below 100 feet depth. The well completion report for deep well number 46 (believed to be the same well) indicates this well was constructed in 1950 to a total depth of 914 feet and with an unknown screened interval (**Table 2**). The standing water level at the time of investigation in September 2003 (HydroScience, 2007) was –20 ft msl or 110 feet bgs. HydroScience (2005) estimated that the well likely had a high original yield in excess of 300 gpm.

Well completion records are available for two other on-Site wells: shallow well number 58, a domestic well installed in 1979 to a total depth of 120 feet and screened from 60 to 120 feet; and deep well number 38, an irrigation well constructed at an unknown date to a total depth of 1,028 feet with an unknown screened interval. The latter well corresponds to well 6N/8W 15-R1, for which water level measurement information was obtained from DWR.

State well 6N/8W 15-R1 (deep well number 38 on **Figure 11**) is located within the northeast portion of the Wilfred Site and near to the northeast Stony Point Site boundary, approximately 1,200 feet south of State well 6N/8W 15-J3 (the exact distance is unknown since DWR (2004b) does not provide coordinates for well 6N/8W 15-R1). State well 6N/8W 15-R1 is 1,028 feet deep (DWR, 1982) and a screened interval is not indicated on the well completion record provided by DWR. Water elevation data are available for spring and fall between October 1974 and April 1984 (DWR, 1987). These data are also plotted on **Figure 17**, and appear to indicate a parallel trend with the shallower well (6N/8W 15-J3) data, though the water elevations were consistently lower (this relationship was also noted by DWR (1982)). Further analysis shows that during the period of October 1974 through April 1984, groundwater elevations were declining in both wells, with the spring and fall water levels in the deeper well (6N/8W 15-R1) declining at a faster rate than those in the shallower well (**Figure 17**). This is likely due to the effect of groundwater pumping in off-Site wells screened in the deeper zone. Although the screened interval of State well 6N/8W 15-R1 is not known, the depth of the well suggests it is likely screened at similar depths to the City wells. Thus, well 6N/8W 15-R1 is more responsive to off-Site pumping than is well 6N/8W 15-J3, which is screened in the shallow groundwater above the screened interval of most of the municipal wells.

The available information suggests that current on-Site pumping is comparatively low-volume, although significant volumes may have been pumped from beneath the Site in the past. It is assumed that the existing on-Site wells will be abandoned if the proposed Graton Rancheria project is constructed.

5.4.2 GROUNDWATER ELEVATION DATA FROM NEARBY OFF-SITE WELLS

There are several wells within 1 mile of the Site for which DWR maintains water level records. In addition, the City of Rohnert Park's municipal well field lies to the east (**Figure 2**).

City of Rohnert Park well 24 is located approximately 300 feet east of the southeastern Site boundary (**Figure 2**). This well is 592 feet deep, and has multiple screened zones between 258 and 582 feet depth. The pumping capacity of the well was 120 gpm. This well is no longer used by the City due to high levels of iron and manganese in the extracted groundwater (HydroScience, 2007). **Figure 13** presents hydrographs for six municipal wells located throughout Rohnert Park's well field, including well 24, during the period of January 1998 through June 2004. The wells are generally screened within the interval of 200 to 1,200 feet bgs. The entire well field's average pumping rate is shown for reference. Groundwater levels and average pumping rates on **Figure 13** are shown at monthly intervals.

Figure 13 shows that there is an inverse relationship between groundwater elevation in the City wells and total pumping from the City well field. The different wells exhibit different response times to changes in pumping rate, with wells 7 and 18 showing the nearest to expected response patterns (*i.e.,* lowest groundwater elevation closely follows highest monthly pumping rate). Those wells showing less clear responses likely had operational pumping schedules that differed from the average pumping of the entire City well field (black line on **Figure 13**). **Figure 14** was derived from **Figure 13**, using the groundwater elevation data from City wells 7 and 18 only. It shows drawdown instead of groundwater elevation on the y-axis and, as well as showing drawdown for wells 7 and 18, it also shows the average drawdown for the two wells. This figure shows that the transition period for groundwater drawdown response to changes in pumping can be up to four months. This is most clearly seen for drawdown decreases that begin around October of each year when pumping rates typically begin to decrease.

Groundwater elevation data for City well 24 are also plotted on **Figure 15**, which shows hydrographs for other wells within 1 mile of the Site. State well 6N/8W 15-J3 is located near to the northeast Site boundary (**Figure 2**, see also shallow well 108 on **Figure 11**). This well was installed in 1946 to a depth of 166 feet and has screened intervals from depths of 65 to 90 feet and 151 to 166 feet (**Table 1**; DWR, 1982). Water elevation measurements for this well are available dating back to 1950 (DWR, 2004). Between 1950 and 1968, the measurements were generally taken in the spring (March), although fall (October) data are available for 1951, 1952 and 1958 through 1960. From 1969 to 1977, regular spring and fall measurements were taken, and after 1977 measurements were generally taken on a monthly basis. These data are plotted on **Figure 15**.

The data plotted on **Figure 15** indicate gradually declining water elevations between 1950 and the early 1970's, then an increased rate of decline until around 1990, when levels appeared to stabilize. The increased rate of groundwater level decline beginning in the 1970's, as exhibited by well 6N/8W 15-J3, correlates with increased pumping by the City of Rohnert Park. **Figure 16** shows a plot of City pumping versus groundwater elevation in well 6N/8W 15-J3 (calculated as the average between spring and fall flows). As can be seen, between 1970 and about 1990, when groundwater levels in this well were in marked decline, Rohnert Park's pumping increased steadily from about 0.8 to 4.3 mgd. Reductions in pumping in recent years (after 2000) appear to be reflected by rising groundwater levels.

During 1990 through 1997, water levels in well 6N/8W 15-J3 stabilized (at an elevation of approximately 41 feet amsl) and the City's pumping rates were also relatively stable (at an annual average rate of about 4.3 mgd) (**Figure 16**). The observation that groundwater levels ceased falling when pumping continued at a rate that had previously caused a water level decline suggests that groundwater recharge (or inflow) increased as water levels fell. According to this interpretation, groundwater levels ceased falling when the increased recharge or inflow rate was sufficient to balance the water budget for the southern Santa Rosa Plain sub-basin, including the increased, stable pumping rate of 4.3 mgd. During the historical period of 1970 through 1982, the City's pumping had not yet reached the peak rates of the 1990's, yet water levels in well in well 6N/8W 15-J3 continued to fall. Thus, groundwater recharge or inflow was apparently insufficient to balance the water budget during that time. Note that this water budget deficiency cannot be entirely explained by a shortage of precipitation because precipitation was above average for the period of 1970 through 1982. The significance of this observation is further discussed in **Section 6.5.** The trend in long-term water levels is also consistent with the observation that water levels stabilize in water production wells operated by the City of Rohnert Park about four months after the onset of pumping. The significance of this latter observation for the evaluation of impacts from groundwater pumping at the Site is further discussed in **Section 6.2.2.**

ote that the correlation between groundwater elevation decline in well 6N/8W 15-J3 and increased City of Rohnert Park pumping, as described above and illustrated on **Figure 16**, does not mean that the groundwater elevation in well 6N/8W 15-J3 has responded exclusively to the City's pumping. In fact, most of the City's pumping wells are approximately 2 miles away. There are other municipal, domestic, and agricultural wells in the southern Santa Rosa Plain groundwater sub-basin, the pumping of which will have an effect, to a lesser or greater degree, on the groundwater elevation in well 6N/8W 15-J3. Nevertheless, there appears to be a clear correlation between pumping by the City of Rohnert Park and shallow groundwater elevations near the Site, as typified by the hydrograph of well 6N/8W 15-J3.

For the purposes of this report, all pumping from off-Site wells in the southern Santa Rosa Plain groundwater sub-basin that affects groundwater levels at the Site (represented by well 6N/8W 15-J3) is referred to as "off-Site pumping". Note that pumping from existing on-Site wells (see **Section 5.4.1**) will be discontinued if the proposed Graton Rancheria project is constructed. In that case, future pumping on-Site would include only the proposed wells (**Section 1.1**).

DWR has limited groundwater level data for other wells in the Site vicinity, including well 6N/8W 22-R1, located to the southeast of the Site, and well 6N/8W 16-K3, located west of the Site, (**Figure 2; Figure 11**). Well 6N/8W 22-R1 corresponds with deep well number 11 shown on **Figure 11**. Based on well completion records provided by DWR, this well was installed to a depth of 407 feet in 1978 and is screened from a depth of 387 to 407 feet. Although a complete State Well number was not legible on the well completion report provided by DWR, it is likely that well 6N/8W 16-K3 corresponds with shallow well 158 (**Figure 10**), which was installed in 1971 to a depth of 79 feet and is screened from a depth of 59 to 79 feet (**Table 1**) The relationship between the groundwater elevations in 6N/8W 16-K3 and 6N/8W 15-J3 appears similar to that between 6N/8W 15-R1 and 6N/8W 15-J3, though the data time periods for wells 6N/8W 16-K3 and 6N/8W 15-R1 do not overlap (**Figure 15**). Therefore, the screened intervals of wells 6N/8W 16-K3 and 6N/8W 15-J3 may be similar. Groundwater elevations in 6N/8W 22-R1 were noticeably lower than those in 6N/8W 15-K3 over the period of record. The groundwater elevations within well 6N/8W 22-R1 are broadly comparable to those measured in Rohnert Park well 24 (**Figure 15**); and indeed, well 6N/8W 22-R1 is screened at a similar interval to Rohnert Park well 24.

5.4.3 CONCLUSIONS FROM GROUNDWATER ELEVATION DATA

Different monitoring wells in the Site vicinity show varying groundwater elevations at any given time according to their screened interval(s), though with generally parallel trends over time that appear to be strongly influenced by pumping of the City of Rohnert Park's well field. The impact caused by pumping will be at a maximum in a given monitoring well if the screened interval in nearby production well(s) is at a similar depth to the screened interval of that monitoring well. However, if the screened interval of the monitoring well is higher than the screened interval in the production well(s), the magnitude of impact will be less. Strong seasonal variations are observed in monitoring wells, with spring and fall shallow groundwater levels typically differing by more than 10 feet in the Site vicinity (**Figure 17**), and considerably more in Rohnert Park's well field (**Figure 13**).

The correlation between groundwater elevations in wells on or near to the Site (6N/8W 15-J3 and 6N/8W 15-R1) and pumping of the City of Rohnert Park's well field indicates that there is horizontal continuity of groundwater-bearing zones in the area. The differences in groundwater elevation between wells screened in shallower and deeper zones are consistent with the presence of less transmissive hydrogeologic zones separating the more productive intervals. The electric borehole log for City of Rohnert Park well 24 reflects this hydrogeologic conceptual model, with intervals having higher electrical resistivity (interpreted to be sand and gravel) alternating with intervals having lower electrical resistivity (interpreted to be silt and clay) over the well's multiple-screened interval between 258 and 582 feet bgs. Nevertheless, data for state well 6N/8W 15-J3 suggest that water levels in shallower groundwater zones near the Site respond to pumping in deeper zones. This contrasts with the results of aquifer testing of the Todd Road well, located about 2.5 miles northwest of the Site, which indicated a hydraulic discontinuity between shallow and deep wells (**Section 3.5**). The conclusion in the WSA that shallow wells in the area generally do not respond to pumping in the deeper zone was based in part on this data from the Todd Road site (Winzler & Kelley, 2005; **Section 3.11**).

The hydrograph for well 6N/8W 15-J3 reflects shallow groundwater elevations very near to, and therefore likely beneath, the Site. The earliest data from this well indicate that in 1950, spring groundwater levels were as high as 94 feet above msl – which is very close to ground surface. Spring levels declined to about 45 feet above msl by the mid-1990's, a drop of approximately 50 feet. Recently, there has been indication that groundwater elevations are beginning to rise (**Figure 15**).

6 POTENTIAL IMPACTS OF USING GROUNDWATER TO SUPPLY THE PROPOSED DEVELOPMENT ALTERNATIVES A, B AND C

This section presents an evaluation of the potential impacts resulting from withdrawing groundwater for use as a water supply for the proposed Alternatives A through C. These alternatives are identical in terms of their water demand and proposed groundwater supply well location. In addition, they represent the average proposed groundwater pumping rates and predicted impacts to water levels and existing wells as discussed in below and in **Section 7.**

6.1 PROJECT GROUNDWATER DEMAND

The proposed groundwater supply wells at the Site will pump at a combined rate of 200 gpm (for Alternatives A through C); however, the total water demand for the proposed development is 250 gpm. The difference between the groundwater pumping rate of 200 gpm and the total demand of 250 gpm will be made up by recycling highly-treated wastewater for on-Site reuse for non-contact purposes and irrigation (HydroScience, 2007). For purposes of predicting impacts in this report, a sustained pumping rate of 200 gpm (0.29 mgd or 323 AFY) from the proposed wells is assumed.

6.2 DEVELOPMENT OF DRAWDOWN MODEL

An analytical drawdown model was developed for predicting water-level impacts due to proposed pumping at the Site. The purpose of the model is to assess potential impacts to adjacent groundwater users from the proposed pumping only. Thus, existing or future impacts due to off-Site pumping wells are not predicted by the analytical model.

Two pumping wells with a combined rate of 200 gpm have been proposed, with recycling, to meet the 250 gpm estimated water demand for the proposed development (HydroScience, 2007). For purposes of simplicity and conservatism, the analytical model described in this Section simulates a single well pumping at 200 gpm. Simulating a single well to represent two nearby wells with the same total pumping rate generally gives a small increase in the predicted off-Site drawdown.

The analytical model uses the Theis non-equilibrium equation (Driscoll, 1986) for describing drawdown from a pumping well. Inputs for the analytical model were derived from the pumping test on the Todd Road well reported by DWR (1987) and described in **Section 3.5**, and other sources cited in this report. The model specifically evaluates drawdown impacts in the deeper zone (*i.e.*, below 200 feet bgs), since that is where the Todd Road well is screened. However, an analysis of hydrographs for two State wells near the Site is presented to show that drawdown impacts will be smaller in the shallower zone than in the deeper zone.

6.2.1 MODEL DATA

The model assumes that the pumping wells would be screened in permeable zones found between 200 and 600 feet bgs (HydroScience, 2007). The Todd Road well (2.5 miles northwest of the Site) was used as the data source representing the proposed pumping well for purposes of model development. The well location is shown on **Figure 2**. The Todd Road well is 815 feet deep and intercepts an aquifer thickness of 160 feet. The Todd Road well is the nearest well to the Site with available aquifer test data. Aquifer parameters (transmissivity and storativity) used for the model were obtained from DWR's analysis of the Todd Road well test (DWR, 1987). The following qualifications apply to the use of these parameters for predicting drawdown at the Site:

- The Todd Road well is 2.5 miles from the Site, and geologic conditions may vary over that distance, resulting in an unknown drift in aquifer parameter values from the area of the Todd Road well to the Site. However, we have not found any hydrogeologic boundaries or changes in geologic formations between the two locations.

- The Todd Road well is screened from 650 to 800 feet bgs, whereas the proposed wells at the Site would be 600 feet deep (HydroScience, 2007). The observation well used for the pumping test is 570 feet deep, in good agreement with the depth for the proposed wells.

- The screened interval of the observation well used in the pumping test is at least 80 feet above the screened interval of the Todd Road well. This discrepancy may have resulted in over-estimation of aquifer transmissivity from the results of the pumping test. However, since the Todd Road test well and monitoring well are both screened in the deeper zone, there are no data available to assess the significance of the potential over-estimation of transmissivity.

- DWR (1987) reported two sets of transmissivity and storativity values for the Todd Road pumping test that were derived using two alternative methods for aquifer test analysis (the Cooper-Jacob and the Hantush-Jacob methods, see **Section 3.5** of this report). However, these sets of values were found to produce similar results in the analytical model, and, the difference between model predictions resulting from DWR's two sets of parameter values was too small to justify separate versions of the model.

- Based on the data available to us (DWR, 1987), it appears that DWR may have overestimated the aquifer transmissivity in its analysis of the Todd Road aquifer test using the Cooper-Jacob method; therefore, the aquifer parameters derived using the Hantush-Jacob method were used to construct the analytical drawdown model used in this report.

An aquifer test is recommended prior to development of an on-Site water supply to clarify the aquifer parameters for the Site and aid in well design. The results of this pumping test could be used to confirm that the transmissivity and storativity from the Todd Road pumping test are applicable to the Site and, if needed, to design a deeper well such that the well transmissivity and predicted impacts are within acceptable levels for the project.

The aquifer parameters used in the analytical model are summarized in the table below.

Aquifer Parameter	Parameter Value	Units	Source
Transmissivity	10,100	gpd/ft	DWR (1987)
Storativity	0.0015	NA	DWR (1987)
Pumping Rate+	200	gpm	HydroScience (2005)
Pumping Time++	122	days	WorleyParsons Komex

+ Assumes rate is time-constant.

++ Corresponds to time required for water levels to stabilize during pumping, estimated from **Figure 11**.

6.2.2 MODEL ASSUMPTIONS

The Theis non-equilibrium well equation incorporates the following standard assumptions:

1. The aquifer being pumped is homogeneous and isotropic.
2. The aquifer is uniform in thickness and infinite in areal extent.
3. The aquifer receives no recharge, thus all flow produced from the pumping well comes from aquifer storage.

4. The pumping well is screened in, and receives water from, the full thickness of the aquifer.
5. Water is released from aquifer storage instantaneously when the water level is lowered.
6. The pumping well is 100 percent efficient.
7. Laminar flow exists throughout the well and aquifer.
8. The water table or potentiometric surface has no slope.

The model assumes that the proposed wells will pump at a constant rate (*i.e.*, without seasonal variations). The drawdown predictions developed for this report assume that water levels near the Site will adjust to the proposed pumping rate of 200 gpm (0.29 mgd) and that stable, though lower, groundwater levels will be reached after a period of about four months. Hydrographs and time-drawdown graphs for wells in the City of Rohnert Park's well field indicate that drawdown tends to stabilize at a new level about four months after a change in pumping. This is most clearly seen for drawdown decreases that begin in October of each year when pumping rates typically decrease (see **Section 5.4.2** and **Figure 14**). The Theis equation is not valid for the Site after the four month period when water elevations have become stable.

The model does not portray any specific boundaries. The Sebastapol Fault (which is southwest of the Site) is not assumed to be a barrier to groundwater flow. As discussed in **Section 2.4.2**, there have been varying interpretations regarding whether this fault is a barrier to groundwater flow. Also the model does not assume that the Laguna de Santa Rosa (which is near the fault) is a recharge boundary. We are not aware of any previous analyses of the surface water-groundwater interactions beneath the Laguna de Santa Rosa. If the fault is a barrier to groundwater flow, or the stream is a recharge boundary affecting groundwater flow, then drawdown from the new water supply wells at the Site will be underestimated or overestimated, respectively, by the drawdown model.

6.2.3 MODEL LIMITATIONS

The analytical model used for this report was developed to predict drawdown using available hydrogeologic data as input. Thus the lack of site-specific data for transmissivity, storativity, and pumping time has been compensated by using data from surrounding areas (*e.g.*, Todd Road and **Figure 14**) to make reasonable estimates of Site conditions. The Theis equation was developed using the eight assumptions listed above in **Section 6.2.2**. The Theis equation is accurate when each of these assumptions is met. Most of the assumptions are considered acceptable for the southern Santa Rosa Plain aquifer: a semi-confined aquifer in which thickness changes are gradual, layering is generally horizontal, the groundwater surface is relatively level, thicknesses of the coarse-grained zones can be mathematically combined to estimate the equivalent thickness of a homogeneous aquifer, and the aquifer has the areal extent shown on **Figure 1**. To the extent that these assumptions are valid for the study area, the analytical model remains accurate.

6.3 GROUNDWATER-LEVEL IMPACTS IN THE SITE VICINITY FROM PROPOSED PUMPING WELLS

The analytical drawdown model was used to predict drawdown impacts in the deeper screened zone in the vicinity of the Site, from operation of the pumping the proposed wells. The model and input parameters are described in **Section 6.2.1.**

Hydrographs and time-drawdown graphs for wells in the City of Rohnert Park's well field (**Figures 13 and 14**) indicate that groundwater levels tend to stabilize at a new level about four months after a change in pumping. Note that the absence of continued water-level declines does not imply the absence of impacts.

Figure 19 is a semi-log distance-drawdown graph showing the model's predicted drawdown for the deeper screened zone. **Figure 19** indicates the predicted drawdown as a function of distance between the pumping well and the point of drawdown measurement. For example, the distance could represent the interval between the proposed pumping well and an off-Site privately-owned well. **Figure 19** can be applied to any potential pumping well location on the Site, but in this report is further used to assess impacts from locating the water production wells for the project on the southern parcel that is shared by both the Wilfred and Stony Point Site configurations, near City of Rohnert Park well 24 (**Figure 2**), which is the location proposed by HydroScience (2005).

For the purposes of our evaluation, we have assumed that the water supply wells for the project will be set back at least 100 feet from the eastern Site boundary. The distance from the proposed pumping well to the Site boundary is represented by the left vertical axis of the chart on **Figure 19**, whose origin occurs at a distance of 100 feet. Drawdown occurring beyond the Site boundary (to the right of this axis) can potentially impact off-Site wells and users. **Figure 19** shows that at the property boundary, the maximum predicted drawdown in the deeper screened zone is 23.0 feet. In addition, the predicted drawdown in the deeper screened zone attenuates to about 3 feet at a distance of about 8.600 feet and 1 foot at a distance of 17,000 feet from the proposed well. Drawdown less than 3 feet is probably insignificant in relation to natural seasonal and longer term variations in groundwater levels.

To gain perspective on the potential difference to the shallow and deep zone from pumping at the Site, we evaluated the relative drawdown response to City of Rohnert Park pumping in a pair of State wells separated by 1,200 feet and located near the northeastern Site boundary. As detailed in **Section 5.4.2**, State well 6N/8W 15-J3 is a shallower well (166 feet deep) and State well 6N/8W 15-R1 is a deeper well (1,028 feet deep). We compared the hydrographs of these wells to evaluate shallower and deeper responses to pumping from off-Site pumping wells, which are generally screened in the deeper zone. **Figure 17** presents drawdown hydrographs showing these shallower and deeper responses during the time period of October 1974 to April 1984. (Peak water levels during the hydrograph period were used as static levels for computing drawdown.) The resulting drawdown data for October 1974 to April 1984 illustrate the empirical relationship of drawdown in shallower and deeper wells at the Site (**Figure 18**).

Figure 18 shows that off-Site pumping causes greater drawdown in the deeper State well (6N/8W 15-R1) as opposed to the shallower State well (6N/8W 15-J3). We expect that pumping from the proposed wells at the Site (screened in the deeper zone) would produce a similar type of effect on the State wells, causing more drawdown in the deeper screened zone versus the shallower screened zone. However, the observed amounts of drawdown in the shallower and deeper State wells due to pumping off-Site wells (**Figure 18**) may not be directly comparable to drawdowns that would be caused by pumping the proposed on-Site wells. For example, the proposed wells will pump at a significantly lower rate than the combined rate the off-Site pumping wells, possibly causing the proposed wells to produce a smaller difference between drawdowns in the shallower and deeper zones.

In summary, drawdown in the deeper zone is expected to be greater than corresponding drawdown in the shallower zone at any given location near the Site. Nevertheless, since a given drawdown impact (e.g. 23.0 feet) is potentially more serious for a shallower well than for a deeper well, we consider it appropriate to apply the drawdown predictions in **Figure 19** to both the shallower and deeper zones.

6.4 INTERFERENCE DRAWDOWN IMPACTS IN OFF-SITE WELLS

6.4.1 TYPES OF IMPACTS

The project-related drawdown at any affected well (interference drawdown) will result in a decreased saturated thickness available to be pumped at that well. In the most extreme case, this could result in drawdown of the water level in a well to a depth below the screen of the well (*i.e.*, the affected well goes dry as a result of project pumping). At the other extreme, the effect of project pumping may be so small that the project-related drawdown is insignificant relative to short term or seasonal fluctuations, or the drawdown could represent an insignificant impact to the well user. The following possible significant impacts could occur:

1. The interference drawdown results in the water level in the aquifer being drawn down below the screen of the well (*i.e.*, the well goes dry as discussed above).

2. The interference drawdown results in the water level in the aquifer being drawn down to a point where the remaining saturated thickness is too small for the affected well to provide an adequate water supply for the intended use, or the pumping water level is too close the intake level of the pump, exposing it to potential damage.

3. The interference drawdown results in the water level in the well during pumping (the well's pumping water level) being drawn near the intake of the pump, requiring lowering of the pump intake in order for the well to remain operational. This is essentially a variation of case 2, but there is space below the pump allowing an adequate flow rate to be restored by lowering the pump. Energy costs would be expected to increase after the pump is lowered.

4. The interference drawdown causes a decrease in saturated thickness such that the well and pump can continue to operate and produce adequate amounts of water, but pumping must occur at either greater frequency or duration, and must lift water for a greater height. As a result more energy is used, resulting in greater operational and maintenance costs. This condition can develop prior to the onset of case 1, 2 or 3.

The hydrogeologic factors that dictate which of the above impacts will occur are the saturated thickness of the well before interference drawdown and the amount of interference drawdown that is applied (which varies with the distance of the impacted well from the project well). The impact from interference drawdown has the potential to be more severe if it represents a higher percentage of the well's initial saturated thickness prior to the onset of interference drawdown. For example, a 10-foot drop in water level has a greater potential to cause Impacts 1 or 2 in a shallower well; whereas, the same drop in water level in a deeper well might result in less serious, but potentially still significant, impacts such as 3 or 4. In general, small variations in saturated thickness are not considered significant when assessing transmissivity values from the interpretation of aquifer test drawdown data (Jacob, 1950). However, in assessing the impacts of interference drawdown to neighboring pumping wells, a small change in saturated thickness (*e.g.*, 3 feet or more) could still cause a significant increase in electrical costs to a high capacity well. These cases are discussed in additional detail in the subsequent sections.

The impacts resulting from interference drawdown are also dependant on several factors that may vary from well to well, even if the wells have the same saturated thickness and applied interference drawdown. These well-specific factors include the following:

- Local variations in the transmissivity of the saturated sediments in which the well is completed (*i.e.*, their ability to yield water to the well with a given amount of drawdown in the aquifer);
- The condition and efficiency of the well (*i.e.*, the water level in the well bore compared to the water level in the aquifer just outside the well, which can be significantly lower if the well is in poor condition or poorly designed);
- The well's pump specifications, including its rating curve, the depth at which the pump intake is set, and the resulting pumping water level in the well during operation;
- The well's screened interval, which usually, but not always, extends to the bottom of a well; and
- The minimum required water production rate of the well.

6.4.2 EVALUATION APPROACH

The factors listed above affect the amount of water a well can produce, the amount of drawdown in the aquifer needed to produce that water, and the pumping water level inside the well while it is operating, which may be lower than the water level in the aquifer. As such, information regarding these factors is important when assessing impacts to individual wells; however, it is not readily available. For this reason, our present evaluation uses saturated thickness and interference drawdown, which can be determined by applying our analytical drawdown model to available information regarding nearby wells, to assess the range of potential impacts that may reasonably be expected.

Our evaluation of interference drawdown related impacts to nearby wells is based on the following specific data:

- The distance from the proposed pumping wells to the off-Site well in question;
- The predicted drawdown in the aquifer at the location of the off-Site well;

- The depth of the off-Site well; and

- The static depth to groundwater in the off-Site well.

Wells screened in the shallower zone are defined for purposes of this report as being less than 200 feet deep. These wells are generally privately-owned domestic wells, with a smaller number of agricultural wells (see **Section 4.6** and **Table 1**). As a result, the shallower wells are more numerous than the deeper wells in the area surrounding the Site, but the shallower wells tend to have lower pumping rates. For evaluation of interference drawdown-related impact, the shallower wells are important because they are sensitive to smaller levels of drawdown than are deeper wells. For this reason, shallow wells are evaluated for Impacts 1 through 4 (listed in **Section 6.4.1**); whereas, deeper wells (greater than 200 feet deep) are not considered to be at risk for Impacts 1 and 2 and are therefore evaluated only for Impacts 3 and 4.

The topographic elevation at the Site is about 90 feet above msl. As shown on **Figure 15**, water levels in the shallower wells near the Site are currently about 50 feet above msl and water levels in the deeper wells are about 40 feet below msl. Thus, the depth to water at the Site is about 40 feet for the shallower wells and 130 feet for the deeper wells. These representative water depths were used to compute the saturated thicknesses penetrated by the shallow and deeper wells in the Site vicinity (**Tables 1 and 2**, respectively). The drawdown curve presented in **Figure 19** was then used to estimate the predicted drawdowns at these wells, given their distance from the proposed well location for the project (**Figures 2, 10 and 11**). As discussed in **Section 6.2**, this drawdown curve shows predicted drawdowns for the deeper aquifer and is applied to predict drawdown for the shallower wells. Finally, the predicted drawdown was subtracted from estimated saturated thickness to estimate the saturated thickness remaining.

For the purposes of this analysis, Impacts 1 and 2 may be grouped together since they both result in a well's being rendered unusable. Impact 3 is best evaluated on a case by case basis during the mitigation phase (**Section 6.6**), but a limited discussion is included in **Section 6.4.5.** Impact 4 can occur in shallow or deeper wells that may or may not be at risk of the first three impacts. It is further discussed in **Section 6.4.6.**

6.4.3 PREDICTED INTERFERENCE DRAWDOWN IN WELLS NEAR THE SITE

Tables 1 and **2** list shallow and deep wells identified from DWR records within approximately 1 ½ miles of the proposed location for the pumping well(s) at the Site. Wells beyond this distance are still predicted to experience interference drawdown, but the likelihood of significant impact is less, and well records therefore were not retrieved. The predicted drawdown at more distant well locations can be estimated using the distance-drawdown relationship presented in **Figure 19**.

Table 1 summarizes data regarding 193 shallower wells located at distances up to 8,900 feet from the Site. The reported depths of these wells range from 43 to 200 feet. All of these wells are predicted to experience drawdown from groundwater pumping for the development. The predicted interference drawdown at these wells ranges from 2.9 to 9.1 feet. The estimated remaining saturated thickness of these wells after deducting the predicted interference drawdown ranges from -1.0 to 154.5 feet.

Table 2 lists 62 deeper wells identified from DWR records at distances up to approximately 8,400 feet from the proposed location of the water supply well(s) at the Site. The reported depths of these wells range from 201 to 1,501 feet. The predicted drawdown at these wells ranges from 3.1 to 17.8 feet, and the remaining saturated thickness of the wells (based on an assumed pre-existing depth to water of 130 feet) ranges from 66 to 1,367 feet.

6.4.4 IMPACTS ON USABILITY OF NEARBY SHALLOW WELLS (IMPACTS 1 AND 2)

To help characterize the risk of shallow wells experiencing Impacts 1 or 2, these wells may be grouped into several categories based on the remaining saturated thickness. Wells with remaining saturated thicknesses of less than 20 feet may be considered at greatest risk for going dry or being rendered unusable by having insufficient available drawdown to support normal (primarily residential) pumping. Eight such wells were identified and are highlighted in **Table 1**. Wells with remaining saturated thicknesses between 20 and 40 feet may have a smaller but still significant risk of experiencing these impacts. There are 32 such wells in the vicinity of the Site. Wells with remaining saturated thicknesses over 60 feet are probably not at significant risk of being dewatered or rendered unusable.

This risk categorization scheme is a simplification and does not consider the fact that information regarding several factors important to assessing well-specific impacts (i.e., well use, pumping rate, condition and type of pump) was not available from the DWR. In addition, other wells may be located in the vicinity of the Site for which DWR records were not available.

6.4.5 IMPACTS REQUIRING PUMP INTAKES TO BE RESET TO GREATER DEPTH (IMPACT 3)

A reduction in the saturated thickness above the well's pump intake can result in a decrease in the amount of water the well can produce. In extreme cases, the pumping water level inside the well can fall below the pump intake, potentially damaging the pump if the pump controls are not equipped to sense this condition and shut the pump down. In cases where the pump intake is set near the bottom of a well and cannot be lowered, this impact is essentially synonymous with Impacts 1 or 2, discussed in **Section 6.4.4**. In other cases, it may be possible to lower the pump intake and continue use of the well.

In shallow or domestic wells, the pump intake is often set near the bottom of the well and evaluation of this impact is synonymous with Impacts 1 and 2, as noted above. Pump intakes for deeper wells and high capacity wells (e.g., municipal wells) are commonly set above the bottom of the well and lowered as needed during the life of the well. Because information regarding pump intake depths and other well-specific factors that contribute to this impact is not readily available, this impact is best evaluated during the mitigation phase.

6.4.6 IMPACTS ON OPERATING COST OF NEARBY SHALLOW AND DEEPER WELLS (IMPACT 4)

Interference drawdown changes the operational characteristics of the pump operating within an existing water well. The additional interference drawdown effectively results in an increase in pump head (the distance the pump must lift the water), which in turn decreases the pump discharge rate, and changes the pump power requirements. The well will have to be pumped for a longer time each day as a result. Thus, more power will be required to pump the same total volume of water. The extent to which a well might be impacted by increased electrical costs may be dependant upon several factors, including the following:

- Distance from the proposed pumping wells to the off-site well of concern (*i.e.*, the amount of interference drawdown;
- Aquifer characteristics;
- Depth of the off-site well;
- Pumping water level in the well prior to the onset of interference drawdown;
- Pump specifications;
- Well condition and efficiency; and
- Nature of pumping (rate and duration/frequency).

Because information regarding the well-specific factors above (except well depth) is not readily available for wells near the Site, several operational scenarios and their associated changes in pumping power requirements were examined in order to add perspective on the range of impacts that might be anticipated. These included:

- A range of interference drawdown to represent varying distance between the pumping wells at the Site and the off-site well;
- Two pumping rates to generally represent well uses for residential, and irrigation/municipal/industrial purposes;
- A range of well depths (pump depths) to represent typical well depths in the area; and
- A range of pumping water levels based on conditions at the Rohnert Park Site and the addition of potential interference drawdown; and
- The assumption of appropriate pumps installed in the wells to produce the designated flow rates under the assumed conditions.

For each scenario, our engineer selected a pump that would be appropriate to supply water at the approximate rate specified given the well depth and water level. Thus, for purposes of this analysis, wells with different pumping water levels were assumed to contain different pumps, in order to maintain a reasonable match in each case between the well's pump, water level, and flow rate. The changes in electrical consumption to pump 1 AF of water were then evaluated for that pump when the different levels of interference drawdown were applied. Additional details regarding our methodology are presented in **Appendix A**.

By attempting to model a range of conditions, we hoped to bracket the real world pumps and ensure that their operating conditions lie within the feasible space of this analysis. While this analysis is not exact and may not be representative of all actual installed pump types and conditions, it does offer some insight as to how much additional power might be required to pump 1 acre-foot of water if additional water table drawdown occurs. If site-specific information regarding water wells and pumps becomes available in the future, this analysis could be adapted to examine power requirement impacts for those specific pumps during the mitigation phase of the project.

Evaluations representing two different well and pump configurations under two different interference drawdown conditions, were made based upon the following ranges of values and boundary conditions:

- Two pumping rates: 10 gpm and 250 gpm;
- For the 10 gpm pumping rate: a pump configuration with the intake depth at 200 feet bgs and an associated pumping water level of 100 feet bgs;

- For the 250 gpm pumping rate: a pump configuration with the intake depth at 500 feet bgs and an associated pumping water level of 265 feet bgs; and
- Two interference drawdown depths: 5 feet, and 20 feet.

The evaluations were combined to produce the following matrix four cells for which the additional incremental power (in kilowatt-hours [kW-hours]) required to pump 1 AF of water was evaluated per the procedures outlined in **Appendix A**.

Additional Power Consumption Caused by Interference Drawdown Under Representative Well Configurations for the Rohnert Park Site Vicinity

Pump Discharge Rate (gpm)		**10**
Pump Installation Depth (feet bgs)		**200**
Pumping Water Level (feet bgs)		**100**
	Interference Drawdown (feet)	**Additional Power Consumption (kW-hours/acre-foot)**
	5	103.1
	20	611.5
Pump Discharge Rate (gpm)		**250**
Pump Installation Depth (feet bgs)		**500**
Pumping Water Level (feet bgs)		**265**
	Interference Drawdown (feet)	**Additional Power Consumption (kW-hours/acre-foot)**
	5	7.0
	20	18.1

The results of our evaluation are discussed below. Additional details are presented in the graphs and charts included in **Appendix A**.

For the pumping case of 10 gpm, and 5.0 feet of interference drawdown, the additional power required was 103.1 kW-hours to pump 1 acre-foot of water. For 20.0 feet of interference drawdown, the additional power required was 611.5 kW-hours for 1 acre-foot of water. For the pumping case of 250 gpm, and 5.0 feet of inerference drawdown, the additional power required was 7.0 kW-hours to pump 1 acre-foot of water. For 20.0 feet of interference drawdown, the additional power required was 18.1 kW-hours for 1 acre-foot of water.

The following conclusions may be drawn from the above results:

- As interference drawdown increases, the additional power required to pump 1 AF of water also increases.
- As the depth to the pumping water level increases, the additional power required to pump 1 AF of water when interference drawdown is applied also increases.
- Wells operated at lower flow rates (10 gpm) may experience a greater increase in the power required to pump an acre-foot of water when interference drawdown occurs than higher capacity wells. Conversely, at higher flow rates (250 gpm), interference drawdown causes less of an increase in power to pump 1 AF of water than for the 10 gpm flow rate.
- Notwithstanding the increase in unit power consumption rates, the overall cost increase resulting from interference drawdown will be greater for higher capacity wells than for lower capacity wells. This is because lower capacity wells are typically associated with residential use, and the annual water volume pumped by a residential user is comparatively small. According to the American Water Works Association (AWWA), the average household in the United Stated uses approximately 1/3 AF of water per year. Another rule of thumb that is commonly used is that 1 AF of water can supply two households for one year. Therefore, the additional electrical cost to domestic well users will probably not be significant. For perspective, assuming an electricity cost of $0.15 per kW-hour, the additional yearly cost to operate the pumps modeled above would range form about $5 to $45 per year. Conversely, water wells pumping at higher rates are typically associated with agricultural, industrial or municipal users, with water requirements in the hundreds or thousands of acre-feet per year. Even though less additional power is generally required per acre-foot of water when interference drawdown occurs in higher capacity wells, the need to pump many acre-feet of water per year results in a larger overall annual cost increase. For perspective, if a municipal water user pumps 400 AF of water from a well in a year (which corresponds to a sustained pumping rate of about 250 gpm), the additional annual power requirement for the scenarios modeled above will be about 2,800 kW-hours with 5 feet of interference drawdown and 7,240 kW-hours with 20 feet of interference drawdown. At $0.15 per kW-hour, the additional cost impact to that user would be approximately $420 and $1,086 per year, respectively. The increase represents an increase of the total pumping cost of about 2 to 5 percent, respectively, for the pumps modeled.

- The difference between the power consumption increases for different pump and drawdown scenarios can be significant, reflecting the importance of using well- and pump-specific information in assessing impacts to wells during the mitigation phase of the project.

6.4.7 IMPACTS ON CITY OF ROHNERT PARK MUNICIPAL WELLS

As summarized in **Table 2**, Rohnert Park wells 3, 7, 16, 23, 24 and 41 are located within approximately 1.1 miles of the proposed pumping well location on the project Site. These wells are predicted to experience interference drawdown from project pumping ranging from approximately 4.7 to 17.8 feet. It should be noted that Rohnert Park well 24 (the well closest to the Site) and well 7 are reported to be out of service and on standby status (HydroScience, 2007). The project-related interference drawdown at the remaining wells is estimated to range from 4.7 to 9.1 feet. As shown on **Figures 13 and 14**, this is generally less than the amount of interference drawdown that the City's wells experience from operating its own groundwater production well system, approximately 20 to 75 feet. However, interference drawdown from Site pumping would be in addition to any interference drawdown induced by other off-Site pumping. Project-induced interference drawdown will result in some increase in electrical costs to operate City wells near the Site, with the cost dependant upon the well-specific factors discussed in **Section 6.4.6**. Information regarding current pump intake depths was not provided by the City for this study; therefore, we are unable to determine whether the pump intakes of nearby wells may need to be lowered. These impacts may be further evaluated based on well-specific information in the mitigation phase of the project.

6.5 CUMULATIVE IMPACTS

6.5.1 REGIONAL PUMPING

The groundwater demand for the project is 200 gpm or approximately 323 AFY. This represents an increase of approximately 0.8 to 1 percent in current groundwater pumping and 1 to 1.7 percent in future groundwater pumping in the Santa Rosa Valley groundwater basin (**Section 4.4**). Rohnert Park's WSA provides several estimates of historical, recent and future total pumping in that report's study area – the upper Laguna de Santa Rosa watershed (see **Section 3.11**). In 2003, the total groundwater pumping in the area was estimated to be 7,078 AFY (Winzler & Kelly, 2005). The report estimates that by 2025, the projected total area pumping will be 7,350 AFY (note that this figure includes 100 AFY attributed to the Graton Rancheria hotel and casino project). Based on these estimates, the project will increase current and future groundwater pumping in the upper Laguna de Santa Rosa watershed by approximately 4.5 percent.

6.5.2 REGIONAL GROUNDWATER LEVELS

The City of Rohnert Park's WSA maintains that "there is no indication that overdraft has occurred" (Winzler & Kelly, 2005); whereas, the O.W.L. Foundation has consistently argued that the declining groundwater levels of the 1970s and 1980s indicate the basin has been in a state of overdraft. The DWR has not made an official finding regarding the basin's overdraft status, and its most recent description of the Santa Rosa Plain sub-basin indicates that "[t]he Santa Rosa Plain ground water basin as a whole is about in balance, with increased ground water levels in the northeast contrasting with decreased ground water levels in the south" (DWR, 2003). Correspondence with USGS staff regarding its ongoing cooperative study of the Santa Rosa Plain groundwater basin indicate that the question of whether the basin is in overdraft will be addressed in its final report based on well hydrograph analysis and the numerical groundwater flow model that will be constructed.

In its ruling on *O.W.L. Foundation v. City of Rohnert Park,* the court found the WSA had wrongly used the DWR's definition of "critical overdraft" in its assessment and that the WSA should instead use the DWR's definition of "overdraft." The DWR's definition of overdraft is contained in Bulletin 118 as follows:

> *Groundwater overdraft is defined as the condition of a groundwater basin or subbasin in which the amount of water withdrawn by pumping exceeds the amount of water that recharges the basin over a period of years, during which the water supply conditions approximate average conditions. Overdraft can be characterized by groundwater levels that decline over a period of years and never fully recover, even in wet years.* (DWR, 2003)

The document goes on to state:

> *The word "overdraft" has been used to designate two unrelated typed of water shortages. The first is "historical overdraft" similar to the type illustrated in Figure 18, which shows that groundwater levels began to decline in the mid 1950s and then leveled off in the mid 1980s, indicating less groundwater extraction or more recharge.*

It is noteworthy that Figure 18 of the quoted report presents a relationship between groundwater pumping and groundwater levels that is very similar to **Figure 16** contained in this report. As such, the pumping history and well hydrographs in the southern Santa Rosa Plain sub-basin are consistent with the DWR's definition of a "historical overdraft" condition.

Based on the comparison in **Section 6.5.1**, the project represents a relatively modest increase in regional groundwater pumping. Basin-wide groundwater pumping is expected to remain relatively stable over the next several decades (**Section 4.4**). In the upper Laguna de Santa Rosa watershed (the southern Santa Rosa Plain), groundwater demand is expected to stay below historical levels that were associated with regional groundwater level declines in the 1980's (**Section 3.11**). Groundwater levels in the southern Santa Rosa Plain have been relatively stable through the 1990s and recently have shown signs of rebounding (**Section 5.4.2**). Under these conditions, it is not likely that the project will contribute to a further decline in regional groundwater levels; however, that does not mean the project will have no regional water level impacts. If current groundwater levels in the area are rebounding from a historical overdraft condition due to decrease in groundwater pumping since the late 1990's, then one regional impact of project groundwater pumping will be to decrease or slow this rebound in proportion to the amount of increase in pumping represented by the project.

6.5.3 GROUNDWATER DIVIDE MIGRATION OR GROUNDWATER INFLOW

Based on the available data, the stabilization in water levels in the vicinity of Rohnert Park in the late 1980's represents a readjustment in a region's water budget that can only be explained by decreased extraction, increased recharge or increased groundwater inflow from adjacent areas. As discussed in **Section 5.4.2**, groundwater levels near Rohnert Park stabilized about 10 years before the City of Rohnert decreased its rate of groundwater extraction (**Figure 16**). Therefore, the remaining explanations for the readjustment in the local water budget are increased recharge or groundwater inflow. There is no clear correlation between groundwater levels and historical precipitation after 1970 (Winzler & Kelly, 2005), therefore, a likely explanation is that the cone of depression associated with City of Rohnert Park pumping expanded until it intercepted sufficient recharge or groundwater inflow to stabilize. The source of this additional recharge or inflow has not been evaluated; however, one plausible explanation would be the possible migration of the groundwater divide between the Santa Rosa Plain groundwater sub-basin and Petaluma Valley groundwater basin that was described in the Todd (Todd, 2004). This possibility is discussed further below; however, it is also possible that additional recharge or inflow is being derived from other, as yet unidentified areas.

Todd indicated that increasing groundwater use by the City of Rohnert Park in the 1970's and 1980's caused a decline in groundwater levels that in turn caused the groundwater divide between the Santa Rosa Plain basin and Petaluma Valley basin to migrate southward, resulting in "groundwater capture" from the adjacent basin. Todd's groundwater budget for the southern Santa Rosa Plain includes an inflow of 65 to 70 AFY from the Petaluma Valley basin. This interpretation is supported by comparison of 1951 groundwater level contours in the USGS' 1958 study of groundwater resources in Sonoma County (Cartwright, 1958) and 2002 groundwater level contours. This comparison suggests that the groundwater divide migrated southward approximately ¾ mile to a current location between Railroad Avenue and Lichau Creek, potentially impinging upon the Lichau Creek watershed.

In response to this report, the San Francisco Regional Water Quality Control Board sent letters to Sonoma County and the Rohnert Park City Council on January 21, 2005 expressing their position that the Draft EIR for the Canon Manor Subdivision and the WSA are inadequate. In each letter, the RWQCB asserted that potential impacts of groundwater pumping in the Santa Rosa Plain sub-basin on Lichau Creek have not been adequately evaluated. Regarding the Canon Manor EIR, the letter stated the following.

> *The EIR acknowledges that groundwater is already being withdrawn from the groundwater basin with the Lichau Creek drainage to augment overdrafted/allocated groundwater in the basin to the north, which has apparently caused a significant relocation of the groundwater basin divide (seperating direction of groundwater flow) to the south, proximate to Lichau Creek.*

At the core of the RWQCB's concern was the fact that shallow groundwater discharges to Licheau Creek, and capture of some of this groundwater by the basin to the north could reduce groundwater discharge and change conditions in this salmonid-bearing (anadramous) stream.

The RWQCB sent a follow-up letter regarding the Canon Manor DEIR on February 7, 2005. This letter stated Sonoma County Public Works Department and SCWA had committed to monitor and investigate potential cumulative groundwater impacts to the Lichau Creek drainage as part of the ongoing cooperative study with USGS, thus satisfying the EIR requirement for evaluating this potential impact. Specifically, the letter states the following.

> *We learned that a mitigation measure, identified in the DEIR, regarding establishment of an inter-agency comprehensive hydrogeologic assessment to be conducted by the USGS, is currently being developed by the SCWA. This hydrogeologic assessment will be designed to monitor long-term cumulative impacts of groundwater use in the Santa Rosa Plain to support groundwater management and minimize future risk of overdraft. The study will be based on inter-agency cooperation, including data sharing and funding. The DEIR acknowledges that groundwater is already being drawn from the groundwater basin within the Lichau Creek drainage to augment*

groundwater extracted from the hydraulically connected groundwater basin to the north. As such, during our February 3 meeting, the County and SCWA staff expressed their commitment to expand the scope of the hydrogeologic assessment to monitor for the effects of potential cumulative impacts in the Lichau Creek watershed, including lowering of the groundwater table and loss of surface water base-flow.

An alternate explanation for the difference between the 1951 and 2002 water level contour maps was offered by Luhdorff & Scalmanini (2004b) in a technical memorandum written in support of the Rohner Park WSA. The memorandum points out that data point used to contour the groundwater level data were relatively widely spaced, that the "...Cardwell report illustrates the estimated location of the groundwater divide with two directional arrows, which are given no more significance than numerous other arrows illustrating the direction of groundwater flow" and the groundwater divide indicated on the 1951 map is not contiguous with the location of the groundwater divide in the adjacent ridges. On this basis, Luhdorff & Scalmanini concluded that a more likely location of the groundwater divide in 1951 would have been the dividing line between the Lichau Creek / Laguna de Santa Rosa watershed boundary, which is near the groundwater divide location inferred by Todd based on the 2002 groundwater level data. They therefore conclude that the groundwater divide has not migrated significantly.

Based upon our review, we agree that groundwater level data in the vicinity of the watershed divide are relatively sparse. In addition, both the hydraulic and topographic gradients are relatively gentle. The location of the groundwater divide could therefore be subject to differing interpretations. However, because the alluvial deposits along the valley axis are likely to be more continuous and permeable than the adjacent alluvial fan deposits to the east and the deposits of the Petaluma Formation and Sonoma Volcanics that underlie the adjacent highlands, the groundwater divide crossing the alluvial valley may not necessarily be contiguous with the location of the divide in the adjacent highlands. In addition, in alluvial valley areas with a gentle topographic and hydraulic gradients, a groundwater divide will not necessarily coincide with a watershed divide, but may be affected by other factors influencing groundwater inflows and outflows in the groundwater basins. In our opinion, the available data are inconclusive as to whether or not the groundwater divide has migrated or groundwater inflow is occurring from Petaluma Valley basin; however, migration of the groundwater divide would be consistent with the formation and expansion of a cone of depression in the southern Santa Rosa Plain sub-basin during the 1970s and 1980s. We understand that data to address this issue will be gathered and evaluated as part of the USGS – SCWA cooperative study.

Pumping near the groundwater divide has a proportionally greater likelihood of impacting the basin to the south than pumping near the City of Rohnert Park; however, pumping at the Site could nevertheless contribute to a cumulative impact to the Petaluma Basin and Lichau Creek watershed. To the extent that such impacts are occurring, the percentage of contribution from the project would likely be no more than the percentage of pumping represented by the project in the southern Santa Rosa Plain sub-basin, which is estimated to be approximately 4.5 percent (**Section 6.5.1**).

7 POTENTIAL IMPACTS OF USING GROUNDWATER TO SUPPLY THE REDUCED INTENSITY ALTERNATIVE (ALTERNATIVES D AND H)

This section presents an evaluation of the reduced-intensity alternatives (Alternatives D and H). Comparison to the results of the preceding analysis of Alternatives A, B and C is provided for perspective. The predicted groundwater impacts associated with Alternative E, the business park alternative, are expected to approximately proportional to the associated groundwater pumping rate, or about 50 percent less than those of Alternatives D and H. Therefore, specific evaluation of Alternative E was not conducted as part of this study.

7.1 PROJECT GROUNDWATER DEMAND

The proposed groundwater supply wells at the Site will pump at a combined average rate of 125 gpm (for Alternatives D and H); however, the average total water demand for the reduced-intensity development is 150 gpm. The difference between the groundwater pumping rate of 125 gpm and the total demand of 150 gpm will be made up by recycling highly-treated wastewater for on-Site reuse for non-contact purposes and irrigation (HydroScience, 2007). For purposes of predicting impacts in this report, a sustained pumping rate of 125 gpm (0.18 mgd or 202 AFY) from the proposed wells is assumed.

7.2 GROUNDWATER-LEVEL IMPACTS IN THE SITE VICINITY FROM PROPOSED PUMPING WELLS

The analytical drawdown model discussed in **Section 6.2** was used to predict drawdown impacts in the deeper screened zone in the vicinity of the Site resulting from the use of groundwater to supply the reduced intensity alternatives. The model, input parameters and limitations are identical to those discussed in **Sections 6.2**; however, a reduced pumping rate of 125 gpm was used.

The predicted distance-drawdown relationship for the in the deeper screened zone in the reduced-intensity alternatives is shown on **Figure 19.** As for Alternatives A, B and C, the distance represents the interval between a proposed pumping well and an off-Site measurement point or well (**Section 6.3**). As discussed previously, **Figure 19** can also be applied to any potential pumping well location on the Site, but in this report is used to assess impacts from locating the water production wells as shown in **Figure 2**. For the purposes of our evaluation, we have assumed that the water supply wells for the project will be set back at least 100 feet from the eastern Site boundary. The distance from the proposed pumping well to the Site boundary is coincides with the distance value of the left vertical axis of the chart on **Figure 19** (100 feet on the X-axis distance scale). **Figure 19** shows that at the Site boundary (100 feet), the predicted drawdown is 14.3 feet. In addition, the predicted drawdown attenuates to about 3 feet at a distance of about 5,500 feet and 1 foot at a distance of 14,000 feet from the proposed well. Drawdown less than 3 feet is probably insignificant in relation to natural seasonal and longer term variations in groundwater levels (**Figures 13, 14, 15 and 17**). Comparison of the drawdown curves on **Figure 19** indicates that at any given distance from the pumping well, the predicted drawdown for the reduced-intensity alternatives is approximately 40 percent less than Alternatives A, B and C.

Similar to the distance-drawdown curve for Alternatives A, B and C, the curve for the reduced-intensity alternatives represents drawdown after a period of four months of pumping, which is the approximate time period after the onset of pumping when water levels in other wells in the area have been observed to stabilize (**Section 6.3**). Also, the drawdown curve represents predictions for water levels in the deeper zone (between about 200 and 1,000 feet deep). There is reason to believe that drawdown in the shallow zone (less than 200 feet deep) may be less; however, since some drawdown has been observed in shallow zone wells when nearby deep wells are pumped and the implications of drawdown in shallow zone wells are potentially more severe, we have assumed that the predicted drawdown shown in **Figure 19** applies to both the shallow and the deep zone (**Section 6.3**).

7.3 INTERFERENCE DRAWDOWN IMPACTS IN OFF-SITE WELLS

7.3.1 TYPES OF IMPACTS AND EVALUATION APPROACH

The potential types of impacts to nearby wells that could result from pumping for the reduced-intensity alternatives are the same as those discussed in **Section 6.4.1** for Alternatives A, B and C. To summarize the discussion presented in **Section 6.4.1**, these potential impacts may be categorized as follows:

1. The well goes dry;

2. The well can no longer produce adequate water for the intended use;

3. The well pump intake requires lowering; and

4. The cost to operate the well increases.

As discussed in **Section 6.2**, the impacts that occur and the timing or severity of their manifestation is dependant on a variety of hydrogeologic and well-specific factors, some of which are not currently known. For this reason, our present evaluation uses saturated thickness and interference drawdown, which are determined by applying available information regarding nearby wells, and our analytical drawdown model, in an assessment of the range of potential impacts that may reasonably be expected. The method for estimating the remaining saturated thickness after deducting the predicted interference drawdown is identical to that discussed for Alternatives A, B and C in **Section 6.2**. The results are presented in **Table 3** for shallow wells (less than 200 feet deep) and in **Table 4** for deep wells (over 200 feet deep).

7.3.2 PREDICTED INTERFERENCE DRAWDOWN IN WELLS NEAR THE SITE

Tables 3 and **4** list shallow and deep wells identified from DWR records within approximately 1 ½ miles of the proposed location for the pumping well(s) at the Site. Wells beyond this distance are still predicted to experience interference drawdown, but the likelihood of significant impact is less, and well records therefore were not retrieved. The predicted drawdown at more distant well locations can be estimated using the distance-drawdown relationship presented in **Figure 19**.

Table 3 summarizes data regarding 193 shallower wells located at distances up to 8,900 feet from the Site. The reported depths of these wells range from 43 to 200 feet. All of these wells are predicted to experience drawdown from groundwater pumping for the development. The predicted interference drawdown at these wells resulting from the reduced-intensity alternative ranges from 1.8 to 5.7 feet (compared to 2.9 to 9.1 feet for Alternatives A, B and C), conservatively assuming that the shallower wells have the same interference drawdown impacts as deeper wells for which **Figure 19** is designed. The estimated remaining saturated thickness of these wells after deducting the predicted interference drawdown ranges from 0.3 to 164.3 feet (compared to -1.0 to 154.5 feet for Alternatives A, B and C).

Table 4 lists 62 deeper wells identified from DWR records at distances up to approximately 8,400 feet from the proposed location of the water supply well(s) at the Site. The reported depths of these wells range from 201 to 1,501 feet. The predicted drawdown at these wells ranges from 1.9 to 11.1 feet (compared to 3.1 to 17.8 feet for Alternatives A, B and C), and the remaining saturated thickness of the wells (based on an assumed pre-existing depth to water of 130 feet) ranges from 68 to 1,368 feet (compared to 66 to 1,367 feet for Alternatives A, B and C).

7.3.3 IMPACTS ON USABILITY OF NEARBY SHALLOW WELLS (IMPACTS 1 AND 2)

The approach used to characterize the risk of shallow wells experiencing Impacts 1 or 2 as a result of the reduced-intensity alternatives is identical to the approach taken for evaluating Alternatives A, B and C described in **Section 6.4.4**. Wells with remaining saturated thicknesses of less than 20 feet are considered at greatest risk for going dry or being rendered unusable by having insufficient available drawdown to support normal (primarily residential) pumping. Eight such wells were identified and are highlighted in **Table 3**. Although the amount of interference drawdown is approximately 40 percent less under the reduced-intensity alternative, these are the same eight wells judged to be at greatest risk for these impacts under our evaluation of Alternatives A, B and C. Wells with remaining saturated thicknesses between 20 and 40 feet may have a smaller but still significant risk of experiencing these impacts. There are 29 such wells in the vicinity of the Site under the reduced intensity alternative, a slight decrease compared to 32 wells under Alternatives A, B and C. Wells with remaining saturated thicknesses over 60 feet are probably not at significant risk of being dewatered or rendered unusable.

As stated in **Section 6.4.4**, this risk categorization scheme is a simplification and does not consider the fact that information regarding several factors important to assessing well-specific impacts (i.e., well use, pumping rate, condition and type of pump) was not available from the DWR. In addition, other wells may be located in the vicinity of the Site for which DWR records were not available.

7.3.4 IMPACTS REQUIRING PUMP INTAKES TO BE RESET TO GREATER DEPTH (IMPACT 3)

As discussed in **Section 6.4.5**, information regarding pump intake depths and other well-specific factors that contribute to this impact is not readily available, so this impact is best evaluated during the mitigation phase. In shallow or domestic wells, the pump intake is often set near the bottom of the well and evaluation of this impact is synonymous with Impacts 1 and 2, as noted above. Pump intakes for deeper wells and high capacity wells (e.g., municipal wells) are commonly set above the bottom of the well and lowered as needed during the life of the well.

7.3.5 IMPACTS ON OPERATING COST OF NEARBY SHALLOW AND DEEPER WELLS (IMPACT 4)

The analysis of interference drawdown on pump operating costs presented in **Section 6.4.6** and the resulting conclusions are also applicable to the low intensity alternatives. That is, the interference drawdown resulting from withdrawing groundwater to supply the hotel and casino development is not likely to cause a significant increase in pump operating costs for nearby domestic well owners due to the relatively low volume of groundwater used by a typical residence. However, the increased electrical cost to operate a higher capacity municipal, industrial or irrigation well could range from several hundred to perhaps several thousand dollars per year, depending on the well-specific factors discussed in **Section 6.4.6** and volume of groundwater produced. Because the drawdown induced by pumping under Alternatives D and H is less than the drawdown under Alternatives A, B and C, the increase in electrical cost will also be less. The relationship between increased electrical usage and interference drawdown is often assumed to be linear (as in the commonly used formula presented in **Section 8.6**), and a linear relationship would mean that the increased electrical costs to nearby well operators resulting from the reduced-intensity alternatives will be roughly 40 percent less than for Alternatives A, B and C. However, as highlighted by the analysis discussed in **Section 6.4.6**, the relationship is not necessarily linear. Electrical consumption by the pumps modeled in that analysis increased at a rate approximately 65 to 150 percent of drawdown increase. Thus, assuming that increased electrical usage will be 40 percent less for Alternatives D and H (compared to Alternatives A, B and C) is a reasonable approximation, but the actual difference will depend on well and pump specific factors that are not known at this time.

7.3.6 IMPACTS ON CITY OF ROHNERT PARK MUNICIPAL WELLS

As shown in **Tables 2 and 4**, Rohnert Park wells 3, 7, 16, 23, 24 and 41 are located within approximately 1.1 miles of the proposed pumping well location on the project Site. These wells are predicted to experience interference drawdown from the reduced-intensity alternatives ranging from approximately 2.8 to 11.1 feet. This compares to approximately 4.7 to 17.8 feet for Alternatives A, B and C. It should be noted that Rohnert Park well 24 (the well closest to the Site) and well 7 are reported to be out of service and on standby status (HydroScience, 2007). The project-related interference drawdown at the remaining four wells is estimated to range from 2.8 to 5.7 feet (compared to approximately 4.7 to 9.1 feet for Alternatives A, B and C). As shown on **Figures 13 and 14**, for both the reduced-intensity and preferred alternatives, the project-related interference drawdown is generally less than the amount of interference drawdown that the City's wells experience from operating its own groundwater production well system, approximately 20 to 75 feet. However, interference drawdown from Site pumping would add to any interference drawdown induced in City of Rohnert Park wells by other off-Site pumping, and would result in increased electrical costs to operate the City wells listed above, with the cost dependant upon the well-specific factors discussed in **Section 6.4.6**. Based on the range of interference drawdown predicted for the City's active wells and the pumps modeled in **Section 6.4.6**, it is reasonable to conclude that electrical costs for operating these wells may increase by approximately 2 percent.

Information regarding current pump intake depths was not provided by the City for this study; therefore, we are unable to determine the potential effect of pump intake depth on electrical costs and whether the pump intakes of nearby wells may need to be lowered. These impacts may be further evaluated based on well-specific information in the mitigation phase of the project.

7.4 CUMULATIVE IMPACTS

7.4.1 REGIONAL PUMPING

The groundwater demand for the reduced intensity alternative is 125 gpm or approximately 202 AFY. This represents an increase of approximately 0.5 to 0.6 percent in current groundwater pumping and 0.6 to 1.1 percent in future groundwater pumping in the Santa Rosa Valley groundwater basin (**Section 4.4**). This relationship compares to an increase of approximately 0.8 to 1 percent of current groundwater pumping and 1 to 1.7 percent of future groundwater pumping for Alternatives A, B and C. Rohnert Park's WSA provides several estimates of historical, recent and future total pumping in that report's study area – the upper Laguna de Santa Rosa watershed (see **Section 3.11**). In 2003, the total groundwater pumping in the area was estimated to be 7,078 AFY (Winzler & Kelly, 2005). The report estimates that by 2025, the total area pumping will be 7,350 AFY (including 100 AFY attributed to the Graton Rancheria hotel and casino project). Based on these estimates, the reduced-intensity alternatives will increase current and future groundwater pumping in the upper Laguna de Santa Rosa watershed by approximately 2.9 percent. This compares to an approximately 4.5 percent increase for Alternatives A, B and C.

7.4.2 REGIONAL GROUNDWATER LEVELS

Based on the comparison in **Sections 6.5.1 and 7.4.1**, each project alternative represents a relatively modest increase in regional groundwater pumping. The increase is about 40 percent less for the reduced-intensity alternatives than for Alternatives A, B and C. Under both scenarios, basin-wide groundwater pumping is expected to remain relatively stable over the next several decades (**Section 4.4**). In the upper Laguna de Santa Rosa watershed (the southern Santa Rosa Plain), groundwater demand is expected to stay below historical levels that were associated with regional groundwater level declines in the 1980's (**Section 3.11**). Groundwater levels in the southern Santa Rosa Plain have been relatively stable through the 1990s and recently have shown signs of rebounding (**Section 5.4.2**). Under these conditions, it is not likely that the project will contribute to a further decline in regional groundwater levels because of the relatively modest increase in regional groundwater pumping; however, this does not mean the project will have no regional water level impacts. Current groundwater levels in the area are rebounding from a historical overdraft condition due to decrease in groundwater pumping since the late 1990's, thus one regional impact of project groundwater pumping will be to decrease or slow this rebound in proportion to the amount of increase in pumping represented by the project. The reduced-intensity alternative will have approximately 40 percent less effect on the pace of groundwater level rebound than Alternatives A, B and C.

7.4.3 GROUNDWATER DIVIDE MIGRATION OR GROUNDWATER INFLOW

Based upon our review, the available data are inconclusive as to whether or not the southern groundwater divide has migrated or groundwater inflow is occurring from Petaluma Valley basin; however, migration of the groundwater divide would be consistent with the historical formation and expansion of a cone of depression in the southern Santa Rosa Plain sub-basin. We understand that data to assess groundwater divide migration will be gathered and evaluated as part of the USGS – SCWA cooperative study. Pumping near the groundwater divide has a greater likelihood of impacting the Petaluma Valley basin than would pumping near the City of Rohnert Park; nevertheless, pumping at the Site could contribute to a cumulative impact to the Petaluma Basin and the Lichau Creek watershed specifically. To the extent that groundwater inflow from the Petaluma Basin to the Santa Rosa Plain groundwater sub-basin is occurring, the percentage of contribution from the project to inducing this inflow would likely be no more than the percentage of pumping represented by the project in the southern Santa Rosa Plain sub-basin (estimated to be approximately 2.8 percent). This is compared to 4.5 percent for the Alternatives A, B and C (**Section 7.4.1**).

8 POTENTIAL MITIGATION MEASURES

8.1 GROUNDWATER PUMPING TEST

The actual drawdown impacts from using groundwater to supply the proposed projects can be verified and more accurately assessed if a pumping test is conducted at the Site to simulate project pumping. The pumping test would preferably be performed using a test well installed at the Site, but could possibly be performed using one of the existing wells, if they are determined to be suitable. During the pumping test, water levels in at least two nearby shallow and two nearby deeper wells should be monitored. If adequate monitoring wells are not available, we recommend that monitoring wells be installed. The results of the pumping test could be used to update the analytical drawdown model prepared for this study with Site-specific data resulting in a more reliable set of drawdown predictions to represent the effects of the proposed pumping.

It is also recommended that as part of the pumping test a very shallow well be installed and monitored near the Laguna de Santa Rosa to help verify whether the groundwater pumping could influence surface water-groundwater interaction in this area. In addition, a very shallow and a shallow well should be monitored near the leaking underground fuel tank (LUFT) sites located northeast of the Site to help evaluate whether project pumping could increase vertical gradients in the vicinity of these sites (WorleyParsons Komex, 2007)._This information would be used in supply well design as discussed below.

8.2 WATER SUPPLY WELL DESIGN

Based on the results of the pumping test described above, the depth and screened intervals of the production wells at the Site could be designed to help confine drawdown impacts to the intermediate or deep water-bearing aquifers beneath the area, and to reduce the drawdown impacts to the shallow and uppermost aquifers, to the extent that the existing stratigraphy would allow such confining effects to occur. The stratigraphy in the southern Santa Rosa Plain suggests that optimized well design can be used to help mitigate the extent of drawdown impacts to shallow wells in the area, as well as potential impacts to surface water and impacts from shallow groundwater contamination incidents (WorleyParsons Komex, 2007).

8.3 GROUNDWATER LEVEL MONITORING

The actual drawdown impacts from using groundwater to supply the proposed projects can only be accurately assessed with the implementation of a properly designed monitoring program. Such a program would allow documentation of the actual distance-drawdown relationship in the vicinity of the Site, local ambient groundwater level trends and the potential influence of interference drawdown from other water users in the area. This information in turn can be used to guide the application and evaluate the effectiveness of the hydrogeological mitigation measures that are being considered as part of the project, and can form the basis for assessment of impacts to well owners in the Site Vicinity.

A groundwater level monitoring program could include existing wells and/or new wells installed for the project. We recommend that a monitoring program be designed based on an evaluation of completion data and lithologic logs for existing wells that may be available for that purpose. The monitoring program should include at least two wells completed at depths shallower than 200 feet and two wells completed at depths between 300 and 600 feet. Ideally, one shallow and one deep monitoring well should be located within ½ mile of the proposed project pumping well(s) to evaluate near-Site drawdown associated with the project. The other shallow and deep monitoring wells should be located between 1 and 2 miles from the pumping well(s), near the estimated lateral limit of significant drawdown associated with the project.

If existing wells are used, they should not be used for water production within one month of being measured. Also, the monitoring wells should not be located near wells that are being actively pumped. We recommend that water level measurements begin at least one year prior to project development to develop sufficient baseline data, and that both spring and fall measurements be taken.

Data from groundwater level monitoring that is conducted by DWR can also be used to assess the ongoing regional groundwater level trend in the Santa Rosa Plain sub-basin and establish a regional baseline.

8.4 ON-SITE HYDROGEOLOGIC MITIGATION MEASURES CONSIDERED AS PART OF SITE DEVELOPMENT

Several mitigation measures are planned that will reduce the impact of on-Site pumping for water supply. Key measures that are planned include Best Management Practices (BMPs) that promote infiltration of storm water runoff from developed portions of the Site, and on-Site disposal of treated wastewater. BMPs for enhancing infiltration of storm water runoff have the potential to increase the rate of natural recharge at the Site, while on-Site disposal of treated wastewater will return groundwater originating from the casino wells back to the aquifer. As discussed below, the effectiveness of these measures to reduce drawdown impacts is directly proportional to the rate of new recharge compared with the pumping rate.

Since buildings and pavements are relatively impermeable to storm water, such "hardscape" development increases runoff and decreases recharge to groundwater relative to pre-development conditions. The primary function of storm water BMPs is usually to decrease the amount or rate of runoff entering waterways from impermeable hardscape development. However, an important secondary function is to recapture as recharge some of the storm water that would otherwise flow from the Site as runoff. Storm water BMPs that are planned for the project include routing of storm water runoff to landscaped areas where feasible, conveying storm water via vegetated swales instead of concrete-lined V-ditches, and constructing a storm water detention basin to retain a portion of the storm water at the Site, where it will evaporate or percolate into the subsurface. The effectiveness of these BMPs in promoting recharge depends on soil and climatic conditions, available space, Site layout and BMP design. Because near surface soil at the Site is relatively fine grained (HydroScience, 2007), designing a storm water basin to percolate more than the pre-development recharge may not be feasible at this location. Therefore, it is not anticipated that these BMPs will have a significant mitigating effect on drawdown. It should be noted, however, that any recharge from surface water percolation will preferentially mitigate drawdown in the shallow aquifer, where wells are potentially most sensitive to drawdown impacts.

Recharge from on-Site application of treated wastewater for irrigation and spray field disposal could also have a mitigating effect on groundwater drawdown. Wastewater from the casino development will be treated in an on-Site wastewater treatment plant to a level meeting or exceeding tertiary treatment standards. The treated wastewater will then be disposed at the Site by spray field irrigation during the dry season (May 15 through September 30) or by a combination of surface water discharge and spray field irrigation during the wet season. A portion of the wastewater will also be recycled to reduce the demand for pumped groundwater, and a portion will be used for on-Site irrigation. The rate of dry season spray field irrigation is estimated to be approximately 185,000 gpd for Alternatives A, B and C, which will be applied at agronomic rates. The amount of this water that percolates into the subsurface will depend on climatic and soil conditions as well as the design of the spray field and irrigation system. Generally, it can be assumed that 5 to 10 percent of the applied water would percolate to recharge the shallow groundwater. This would represent a relatively modest mitigation of drawdown (proportional to the ratio of the percolated wastewater to the extracted groundwater) of 3 percent to 6 percent. The percentage of drawdown mitigated for the reduced-intensity alternative would be similar. In both cases, the effect of this mitigation would be more pronounced in the shallow than in the deeper aquifer.

8.5 POTENTIAL OFF-SITE HYDROGEOLOGIC MITIGATION MEASURES

In addition to mitigation measures that are being considered as part of Site development, participation in off-Site artificial recharge projects is currently being considered. This could be accomplished by purchasing rights to wells that are currently pumping and then not using them (*in lieu* recharge) or underwriting local water conservation programs.

8.6 POTENTIAL MITIGATION MEASURES FOR IMPACTS TO NEARBY WELLS

The amount of project-related interference drawdown that may be expected has been predicted as a function of distance (**Figure 19**) and at the known off-Site well locations (**Tables 1** through **4**). The actual amount of interference drawdown associated with the project will be estimated from the proposed pumping test and groundwater level monitoring program (**Sections 8.1 and 8.3**). We recommend that these data be used in the proposed mitigation program to distinguish the portion of impacts to nearby wells that is project related vs. the portion that is attributable to interference drawdown from other nearby high-capacity wells. At least one year of baseline data and one year of data after project pumping begins should be collected prior to implementation of the mitigation/cost reimbursement program outlined below.

The following mitigation measures for impacts to nearby wells are proposed:

- Well Usability (Impacts 1 and 2) – The tribe would reimburse the owners of wells that become unusable within 3 years[4] of the onset of project pumping for a portion of the prevailing, customary cost for well replacement, rehabilitation or deepening. The mitigation method for which reimbursement is made would be the lowest-cost customary and reasonable method to restore the lost well capacity. The percentage of the cost reimbursed by the tribe would depend upon the degree to which the impact is caused by project pumping vs. pumping by other nearby operators of high capacity wells. Reimbursement would be for replacement in-kind; that is, for a well of similar

[4] As discussed in **Section 6.2.2**, hydrographs and time-drawdown graphs for wells in the City of Rohnert Park's well field indicate that drawdown tends to stabilize at a new level about four months after a change in pumping; therefore, three years was selected as a conservative amount of time for interference drawdown impacts to manifest themselves and for well owners to become aware of them and submit a claim.

construction, but deepened so as to restore the lost well capacity. A depreciation allowance would be subtracted from the reimbursement amount for wells or pumps that have condition issues. In order to be eligible, the well owner would need to provide the tribe with documentation of the well location and construction (diameter, depth, screened interval, pump type, etc.), and that the well was constructed and usable before project pumping was initiated.

- Groundwater level falling near or below pump intake (Impact 3) – Whether a pump intake requires lowering depends on a number of well-specific factors that are not known at this time. The tribe would reimburse the owners of wells with pumps that require lowering within 3 years[3] of the onset of project pumping for a portion of the prevailing, customary cost for this service. The percentage of the cost reimbursed by the tribe would take into consideration the degree to which the impact is caused by project pumping vs. pumping by other nearby operators of high capacity wells, and the degree to which a well's capacity may have been reduced in the absence of project pumping due to shallow placement of the pump intake. Replacement discharge piping would not be reimbursed, and replacement of pumps would not be reimbursed unless the pump was damaged due to project-related interference drawdown. In order to be eligible, the well owner would need to provide the tribe with documentation of the well location and construction, including pump intake depth, and that the well was constructed and usable before project pumping was initiated. The tribe must be made aware of the cost reimbursement claim prior to lowering of the pump intake, so that the need for possible well deepening, replacement or rehabilitation can be assessed. At the tribe's discretion, compensation may be paid toward well deepening, replacement or rehabilitation in lieu of toward lowering the pump intake.

- Increased Electrical and Maintenance Cost (Impact 4) – Based on our analysis, operators of wells utilized for domestic purposes and limited agricultural or industrial pumpers are not expected to experience significant increases in their electrical costs. The tribe would reimburse well owners pumping more than 100 AF/year for their additional annual electrical costs at the prevailing electrical rate based on the following formula[5]:

$$\text{KWhr/year} = \frac{\text{(gallons Pumped/year)} \times \text{(feet of interference drawdown)}}{1{,}621{,}629}$$

  In order to qualify for reimbursement, the well owner must provide proof of the actual annual volume of water pumped and/or the electrical usage associated with the pumping. As an alternative to annual payments, a one-time lump sum payment of a mutually agreeable amount could be made.

- No reimbursement would be made available for wells installed after operation of the project wells commences.

- For any of the above impacts, the tribe may choose at its discretion to provide the well owner with a connection to a local public or private water supply system *in lieu* of the above mitigation measures, at reduced cost in proportion to the extent the impact was caused by project pumping.

The known owners of identified wells within 2 miles of the proposed project pumping well(s) would be notified of the mitigation program outlined above before project pumping begins. We recommend that the tribe contract with a third party such as the County of Sonoma to oversee this mitigation program.

[5] This formula is derived from combining the following two formulas (CEC, 2005):

KW input = ([Pump brake horsepower] x 0.7457) / (motor efficiency)

Pump brake horsepower = ([gpm] x [feet of water] x [specific gravity]) / (3960 x [pump efficiency])

Where:
specific gravity = 1;
typical motor efficiency = 85%; and
typical pump efficiency = 60%

9 CONCLUSIONS

- Under Alternatives A, B and C, the proposed groundwater supply wells at the Site will pump at a rate of approximately 200 gpm, which is equivalent to 0.29 mgd or 323 AFY. This represents an increase of approximately 0.8 to 1 percent in current groundwater pumping and 1 to 1.7 percent in future groundwater pumping in the Santa Rosa Valley groundwater basin, and a 4.5 percent increase in current and projected future pumping in the southern portion of the Santa Rosa Plain sub-basin. Under the reduced intensity alternatives (Alternatives D and H), the proposed groundwater supply wells will pump at a rate of 125 gpm, which is equivalent to 0.18 mgd or 202 AFY. This represents an increase of approximately 0.5 to 0.6 percent in current groundwater pumping and 0.6 to 1.1 percent in future groundwater pumping in the Santa Rosa Valley groundwater basin, and a 2.9 percent increase in current and projected future pumping in the southern portion of the Santa Rosa Plain sub-basin. Under Alternative E, the business park alternative, the increased groundwater demand would be about half of the reduced intensity alternatives. Under each scenario, the basin-wide and local increase in groundwater demand is relatively modest.

- Between 1970 and about 1990, when groundwater levels were in marked decline, and Rohnert Park's pumping increased steadily, groundwater recharge or inflow was likely increasing. According to this interpretation, additional recharge or inflow to the sub-basin was induced by the declining groundwater levels. Groundwater levels ceased falling when the increased recharge or inflow rate was sufficient to balance the water budget for the Santa Rosa Plain groundwater sub-basin including the increased pumping rate, which reached a stable peak value in about 1990. We lack sufficient information to positively identify the source of this additional recharge or inflow; however, the implication is continued annual pumping of 4.3 mgd at the City's well field could occur without causing long-term water-level declines below the elevations reached during 1990 through 1997 (providing that all other pumping rates and hydrologic conditions in the southern Santa Rosa Plain groundwater sub-basin remained similar to conditions during 1990 through 1997). Note that the absence of continued water-level declines does not imply the absence of impacts.

- Hydrographs and time-drawdown graphs for wells in the City of Rohnert Park's well field indicate that drawdown in the pumping wells tends to stabilize at a new level about four months after a change in pumping. This is most clearly seen for drawdown decreases that begin in October of each year when pumping rates typically decrease.

- Drawdown due to pumping the proposed wells was predicted using an analytical model based on the Theis equation. For Alternatives A, B and C, the predicted drawdown at the Site boundary in the deeper screened zone is 23.0 feet. In addition, the predicted drawdown in the deeper screened zone attenuates to about 1 foot at a distance of 17,000 feet from the proposed wells. For the reduced-intensity alternatives (D and H) the predicted drawdown in the deeper screened zone at the Site boundary is 14.3 feet and the predicted drawdown attenuates to about 1 foot at a distance of 14,000 feet. The drawdown for Alternatives D and H is about 40 percent less than for Alternatives A, B and C. The drawdown for the Alternative E would be roughly half that for Alternatives for D and H, assuming an approximately proportional relationship between pumping and drawdown response .

- Off-Site pumping causes greater drawdown near the Site in the deeper State well (6N/8W 15-R1) as opposed to the shallower State well (6N/8W 15-J3). We expect that pumping from the proposed wells at the Site (screened below 200 feet) would produce a similar type of effect on these two State wells, causing more drawdown in the deeper screened zone (below 200 feet bgs) versus the shallower screened zone (up to 200 feet bgs). However, the effects on shallow wells of pumping in deep wells (such as the proposed wells) cannot be quantified with the available information. Thus the drawdowns in shallower and deeper wells are considered to be equal for the purposes of this report. The same drawdown can result in more serious impacts for shallower wells due to the smaller saturated intervals.

- Water levels in the shallower wells near the Site are about 50 feet above msl and water levels in the deeper wells are about 40 feet below msl. This corresponds to depths to water at the Site of about 40 feet for the shallower wells (completed at depths up to 200 feet bgs) and 130 feet for the deeper wells (completed at depths greater than 200 feet). Most recently, water levels have continued to recover above this elevation, as pumping the City of Rohnert Park has decreased. Records obtained from the DWR indicate there are at least 193 shallower wells and 61 deeper wells located within approximately 1.5 miles of the geographic center of the Site. It is not known how many of these wells are still being actively used, or whether there are other wells for which records were not available. All of these wells are predicted to experience some drawdown impacts (interference drawdown) and a resulting proportional decrease in well yield or efficiency, pumping cost and pump life. In the absence of well-specific data regarding transmissivity, use, condition and efficiency, these impacts may be assumed to be generally proportional to the amount of interference drawdown and the remaining saturated thickness of the well after interference drawdown.

- The most serious impact that could be experienced by a nearby groundwater user would be having their well go dry or rendered unusable because the remaining saturated thickness after drawdown is too small to support pumping at the required rate. The wells most at potential risk for this impact are expected to be primarily shallow domestic wells near the site. For perspective, we have grouped the wells reported in the vicinity of the Site into several categories based upon the saturated thickness after interference drawdown. Shallow wells with a saturated thicknesses of less than 20 feet during the proposed pumping are considered at greatest risk for going dry or being rendered unusable by having insufficient available drawdown to support normal pumping. Eight such wells were identified under both Alternatives A, B and C and the reduced-intensity alternatives (D and H). Wells with saturated thicknesses between 20 and 40 feet during the proposed pumping may have a smaller but still potentially significant risk of experiencing these impacts. There were 32 such wells under Alternatives A, B and C and 29 under the reduced intensity alternatives. Wells with saturated thicknesses over 40 feet during the proposed pumping are at much lower risk of being dewatered or rendered unusable. All of the deeper wells fall into this category.

- In some wells, if water levels fall to a point where the well is in danger of going dry or becoming unusable, the pump intakes can be lowered to extend the life of the well. Without more specific information regarding well construction and pump depth, it is not possible to estimate how many wells may be at risk of experiencing this impact. However, pump intakes for shallow or domestic wells are generally set near the well bottoms and cannot be lowered; whereas, pump intakes for deeper municipal, industrial or agricultural wells are sometimes set at a relatively shallow depth and could require lowering if the wells are located near the Site.

- Interference drawdown will cause an increase in the electrical cost to pump a unit volume of groundwater from a well (**Sections 6.4.6 and 7.3.5**). This cost increase is not expected to be significant for domestic wells because of the relatively low volume of groundwater pumped by a typical household, but could be significant (ranging from several hundred to several thousand dollars) for higher capacity agricultural, industrial or municipal wells near the Site. For the pumps modeled, the increased costs for higher capacity pumping represented approximately a 2 to 5 percent increase in overall pumping costs. This analysis is applicable to Alternatives A, B and C as well as to the reduced-intensity alternatives, although the increase in pumping costs for off-site wells resulting from the reduced-intensity alternative is expected to be lower.

- The overall groundwater level trend since the 1970's fits the California Department of Water Resource's definition of "historical overdraft;" however, groundwater levels appear to be recovering slightly as groundwater pumping has decreased. The groundwater divide between the Santa Rosa Plain groundwater sub-basin and the Petaluma Valley groundwater basin to the south may have migrated southward during the historical overdraft period, resulting in capture of some groundwater from the adjacent basin. The proposed project pumping represents a small increase to the overall regional current and future groundwater pumping rate (approximately 0.5 to 1.7 percent basin-wide and 2.9 to 4.5 percent in the southern Santa Rosa Plain, depending on the development alternative), and projected groundwater pumping rates in the southern Santa Rosa Plain sub-basin are expected to stay below the peak levels of the 1980s and 1990s. It is therefore unlikely that groundwater pumping for the project will cause a resumption of declining groundwater level trends or further migration of the groundwater divide, but that does not mean the project will have no regional hydrogeologic impacts. Project pumping would be expected to result in a small decrease in the rate of recovery from the historical overdraft condition (proportional to the ratio of Site pumping to regional pumping). In addition, to the extent that water groundwater capture is occurring from the adjacent basin, pumping at the Site could contribute to a fraction of this capture.

10 CLOSURE/LIMITATIONS

This report has been prepared for the exclusive use of Analytical Environmental Services, Inc. as it pertains to the assessment of the effects of the use of groundwater to supply the proposed Graton Rancheria hotel and casino development near Rohnert Park, Sonoma County, California. Our services have been performed using that degree of care and skill ordinarily exercised under similar circumstances by reputable, qualified environmental consultants practicing in this or similar locations. No other warranty, either express or implied, is made as to the professional advice included in this report. These services were performed consistent with our agreement with our client.

Opinions and recommendations contained in this report apply to conditions existing when services were performed and are intended only for the client, purposes, locations, time frames, and project parameters indicated. We do not warrant the accuracy of information supplied by others or the use of segregated portions of this report. Our evaluations were performed on the basis of information that was reasonably available within the time frame and constraints of the project, and may not include all of the data that are available. Reasonable extrapolations and interpretations were made; however, these are not a substitute for Site-specific data. Such data are typically gathered during the design phase of a Site-specific water supply well and the mitigation phase of a project, and can be used to more accurately predict well yields and drawdown impacts, and to optimize well design for these factors.

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Table 1 - Construction Details for Shallow Wells Near the Site and Predicted Interference Drawdown and Saturated Thickness under Alternatives A, B and C

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LOCATION: Graton Rancheria Hotel and Casino, Rohnert Park, CA
PROJECT DESCRIPTION: Groundwater Study

DATE: 3-Dec-06
BY: Alan Blakemore
REVISION: Mike Tietze

Well Reference Number [1]	Year Installed	Well Type	Depth (feet bgs) [2]	Screened Interval (feet bgs)	Approximate Distance from Pumping Well (miles) [3]	Predicted Drawdown (feet) [4]	Saturated Thickness (feet)	Predicted Saturated Thickness after Drawdown (feet) [5]
1	1987	Domestic	90	29-90	1.3	4.0	50.0	46.0
2	1980	Domestic	81	36-81	1.3	4.0	41.0	37.0
3	1972	Domestic	120	75-120	0.5	8.2	80.0	71.8
4	1946	Irrigation/Stock	160	Unknown	0.8	6.0	120.0	114.0
5	1974	Domestic	95	55-95	1.2	4.4	55.0	50.6
6	1972	Domestic	80	40-80	1.4	3.8	40.0	36.2
7	1953	Domestic	137	116-136	1.3	4.0	97.0	93.0
8	1945	Unknown	115	Unknown	1.3	4.0	75.0	71.0
9	1978	Domestic	79	39-79	1.4	3.8	39.0	35.2
10	1979	Domestic	160	35-160	0.7	6.7	120.0	113.3
11	1980	Domestic	160	40-160	0.7	6.7	120.0	113.3
12	1975	Domestic	172	129-172	1	5.1	132.0	126.9
13	1971	Domestic	68	48-68	0.8	6.0	28.0	22.0
14	1971	Domestic	84	64-84	0.8	6.0	44.0	38.0
15	1969	Domestic	82	62-82	1.2	4.4	42.0	37.6
16	1974	Domestic	128	108-128	1.2	4.4	88.0	83.6
17	1976	Domestic	92	52-92	1.3	4.0	52.0	48.0
18	1971	Domestic	79	59-79	1	5.1	39.0	33.9
19	1971	Domestic	76	56-76	0.8	6.0	36.0	30.0

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21	1949	Unknown	157	Unknown	1.1	4.7	117.0	112.3
22	1949	Unknown	107	Unk	0.9	5.5	67.0	61.5
23	1982	Domestic	150	50-150	1.2	4.4	110.0	105.6
24	1986	Domestic	176	96-176	1.1	4.7	136.0	131.3
25	1987	Domestic	157	54-154	1.3	4.0	117.0	113.0
26	1976	Domestic	110	40-110	1.1	4.7	70.0	65.3
27	1972	Domestic	146	62-146	1.5	3.4	106.0	102.6
28	1967	Domestic	100	80-100	1.2	4.4	60.0	55.6
29	1973	Domestic	114	104-114	0.8	6.0	74.0	68.0
30	1974	Domestic	100	60-100	1.1	4.7	60.0	55.3
31	1974	Domestic	128	108-128	1.2	4.4	88.0	83.6
32	1972	Domestic	118	98-118	1.1	4.7	78.0	73.3
33	1973	Domestic	80	60-80	0.9	5.5	40.0	34.5
34	1975	Domestic	100	80-100	1.2	4.4	60.0	55.6
35	1974	Domestic	140	120-140	1.7	2.9	100.0	97.1
36	1985	Domestic	143	95-143	1.6	3.1	103.0	99.9
37	1954	Domestic	80	Unknown	1.6	3.1	40.0	36.9
38	1937	Unknown	100	38-100	1.6	3.1	60.0	56.9
39	1948	Unknown	114	Unknown	1.6	3.1	74.0	70.9

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40	1951	Domestic	108	98-108	1.7	2.9	68.0	65.1
41	1954	Domestic	120	60-120	1.6	3.1	80.0	76.9
42	1997	Domestic/Industrial	149	89-149	1.4	3.8	109.0	105.2
43	1973	Domestic	166	147-166	1.4	3.8	126.0	122.2
44	1962	Domestic	144	93-142	1.6	3.1	104.0	100.9
45	1964	Domestic	59	39-59	1.1	4.7	19.0	14.3
46	1989	Domestic	162	102-162	1.5	3.4	122.0	118.6
47	1979	Domestic	178	158-178	1.5	3.4	138.0	134.6
48	1964	Domestic	47	10-47	1.6	3.1	7.0	3.9
49	1985	Domestic	140	80-140	1.6	3.1	100.0	96.9
50	1985	Domestic	140	100-140	1.7	2.9	100.0	97.1
51	1976	Domestic	165	128-165	1.6	3.1	125.0	121.9
52	1967	Domestic	127	107-127	1.4	3.8	87.0	83.2
53	1956	Domestic	90	55-90	1.6	3.1	50.0	46.9
54	1953	Domestic	92	82-92	1.6	3.1	52.0	48.9
55	1966	Domestic	60	50-60	1.5	3.4	20.0	16.6
56	1980	Domestic	103	83-103	0.7	6.7	63.0	56.3
57	1980	Domestic	125	85-125	0.9	5.5	85.0	79.5
58	1979	Domestic	120	60-120	0.6	7.4	80.0	72.6

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59	1954	Domestic	112	93-111	1	5.1	72.0	66.9
60	1977	Domestic	98	59-99	1	5.1	58.0	52.9
61	1966	Domestic	123	Unknown	1	5.1	83.0	77.9
62	1974	Domestic	92	66-92	1	5.1	52.0	46.9
63	1984	Domestic	150	55-150	1.1	4.7	110.0	105.3
64	1986	Domestic	130	110-130	1	5.1	90.0	84.9
65	1987	Domestic	140	100-140	0.9	5.5	100.0	94.5
66	1954	Domestic	120	80-120	1.3	4.0	80.0	76.0
67	1979	Domestic	125	80-125	0.9	5.5	85.0	79.5
68	1965	Domestic	84	74-84	1.4	3.8	44.0	40.2
69	1970	Domestic	84	45-84	1.1	4.7	44.0	39.3
70	1973	Domestic	171	49-171	1.4	3.8	131.0	127.2
71	1977	Domestic	95	35-95	1.4	3.8	55.0	51.2
72	1968	Domestic	87	67-87	1.2	4.4	47.0	42.6
73	1969	Domestic	64	44-64	0.5	8.2	24.0	15.8
74	1977	Domestic	95	55-95	0.4	9.1	55.0	45.9
75	1961	Domestic	80	40-80	1.2	4.4	40.0	35.6
76	1962	Domestic	152	None	1.4	3.8	112.0	108.2
77	1971	Domestic	71	51-71	0.4	9.1	31.0	21.9

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78	1956	Domestic	121	80-120	1.3	4.0	81.0	77.0
79	1956	Domestic	160	140-160	0.9	5.5	120.0	114.5
80	1952	Domestic	111	91-111	0.8	6.0	71.0	65.0
81	1942	Unknown	90	70-90	0.9	5.5	50.0	44.5
82	1957	Municipal	80	27-80	1	5.1	40.0	34.9
83	1969	Domestic	107	87-107	1.1	4.7	67.0	62.3
84	1973	Domestic	100	78-100	1.3	4.0	60.0	56.0
85	1954	Domestic	43	29-43	1.2	4.4	3.0	-1.4
86	1967	Domestic	78	57-77	0.8	6.0	38.0	32.0
87	1983	Domestic	160	45-160	0.6	7.4	120.0	112.6
88	1943	Unknown	183	155-183	0.7	5.6	143.0	137.4
89	1930	Domestic/Stock	183	Unknown	0.8	6.0	143.0	137.0
90	1974	Domestic	200	180-200	0.6	7.4	160.0	152.6
91	1939	Unknown	79	Unknown	1.3	4.0	39.0	35.0
92	1948	Unknown	181	141-181	1.3	4.0	141.0	137.0
93	1983	Domestic	190	50-190	0.8	6.0	150.0	144.0
94	1974	Domestic	200	180-200	0.8	6.0	160.0	154.0
95	1974	Domestic	200	160-200	0.9	5.5	160.0	154.5
96	191	Domestic	191	155-191	0.9	5.5	151.0	145.5

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97	1997	Domestic	192	72-192	1.3	4.0	152.0	148.0
98	1948	Unknown	180	175-180	1.3	4.0	140.0	136.0
99	1941	Unknown	183	169-183	1.1	4.7	143.0	138.3
100	1937	Unknown	184	Unknown	0.9	5.5	144.0	138.5
101	1969	Domestic	180	120-180	1.1	4.7	140.0	135.3
102	1987	Domestic	197	114-197	1	5.1	157.0	151.9
103	1988	Domestic	197	77-197	1	5.1	157.0	151.9
104	1985	Domestic	196	76-196	1.1	4.7	156.0	151.3
105	1986	Domestic	195	140-195	1	5.1	155.0	149.9
106	1976	Domestic	189	112-189	1	5.1	149.0	143.9
107	1976	Domestic	193	113-193	1.1	4.7	153.0	148.3
108	1946	Unknown	166	65-166	0.8	6.0	126.0	120.0
109	1986	Domestic	200	160-200	1.5	3.4	160.0	156.6
110	1971	Domestic	198	178-198	1.1	4.7	158.0	153.3
111	1989	Domestic	197	135-195	1	5.1	157.0	151.9
112	1977	Domestic	200	140-160	0.8	6.0	160.0	154.0
113	1972	Domestic	79	59-79	0.6	7.4	39.0	31.6
114	1972	Domestic	79	59-79	0.8	6.0	39.0	33.0
115	1972	Domestic	83	63-83	0.6	7.4	43.0	35.6

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116	1972	Domestic	88	68-88	0.9	5.5	48.0	42.5
117	1972	Domestic	79	59-79	0.6	7.4	39.0	31.6
118	1972	Domestic	84	64-84	0.5	8.2	44.0	35.8
119	1972	Domestic	79	59-79	0.9	5.5	39.0	33.5
120	1972	Unknown	200	180-200	0.6	7.4	160.0	152.6
121	1986	Domestic	180	130-180	0.9	5.5	140.0	134.5
122	1962	Domestic	100	Unknown	1.3	4.0	60.0	56.0
123	1958	Domestic	156	120-154	1	5.1	116.0	110.9
124	1959	Domestic	72	50-72	1.4	3.8	32.0	28.2
125	1954	Domestic	104	96-104	1.2	4.4	64.0	59.6
126	1978	Domestic	105	95-105	1.1	4.7	65.0	60.3
127	1989	Domestic	150	90-150	1	5.1	110.0	104.9
128	1988	Domestic	207	Unknown	1.2	4.4	167.0	162.6
129	1954	Domestic	116	97-116	0.9	5.5	76.0	70.5
130	1956	Domestic	115	95-115	0.8	6.0	75.0	69.0
131	1986	Domestic	137	76-136	0.7	6.7	97.0	90.3
132	1955	Domestic	88	Unknown	1.2	4.4	48.0	43.6
133	1982	Domestic	104	48-104	1.1	4.7	64.0	59.3
134	1981	Irrigation	160	40-160	1.4	3.8	120.0	116.2

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135	1989	Domestic	160	100-160	1.4	3.8	120.0	116.2
136	1984	Domestic	85	25-85	1.1	4.7	45.0	40.3
137	1985	Domestic	135	75-135	1.4	3.8	95.0	91.2
138	1985	Domestic	170	130-170	1.2	4.4	130.0	125.6
139	1984	Domestic	175	115-175	1	5.1	135.0	129.9
140	1971	Domestic	76	54-76	1.2	4.4	36.0	31.6
141	1968	Domestic	141	58-141	1.3	4.0	101.0	97.0
142	1968	Domestic	116	108-116	1.1	4.7	76.0	71.3
143	1974	Domestic	100	80-100	1	5.1	60.0	54.9
144	1974	Domestic	100	80-100	1	5.1	60.0	54.9
145	1975	Domestic	170	50-170	1.2	4.4	130.0	125.6
146	1966	Domestic	59	39-59	1.2	4.4	19.0	14.6
147	1977	Domestic	80	40-80	1.1	4.7	40.0	35.3
148	1982	Domestic	147	30-147	1	5.1	107.0	101.9
149	1979	Domestic	107	77-107	0.7	5.6	67.0	61.4
150	1979	Domestic	140	60-140	0.9	5.5	100.0	94.5
151	1986	Domestic	159	129-159	1	5.1	119.0	113.9
152	1962	Domestic	136	128-136	1.4	3.8	96.0	92.2
153	1963	Domestic	155	127-141	1	5.1	115.0	109.9

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154	1962	Domestic	136	124-134	1.4	3.8	96.0	92.2
155	1962	Domestic	75	55-75	1.7	2.9	35.0	32.1
156	1962	Domestic	120	70-120	1.6	3.1	80.0	76.9
157	1961	Domestic	104	96-104	1.6	3.1	64.0	60.9
158	1971	Domestic	79	59-79	1.2	4.4	39.0	34.6
159	1972	Domestic	69	29-69	1.6	3.1	29.0	25.9
160	1953	Domestic	84	68-82	1.2	4.4	44.0	39.6
161	1953	Domestic	106	92-106	1.2	4.4	66.0	61.6
162	1953	Domestic	108	100-108	1.2	4.4	68.0	63.6
163	1950	Irrigation	90	Open Bottom	1.1	4.7	50.0	45.3
164	1953	Domestic	68	58-68	1.6	3.1	28.0	24.9
165	1953	Domestic	136	60-136	1.4	3.8	96.0	92.2
166	1953	Irrigation	109	25-109	1.2	4.4	69.0	64.6
167	1948	Unknown	140	Unknown	1.5	3.4	100.0	96.6
168	1953	Irrigation	176	116-176	0.9	5.5	136.0	130.5
169	1945	Unknown	110	90-110	1	5.1	70.0	64.9
170	1941	Unknown	76	Unknown	1.5	3.4	36.0	32.6
171	1980	Domestic	140	80-140	1.5	3.4	100.0	96.6
172	1978	Irrigation	141	85-141	1.4	3.8	101.0	97.2

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173	1957	Domestic	136	128-136	1.2	4.4	96.0	91.6
174	1955	Domestic	180	Unknown	1.3	4.0	140.0	136.0
175	1981	Domestic	149	50-149	1.5	3.4	109.0	105.6
176	1983	Domestic	125	25-125	1	5.1	85.0	79.9
177	1982	Domestic/ Irrigation	150	110-150	1.6	3.1	110.0	106.9
178	1986	Domestic	177	136-176	1.1	4.7	137.0	132.3
179	1986	Domestic	150	80-150	1.3	4.0	110.0	106.0
180	1984	Domestic	157	97-157	1.6	3.1	117.0	113.9
181	1985	Domestic	155	75-155	1.5	3.4	115.0	111.6
182	1985	Domestic	173	113-173	1.5	3.4	133.0	129.6
183	1984	Domestic	150	80-150	1.4	3.8	110.0	106.2
184	1975	Domestic	155	75-155	1.1	4.7	115.0	110.3
185	1977	Unknown	71	30-40	1.5	3.4	31.0	27.6
186	1979	Domestic	185	126-185	1.1	4.7	145.0	140.3
187	1968	Domestic	116	108-116	1.4	3.8	76.0	72.2
188	1966	Domestic	120	60-120	1.6	3.1	80.0	76.9
189	1974	Domestic	155	110-155	1.5	3.4	115.0	111.6
190	1964	Domestic	152	72-152	1.3	4.0	112.0	108.0
191	1966	Domestic	60	40-60	1.3	4.0	20.0	16.0

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Table 1 - Construction Details for Shallow Wells Near the Site and Predicted Interference Drawdown and Saturated Thickness under Alternatives A, B and C

CLIENT: AES
PROJECT No.: N0410C
LOCATION: Graton Rancheria Hotel and Casino, Rohnert Park, CA
PROJECT DESCRIPTION: Groundwater Study

DATE: 3-Dec-06
BY: Alan Blakemore
REVISION: Mike Tietze

Well Reference Number [1]	Year Installed	Well Type	Depth (feet bgs) [2]	Screened Interval (feet bgs)	Approximate Distance from Pumping Well (miles) [3]	Predicted Drawdown (feet) [4]	Saturated Thickness (feet)	Predicted Saturated Thickness after Drawdown (feet) [5]
192	1978	Domestic	159	79-159	1.1	4.7	119.0	114.3
193	1974	Domestic	178	32-178	1.3	4.0	138.0	134.0
20 [6]	1948	Unknown	61	Unknown	1.1	4.7	21.0	16.3

Notes:
1. Approximate locations of wells are plotted on Figure 10.
2. bgs = below ground surface
3. Approximate distance from pumping well rounded to the nearest 0.1 mile.
4. Predicted drawdown interpolated from Figure 19. Drawdown in shallow wells is expected to be less than this amount.
5. Saturated thickness is calculated assuming a depth to water of 40 feet and subtracting predicted drawdown.
6. Highlighed wells are at greatest risk of going dry or experiencing significant reduction in capacity.

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Table 2 - Construction Details for Deep Wells Near the Site and Predicted Interference Drawdown and Saturated Thickness under Alternatives A, B and C

CLIENT: AES
PROJECT No.: N0410C
LOCATION: Graton Rancheria Hotel and Casino, Rohnert Park, CA
PROJECT DESCRIPTION: Groundwater Study

DATE: 3-Dec-06
BY: Alan Blakemore
REVISION: Mike Tietze

Well Reference Number [1]	Year Installed	Well Type	Depth (feet bgs) [2]	Screened Interval (feet bgs)	Approximate Distance from Pumping Well (miles) [3]	Saturated Thickness (feet)	Predicted Drawdown (feet) [4]	Predicted Saturated Thickness after Drawdown (feet) [5]	Remarks
1	1971	Municipal	475	128-460	1.1	345	4.7	340	Rohnert Park No. 07
2	1936	Unknown	375	100-375	0.7	245	5.6	239	
3	1931	Unknown	438	Unknown	0.7	308	5.6	302	
4	1977	Municipal	1500	300-1500	1.1	1,370	4.7	1,365	Rohnert Park No. 16
5	1980	Municipal	590	190-580	0.4	460	9.1	451	Rohnert Park No. 23
6	1971	Unknown	314	Unknown	0.9	184	4.9	179	
7	1958	Municipal	805	271-805	1.1	675	4.7	670	Rohnert Park No. 03
8	1942	Unknown	201	180-201	0.9	71	5.5	66	
9	1987	Unknown	260	160-260	0.6	130	7.4	123	
10	1980	Municipal	592	258-582	0.06	462	17.8	444	Rohnert Park No. 24
11	1978	Domestic	407	387-407	0.5	277	8.2	269	
12	1958	Domestic	201	180-201	1.3	71	4.0	67	
13	1971	Domestic	208	188-208	1.3	78	4.0	74	
14	1975	Domestic	253	193-253	0.8	123	6.0	117	
15	1979	Domestic	230	160-230	1	100	5.1	95	
16	1979	Domestic	219	87-219	1	89	5.1	84	
17	1977	Domestic	269	164-269	1.2	139	4.4	135	
18	1981	Domestic	224	212-222	1.3	94	4.0	90	

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Table 2 - Construction Details for Deep Wells Near the Site and Predicted Interference Drawdown and Saturated Thickness under Alternatives A, B and C

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PROJECT DESCRIPTION: Groundwater Study

DATE: 3-Dec-06
BY: Alan Blakemore
REVISION: Mike Tietze

Well Reference Number [1]	Year Installed	Well Type	Depth (feet bgs) [2]	Screened Interval (feet bgs)	Approximate Distance from Pumping Well (miles) [3]	Saturated Thickness (feet)	Predicted Drawdown (feet) [4]	Predicted Saturated Thickness after Drawdown (feet) [5]	Remarks
19	1969	Domestic	215	195-215	0.9	85	5.5	80	
20	1983	Domestic	504	480-504	1.3	374	4.0	370	
21	1989	Domestic	270	230-270	1.2	140	4.4	136	
22	1986	Domestic	280	120-280	1.2	150	4.4	146	
23	1977	Domestic	231	195-231	1.1	101	4.7	96	
24	1976	Domestic	250	130-250	1	120	5.1	115	
25	1941	Unknown	258	Unknown	1.1	128	4.7	123	
26	1940	Unknown	216	176-216	1.2	86	4.4	82	
27	1986	Domestic	350	130-350	1.2	220	4.4	216	
28	1987	Domestic	270	170-270	1.1	140	4.7	135	
29	1956	Domestic	220	36-220	1.2	90	4.4	86	
30	1986	Domestic	295	135-295	1.3	165	4.0	161	
31	1985	Domestic	252	207-247	1.1	122	4.7	117	
32	1985	Domestic	204	64-204	1.1	74	4.7	69	
33	1968	Domestic	240	None	1	110	5.1	105	
34	1968	Domestic	250	230-250	0.8	120	6.0	114	
35	Unknown	Irrigation	725	Unknown	0.8	595	6.0	589	
36	1978	Domestic	204	132-204	1.2	74	4.4	70	

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Table 2 - Construction Details for Deep Wells Near the Site and Predicted Interference Drawdown and Saturated Thickness under Alternatives A, B and C

CLIENT: AES
PROJECT No.: N0410C
LOCATION: Graton Rancheria Hotel and Casino, Rohnert Park, CA
PROJECT DESCRIPTION: Groundwater Study

DATE: 3-Dec-06
BY: Alan Blakemore
REVISION: Mike Tietze

Well Reference Number [1]	Year Installed	Well Type	Depth (feet bgs) [2]	Screened Interval (feet bgs)	Approximate Distance from Pumping Well (miles) [3]	Saturated Thickness (feet)	Predicted Drawdown (feet) [4]	Predicted Saturated Thickness after Drawdown (feet) [5]	Remarks
37	1977	Irrigation	345	177-345	1	215	5.1	210	
38	Unknown	Irrigation	1028	Unknown	0.5	898	8.2	890	
39	1982	Domestic/Irrigation	218	110-218	1.5	88	3.4	85	
40	1980	Domestic	214	184-214	0.7	84	6.7	77	
41	1977	Irrigation	517	219-517	1.3	387	4.0	383	
42	1977	Municipal	1501	351-1491	1.2	1,371	4.4	1,367	Rohnert Park No. 15
43	1987	Industrial	260	180-260	1.2	130	4.4	126	
44	1997	Domestic	216	181-216	0.9	86	5.5	81	
45	1987	Domestic	210	287-310	0.8	80	6.0	74	
46	1950	Irrigation	914	Unknown	0.7	784	6.7	777	
47	1972	Irrigation	415	115-415	1.2	285	4.4	281	
48	1948	Unknown	Unknown	Unknown	1.2	Unknown	4.4	Unknown	
49	1947	Unknown	560	Unknown	0.9	430	5.5	425	
50	1951	Irrigation	204	0-204	1.5	74	3.4	71	
51	1978	Domestic	220	200-220	1.4	90	3.8	86	
52	1977	Domestic	389	379-389	1.6	259	3.1	256	
53	1988	Irrigation	360	220-360	1.5	230	3.4	227	
54	1987	Domestic	207	107-207	1.4	77	3.8	73	

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PROJECT DESCRIPTION: Groundwater Study

DATE: 3-Dec-06
BY: Alan Blakemore
REVISION: Mike Tietze

Well Reference Number [1]	Year Installed	Well Type	Depth (feet bgs) [2]	Screened Interval (feet bgs)	Approximate Distance from Pumping Well (miles) [3]	Saturated Thickness (feet)	Predicted Drawdown (feet) [4]	Predicted Saturated Thickness after Drawdown (feet) [5]	Remarks
55	1987	Domestic	320	260-320	0.9	190	5.5	185	
56	1989	Domestic	225	205-225	1.3	95	4.0	91	
57	1971	Irrigation	411	131-411	1.3	281	4.0	277	
58	1979	Domestic	500	423-463	1.1	370	4.7	365	
59	1980	Domestic	205	180-200	1.2	75	4.4	71	
60	1979	Domestic	260	40-260	1.4	130	3.8	126	
61	1950	Irrigation	1204	781-1204	0.9	1,074	5.5	1,069	
62	1992	Municipal	715	75-715	0.9	585	7.0	578	Rohnert Park No. 41

Notes:
1. Approximate locations of wells are plotted on Figure 11.
2. bgs = below ground surface
3. Approximate distance from pumping well rounded to the nearest 0.1 mile.
4. Predicted drawdown at the well interpolated from Figure 19.
5. Remaining saturated thickness is calculated assuming a depth to water of 130 feet and subtracting predicted drawdown

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Table 3 - Predicted Interference Drawdown and Saturated Thickness for Shallow Wells Near the Site under the Reduced- Intensitity Alternatives

CLIENT: AES
PROJECT No.: N0410C
LOCATION: Graton Rancheria Hotel and Casino, Rohnert Park, CA
PROJECT DESCRIPTION: Groundwater Study

DATE: 3-Dec-06
BY: Alan Blakemore
REVISION: Mike Tietze

Well Reference Number [1]	Well Type	Depth (feet bgs) [2]	Approximate Distance from Pumping Well (miles) [3]	Predicted Drawdown (feet) [4]	Saturated Thickness (feet)	Predicted Saturated Thickness after Drawdown (feet) [5]
1	Domestic	90	1.3	2.5	50.0	47.5
2	Domestic	81	1.3	2.5	41.0	38.5
3	Domestic	120	0.5	5.1	80.0	74.9
4	Irrigation/Stock	160	0.8	3.7	120.0	116.3
5	Domestic	95	1.2	2.8	55.0	52.3
6	Domestic	80	1.4	2.4	40.0	37.6
7	Domestic	137	1.3	2.5	97.0	94.5
8	Unknown	115	1.3	2.5	75.0	72.5
9	Domestic	79	1.4	2.4	39.0	36.6
10	Domestic	160	0.7	4.2	120.0	115.8
11	Domestic	160	0.7	4.2	120.0	115.8
12	Domestic	172	1	3.2	132.0	128.8
13	Domestic	68	0.8	3.7	28.0	24.3
14	Domestic	84	0.8	3.7	44.0	40.3

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Well Reference Number [1]	Well Type	Depth (feet bgs) [2]	Approximate Distance from Pumping Well (miles) [3]	Predicted Drawdown (feet) [4]	Saturated Thickness (feet)	Predicted Saturated Thickness after Drawdown (feet) [5]
15	Domestic	82	1.2	2.8	42.0	39.3
16	Domestic	128	1.2	2.8	88.0	85.3
17	Domestic	92	1.3	2.5	52.0	49.5
18	Domestic	79	1	3.2	39.0	35.8
19	Domestic	76	0.8	3.7	36.0	32.3
21	Unknown	157	1.1	2.9	117.0	114.1
22	Unknown	107	0.9	3.4	67.0	63.6
23	Domestic	150	1.2	2.8	110.0	107.3
24	Domestic	176	1.1	2.9	136.0	133.1
25	Domestic	157	1.3	2.5	117.0	114.5
26	Domestic	110	1.1	2.9	70.0	67.1
27	Domestic	146	1.5	2.1	106.0	103.9
28	Domestic	100	1.2	2.8	60.0	57.3
29	Domestic	114	0.8	3.7	74.0	70.3

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Well Reference Number [1]	Well Type	Depth (feet bgs) [2]	Approximate Distance from Pumping Well (miles) [3]	Predicted Drawdown (feet) [4]	Saturated Thickness (feet)	Predicted Saturated Thickness after Drawdown (feet) [5]
30	Domestic	100	1.1	2.9	60.0	57.1
31	Domestic	128	1.2	2.8	88.0	85.3
32	Domestic	118	1.1	2.9	78.0	75.1
33	Domestic	80	0.9	3.4	40.0	36.6
34	Domestic	100	1.2	2.8	60.0	57.3
35	Domestic	140	1.7	1.8	100.0	98.2
36	Domestic	143	1.6	1.9	103.0	101.1
37	Domestic	80	1.6	1.9	40.0	38.1
38	Unknown	100	1.6	1.9	60.0	58.1
39	Unknown	114	1.6	1.9	74.0	72.1
40	Domestic	108	1.7	1.8	68.0	66.2
41	Domestic	120	1.6	1.9	80.0	78.1
42	Domestic/Industrial	149	1.4	2.4	109.0	106.6
43	Domestic	166	1.4	2.4	126.0	123.6

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Well Reference Number [1]	Well Type	Depth (feet bgs) [2]	Approximate Distance from Pumping Well (miles) [3]	Predicted Drawdown (feet) [4]	Saturated Thickness (feet)	Predicted Saturated Thickness after Drawdown (feet) [5]
44	Domestic	144	1.6	1.9	104.0	102.1
45	Domestic	59	1.1	2.9	19.0	16.1
46	Domestic	162	1.5	2.1	122.0	119.9
47	Domestic	178	1.5	2.1	138.0	135.9
48	Domestic	47	1.6	1.9	7.0	5.1
49	Domestic	140	1.6	1.9	100.0	98.1
50	Domestic	140	1.7	1.8	100.0	98.2
51	Domestic	165	1.6	1.9	125.0	123.1
52	Domestic	127	1.4	2.4	87.0	84.6
53	Domestic	90	1.6	1.9	50.0	48.1
54	Domestic	92	1.6	1.9	52.0	50.1
55	Domestic	60	1.5	2.1	20.0	17.9
56	Domestic	103	0.7	4.2	63.0	58.8
57	Domestic	125	0.9	3.4	85.0	81.6

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58	Domestic	120	0.6	4.6	80.0	75.4
59	Domestic	112	1	3.2	72.0	68.8
60	Domestic	98	1	3.2	58.0	54.8
61	Domestic	123	1	3.2	83.0	79.8
62	Domestic	92	1	3.2	52.0	48.8
63	Domestic	150	1.1	2.9	110.0	107.1
64	Domestic	130	1	3.2	90.0	86.8
65	Domestic	140	0.9	3.4	100.0	96.6
66	Domestic	120	1.3	2.5	80.0	77.5
67	Domestic	125	0.9	3.4	85.0	81.6
68	Domestic	84	1.4	2.4	44.0	41.6
69	Domestic	84	1.1	2.9	44.0	41.1
70	Domestic	171	1.4	2.4	131.0	128.6
71	Domestic	95	1.4	2.4	55.0	52.6

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Well Reference Number [1]	Well Type	Depth (feet bgs) [2]	Approximate Distance from Pumping Well (miles) [3]	Predicted Drawdown (feet) [4]	Saturated Thickness (feet)	Predicted Saturated Thickness after Drawdown (feet) [5]
72	Domestic	87	1.2	2.8	47.0	44.3
73	Domestic	64	0.5	5.1	24.0	18.9
74	Domestic	95	0.4	5.7	55.0	49.3
75	Domestic	80	1.2	2.8	40.0	37.3
76	Domestic	152	1.4	2.4	112.0	109.6
77	Domestic	71	0.4	5.7	31.0	25.3
78	Domestic	121	1.3	2.5	81.0	78.5
79	Domestic	160	0.9	3.4	120.0	116.6
80	Domestic	111	0.8	3.7	71.0	67.3
81	Unknown	90	0.9	3.4	50.0	46.6
82	Municipal	80	1	3.2	40.0	36.8
83	Domestic	107	1.1	2.9	67.0	64.1
84	Domestic	100	1.3	2.5	60.0	57.5
85	Domestic	43	1.2	2.8	3.0	0.3

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Well Reference Number [1]	Well Type	Depth (feet bgs) [2]	Approximate Distance from Pumping Well (miles) [3]	Predicted Drawdown (feet) [4]	Saturated Thickness (feet)	Predicted Saturated Thickness after Drawdown (feet) [5]
86	Domestic	78	0.8	3.7	38.0	34.3
87	Domestic	160	0.6	4.6	120.0	115.4
88	Unknown	183	0.7	3.5	143.0	139.5
89	Domestic/Stock	183	0.8	3.7	143.0	139.3
90	Domestic	200	0.6	4.6	160.0	155.4
91	Unknown	79	1.3	2.5	39.0	36.5
92	Unknown	181	1.3	2.5	141.0	138.5
93	Domestic	190	0.8	3.7	150.0	146.3
94	Domestic	200	0.8	3.7	160.0	156.3
95	Domestic	200	0.9	3.4	160.0	156.6
96	Domestic	191	0.9	3.4	151.0	147.6
97	Domestic	192	1.3	2.5	152.0	149.5
98	Unknown	180	1.3	2.5	140.0	137.5
99	Unknown	183	1.1	2.9	143.0	140.1

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Well Reference Number [1]	Well Type	Depth (feet bgs) [2]	Approximate Distance from Pumping Well (miles) [3]	Predicted Drawdown (feet) [4]	Saturated Thickness (feet)	Predicted Saturated Thickness after Drawdown (feet) [5]
100	Unknown	184	0.9	3.4	144.0	140.6
101	Domestic	180	1.1	2.9	140.0	137.1
102	Domestic	197	1	3.2	157.0	153.8
103	Domestic	197	1	3.2	157.0	153.8
104	Domestic	196	1.1	2.9	156.0	153.1
105	Domestic	195	1	3.2	155.0	151.8
106	Domestic	189	1	3.2	149.0	145.8
107	Domestic	193	1.1	2.9	153.0	150.1
108	Unknown	166	0.8	3.7	126.0	122.3
109	Domestic	200	1.5	2.1	160.0	157.9
110	Domesiic	198	1.1	2.9	158.0	155.1
111	Domestic	197	1	3.2	157.0	153.8
112	Domestic	200	0.8	3.7	160.0	156.3
113	Domestic	79	0.6	4.6	39.0	34.4

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Table 3 - Predicted Interference Drawdown and Saturated Thickness for Shallow Wells Near the Site under the Reduced- Intensitity Alternatives

CLIENT: AES
PROJECT No.: N0410C
LOCATION: Graton Rancheria Hotel and Casino, Rohnert Park, CA
PROJECT DESCRIPTION: Groundwater Study

DATE: 3-Dec-06
BY: Alan Blakemore
REVISION: Mike Tietze

Well Reference Number [1]	Well Type	Depth (feet bgs) [2]	Approximate Distance from Pumping Well (miles) [3]	Predicted Drawdown (feet) [4]	Saturated Thickness (feet)	Predicted Saturated Thickness after Drawdown (feet) [5]
114	Domestic	79	0.8	3.7	39.0	35.3
115	Domestic	83	0.6	4.6	43.0	38.4
116	Domestic	88	0.9	3.4	48.0	44.6
117	Domestic	79	0.6	4.6	39.0	34.4
118	Domestic	84	0.5	5.1	44.0	38.9
119	Domestic	79	0.9	3.4	39.0	35.6
120	Unknown	200	0.6	4.6	160.0	155.4
121	Domestic	180	0.9	3.4	140.0	136.6
122	Domestic	100	1.3	2.5	60.0	57.5
123	Domestic	156	1	3.2	116.0	112.8
124	Domestic	72	1.4	2.4	32.0	29.6
125	Domestic	104	1.2	2.8	64.0	61.3
126	Domestic	105	1.1	2.9	65.0	62.1
127	Domestic	150	1	3.2	110.0	106.8

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Well Reference Number [1]	Well Type	Depth (feet bgs) [2]	Approximate Distance from Pumping Well (miles) [3]	Predicted Drawdown (feet) [4]	Saturated Thickness (feet)	Predicted Saturated Thickness after Drawdown (feet) [5]
128	Domestic	207	1.2	2.8	167.0	164.3
129	Domestic	116	0.9	3.4	76.0	72.6
130	Domestic	115	0.8	3.7	75.0	71.3
131	Domestic	137	0.7	4.2	97.0	92.8
132	Domestic	88	1.2	2.8	48.0	45.3
133	Domestic	104	1.1	2.9	64.0	61.1
134	Irrigation	160	1.4	2.4	120.0	117.6
135	Domestic	160	1.4	2.4	120.0	117.6
136	Domestic	85	1.1	2.9	45.0	42.1
137	Domestic	135	1.4	2.4	95.0	92.6
138	Domestic	170	1.2	2.8	130.0	127.3
139	Domestic	175	1	3.2	135.0	131.8
140	Domestic	76	1.2	2.8	36.0	33.3
141	Domestic	141	1.3	2.5	101.0	98.5

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Well Reference Number [1]	Well Type	Depth (feet bgs) [2]	Approximate Distance from Pumping Well (miles) [3]	Predicted Drawdown (feet) [4]	Saturated Thickness (feet)	Predicted Saturated Thickness after Drawdown (feet) [5]
142	Domestic	116	1.1	2.9	76.0	73.1
143	Domestic	100	1	3.2	60.0	56.8
144	Domestic	100	1	3.2	60.0	56.8
145	Domestic	170	1.2	2.8	130.0	127.3
146	Domestic	59	1.2	2.8	19.0	16.3
147	Domestic	80	1.1	2.9	40.0	37.1
148	Domestic	147	1	3.2	107.0	103.8
149	Domestic	107	0.7	3.5	67.0	63.5
150	Domestic	140	0.9	3.4	100.0	96.6
151	Domestic	159	1	3.2	119.0	115.8
152	Domestic	136	1.4	2.4	96.0	93.6
153	Domestic	155	1	3.2	115.0	111.8
154	Domestic	136	1.4	2.4	96.0	93.6
155	Domestic	75	1.7	1.8	35.0	33.2

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Well Reference Number [1]	Well Type	Depth (feet bgs) [2]	Approximate Distance from Pumping Well (miles) [3]	Predicted Drawdown (feet) [4]	Saturated Thickness (feet)	Predicted Saturated Thickness after Drawdown (feet) [5]
156	Domestic	120	1.6	1.9	80.0	78.1
157	Domestic	104	1.6	1.9	64.0	62.1
158	Domestic	79	1.2	2.8	39.0	36.3
159	Domestic	69	1.6	1.9	29.0	27.1
160	Domestic	84	1.2	2.8	44.0	41.3
161	Domestic	106	1.2	2.8	66.0	63.3
162	Domestic	108	1.2	2.8	68.0	65.3
163	Irrigation	90	1.1	2.9	50.0	47.1
164	Domestic	68	1.6	1.9	28.0	26.1
165	Domestic	136	1.4	2.4	96.0	93.6
166	Irrigation	109	1.2	2.8	69.0	66.3
167	Unknown	140	1.5	2.1	100.0	97.9
168	Irrigation	176	0.9	3.4	136.0	132.6
169	Unknown	110	1	3.2	70.0	66.8

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Well Reference Number [1]	Well Type	Depth (feet bgs) [2]	Approximate Distance from Pumping Well (miles) [3]	Predicted Drawdown (feet) [4]	Saturated Thickness (feet)	Predicted Saturated Thickness after Drawdown (feet) [5]
170	Unknown	76	1.5	2.1	36.0	33.9
171	Domestic	140	1.5	2.1	100.0	97.9
172	Irrigation	141	1.4	2.4	101.0	98.6
173	Domestic	136	1.2	2.8	96.0	93.3
174	Domestic	180	1.3	2.5	140.0	137.5
175	Domestic	149	1.5	2.1	109.0	106.9
176	Domestic	125	1	3.2	85.0	81.8
177	Domestic/ Irrigation	150	1.6	1.9	110.0	108.1
178	Domestic	177	1.1	2.9	137.0	134.1
179	Domestic	150	1.3	2.5	110.0	107.5
180	Domestic	157	1.6	1.9	117.0	115.1
181	Domestic	155	1.5	2.1	115.0	112.9
182	Domestic	173	1.5	2.1	133.0	130.9

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Well Reference Number [1]	Well Type	Depth (feet bgs) [2]	Approximate Distance from Pumping Well (miles) [3]	Predicted Drawdown (feet) [4]	Saturated Thickness (feet)	Predicted Saturated Thickness after Drawdown (feet) [5]
183	Domestic	150	1.4	2.4	110.0	107.6
184	Domestic	155	1.1	2.9	115.0	112.1
185	Unknown	71	1.5	2.1	31.0	28.9
186	Domestic	185	1.1	2.9	145.0	142.1
187	Domestic	116	1.4	2.4	76.0	73.6
188	Domestic	120	1.6	1.9	80.0	78.1
189	Domestic	155	1.5	2.1	115.0	112.9
190	Domestic	152	1.3	2.5	112.0	109.5
191	Domestic	60	1.3	2.5	20.0	17.5
192	Domestic	159	1.1	2.9	119.0	116.1
193	Domestic	178	1.3	2.5	138.0	135.5
20 [6]	Unknown	61	1.1	2.9	21.0	18.1

Notes:
1. Approximate locations of wells are plotted on Figure 10.

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PROJECT DESCRIPTION: Groundwater Study

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Well Reference Number [1]	Well Type	Depth (feet bgs) [2]	Approximate Distance from Pumping Well (miles) [3]	Predicted Drawdown (feet) [4]	Saturated Thickness (feet)	Predicted Saturated Thickness after Drawdown (feet) [5]

2. bgs = below ground surface
3. Approximate distance from pumping well rounded to the nearest 0.1 mile.
4. Predicted drawdown interpolated from Figure 19. Drawdown in shallow wells is expected to be less than this amount.
5. Saturated thickness is calculated assuming a depth to water of 40 feet and subtracting predicted drawdown.
6. Highlighed wells are at greatest risk of going dry or experiencing significant reduction in capacity.

APPENDIX A

PUMP POWER ANALYSIS

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Table 4 - Predicted Interference Drawdown and Saturated Thickness for Deep Wells Near the Site under the Reduced-Intensity Alternatives

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PROJECT DESCRIPTION: Groundwater Study

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Well Reference Number [1]	Well Type	Depth (feet bgs) [2]	Approximate Distance from Pumping Well (miles) [3]	Saturated Thickness (feet)	Predicted Drawdown (feet) [4]	Predicted Saturated Thickness after Drawdown (feet) [5]	Remarks
1	Municipal	475	1.1	345	2.9	342	Rohnert Park No. 07
2	Unknown	375	0.7	245	3.5	242	
3	Unknown	438	0.7	308	3.5	305	
4	Municipal	1500	1.1	1,370	2.9	1,367	Rohnert Park No. 16
5	Municipal	590	0.4	460	5.7	454	Rohnert Park No. 23
6	Unknown	314	0.9	184	3.1	181	
7	Municipal	805	1.1	675	2.9	672	Rohnert Park No. 03
8	Unknown	201	0.9	71	3.4	68	
9	Unknown	260	0.6	130	4.6	125	
10	Municipal	592	0.06	462	11.1	451	Rohnert Park No. 24

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PROJECT DESCRIPTION: Groundwater Study

DATE: 3-Dec-06
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Well Reference Number [1]	Well Type	Depth (feet bgs) [2]	Approximate Distance from Pumping Well (miles) [3]	Saturated Thickness (feet)	Predicted Drawdown (feet) [4]	Predicted Saturated Thickness after Drawdown (feet) [5]	Remarks
11	Domestic	407	0.5	277	5.1	272	
12	Domestic	201	1.3	71	2.5	69	
13	Domestic	208	1.3	78	2.5	76	
14	Domestic	253	0.8	123	3.7	119	
15	Domestic	230	1	100	3.2	97	
16	Domestic	219	1	89	3.2	86	
17	Domestic	269	1.2	139	2.8	136	
18	Domestic	224	1.3	94	2.5	92	
19	Domestic	215	0.9	85	3.4	82	
20	Domestic	504	1.3	374	2.5	372	

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Well Reference Number [1]	Well Type	Depth (feet bgs) [2]	Approximate Distance from Pumping Well (miles) [3]	Saturated Thickness (feet)	Predicted Drawdown (feet) [4]	Predicted Saturated Thickness after Drawdown (feet) [5]	Remarks
21	Domestic	270	1.2	140	2.8	137	
22	Domestic	280	1.2	150	2.8	147	
23	Domestic	231	1.1	101	2.9	98	
24	Domestic	250	1	120	3.2	117	
25	Unknown	258	1.1	128	2.9	125	
26	Unknown	216	1.2	86	2.8	83	
27	Domestic	350	1.2	220	2.8	217	
28	Domestic	270	1.1	140	2.9	137	
29	Domestic	220	1.2	90	2.8	87	
30	Domestic	295	1.3	165	2.5	163	

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Well Reference Number [1]	Well Type	Depth (feet bgs) [2]	Approximate Distance from Pumping Well (miles) [3]	Saturated Thickness (feet)	Predicted Drawdown (feet) [4]	Predicted Saturated Thickness after Drawdown (feet) [5]	Remarks
31	Domestic	252	1.1	122	2.9	119	
32	Domestic	204	1.1	74	2.9	71	
33	Domestic	240	1	110	3.2	107	
34	Domestic	250	0.8	120	3.7	116	
35	Irrigation	725	0.8	595	3.7	591	
36	Domestic	204	1.2	74	2.8	71	
37	Irrigation	345	1	215	3.2	212	
38	Irrigation	1028	0.5	898	5.1	893	
39	Domestic/Irrigation	218	1.5	88	2.1	86	
40	Domestic	214	0.7	84	4.2	80	

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Well Reference Number [1]	Well Type	Depth (feet bgs) [2]	Approximate Distance from Pumping Well (miles) [3]	Saturated Thickness (feet)	Predicted Drawdown (feet) [4]	Predicted Saturated Thickness after Drawdown (feet) [5]	Remarks
41	Irrigation	517	1.3	387	2.5	385	
42	Municipal	1501	1.2	1,371	2.8	1,368	Rohnert Park No. 15
43	Industrial	260	1.2	130	2.8	127	
44	Domestic	216	0.9	86	3.4	83	
45	Domestic	210	0.8	80	3.7	76	
46	Irrigation	914	0.7	784	4.2	780	
47	Irrigation	415	1.2	285	2.8	282	
48	Unknown	Unknown	1.2	Unknown	2.8	Unknown	
49	Unknown	560	0.9	430	3.4	427	
50	Irrigation	204	1.5	74	2.1	72	

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Well Reference Number [1]	Well Type	Depth (feet bgs) [2]	Approximate Distance from Pumping Well (miles) [3]	Saturated Thickness (feet)	Predicted Drawdown (feet) [4]	Predicted Saturated Thickness after Drawdown (feet) [5]	Remarks
51	Domestic	220	1.4	90	2.4	88	
52	Domestic	389	1.6	259	1.9	257	
53	Irrigation	360	1.5	230	2.1	228	
54	Domestic	207	1.4	77	2.4	75	
55	Domestic	320	0.9	190	3.4	187	
56	Domestic	225	1.3	95	2.5	93	
57	Irrigation	411	1.3	281	2.5	279	
58	Domestic	500	1.1	370	2.9	367	
59	Domestic	205	1.2	75	2.8	72	
60	Domestic	260	1.4	130	2.4	128	

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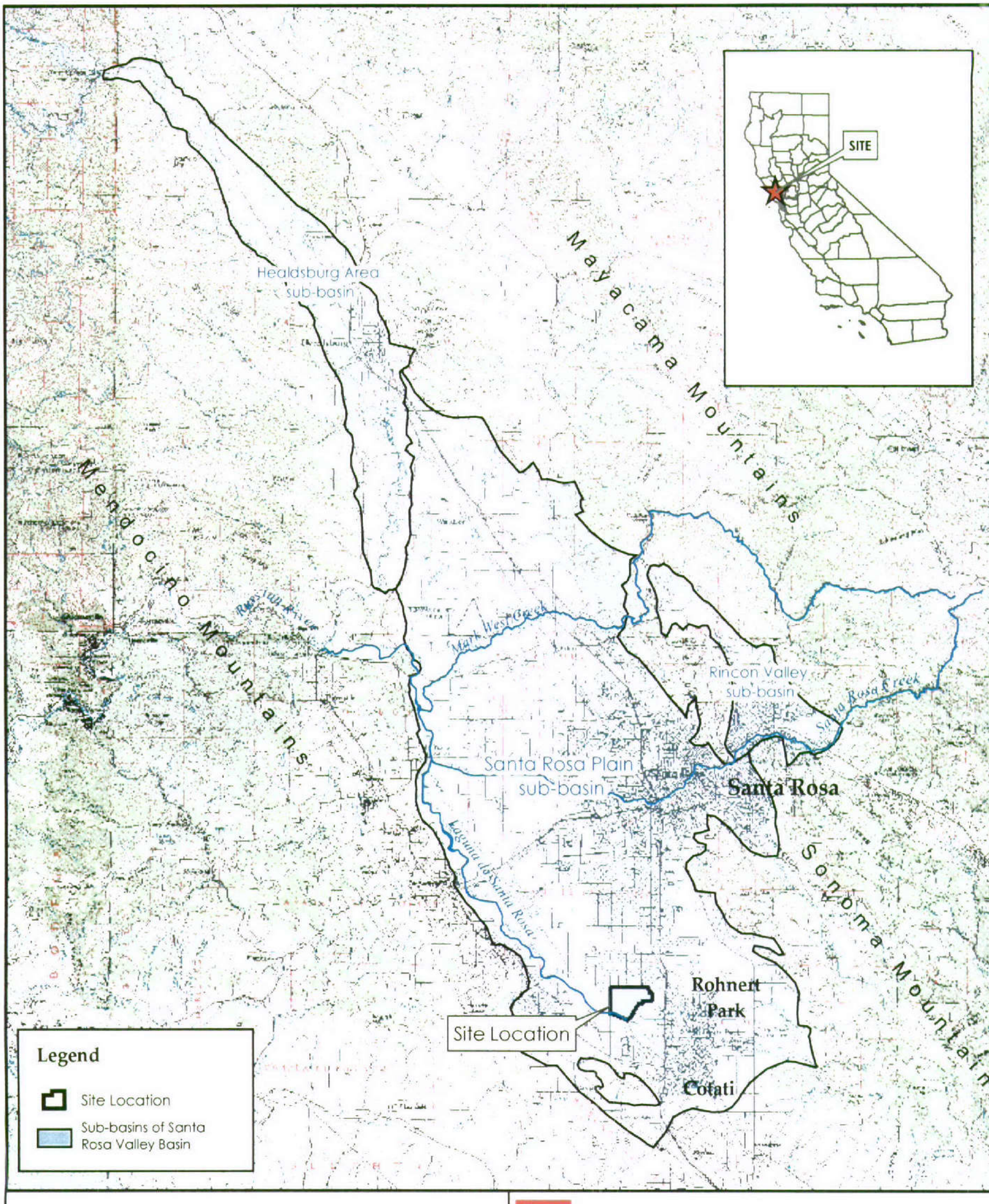
Well Reference Number [1]	Well Type	Depth (feet bgs) [2]	Approximate Distance from Pumping Well (miles) [3]	Saturated Thickness (feet)	Predicted Drawdown (feet) [4]	Predicted Saturated Thickness after Drawdown (feet) [5]	Remarks
61	Irrigation	1204	0.9	1,074	3.4	1,071	
62	Municipal	715	0.9	585	4.4	581	Rohnert Park No. 41

Notes:
1. Approximate locations of wells are plotted on Figure 11.
2. bgs = below ground surface
3. Approximate distance from pumping well rounded to the nearest 0.1 mile.
4. Predicted drawdown at the well interpolated from Figure 19.
5. Remaining saturated thickness is calculated assuming a depth to water of 130 feet and subtracting predicted drawdown



PROPOSED GRATON RANCHERIA

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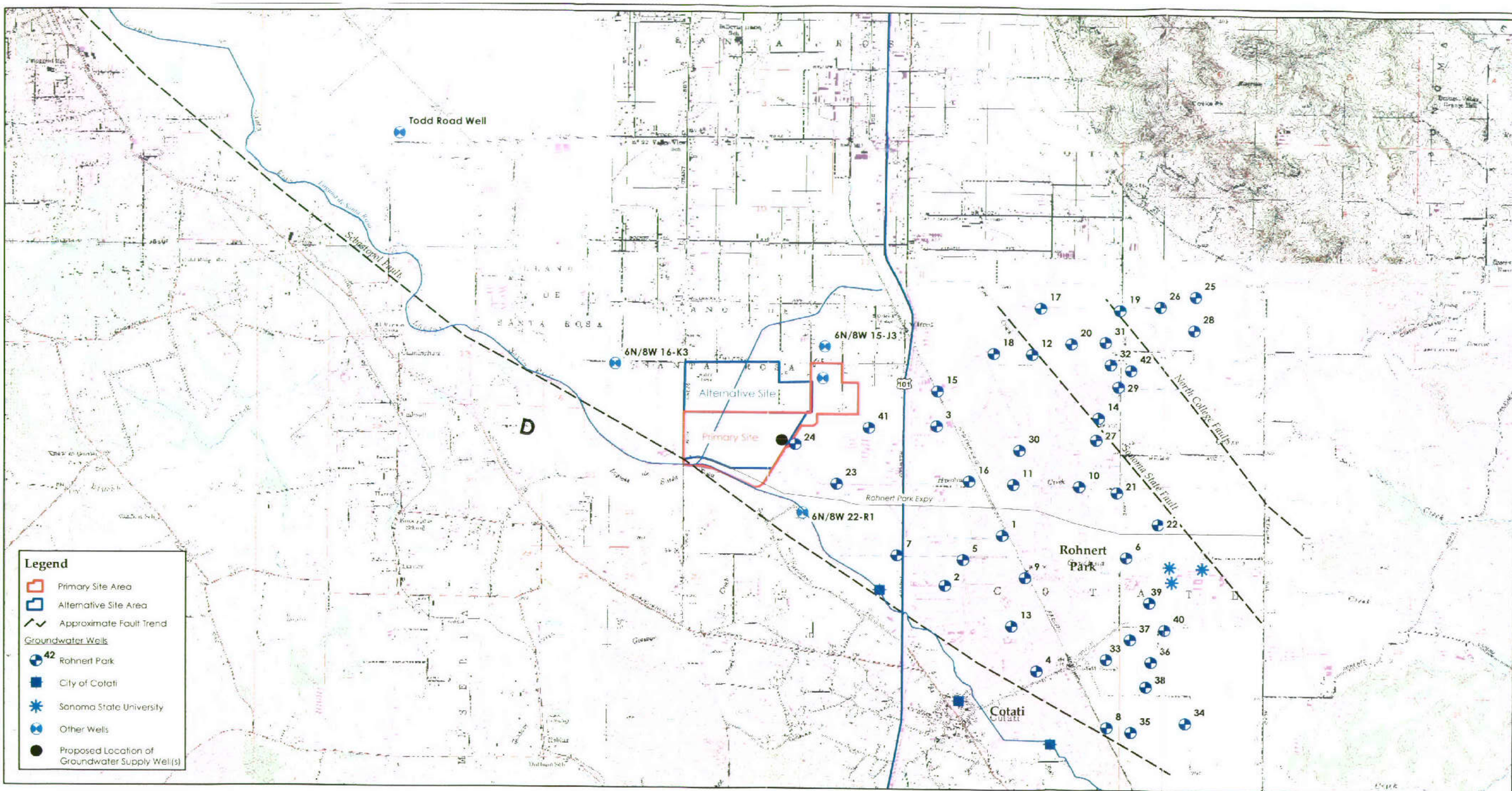
SWL	AB	10/2005
N0410		1

SITE LOCATION PLAN



Notes:
1 - City of Rohnert Park, Cotati, and Sonoma State University well locations from Dyett & Bhatia (2000a).
2 - Other well locations from DWR (2004b).
3 - Approximate locations of faults from DWR (1982).
4 - Basemap data source USGS 7.5 Series: Cotati (1980), Two Rock (1971), Sebastopol (1980), and Santa Rosa (1999) Quadrangles.
5 - All well locations are approximate and were not field verified.

PROPOSED GRATON RANCHERIA

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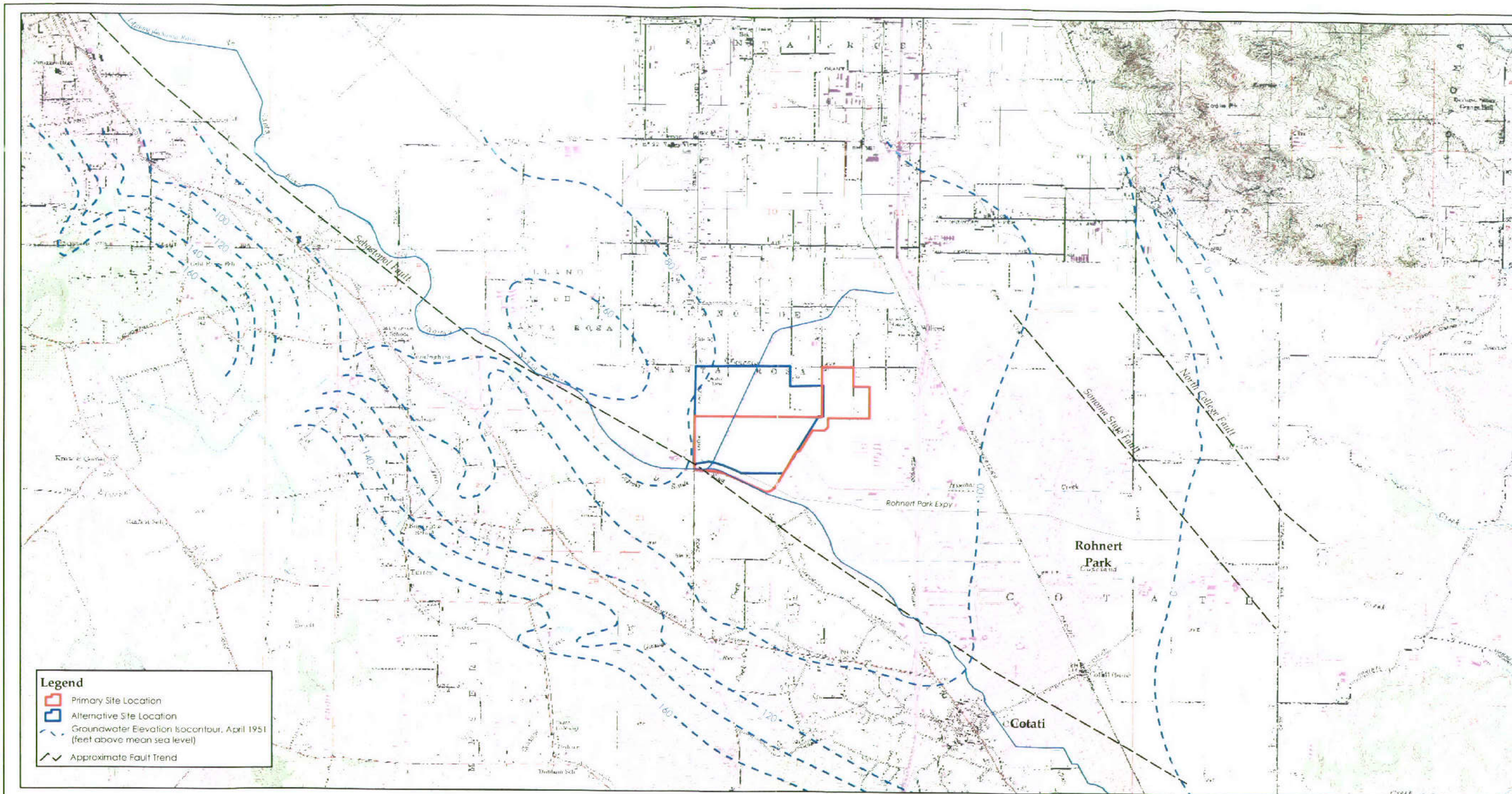
SITE AREA

SWL	AB	09/2004
ND410		2

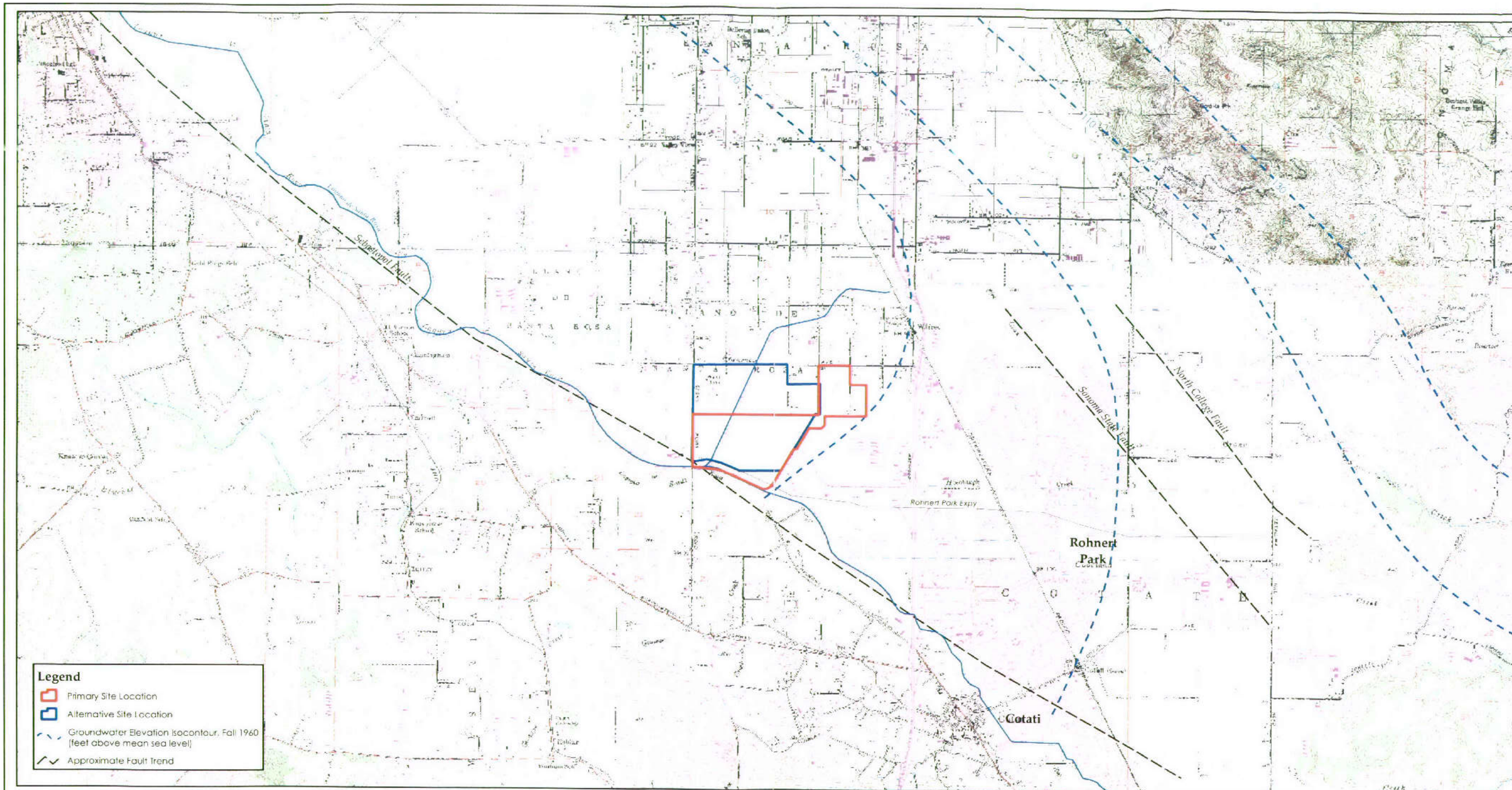


Notes:
1 - Approximate locations of faults from DWR (1982).
2 - Groundwater elevation contours from Cardwell (1958) have been superimposed on the current USGS basemap.
3 - Basemap data source USGS 7.5' Series: Cotati (1980), Two Rock (1971), Sebastopol (1980), and Santa Rosa (1999) Quadrangles.

N W E S

0 3,500 7,000
Feet

PROPOSED GRATON RANCHERIA		WorleyParsons Komex resources & energy	
SPRING 1951 GROUNDWATER ELEVATIONS	SWL	AB	10/2005
	N0410	3	



Notes:
1 - Approximate locations of faults from DWR (1982).
2 - Groundwater elevation contours from DWR (1982) have been superimposed on the current USGS basemap.
3 - Basemap data source USGS 7.5' Series: Cotati (1980), Two Rock (1971), Sebastopol (1980), and Santa Rosa (1999) Quadrangles.

0 3,500 7,000
Feet

PROPOSED GRATON RANCHERIA

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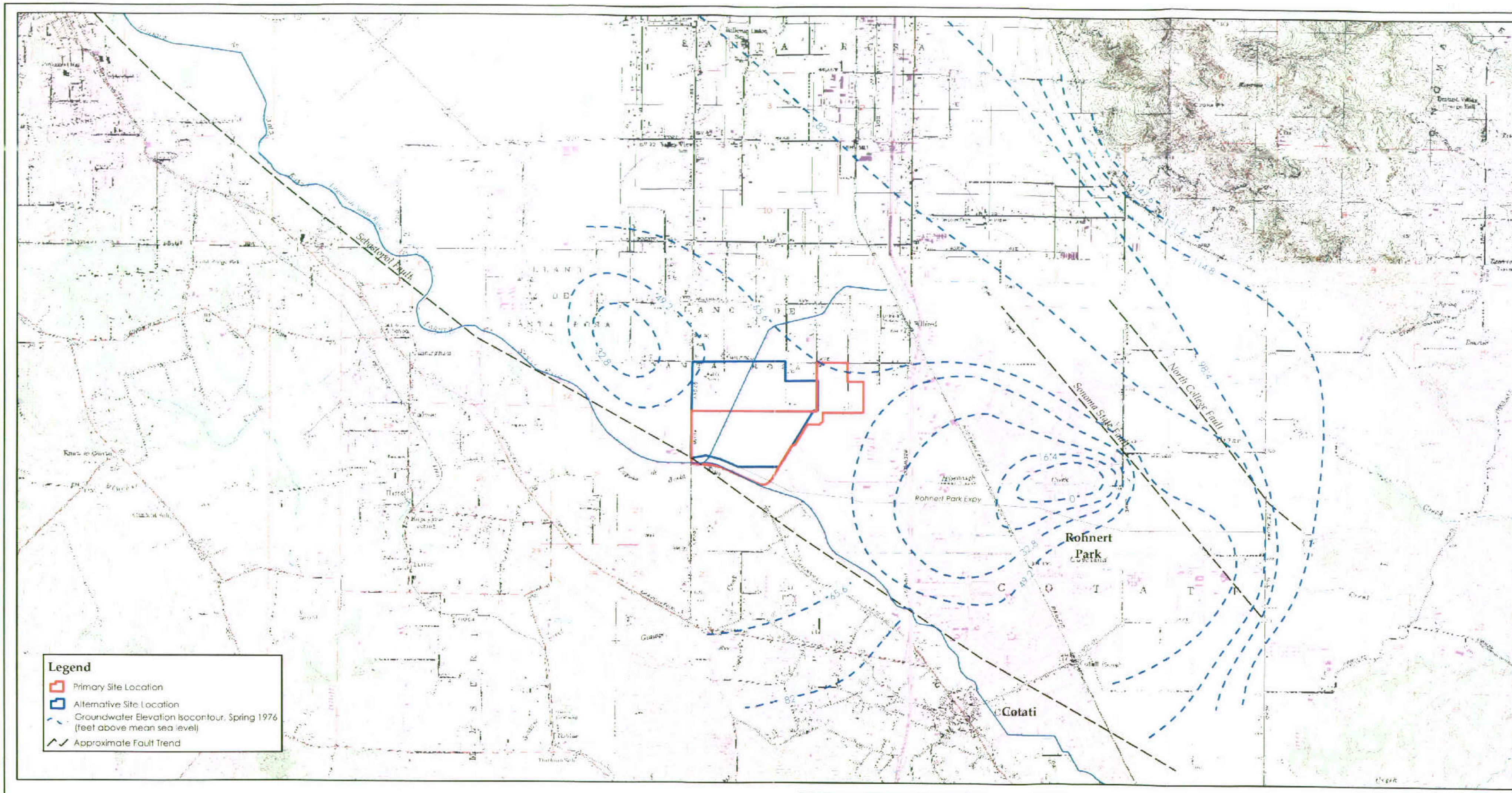
FALL 1960 GROUNDWATER ELEVATIONS

SWL	AB	10/2005
N0410		4



Notes:
1 - Approximate locations of faults from DWR (1982).
2 - Groundwater elevation contours from DWR (1979) have been superimposed on the current USGS basemap. Original contour labels were expressed in meters and have been converted to feet.
3 - Basemap data source USGS 7.5' Series: Cotati (1980), Two Rock (1971), Sebastopol (1980), and Santa Rosa (1999) Quadrangles.

N W E S

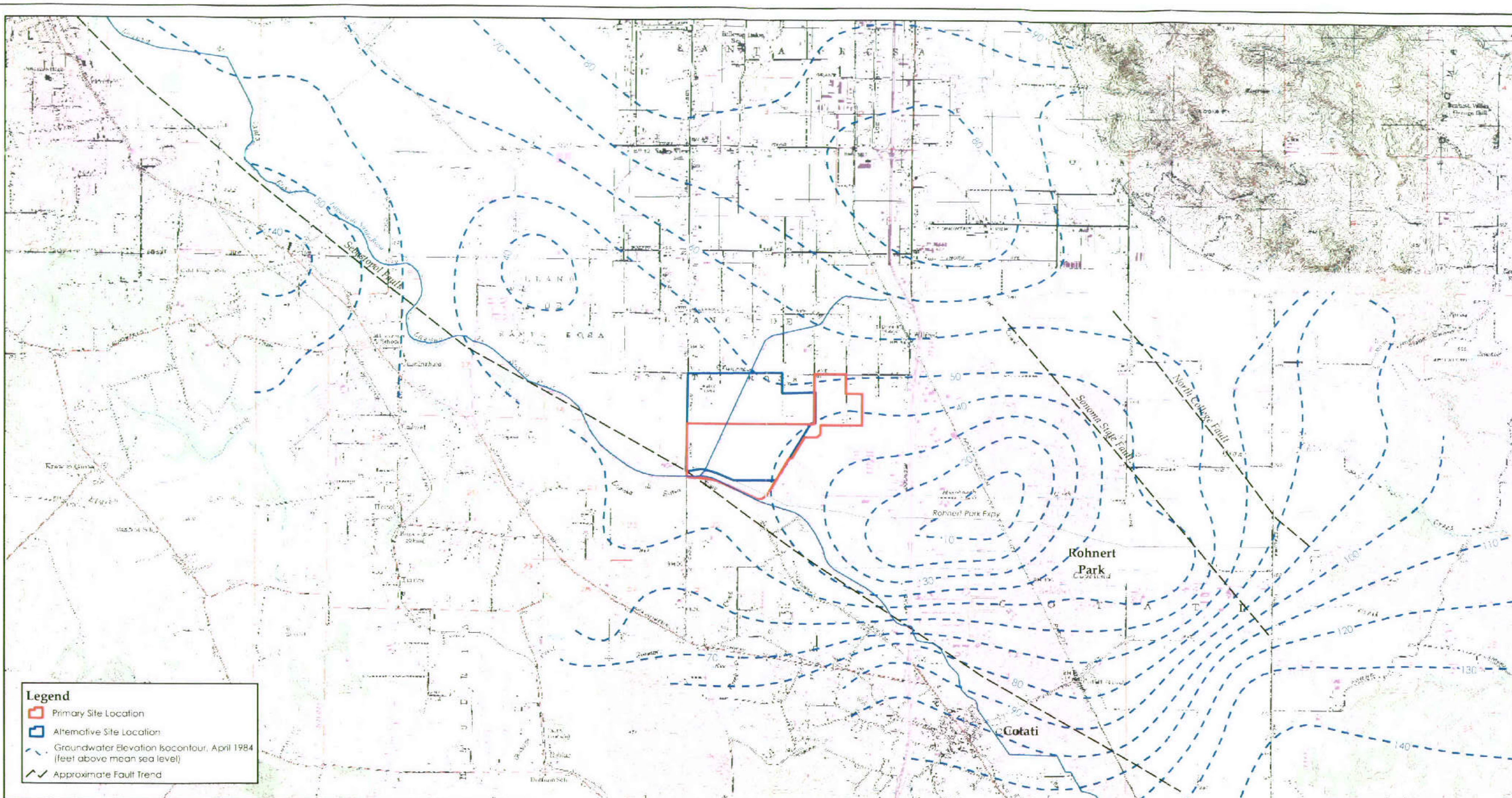
0 3,500 7,000
Feet

PROPOSED GRATON RANCHERIA

WorleyParsons Komex
resources & energy

SPRING 1976 GROUNDWATER ELEVATIONS

SWL	AB	10/2005
N0410	5	

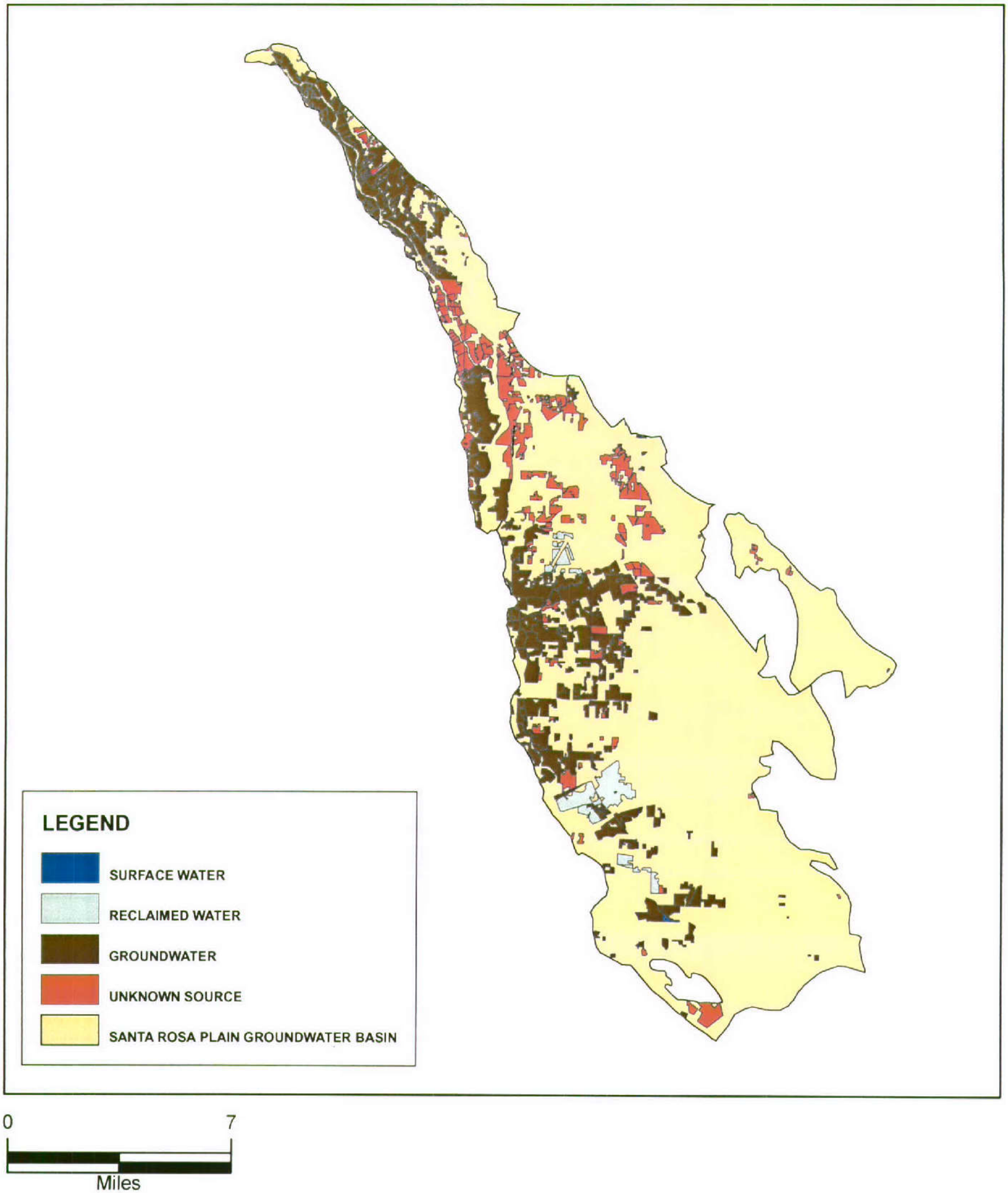


Notes:
1 - Approximate locations of faults from DWR (1982).
2 - Groundwater elevation contours from DWR (1987) have been superimposed on the current USGS basemap.
3 - Basemap data source USGS 7.5' Series: Cotati (1980), Two Rock (1971), Sebastopol (1980), and Santa Rosa (1999) Quadrangles.

N W E S

0 3,500 7,000
Feet

PROPOSED GRATON RANCHERIA	WorleyParsons Komex resources & energy		
SPRING 1984 GROUNDWATER ELEVATIONS	SWL	AB	10/2005
	N0410	6	

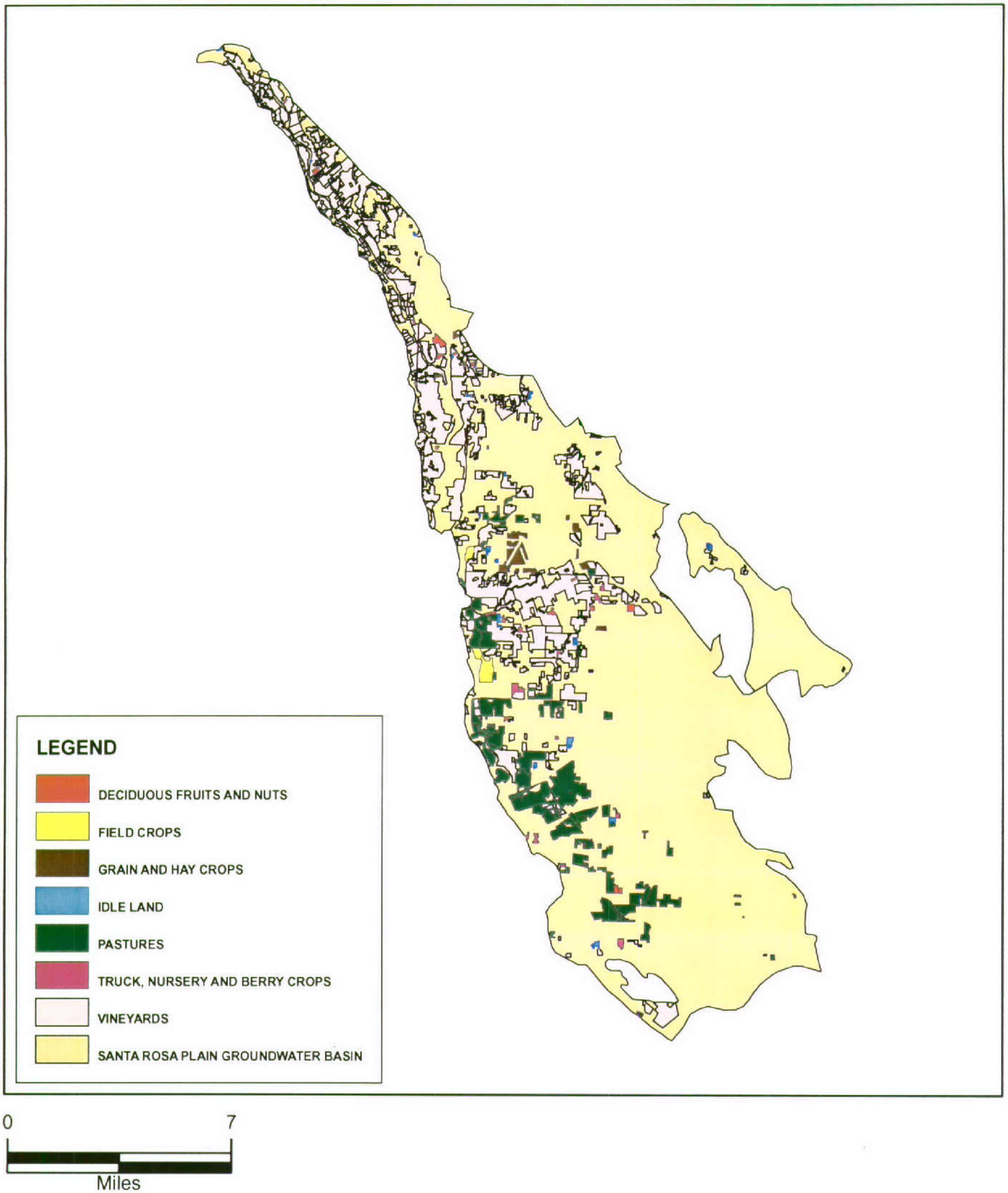


PROPOSED GRATON RANCHERIA CASINO AND HOTEL

WorleyParsons Komex
resources & energy

Agricultural Water Use in Santa Rosa Plain Basin

MB	MT	11/2006
N0410C		7

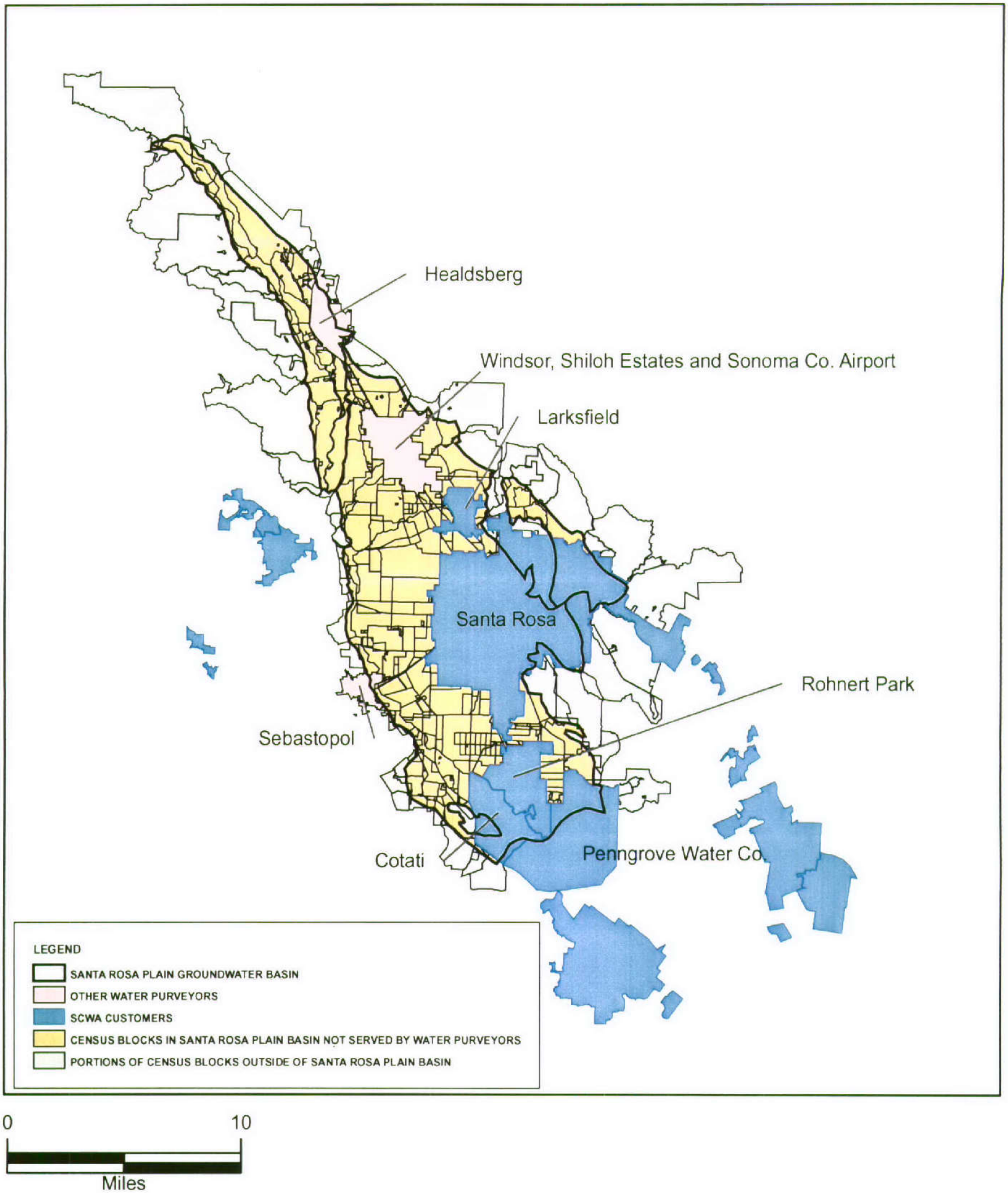


PROPOSED GRATON RANCHERIA CASINO AND HOTEL

WorleyParsons Komex

resources & energy

Irrigated Agricultural Land Use in Santa Rosa Plain Basin	MB	MT	11/2006
	N0410C		8

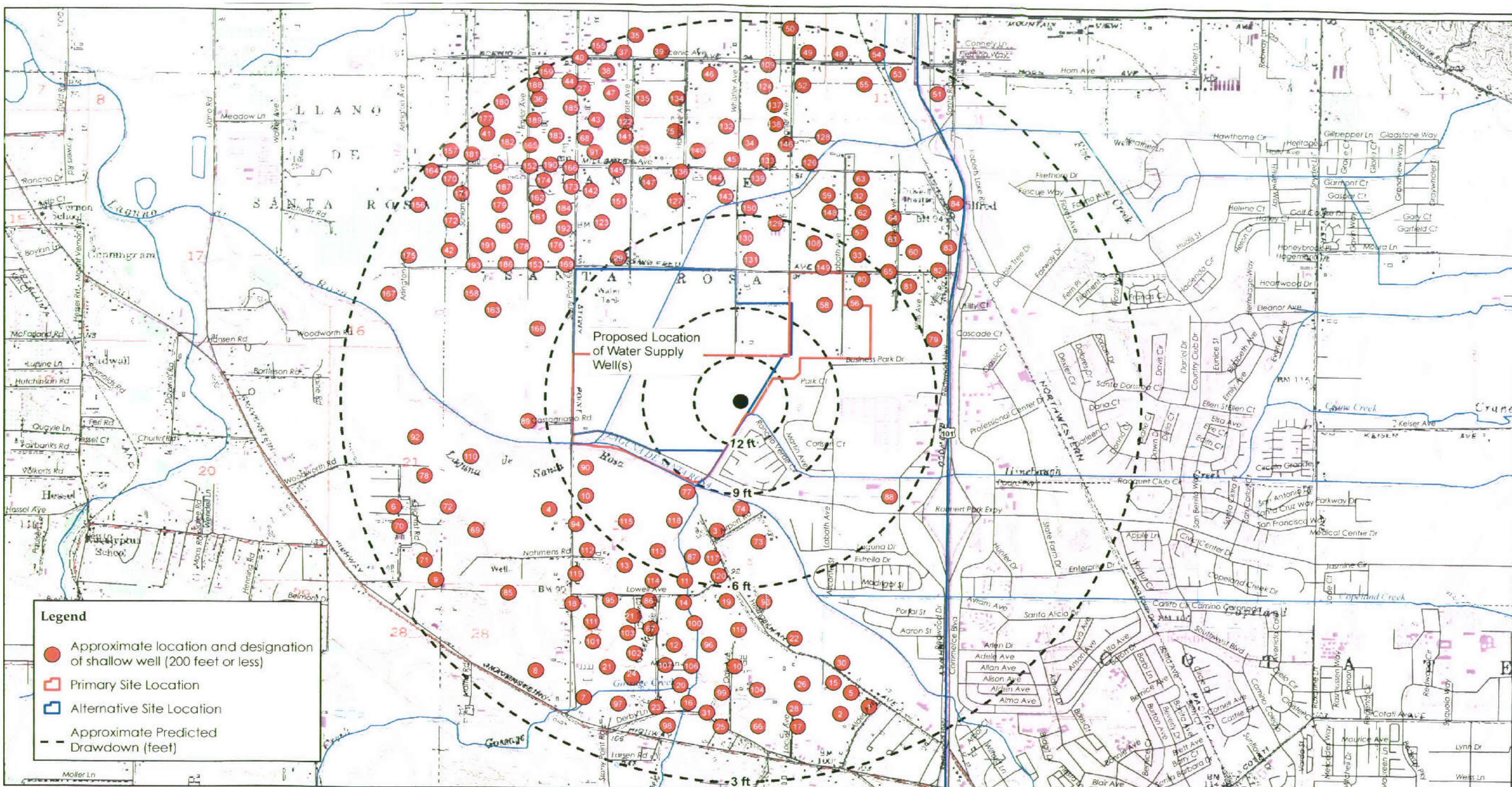


PROPOSED GRATON RANCHERIA CASINO AND HOTEL

WorleyParsons Komex
resources & energy

Domestic Water Sources within Santa Rosa Plain Basin

MB	MT	11/2006
N0410C		9

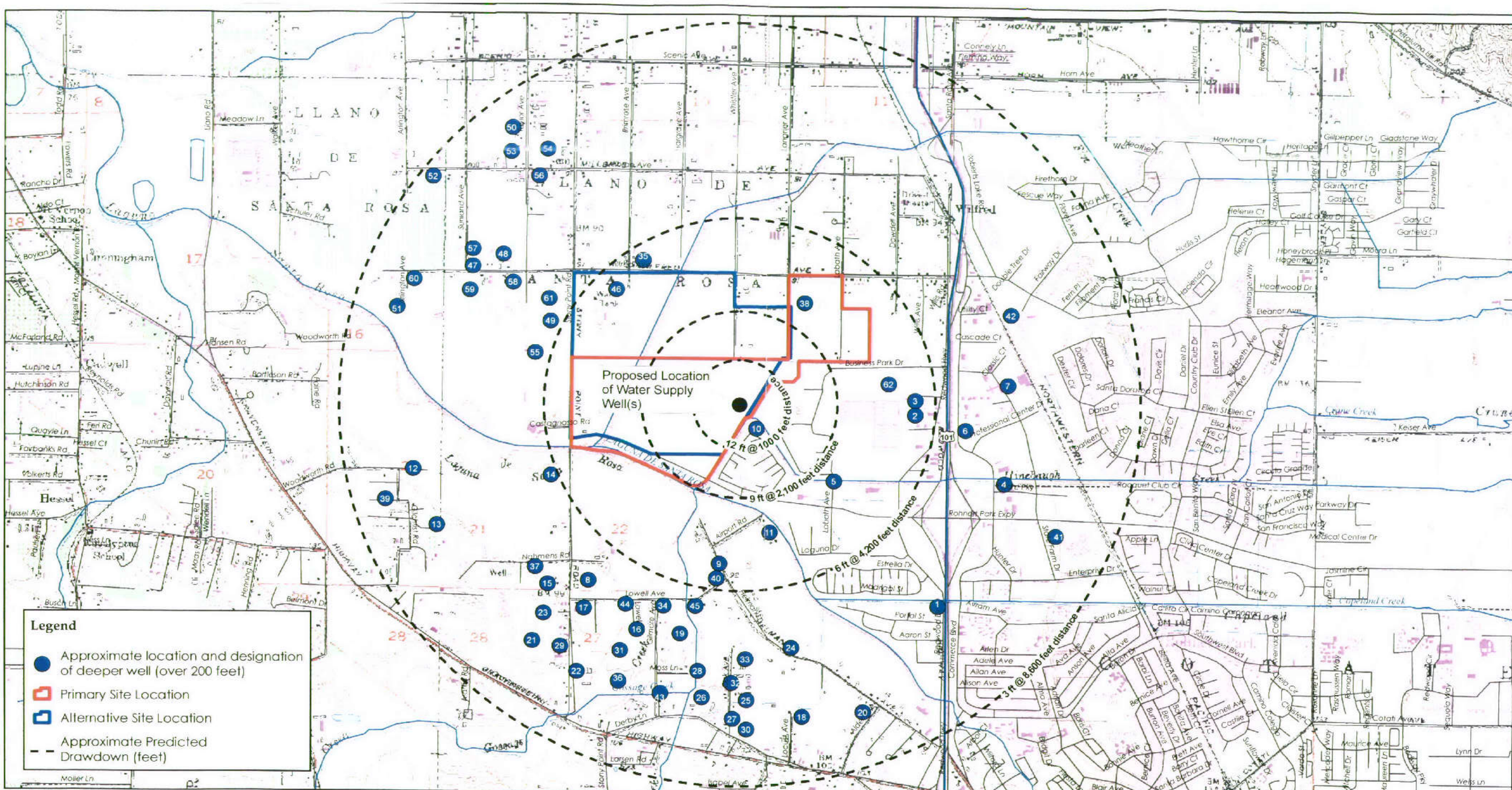


Source: USGS 7.5' Series:
Cotati, Two Rock, Sepastopol, and Santa Rosa Quadrangles.
All well locations are approximate and not field verified.
See Table 1 for well completion data.

Note:
Predicted drawdown contours are approximate and based on the limitations and assumptions described in the text.

0 0.375 0.75 1.5
Miles

PROPOSED GRATON RANCHERIA	WorleyParsons Komex resources & energy		
APPROXIMATE LOCATION OF SHALLOW WELLS AND PREDICTED DRAWDOWN FOR ALTERNATIVES A, B, AND C	SWL	MT	07/2007
	N0410		10



Source: USGS 7.5' Series:
Cotati, Two Rock, Sepastopol, and Santa Rosa Quadrangles.
All well locations are approximate and not field verified.
See Table 2 for well completion data.

Note:
Predicted drawdown contours are approximate and based on the limitations and assumptions described in the text.

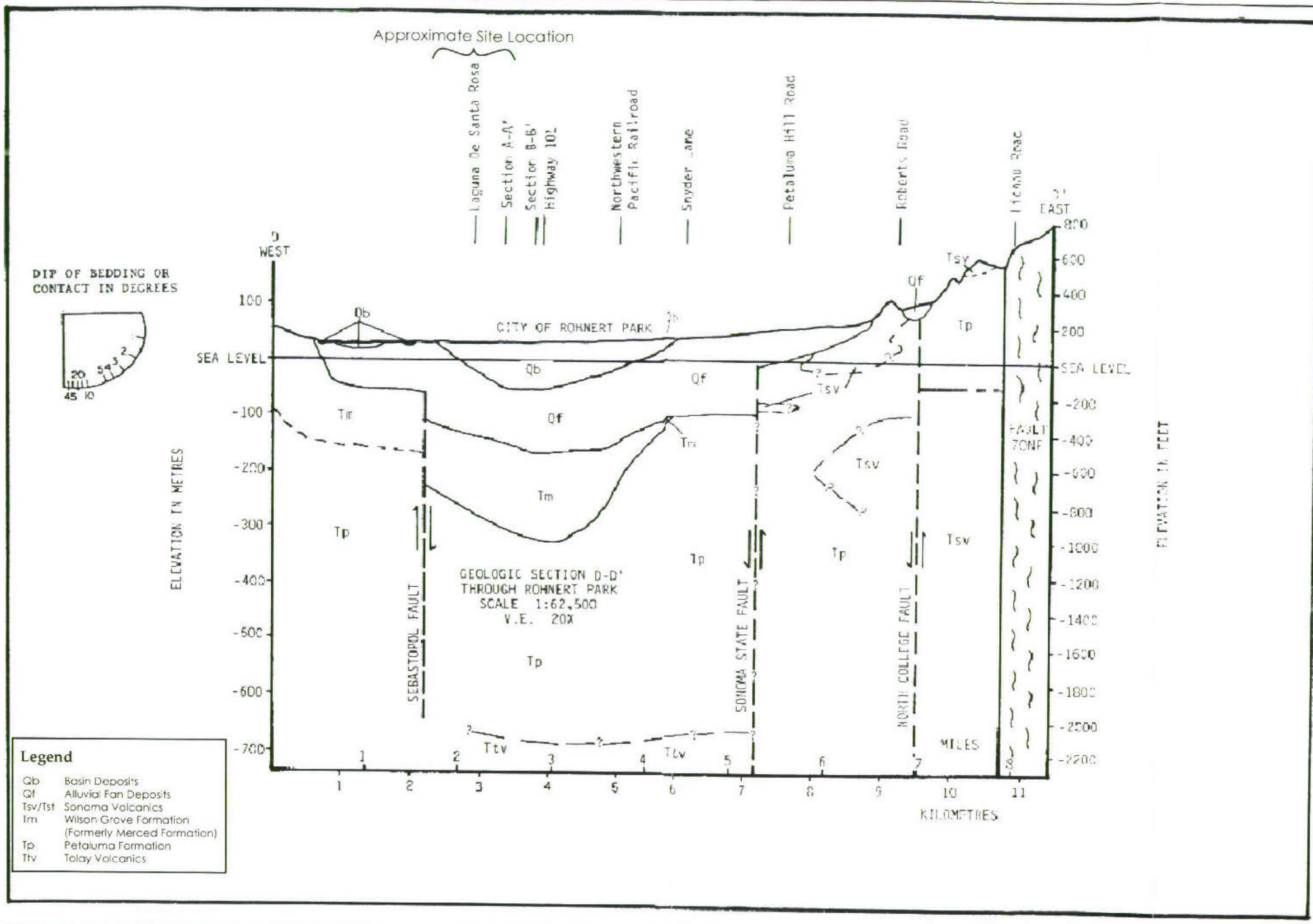
0 0.375 0.75 1.5
Miles

PROPOSED GRATON RANCHERIA

WorleyParsons Komex
resources & energy

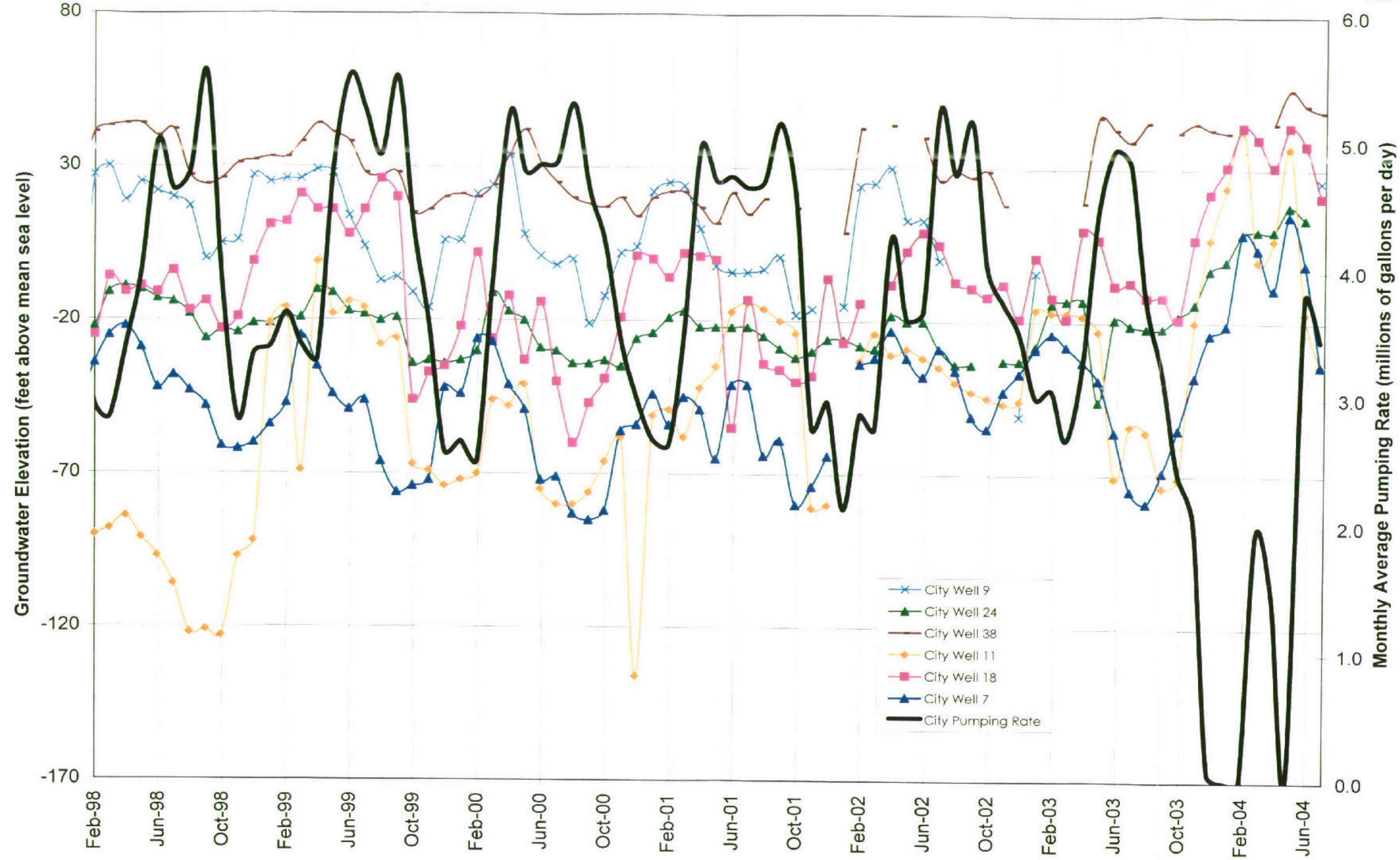
APPROXIMATE LOCATION OF DEEPER WELLS AND PREDICTED DRAWDOWN FOR ALTERNATIVES A, B, AND C

SWL	AB	07/2007
N0410	11	



Cross section from DWR (1982).

PROPOSED GRATON RANCHERIA	WorleyParsons Komex resources & energy		
EAST - WEST GEOLOGICAL CROSS SECTION	SWL	AB	01/2007
	N0466	12	

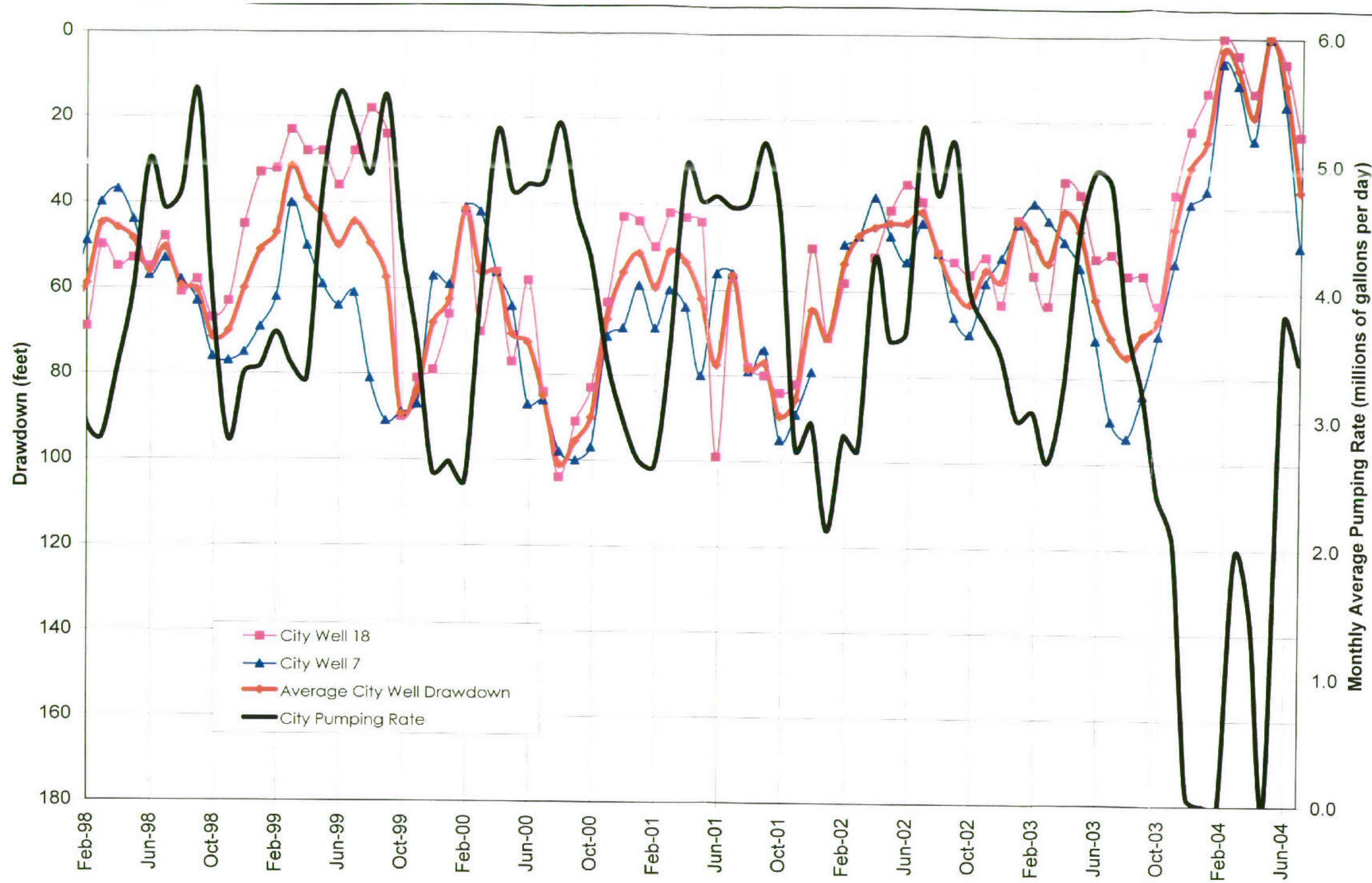


PROPOSED GRATON RANCHERIA

WorleyParsons Komex
resources & energy

HYDROGRAPHS FOR SELECTED CITY OF ROHNERT PARK WELLS VERSUS CITY OF ROHNERT PARK TOTAL PUMPING RATE

SWL	AB	10/2005
N0410	13	

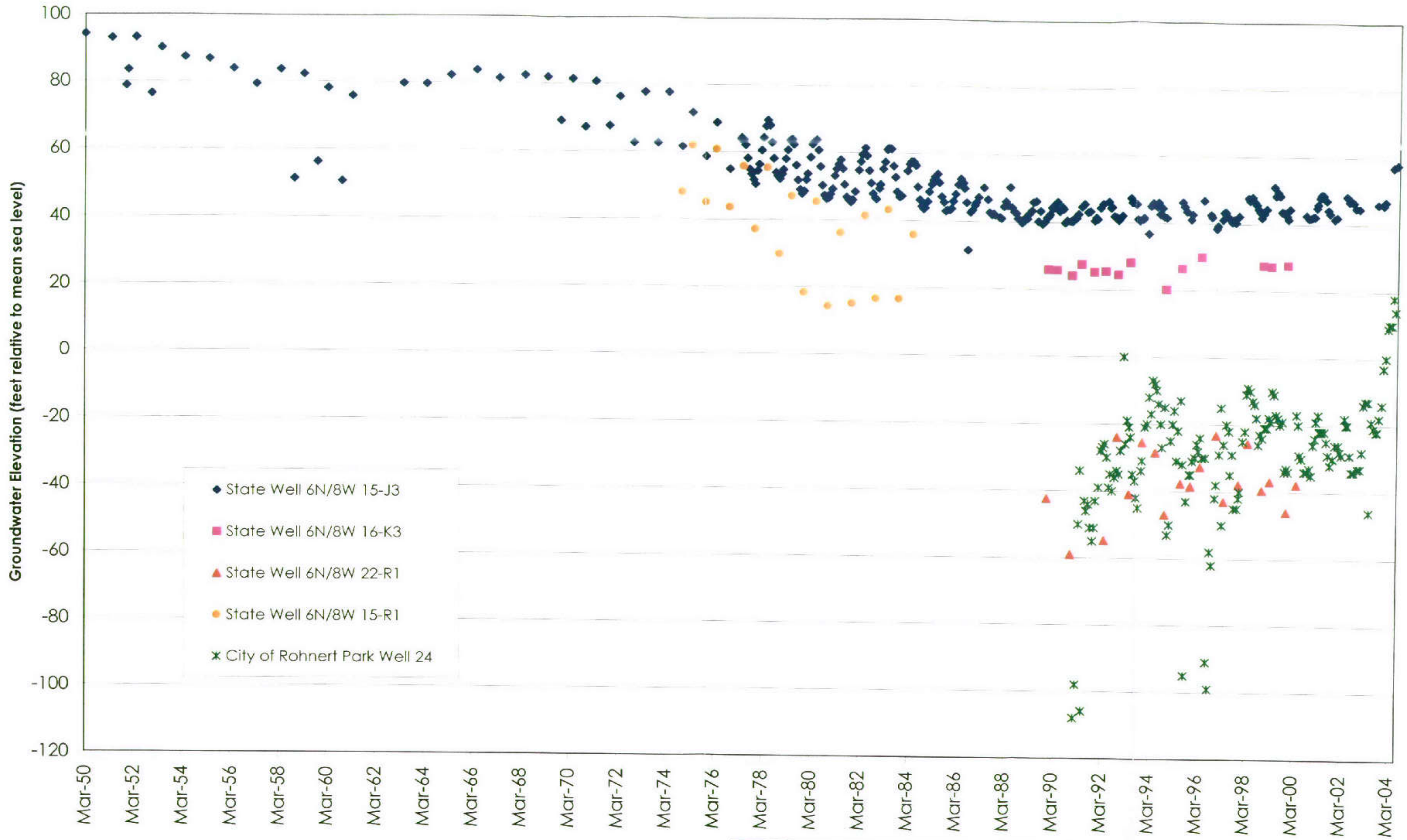


PROPOSED GRATON RANCHERIA

WorleyParsons Komex
resources & energy

DRAWDOWN FOR CITY OF ROHNERT PARK WELLS 7 AND 18 VERSUS CITY OF ROHNERT PARK TOTAL PUMPING RATE

SWL	AB	10/2004
N0410	14	

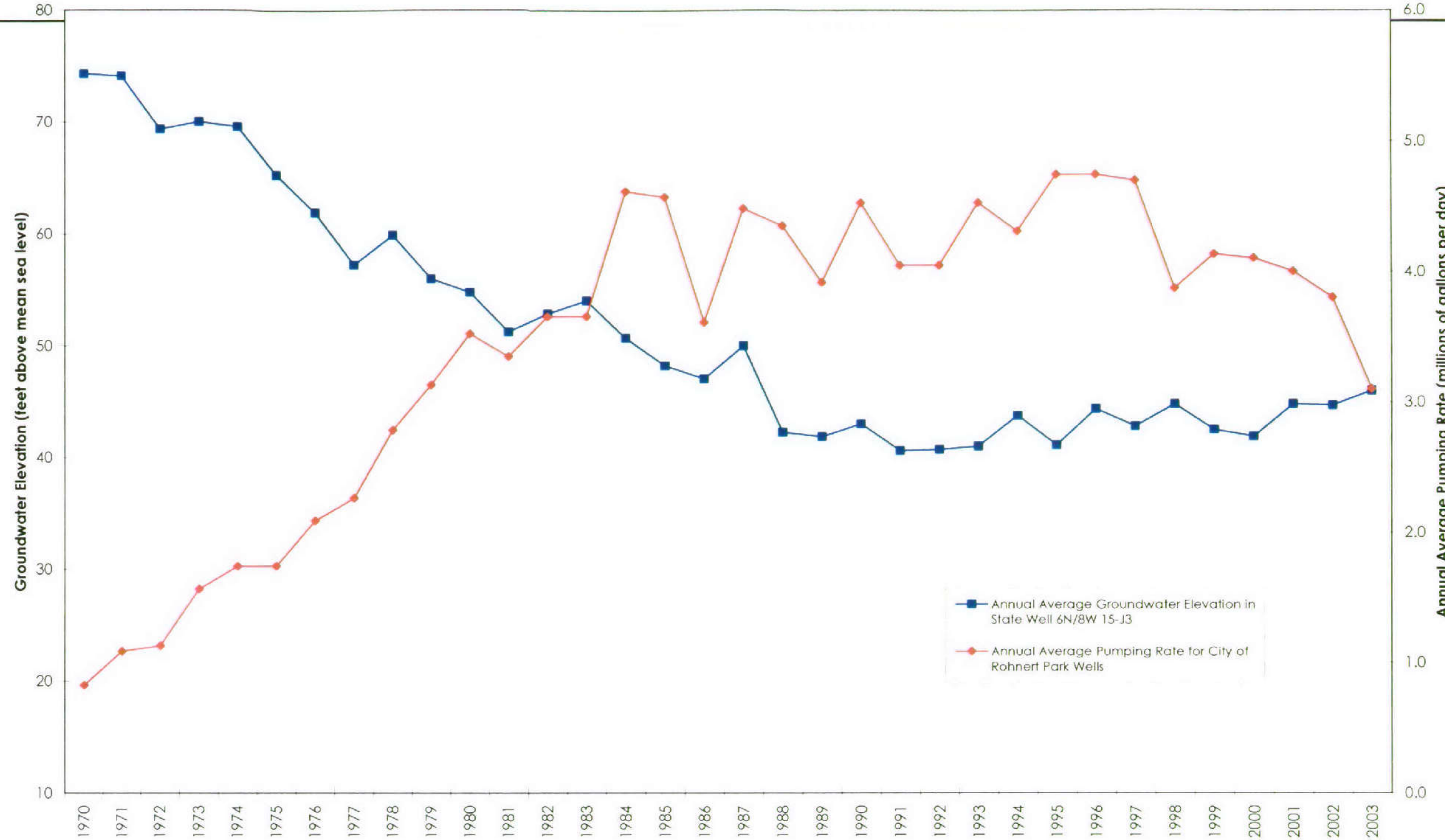


PROPOSED GRATON RANCHERIA

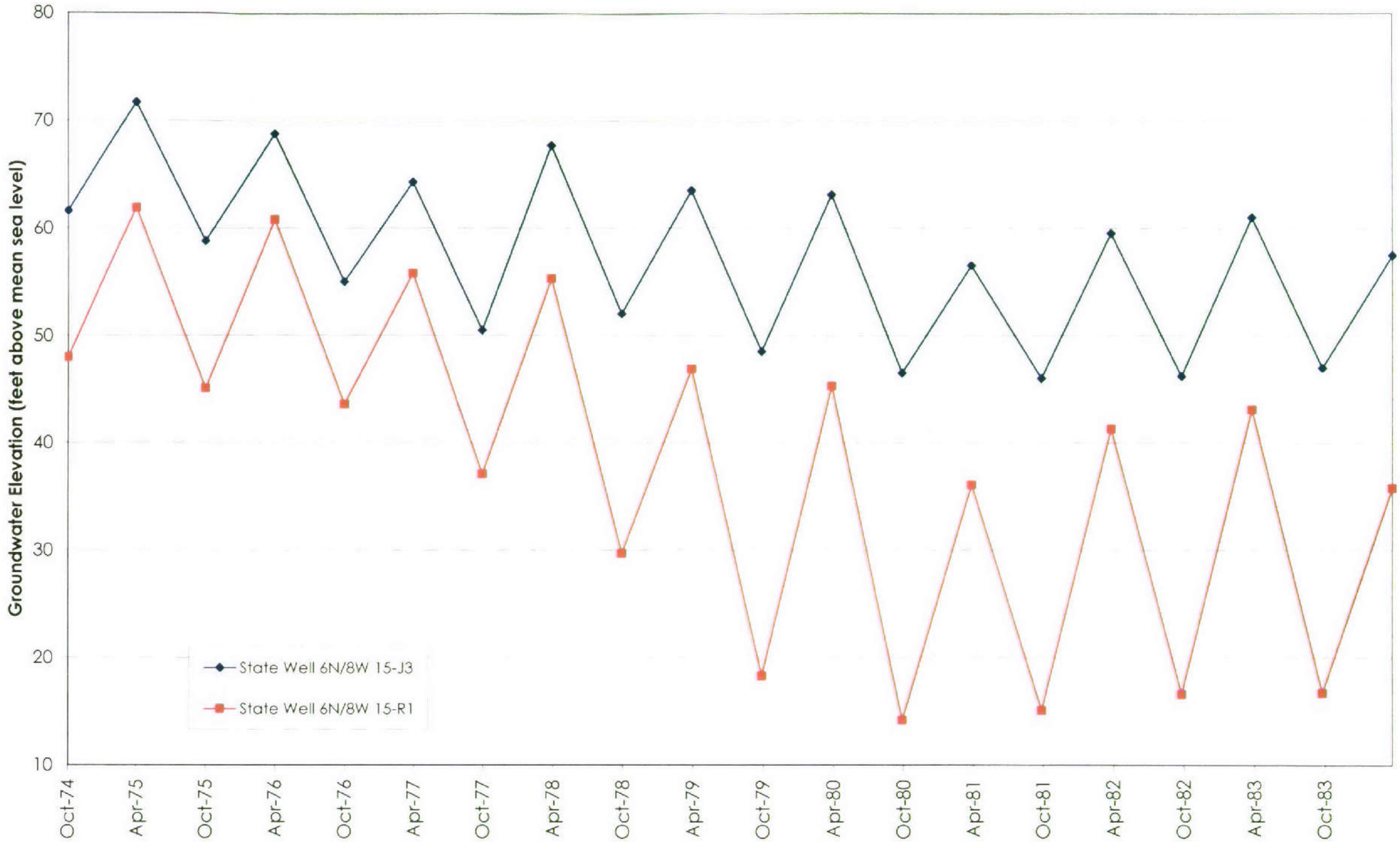
WorleyParsons Komex
resources & energy

HYDROGRAPHS FOR WELLS
IN THE SITE VICINITY

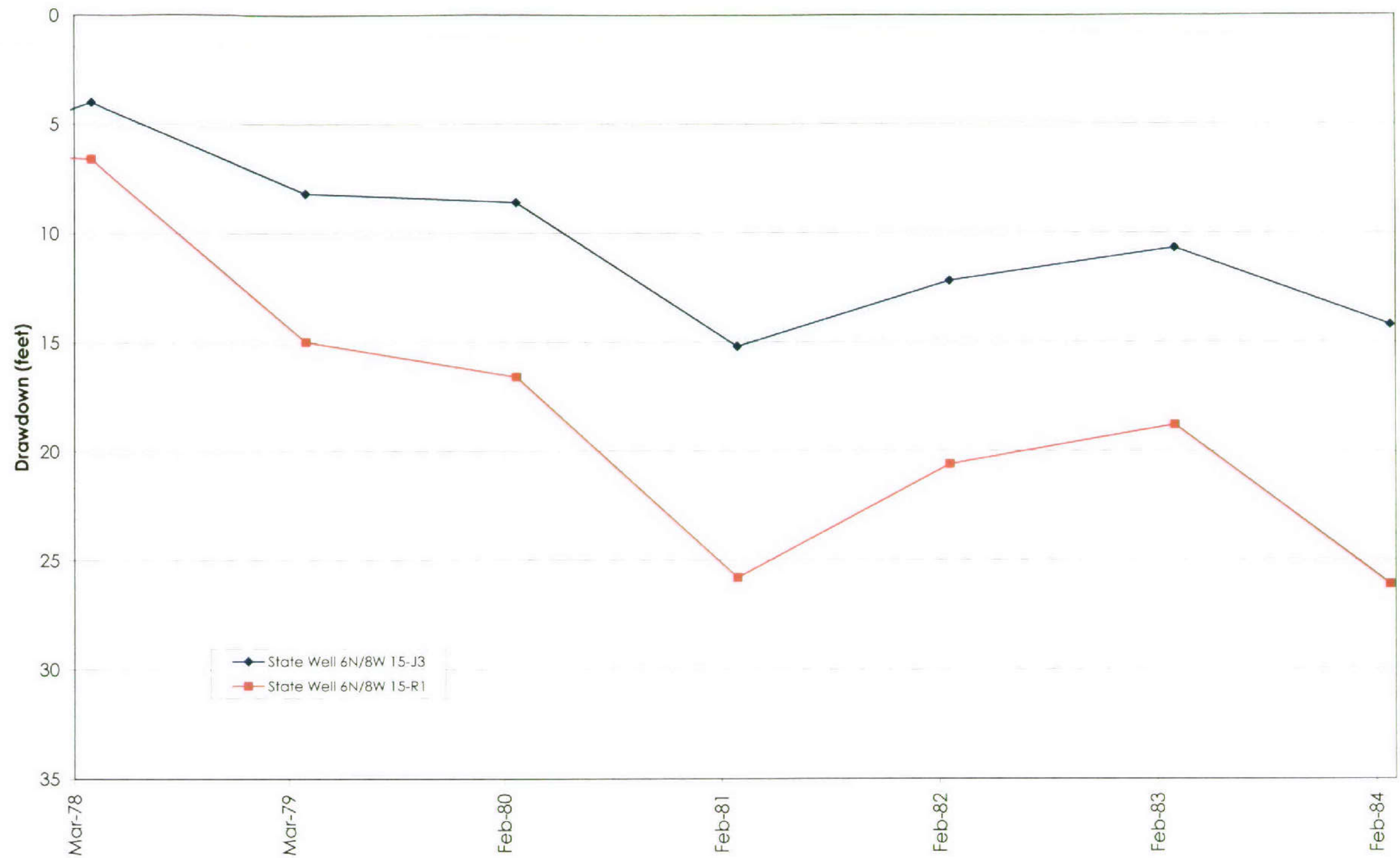
SWL	AB	10/2004
N0410	15	



PROPOSED GRATON RANCHERIA	WorleyParsons Komex resources & energy		
LONG-TERM HYDROGRAPH FOR STATE WELL 6N/8W 15-J3 (SHALLOW) VERSUS CITY OF ROHNERT PARK TOTAL PUMPING RATE	SWL	AB	10/2004
	N0410	16	

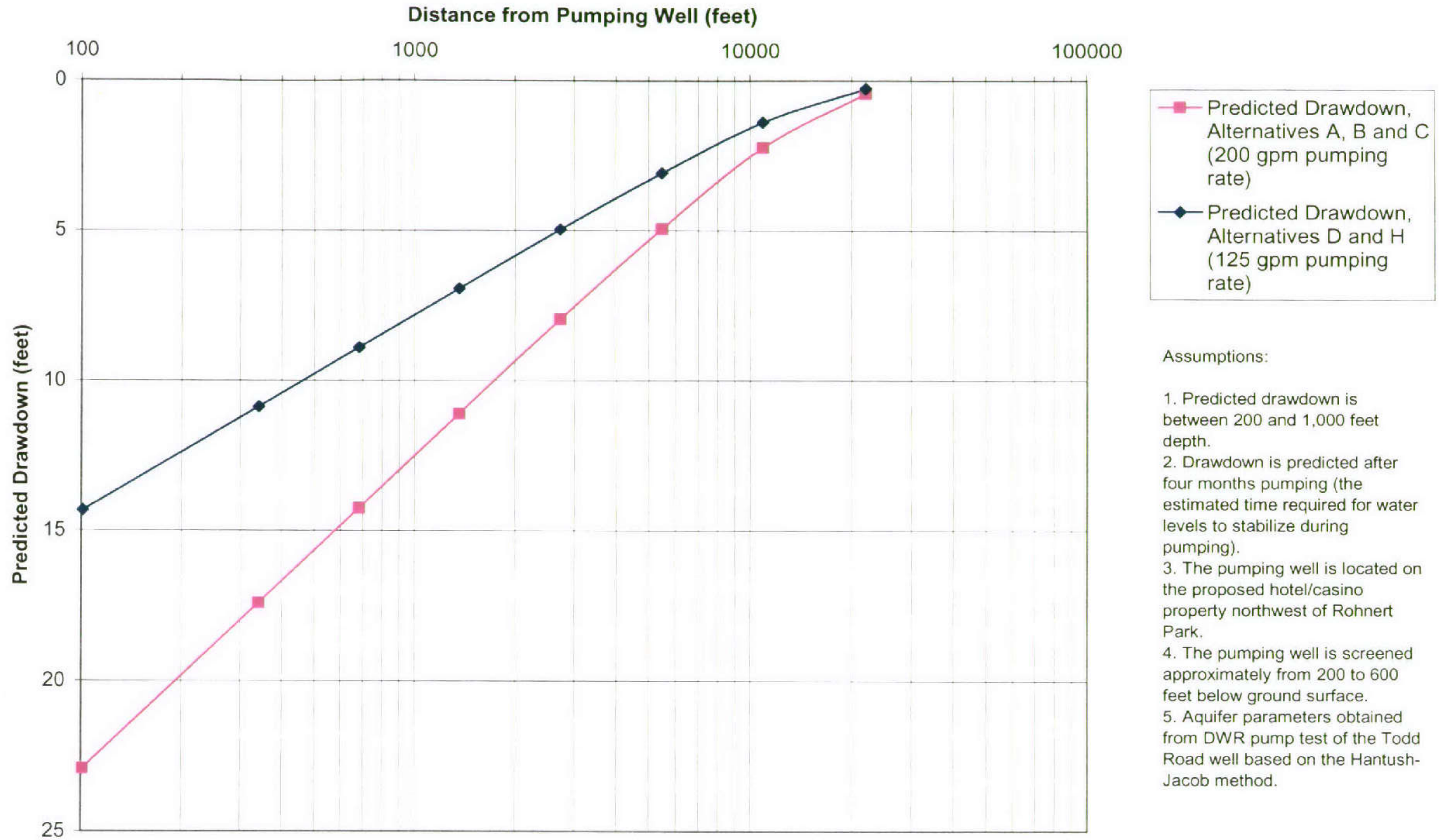


PROPOSED GRATON RANCHERIA	WorleyParsons Komex resources & energy		
HYDROGRAPHS FOR STATE WELLS 6N/8W 15-J3 (SHALLOW) AND 6N/8W 15-R1 (DEEP)	SWL	AB	10/2004
	N0410	**17**	



PROPOSED GRATON RANCHERIA	WorleyParsons Komex resources & energy			
SPRING DRAWDOWN IN STATE WELLS 6N/8W 15-J3 (SHALLOW) AND 6N/8W 15-R1 (DEEP)	SWL	AB	10/2004	
	N0410	18		

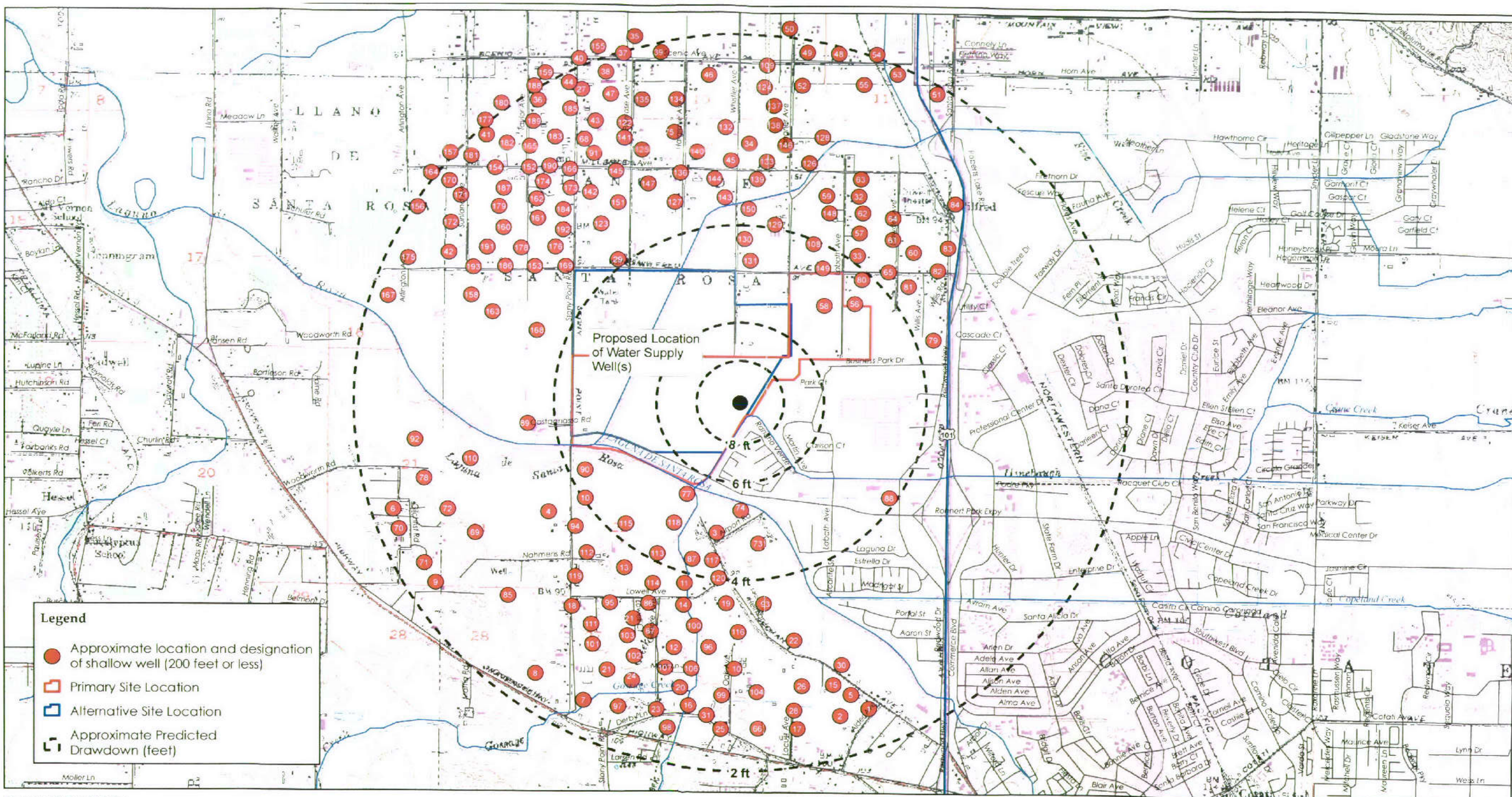
Distance - Drawdown Graph for Proposed Pumping Well



Assumptions:

1. Predicted drawdown is between 200 and 1,000 feet depth.
2. Drawdown is predicted after four months pumping (the estimated time required for water levels to stabilize during pumping).
3. The pumping well is located on the proposed hotel/casino property northwest of Rohnert Park.
4. The pumping well is screened approximately from 200 to 600 feet below ground surface.
5. Aquifer parameters obtained from DWR pump test of the Todd Road well based on the Hantush-Jacob method.

PROPOSED GRATON RANCHERIA	WorleyParsons Komex resources & energy		
DISTANCE-DRAWDOWN PREDICTION FOR PROPOSED PUMPING WELL AT THE SITE	SWL	AB	10/2005
	N0410	19	



Source: USGS 7.5' Series:
Cotati, Two Rock, Sepastopol, and Santa Rosa Quadrangles.
All well locations are approximate and not field verified.
See Table 1 for well completion data.

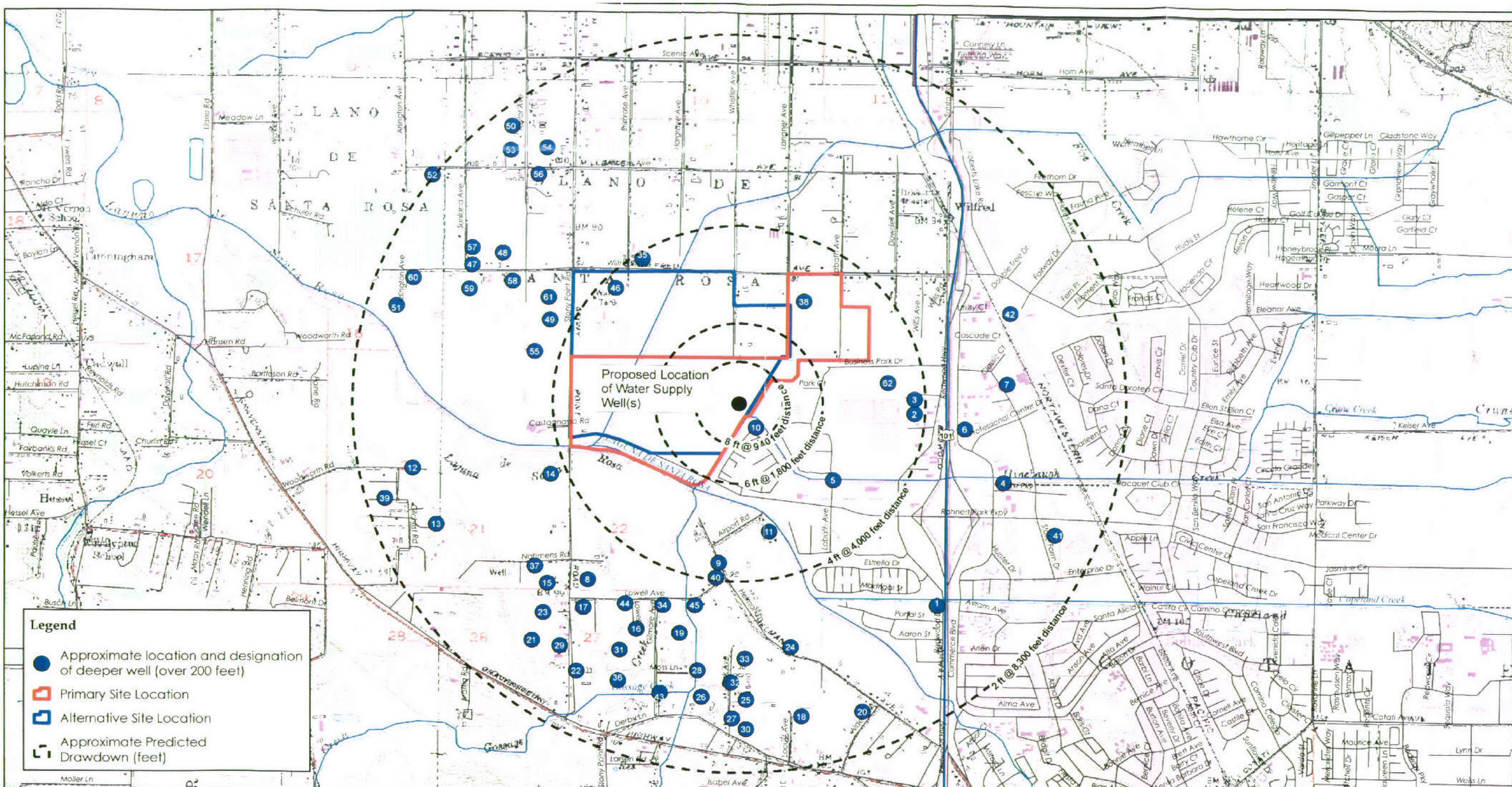
Note:
Predicted drawdown contours are approximate and based on the limitations and assumptions described in the text.

PROPOSED GRATON RANCHERIA

WorleyParsons Komex
resources & energy

APPROXIMATE LOCATION OF SHALLOW WELLS AND PREDICTED DRAWDOWN FOR ALTERNATIVES D AND H

SWL	MT	07/2007
N0410	20	



Source: USGS 7.5' Series:
Cotati, Two Rock, Sepastopol, and Santa Rosa Quadrangles.
All well locations are approximate and not field verified.
See Table 2 for well completion data.

Note:
Predicted drawdown contours are approximate and based on the limitations and assumptions described in the text.

0 0.375 0.75 1.5
Miles

PROPOSED GRATON RANCHERIA	WorleyParsons Komex resources & energy		
APPROXIMATE LOCATION OF DEEPER WELLS AND PREDICTED DRAWDOWN FOR ALTERNATIVES D AND H	SWL	AB	07/2007
	N0410	21	

A. ROHNERT PARK SITE - PUMP POWER IMPACT ANALYSIS

A.1 Introduction

The purpose of this analysis was to estimate potential impacts to pump power requirements on nearby water wells. These impacts might be caused by water table drawdown resulting from the installation and operation of a new water well that is being considered as part of this development.

Additional water table drawdown in the vicinity of an existing water well changes the operational characteristics of the pump operating within that water well. Additional water table drawdown effectively results in an increase in pump head, which in turn decreases the pump discharge rate, and changes the pump power requirements. On that basis, several operational scenarios were examined in an attempt to quantify, in general terms, the additional power required by nearby water well pumps that might be impacted by additional water table drawdown.

A detailed description of the methodology that was utilized to carry out this analysis, as well as a discussion of the results, is included in the sections that follow.

A.2 Methodology

For the Rohnert Park Site, two evaluations were made based upon the following ranges of values and boundary conditions:

- Two (2) pumping rates — 10 gallons per minute (gpm), and 250 gpm;
- Two (2) pump installation depths — 200 feet below ground surface (bgs), and 500 feet bgs;
- Two (2) initial pumping water levels — 100 feet bgs, and 265 feet bgs; and
- Two (2) interference drawdowns — 5 feet, and 20 feet.

As we have no knowledge of the actual pumps and installation conditions for water wells that surround the site, we based our analysis on selecting pumps appropriate for each of the discharge conditions noted above. By attempting to model a range of conditions, we hoped to bracket the real world pumps and ensure that their operating conditions lie within the feasible space of this analysis. While this analysis is not exact and may not be representative of actual installed pump types and conditions, it does offer some insight as to how much additional power might be required to pump one acre-foot of water if additional water table drawdown occurs. If site-specific information regarding water wells and pumps becomes available in the future, this analysis could be adapted to examine power requirement impacts for those specific pumps.

For each case, an appropriate pump was selected that met the baseline condition of zero feet of initial additional drawdown at the pump discharge rates, pump installation depths, and pumping water levels shown above.

Pumps were selected by using the Goulds Pumps website. In particular, pumps were selected using the Design Point Search by inputting flow rate and total head into the selector at the following internet URL:

http://www.pump-flo.com/select/centrifugal/criteria.aspx?DirName=goulds&catName=Goulds%20GL

On the basis of the flow rate and the calculated total head, recommended pump curves were displayed. The most appropriate pump curve for the input conditions was selected and downloaded to start the pump power impact analysis.

Each pump curve was imported into a CAD program so that precise scaling of distances along the Q, H, and P axes could be performed as part of the analysis. Use of the CAD program became critical when determining the effects of additional drawdown on the Q and P of a particular pump. As the incremental changes were typically small for H and P, precise scaling of the changes was required to accurately estimate the overall effect on each system.

Pump curves for each of the pumps selected for this analysis are included. A summary table of pump model for each analysis is shown below.

	Pump	Pumping	Baseline	Baseline	Baseline
Pump Type	Installation	Water	Pumping	Pump	Pump
& Size	Depth	Level	Rate	Head	Power
	(feet bgs)	(feet bgs)	(gpm)	(feet)	(h.p.)
VERT-IMMERS, 1SL	200	100	10	117.8	1.80
MULTISTAGE, 1HMS	500	265	250	276.9	22.17

Figure 1 presents a graphical and text summary of the procedure for estimating incremental differences in Q and P for imposed changes in H. The text summary from this figure is also included below.

All of the steps listed in the methodology shown on **Figure 1** were performed within a CAD program for accurate estimation of incremental differences. Please refer to the ten points shown on the figure in conjunction with the following:

- Once a suitable pump has been found that satisfies the discharge and head requirements for a particular case, the discharge (Q), head (H), and power (P) analysis starts at **Point #1** shown on the figure.
- From **Point #1** at the required Q, a vertical line is drawn upward to intersect the pump curve at **Point #2.**
- From **Point #2**, a horizontal line is drawn to the left (to **Point #3**) and the H associated with that particular Q is determined.
- From **Point #1**, a vertical line is drawn downward to intersect the power curve at **Point #4.**
- From Point #4, a horizontal line is drawn to the left (to **Point #5**) and the P associated with that particular Q is determined.
- The Q, H, and P determined by the steps shown above represent the baseline conditions for a particular case.
- Additional drawdown is then imposed, and the incremental effects on the Q, H, and P are then determined. This change in value of the parameters Q, H, and P are used later to estimate the impact to the power requirements to pump one acre-foot of water.

- Additional drawdown is the equivalent of moving higher up the pump curve (higher head).
- From **Point #3**, a vertical line is drawn upwards to **Point #6**. The length of this line is the imposed drawdown and will either be 5.0 feet or 20.0 feet.
- From **Point #6**, a horizontal line is drawn to the right to intersect the pump curve at **Point #7**.
- From **Point #7**, a vertical line is drawn downwards to **Point #8**. Where this line intersects the Q axis determines what the new Q for the pump will be given the additional head (imposed drawdown) for that case. For monotonically decreasing pump curves, as H increases, Q decreases, and conversely, as H decreases, Q increases.
- From **Point #8**, a vertical line is drawn downwards to intersect the power curve at **Point #9**.
- From **Point #9**, a horizontal line is drawn to the left (to **Point #10**) and the new P associated with the new Q is determined.

An example has been prepared for the case of Q = 250 gpm, with the pump installation depth at 500 feet bgs, and the pumping water level at 265 feet bgs.

Baseline

- Q = 250.0 gpm, H = 276.9 feet, P = 22.17 h.p. (**Point #1**, **#3**, and **#5**, respectively).

Impose an additional drawdown of 20.0 feet (**Point #3** to **#6**).

New

- Q = 228.5 gpm, H = 296.9 feet, P = 21.29 h.p. (Point #8, #6, and #10, respectively).

Incremental Difference

- Q = -21.5 gpm, H = +20.0 feet, P = -0.88 h.p.

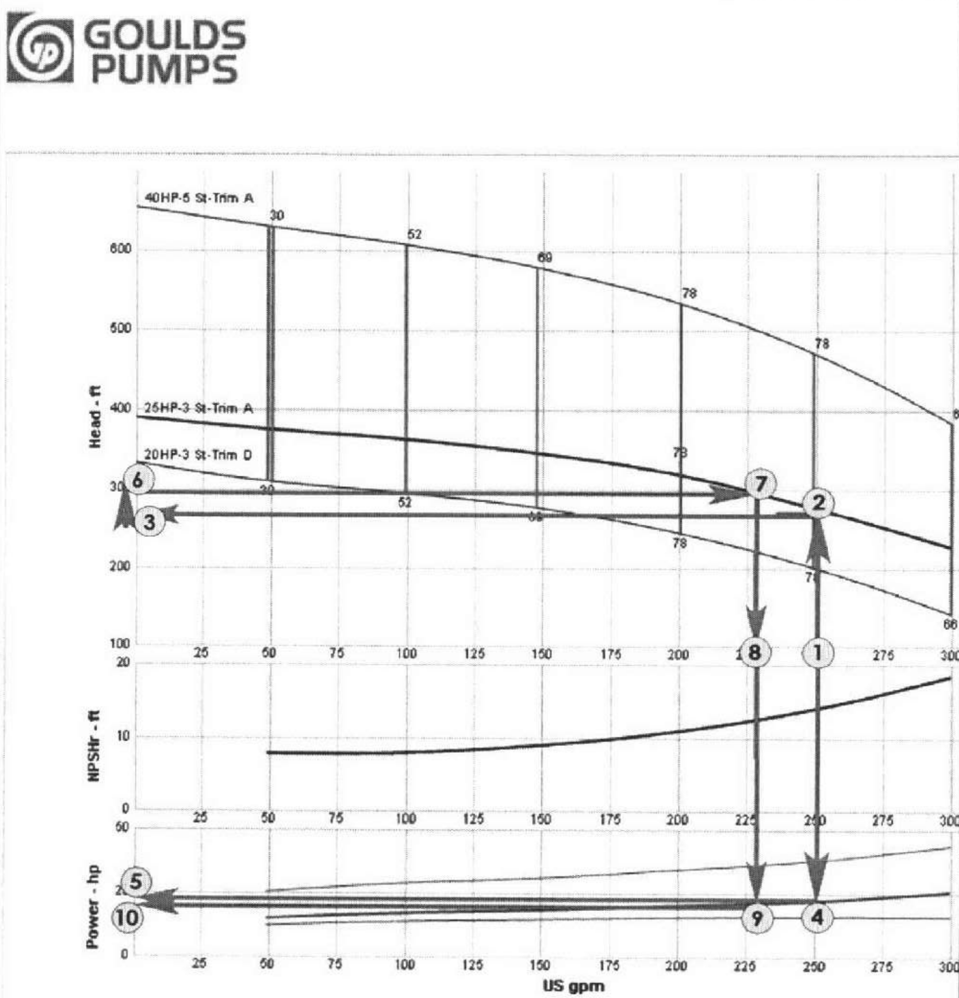
From this analysis, the additional time and/or power required to pump one acre-foot of water for additional drawdowns of 5.0 and 20.0 feet were calculated.

A.3 Results

One acre-foot of water is 325,851 gallons. The differential time to pump that volume of water using the new Q (that resulted from additional drawdown forcing the condition point further up the pump curve), relative to the baseline Q was calculated. This differential time was multiplied by the new power P (in horsepower [h.p]), with the result being incremental power consumption in kilowatt-hours (kW-hours) to pump one acre-foot of water. The table below presents the incremental power consumption results.

Pump Discharge Rate (gpm)		10
Pump Installation Depth (feet bgs)		200
Pumping Water Level (feet bgs)		100
Additional Water Table Drawdown (feet)	0.0	0.0
	5.0	103.1
	20.0	611.5

Pump Discharge Rate (gpm)		250
Pump Installation Depth (feet bgs)		500
Pumping Water Level (feet bgs)		265
Additional Water Table Drawdown (feet)	0.0	0.0
	5.0	7.0
	20.0	18.1



Methodology Summary

Once a suitable pump has been found that satisfies the discharge and head requirements for a particular case, the discharge (Q), head (H) , and power (P) analysis starts at Point #1 shown on the figure.

From Point #1 at the required Q, a vertical line is drawn upward to intersect the pump curve at Point #2.

From Point #2, a horizontal line is drawn to the left (to Point #3) and the H associated with that particular Q is determined.

From Point #1, a vertical line is drawn downward to intersect the power curve at Point #4.

From Point #4, a horizontal line is drawn to the left (to Point #5) and the P associated with that particular Q is determined.

The Q, H, and P determined by the steps shown above represent the baseline conditions for a particular case.

Additional drawdown is then imposed, and the incremental effects on the Q, H, and P are then determined. This change in value of the parameters Q, H, and P are used later to estimate the impact to the power requirements to pump one acre-foot of water.

Additional drawdown is the equivalent of moving higher up the pump curve (higher head).

From Point #3, a vertical line is drawn upwards to Point #6. The length of this line is the imposed drawdown and will either be 5.0 feet or 20.0 feet.

From Point #6, a horizontal line is drawn to the right to intersect the pump curve at Point #7.

From Point #7, a vertical line is drawn downwards to Point #8. Where this line intersects the Q axis determines what the new Q for the pump will be given the additional head (imposed drawdown) for that case. For monotonically decreasing pump curves, as H increases, Q decreases, and conversely, as H decreases, Q increases.

From Point #8, a vertical line is drawn downwards to intersect the power curve at Point #9.

From Point #9, a horizontal line is drawn to the left (to Point #10) and the new P associated with the new Q is determined.

Example

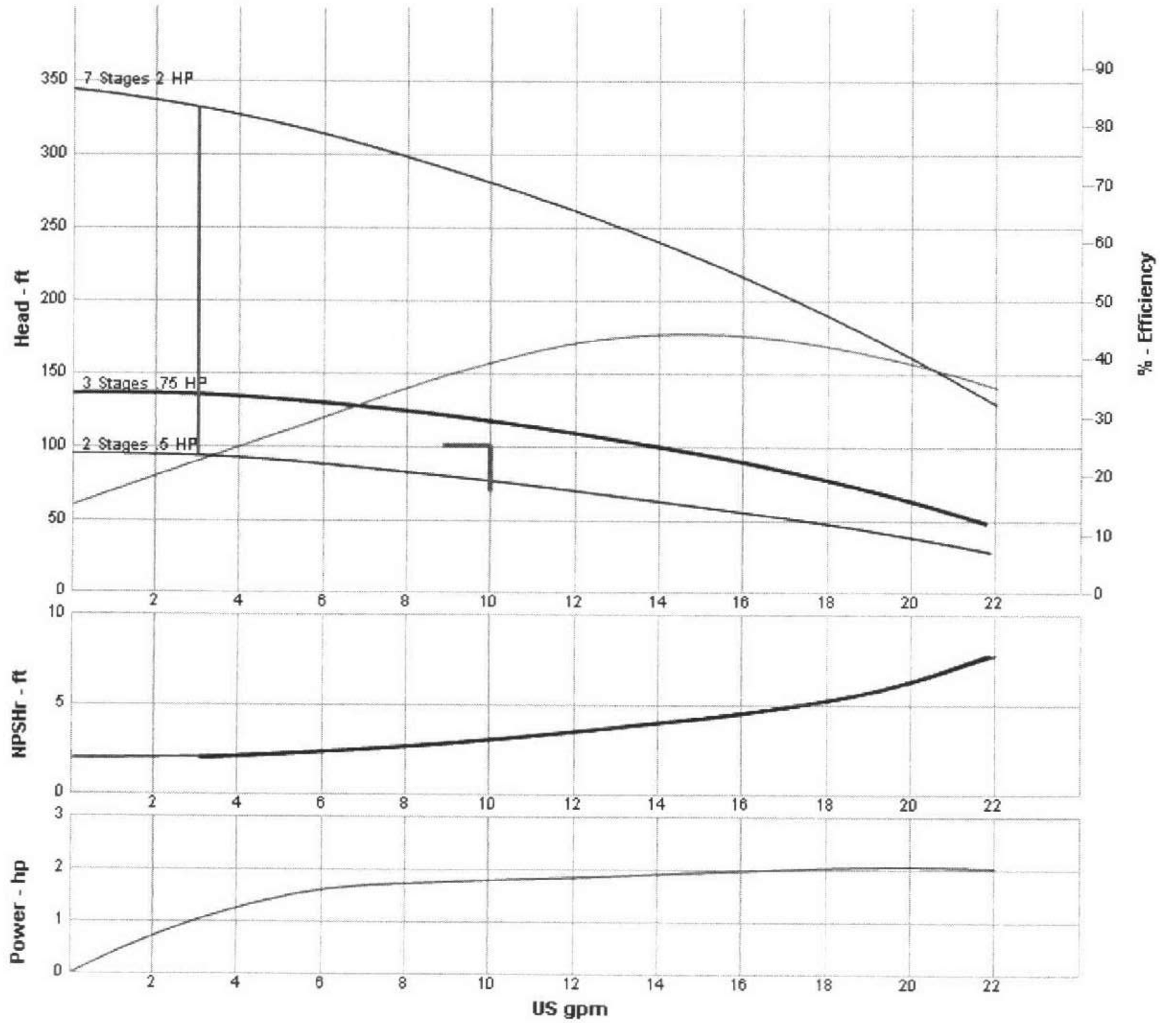
Baseline
Q = 250.0 gpm, H = 276.9 feet, P = 22.17 h.p. (Point #1, #3, and #5, respectively).

Impose an additional drawdown of 20.0 feet (Point #3 to #6).

New
Q = 228.5 gpm, H = 296.9 feet, P = 21.29 h.p. (Point #8, #6, and #10, respectively).

Incremental Difference
Q = -21.5 gpm, H = +20.0 feet, P = -0.88 h.p.

ROHNERT PARK SITE	WorleyParsons Komex resources & energy		
METHODOLOGY SUMMARY	DRAWN BY: WM	EDITED BY: WM	DATE: 12/2006
	APPROVED: MT		1



Request for Quote

Submit your selection to Goulds Pumps.

Pump

Type	MULTISTAGE
Size	1SV_2-7
Curve	Goulds SSV
Speed	3450 rpm
Line	3 Stages .75 H

Redraw Reset

Display Options ?

- (●) Manufacturer settings
- ( ) Custom

Multiple Pumps

Number of pumps 1

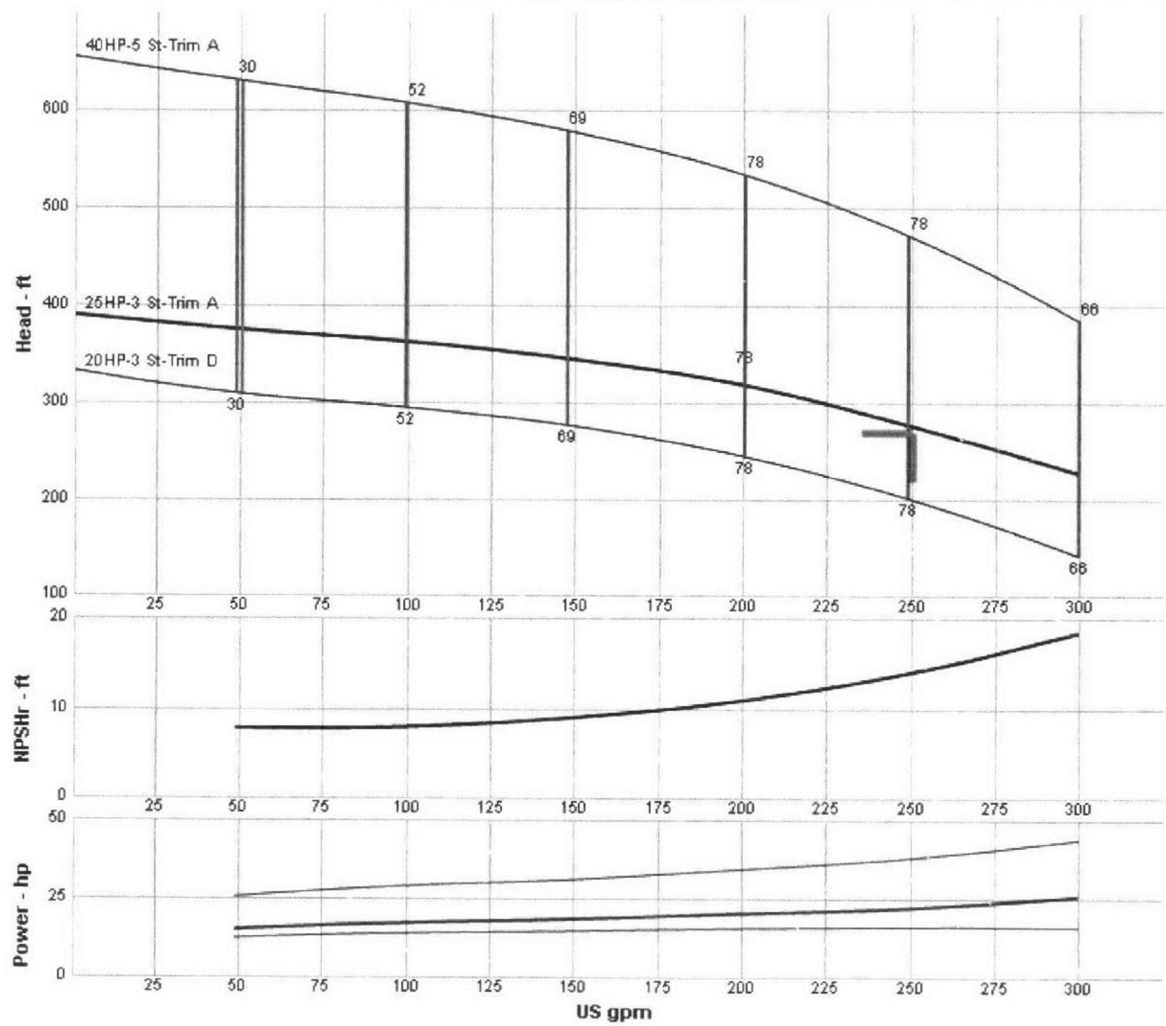
(●) Parallel ( ) Series

Multiple Speeds (rpm)

2900 rpm - 3450 rpm

Flow	Head	Eff	BEP	NPSHr	Power	Motor	Frame	Min flow	Sphere
10 US gpm	117 ft	---	---	3.05 ft	---	---	---	3 US gpm	0.0312 in

GOULDS PUMPS



Request for Quote

Submit your selection to Goulds Pumps.

Pump

Type	MULTISTAGE
Size	2SMA-3525 RPM
Curve	GP SMVT
Speed	3525 rpm
Line	25HP-3 St-Trim

Redraw Reset

Display Options ?

◉ Manufacturer settings
○ Custom

Multiple Pumps

Number of pumps 1

◉ Parallel ○ Series

Multiple Speeds (rpm)

1180 rpm - 3525 rpm

Flow	Head	Eff	BEP	NPSHr	Power	Motor	Frame	Min flow	Sphere
250 US gpm	275 ft	77 %	78 %	14.1 ft	22.5 hp	---	---	50 US gpm	0.25 in

Groundwater Study – Lakeville Site

WorleyParsons Komex

resources & energy

GROUNDWATER STUDY: PROPOSED GRATON RANCHERIA CASINO AND HOTEL – LAKEVILLE ALTERNATIVE SITE

SONOMA COUNTY, CALIFORNIA

PREPARED FOR:

Analytical Environmental Services

2021 N Street, Suite 200

Sacramento, California 95814

PREPARED BY:

WorleyParsons Komex

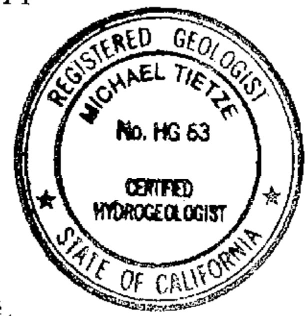
2330 East Bidwell Street, Suite 150

Folsom, California 95630

January 2007

N0410C

Michael Tietze and Dennis Jamison California Professional Geologists and Certified Hydrogeologists, as employees of WorleyParsons Komex, with expertise in groundwater hydrology, have reviewed the report with the title "Groundwater Study: Proposed Graton Rancheria Casino and Hotel, Rohnert Park, California." Their signatures and stamps appear below.



Michael Tietze
Professional Geologist 5698
Certified Hydrogeologist 63
January 2007

Dennis Jamison
Professional Geologist 5787
Certified Hydrogeologist 471
January 2007

EXECUTIVE SUMMARY

The Federated Indians of Graton Rancheria have proposed to establish a hotel and casino development in Sonoma County. One of the alternative sites being considered is located on unincorporated land near Lakeville in southern Sonoma County, California (the Site, **Figure 1**). An Environmental Impact Statement (EIS) is to be prepared by Analytical Environmental Services (AES) as required by the National Environmental Policy Act.

HydroScience Engineers, Inc. (HydroScience) is performing a water and wastewater feasibility study for the proposed development, and has recommended that the development be supplied from two existing on-Site groundwater wells (augmented, if necessary, by additional new on-Site wells) with a capacity of 200 gallons per minute (gpm).

WorleyParsons Komex was contracted by AES to undertake a study relating to the proposed use of groundwater from the on-Site wells to supply the development. The objective of the study is to assess how the use of groundwater could affect local groundwater levels, existing wells and (on a qualitative basis) potential seawater intrusion in the southern Petaluma Valley. Our assessment was restricted to the review and interpretation of existing information on hydrogeology in the area of the Site. A Site visit was not carried out as part of the study.

The approximately 320-acre Site is undeveloped and located at the foot of the Sonoma Mountains in the lower Petaluma Valley near San Pablo Bay. Petaluma River, a tidal slough, is located approximately 2 miles west of the Site. Elevations across the Site range from 40 feet above mean seal level (amsl) to near sea level on the valley floor on the west side of the Site.

The Site lies within the Petaluma Valley Groundwater Basin. Petaluma Valley is a north-northwest trending valley underlain by a basement complex consisting primarily of marine, metamorphic and igneous rocks belonging to the Mesozoic Franciscan Assemblage. Overlying or structurally juxtaposed against these rocks in the southern Petaluma Valley is the Pliocene Petaluma formation, which consists of claystone with some sands and gravels of marine and continental origins. It is into these Mesozoic and Cenozoic rock formations that the ancestral Petaluma River incised a relatively deep and narrow valley during periods of low sea level in late Quaternary to Recent time. This valley was subsequently filled with Holocene and Recent alluvial and estuarine sediments when sea levels rose. The resulting valley fill consists of estuarine deposits (locally referred to as the Bay Mud) that are interbedded with (near the valley margins) and underlain by unconsolidated alluvial fan and floodplain alluvial deposits. The Quaternary to Recent basin fill deposits and the Petaluma formation contain the principal aquifers in the basin. Consolidated Mesozoic basement rocks that underlie the entire area have little permeability and form the boundaries of the groundwater flow system.

The permeability and extent of water-yielding deposits in the Petaluma Valley varies considerably. Groundwater is unconfined, or occurs under water-table conditions, in the shallow alluvial deposits exposed near the valley margins. The groundwater is confined or semiconfined in deeper parts of the basin. The continental deposits of sand and gravel are interbedded with fine-grained silt and clay, which locally may act as confining layers. Recharge is primarily through infiltration of runoff from precipitation in the surrounding mountains: infiltration occurs along streambeds and in the permeable sediments of the valley floor at the basin margins. Groundwater flow in the aquifers is generally parallel to the long axis of the basin or away from the flanking ridges to discharge areas along the tidal sloughs and San Pablo Bay.

The Bay Mud, which consists of estuarine and marsh deposits composed of mud, silty mud, silt, and small amounts of sand, all rich in organic matter, underlies the southern Petaluma Valley. Artificial fill covers the Bay Mud in many areas. Sea water was trapped in the Bay Mud as it was deposited, and water pumped from this formation is generally brackish. The thickness of the Bay Mud deposits between the Site and San Pablo Bay ranges from approximately 50 to at least 239 feet, but may be up to several hundred feet deep These deposits have a low permeability and the few water yielding zones are generally assumed to lack horizontal and vertical continuity. Due to the low permeability of these deposits, the hydraulic connection between the Bay Mud and the underlying alluvial fan deposits is expected to be relatively weak; however, it has been recognized that significant pumping from the underlying deposits could induce leakage of brackish water from the Bay Mud into the alluvial fan deposits. The thickness of Bay Mud encountered at the Site varies from 0 feet near the Lakeville Highway to approximately 150 feet at the western extent of the Site at existing well North #2.

Alluvial deposits occur at the surface along the eastern edge of the southern Petaluma Valley in the vicinity of the Site and extend beneath the Bay Mud. These deposits consist of poorly sorted heterogeneous mixtures of clay, silt, sand and some gravel. The proportion of sand and gravel is expected to be greater closer to the mountains; even so, review of local driller's logs indicates that fine grained deposits are still abundant in these areas. The thickness of the younger and older alluvium is reported to be as much as 200 and 300 feet in the southern Petaluma Valley, respectively; however, the alluvium may be difficult to distinguish from the underlying Petaluma formation in driller's logs. In addition, the published literature contains conflicting information regarding the thickness of the alluvium in the vicinity of the Site and the depth at which the underlying Petaluma formation may be encountered. For these reasons, the true thickness of alluvium at the Site cannot be determined without further study.

Yields from the alluvial aquifer are variable, but it is generally thought to be capable of supporting domestic uses and small irrigation projects. Although the alluvial deposits are quite heterogeneous, they are regarded as a single aquifer unit for the purposes of this study. Groundwater within the alluvial aquifer may occur under unconfined conditions near the heads of alluvial fan deposits, but semi-confined and possibly confined conditions are expected to occur in distal areas where finer grained deposits are more prominent or where the formation is overlain by the Bay Mud.

Water quality data indicate seawater intrusion has occurred locally into alluvial deposits near and possibly adjacent to the Site. Although groundwater from the alluvial deposits, especially near the valley margins and sometimes at depth beneath the Bay Mud, is often of good quality, it is of limited extent.

The Petaluma formation is exposed in the mountains to the east of the Site and underlies the alluvium beneath the Site. It is characterized by an abundance of clay and consists chiefly of shallow marine to brackish-water deposits and continental deposits of clay, shale and sandstone, with lesser amounts of conglomerate and nodular limestone. As mentioned above, the contact between the alluvium and the underlying Petaluma formation may be difficult to distinguish from drill cuttings; therefore, the depth at which the Petaluma formation occurs beneath the Site is not known. It seems likely that some of the deeper wells in the Site's vicinity may include screened zones in the Petaluma formation.

Because of its large percentage of clay, the Petaluma formation is generally of low permeability; however, wells drawing from coarser grained units can yield moderate flow rates. Hydraulic heads are often variable in wells tapping the Petaluma formation, indicating that the water-bearing coarser gained lenses can be hydraulically isolated by the abundance of clay. In these water-bearing lenses, groundwater can occur under confined and semi-confined conditions.

At the southern end of the Petaluma Valley, and underlying or near the Site, a zone of poor quality groundwater has been identified at depths from approximately 150 to 700 feet. This poor quality water has been variously interpreted as connate water from sea water trapped in the fine-grained sediments of the Petaluma formation at the time of deposition, or the result of seawater intrusion into the alluvial aquifers. Several references discuss seawater intrusion as impacting aquifers in the southern Petaluma Valley in the Vicinity of the Site, and indicate that increased groundwater withdrawal in the area could result in additional seawater intrusion.

Fifty-seven wells have been identified within about 1 ½ miles of the Site from available records. The wells range in depth from 12 to 736 feet, with an average depth of 243 feet. Most of the wells are located along the base of the mountains or on the valley floor, and are completed in alluvial fan deposits underlying the Bay Mud. Various deeper wells, including some wells along the mountain front and those within the Sonoma Mountains, are likely completed in the Petaluma formation. Reported well uses include domestic (35), stock watering/dairy (six), irrigation (two), monitoring (two) and "other" (one). The use of eight of the identified wells is not reported, and three of these wells are reported as being abandoned. There are no municipal water supply wells in the lower Petaluma Valley.

HydroScience identified two wells as being located at the Site. North #1 is located near the Sonoma Mountains on the east side of the Site and is 413 feet deep. North #2 is located on the west side of the Site and is 650 feet deep. Both wells are screened in the alluvial aquifer and possibly in the underlying Petaluma formation. HydroScience reported the Bay Mud as being 150 feet thick at North #2 and bedrock being encountered at 650 feet. Water levels in both wells were reported to be artesian in 2003.

Review of hydrographs for three nearby wells completed in the alluvial aquifer indicates that water levels are close to sea level and generally fluctuate within a relatively narrow range of a few feet. Both seasonal and climatic trends are apparent. Comparison with cumulative departure from average annual precipitation indicates that each well responds to periods of below and above average precipitation.

A much greater difference in water levels (approximately 60 feet) is apparent for two wells completed in the Petaluma formation. In addition, water level fluctuations are much greater for these wells – fluctuations range from approximately 16 feet to over 110 feet. This is consistent with the interpretation that the clayey character of the Petaluma formation can result in hydraulic isolation of individual water-bearing zones. Nevertheless, both seasonal and climatic water level trends are apparent. A slight declining trend in water levels is apparent in the hydrographs of both of these wells that does not appear related to climatic influences and may be related to regional water pumping.

Specific capacities for two wells at the Site and four wells in the Site vicinity are reported by HydroScience to range from 0.3 to 3 gallons per minute (gpm)/foot of drawdown. This is in the range of values reported for 24 wells tested in the Site vicinity, which is 0.03 to 30 gpm/foot. Analysis of pumping test data provided by HydroScience indicates a transmissivity of approximately 0.5 square foot per day for North #1. This corresponds with a hydraulic conductivity of approximately 0.25 foot/day, which is typical of strata dominated by fine grained sediments (*i.e.*, silt and clay).

Due to the relatively complex hydrogeologic setting of the Site and the limited data available, it is not possible to quantitatively estimate drawdown impacts from the proposed pumping. To evaluate one scenario of potential drawdown impacts, an analytical element model was prepared using the U.S. Environmental Protection Agency's WhAEM2000 code. The model was set up using a simplified set of assumed boundary conditions to simulate the influence of the relatively long and narrow configuration of the valley and recharge that may be induced from the Petaluma River and San Pablo Bay. Hydrogeologic parameters for the analytical model were derived from the available data at the time of this study.

The hydraulic conductivity of the sediments tapped by the on-Site wells is estimated to be approximately 0.25 foot/day; however, when this value was used in the model, the wells dewatered. For this reason, a hydraulic conductivity of 0.5 foot/day was used in the model. One clear implication is that the sediments penetrated by the wells may not be transmissive enough to yield water to the existing wells at a sufficient rate to meet the water demand for the proposed development. However, doubling the hydraulic conductivity may be within the reasonable range of certainty for aquifer parameter estimation, especially when analyzing a pumping test performed near a no-flow boundary using only data from the pumping well. In addition, doubling of the hydraulic conductivity allows evaluation of drawdown impacts that would occur if the water demand can be met by existing wells or by additional wells installed at the Site.

Based on the model results, the evaluated scenario indicates that pumping at the Site could result in drawdowns measuring 10's of feet between the Site and the river and over 100 feet near the Site. An alternate scenario could also be realistic would assume that the amount of leakance that can be induced form Petaluma River and San Pablo Bay is more limited, and the hydraulic conductivity of the alluvial valley aquifer is much greater than calculated based on the North #1 well pumping test. Under such a scenario, the drawdown near the Site would be less, but drawdowns in the range of 10 feet or so might be expected to extend up to several miles beyond the Petaluma River and beneath San Pablo Bay.

These estimates are approximate; however, it is clear that drawdowns of the magnitude shown on **Figure 8** would significantly impact wells in the vicinity of the Site. **Table 1** lists 57 wells identified from DWR records and published reports in the Site vicinity. The reported depths of these wells range from 12 to 736 feet. All of these wells except well 12 (which is completed across the river from the Site) and wells 26, 27, and 28 (completed in fractured bedrock) may be expected to experience drawdown impacts of at least 10 and in some cases over 100 feet. However, as noted above, it is also possible that drawdown near the Site will be less and that significant drawdown will be experienced beyond the Petaluma River from the Site.

Wells in the Site vicinity are predicted to experience some drawdown impacts (interference drawdown) and a resulting proportional decrease in well yield or efficiency, pumping cost and pump life. In the absence of well-specific data regarding transmissivity, use, condition and efficiency, these impacts may be assumed to be generally proportional to the amount of interference drawdown and the remaining saturated thickness of the well after interference drawdown.

The most serious impact that could be experienced by a nearby groundwater user would be having their well go dry or rendered unusable because the remaining saturated thickness after drawdown is too small to support pumping at the required rate. The wells most at potential risk for this impact are expected to be primarily shallow wells near the Site. Deeper wells or wells located at increased distance form the Site are at lower risk of being dewatered or rendered unusable.

In some wells, if water levels fall to a point where the well is in danger of going dry or becoming unusable, the pump intakes can be lowered to extend the life of the well. Without more specific information regarding well construction and pump depth, it is not possible to estimate how many wells may be at risk of experiencing this impact. However, pump intakes for shallow or domestic wells are generally set near the well bottoms and cannot be lowered; whereas, pump intakes for deeper municipal, industrial or agricultural wells are sometimes set at a relatively shallow depth and could require lowering if the wells are located near the Site.

Interference drawdown will cause an increase in the electrical cost to pump a unit volume of groundwater from a well. This cost increase is not expected to be significant for domestic wells because of the relatively low volume of groundwater pumped by a typical household, but could be significant (ranging from several hundred to several thousand dollars) for higher capacity agricultural or industrial wells near the Site. For the pumps modeled, the increased costs for higher capacity pumping represented approximately a 1 to 11 percent increase in overall pumping costs.

As discussed in several hydrogeologic references reviewed during this study, significant pumping of freshwater aquifers in the lower Petaluma Valley has the potential to cause seawater intrusion. The risk of inducing seawater intrusion and the extent and rate of migration of salinity impacts depends largely on the degree of connectedness between the Petaluma River and San Pablo Bay and the aquifers that will be pumped for the proposed project, which is not known at this time. Nevertheless, based on the reported hydrostratigraphy of the area, the detection of salinity impacts in the alluvial aquifers near the Site, and documentation of historical pumping-induced seawater intrusion in Petaluma, it is reasonable to assume that some seawater intrusion will occur.

Seawater intrusion, if induced by pumping for the proposed development, would impact off-Site wells between the Site and the Petaluma River or the Bay. The consequences of seawater intrusion include well, pump and pipe corrosion, rendering water objectionable or unusable, or creating the need for water treatment prior to use. In addition, seawater intrusion could trigger regulatory requirements to cease pumping and possibly to restore affected groundwater.

Other drawdown-related impacts could include the compaction of depressurized or dewatered clay deposits and resulting subsidence. The risk of subsidence depends on whether unconsolidated or semi-consolidated clay deposits are depressurized or dewatered, which is a function of the interconnectedness between subsurface strata, the rate of lateral groundwater inflow and the rate of recharge in both the pumped alluvial strata as well as the overlying Bay Mud. Evaluation of the potential for subsidence is beyond the scope of this report.

A more sophisticated model, based on better knowledge of the hydrostratigraphy, groundwater flow, recharge and degree of interconnection between surface water and groundwater in the Site vicinity, could be used to predict drawdown impacts and related impacts due to seawater intrusion and subsidence with greater certainty. If greater certainty is required, we recommend that field work including subsurface exploration, additional aquifer testing and surface reconnaissance be performed to support a numerical modeling study.

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LIST OF ACRONYMS AND ABBREVIATIONS

AFY	acre-feet per year
amsl	above mean sea level
AES	Analytical Environmental Services
bgs	below ground surface
DAU	Detailed Analysis Unit
DEIS	Draft Environmental Impact Statement
DWR	California Department of Water Resources
EIS	Environmental Impact Statement
EPA	Environmental Protection Agency
ft^2/day	square feet per day
gpd	gallons per day
gpd/foot	gallons per day per foot
gpm	gallons per minute
HydroScience	HydroScience Engineers, Inc.
mg/L	milligrams per liter
SCWA	Sonoma County Water Agency
SP	Spontaneous Potential
TDS	Total Dissolved Solids
USGS	United States Geological Survey

1 INTRODUCTION

1.1 PROJECT BACKGROUND

The Federated Indians of Graton Rancheria have proposed to establish a casino and hotel development in Sonoma County, California. Several alternatives are being considered for the project. Alternative F, which is the subject of this report, proposes development of an approximately 320-acre property located east of Lakeville in southern Sonoma County ("the Site;" **Figure 1**). An Environmental Impact Statement (EIS) is to be prepared by Analytical Environmental Services (AES) as required by the National Environmental Policy Act.

HydroScience Engineers, Inc. (HydroScience) is performing a water and wastewater feasibility study for the proposed development and has recommended that the development be supplied from on-Site groundwater wells a sustained long-term pumping rate of 200 gallons per minute (gpm) (HydroScience, 2005b).

1.2 PROJECT SCOPE

WorleyParsons Komex was contracted by AES to undertake a study relating to the proposed use of groundwater to supply the new development, as set out in our proposal dated 8 July 2005. Additional studies to address comments on the Preliminary Draft EIS (DEIS) received from cooperating agencies were outlined in a contract amendment dated 12 October 2006. The objective of the study was to assess the how the use of groundwater to supply the new development will potentially affect water resources near the Site. Our assessment has been restricted to the review and interpretation of existing information on hydrogeology in the area of the Site. A Site visit was not carried out as part of the study.

1.3 REPORT ORGANIZATION

This report is subdivided as follows:

Section 1: Introduction.

Section 2: Regional Hydrogeologic Setting. An overview of the topography, drainage, climate, geology and hydrogeology of the region.

Section 3: Available Hydrogeologic Information. A summary of the sources of information used in undertaking the study, with particular emphasis on hydrogeology, groundwater elevations, seawater intrusion and trends in water levels and quality in the general Site area.

Section 4: Site Evaluation. An interpretation of geologic and hydrogeologic conditions pertaining to the Site, with discussion of groundwater resources.

Section 5: Potential Impacts of using Groundwater to Supply the Lakeville Alternative Site. An evaluation of the likely effects of groundwater pumping to supply the proposed development on groundwater levels and wells in the vicinity, and the groundwater basin in which the Site is located.

Section 6: Conclusions.

Section 7: Closure/Limitations.

Section 8: References.

2 REGIONAL HYDROGEOLOGIC SETTING

2.1 SITE LOCATION, TOPOGRAPHY AND LAND USE

The Site is located north of Highway 37 near Lakeville, approximately 2 miles east of the Petaluma River and 1.7 miles north of San Pablo Bay at the northern extent of San Francisco Bay in Sonoma County. The Site lies along the western base of the Sonoma Mountains near the southeastern end of Petaluma Valley, an intermontane basin in the Coast Range of northern California (**Figures 1** and **2**).

The approximately 320-acre Site is presently undeveloped and portions are used for agricultural purposes. Elevations across the Site range from 40 ft above mean seal level (amsl) along Lakeville Highway to near sea level along the southwest portion of the Site. The topography of the valley floor south and west of the Site is generally flat, formed primarily on flood plain and tidal marsh deposits. The elevation of the valley floor in this area is typically at or below sea level (–2 feet amsl). A series of levees and drainage channels are depicted on the topographic map which protect these lands from inundation by San Pablo Bay.

2.2 CLIMATE

The climate along the coast of California is moderated by the Pacific Ocean and is essentially Mediterranean, characterized by mild, wet winters and warm, dry summers. Precipitation is seasonal and usually in the form of rain. The greatest amounts of precipitation fall during late autumn, winter, and early spring. Altitude influences precipitation patterns as the greatest amounts of precipitation fall in the mountains. The average annual precipitation at Petaluma, located approximately 9 miles north of the Site, over the period from 1949 through 2004 was approximately 24.6 inches (WRCC, 2005).

2.3 DRAINAGE

The Petaluma River and its tributaries (Adobe Creek, Willow Creek and San Antonio Creek are noteworthy) drain a basin approximately 146 square miles in size southward into the San Pablo Bay. Petaluma River is essentially a tidally influenced slough from the Bay approximately 11 miles upstream to the city of Petaluma, and tidal marshes are located along side this reach of the river (**Figure 1**). Above the river reach that is tidally influenced, flow in Petaluma River (Petaluma Creek in some reports) is reported to be seasonal (Cardwell, 1958).

The topographic map (**Figures 1** and **2**) suggests that at least three ephemeral streams originating in the Sonoma Mountains cross the northern Site boundary; however, these channels drain relatively small, low and narrow drainage basins in the hills east of the Site and are not expected to discharge significant volumes of water except possibly after intense storms or prolonged wet periods. Apparently, these streams discharge into the drainage network on the valley floor and are tributary to the Petaluma River.

2.4 GEOLOGY, HYDROGEOLOGY AND WATER SUPPLY

The following information is summarized primarily from the U.S. Geological Survey (USGS) Ground Water Atlas of the United States – Segment 1, California, Nevada (USGS, 1995) and the latest comprehensive DWR study of the area (DWR, 1982). The intermontane basins in the coastal mountains of California are structural troughs or depressions that parallel the coastline (trend northwestward) and formed as a result of folding and faulting from the deformation of older rocks by the intense pressures of colliding continental plates. The rocks that underlie the basins and form the core complex of the surrounding mountains are primarily sedimentary rocks of marine origin, and metamorphic rocks and igneous rocks of Mesozoic age. Locally overlying or structurally juxtaposed against the Mesozoic complex are Cenozoic marine (and to a lesser extent continental) sedimentary rocks and volcanic rocks. In the vicinity of the Site, these Cenozoic rocks include consolidated and semi-consolidated Pliocene sediments belonging to the Petaluma formation and the Wilson Grove formation (formerly known as the Merced formation). Basalt and tuff deposits belonging to the Sonoma Volcanics also occur in the area. It is into these Mesozoic and Cenozoic rock formations that the ancestral Petaluma River incised a relatively deep and narrow valley during periods of low sea level in late Quaternary and Recent time. This valley was subsequently filled with Holocene and Recent alluvial and estuarine sediments when sea levels rose. The resulting valley fill consists of estuarine deposits (locally referred to as the Bay Mud) that are interbedded with (near the valley margins) and underlain by unconsolidated alluvial fan and floodplain deposits.

The Quaternary and Recent basin fill deposits and the Pliocene sedimentary and volcanic formations described above collectively contain the principal aquifers in the basin. Consolidated basement rocks of Cretaceous and Jurassic age that underlie the entire area have little permeability and form the boundaries of the groundwater flow system. The permeability and extent of water-yielding deposits in the Petaluma Valley varies considerably. Alluvial fan and stream channel deposits compose the major part of the aquifer. Locally, marine and estuarine sand deposits are an important source of groundwater. Volcanic tuff of Pliocene age in the areas of volcanic rocks also locally yields water to wells.

The Site is located in the Petaluma Valley Groundwater Basin, which is approximately 41 square miles in size. Groundwater is unconfined, or occurs under water-table conditions, in the shallow alluvial deposits exposed near the valley margins. The groundwater is confined or semiconfined in deeper parts of the basin. The continental deposits of sand and gravel are interbedded with fine-grained silt and clay, which locally may act as confining layers.

Recharge is primarily through infiltration of runoff from precipitation in the surrounding mountains: infiltration occurs along streambeds and in the permeable sediments of the valley floor at the basin margins. Most of the precipitation that falls on the valley floor evaporates or is transpired by plants, and does not significantly contribute to recharge. Thus, runoff from the mountains and infiltration through streambeds provide the largest amounts of water to the aquifer system. Groundwater flow in the aquifers is generally parallel to the long axis of the basin. Prior to development, groundwater discharged to the tidal sloughs and San Pablo Bay. Locally, groundwater may discharge to streams, springs, evapotranspiration, and wells.

The quality of groundwater is generally suitable in most areas for most purposes. However, some problems, such as locally elevated concentrations of chloride, sodium, boron, nitrate, iron, and manganese, restrict use of groundwater in some areas and for some applications. Historically, groundwater pumping reversed the freshwater gradient toward the river and induced the intrusion of saltwater in the lower parts of the Petaluma Valley. Seawater intrusion is the primary source of elevated chloride and sodium in the alluvial fan deposits at the southern end of the Petaluma Valley.

Northwest of Petaluma, nitrate concentrations are as high as three times the maximum allowed for drinking water. The probable sources appear to be septic-tank leachate as well as livestock and poultry manure that was historically placed in unlined pits in the area.

3 AVAILABLE HYDROGEOLOGIC INFORMATION

This Section briefly summarizes the primary information sources used in undertaking the hydrogeologic evaluation.

3.1 U.S. GEOLOGICAL SURVEY WATER-SUPPLY PAPER, 1958

The USGS published the results of the study "Geology and Ground Water in the Santa Rosa and Petaluma Valley Areas, Sonoma County, California" in Water-Supply Paper 1427 (Cardwell, 1958). This study focused on the subsurface geologic conditions and occurrence of groundwater, and represents the first comprehensive work on the groundwater resources of these two valleys north of San Francisco Bay. Basic geologic and hydrologic data were compiled and are tabulated in the report, including information from the logs of over 1,000 wells. The study presents a geologic map based mainly on previous work; however, the geology was "field checked, revised, supplemented, and correlated throughout the area."

The Petaluma Valley contains about 45 miles of alluvial plains of which about 10 miles south of the City of Petaluma are tidal marsh. A water level contour map for the Petaluma Valley presented in the report does not include contours for the this southern portion of the Petaluma Valley. The depth to water is reported to be within a few feet of the land surface in the valley trough and 20 to 50 feet or more on the valley slopes in this area. Wells in the southern Petaluma Valley range in depth from 15 to 736 feet, and yields range from a few gpm to 150 gpm. The aquifer system is recharged by infiltration of rainfall in outcrop areas and infiltration of streamflow from mountain runoff. Groundwater flow is generally toward the Petaluma River and down valley to discharge into San Pablo Bay. In the central and southern parts of Petaluma Valley, water levels respond to tidal fluctuations and seasonal effects are masked. In the northern part of the Valley, seasonal groundwater level trends are apparent.

Principal water-bearing deposits are the younger and older alluvium, and the Merced formation (now referred to as the Wilson Grove Formation by most references), although locally, groundwater is also produced from the Petaluma Formation and Sonoma Volcanics. Groundwater is generally unconfined or semiconfined, but may be confined and produce artesian heads locally. General descriptions of the main water-bearing formations are presented in the following paragraphs.

The Petaluma formation, of middle Pliocene age, is composed of continental and marine deposits of clay, shale, sand and sandstone, and contains some conglomerate and nodular limestone. The formation is characterized by a "great abundance of clay." The maximum thickness of this formation in the Petaluma Valley area probably exceeds 3,000 feet, and the formation is overlain by the Sonoma Volcanics, Merced formation and alluvium. Groundwater in the Petaluma formation occurs in lenses or beds of sand or poorly consolidated sandstone separated by clay beds. Yields are generally low, although in the north-central part of the Petaluma Valley, wells may obtain enough water for small-scale irrigation developments. Groundwater generally occurs under confined conditions in the Petaluma formation. Hydraulic heads are generally lower than in the overlying alluvial aquifers, in some cases by 100 feet or more; however, some wells have artesian heads apparently produced by structures related to folds and faults that have deformed or offset low permeability beds.

The Sonoma Volcanics consist of a series of lava flows, tuffs, and sediments of volcanic debris. They unconformably overlie the Petaluma formation and underlie the Merced formation, and are believed to be of late Pliocene age. Lava flows of very low permeability act as confining units which restrict the movement of groundwater; however, small amounts of water are obtained from fractures. Tuff deposits form a large part of the formation and may yield significant flows to wells that tap thick beds of tuffaceous material, generally referred to as "ash" by drillers. The thickness and distribution of the Sonoma Volcanics in the Petaluma Valley basin is not uniform, and the principal areas where wells penetrate these rocks are in the upper Petaluma Valley near Petaluma and Penngrove.

The Merced formation (later re-named the Wilson Grove formation) is of late Pliocene age and consists of massive beds of fine sand and sandstone and thin interbeds of clay and silty clay deposited under marine conditions. It includes lenses of gravel and pebble stringers, and is locally fossiliferous. Cardwell estimates the maximum thickness of the formation as 2,000 feet. He further states that the Merced formation is exposed in the vicinity of Lakeville southward to the point where the Sonoma Mountains merge with the alluvial plain, and that it contains tuffaceous strata in this region. In Petaluma Valley, the Merced deposits are described as finer grained than in the north, and both the exposed and subsurface units are described as being thinner than in Santa Rosa Valley. Information regarding the distribution and character of the Merced formation in the Petaluma Valley conflicts with, and is updated in, later reports.

The Merced formation is probably the most productive aquifer in the area, but the fine sands and the small percentage of coarse materials which characterize the formation result in only moderate transmissivity, and substantial drawdown is required to produce good yields. Based on drillers' observations, some of the most prolific water-bearing zones in the Merced formation are found in fossiliferous strata, which consist of shells and sand representative of the near-shore environment and which may include coarse sand and gravel of high permeability.

The older alluvium consists chiefly of alluvial fan deposits but includes some terrace deposits and old valley fill. These deposits are considered to be of late Pleistocene age. They consist of unconsolidated and poorly sorted sand, or sand and gravel, and interbedded silt and silty clay. The subsurface thickness in Petaluma Valley is estimated to be as much as 200 feet, with the greatest thickness occurring in the southern part of the Valley. The older alluvium is considered a principal aquifer in the upper part of Petaluma Valley, where it supplies moderate to fair well yields, but specific capacities (the amount of yield per unit of drawdown) are reported to be low.

Younger alluvium of Holocene and Recent age forms the alluvial fan deposits at the margins of the valleys and overlies the older alluvium across the valley. The younger alluvium is thickest near the Bay (possibly as much as 300 feet). The deposits are described as predominantly fine grained, and consist of silt, sandy clay, some sand and scattered beds of thin gravel. In the southern Petaluma Valley, based on the logs of a few scattered wells, the younger alluvium consists mostly of clay and silt, although scattered gravel beds have been encountered. Groundwater yields to wells in this area are relatively low.

Groundwater quality varies widely from place to place in the Petaluma Valley. In the northern part of the valley, the quality of water obtained from wells tapping the principal aquifers is good. Beginning a short distance southeast of Petaluma and continuing down-valley, many wells tap water which seems to be contaminated by intrusion of brackish bay water or unflushed connate water of similar chemical character (*e.g.*, in some parts of the Petaluma formation). In the southern Petaluma Valley, local interchange of groundwater and surface water caused by tidal fluctuations has contributed to water quality degradation, and in the vicinity of Petaluma, heavy groundwater pumping locally reversed groundwater gradients and is believed to have resulted in the intrusion of brackish water from the Petaluma River. Where water levels are depressed below sea level for considerable periods of time near tidally-influenced surface water, encroachment of sea water and eventual deterioration of groundwater quality may result. The quality of water in the alluvial aquifers is poor in a large portion of the southern Petaluma Valley; however, along the sides of the valley and locally at depth, water of fair or good quality is obtained in small quantities.

Cardwell's (1958) study also tabulated data on groundwater pumping. More recent studies, described in the following sections, update this information. Thus, a summary of Cardwell's findings on this subject is omitted here.

3.2 DEPARTMENT OF WATER RESOURCES BULLETIN 118, 1975

California Department of Water Resources (DWR) Bulletin No. 118, "California's Ground Water," identified, provided an inventory of, and profiled the status of California's groundwater basins and areas of potential groundwater storage. This report is concerned with utilization and protection of California's groundwater resources, especially with regards to basin management. It also provides a description of groundwater concepts with examples from basins in California.

The younger and older alluvium are described as the principal water-bearing materials in the Petaluma Valley groundwater basin. Average well yields are 40 gpm; the maximum reported is 650 gpm. The total storage capacity of the basin to a depth of 900 feet is estimated as 2,100,000 acre-feet. The basin has been subject to intensive development for domestic use (mostly in rural areas), and moderate development for stock watering, municipal, irrigation and industrial use. Problems identified include hard water, high chloride and total dissolved solids (TDS). It is stated that any appreciable increase in groundwater draft in the bayward segment of the basin will result in seawater intrusion. Figure 17 of this report shows the extent of seawater intrusion in Petaluma Valley as extending near the Site.

3.3 DEPARTMENT OF WATER RESOURCES BULLETIN 118-4, VOLUME 1, 1975

DWR, in cooperation with the Sonoma County Planning Department, prepared an evaluation of the groundwater resources in Sonoma County (Ford, 1975). Essentially, this report provided an update to the USGS Water-Supply Paper 1427 (Cardwell, 1958) described previously. The report describes the following conditions in the vicinity of the Site:

- The Site area is characterized as being within the Lakeville Storage Unit with a low density of wells per square mile section (five to 15) in the vicinity of the Site.
- Specific capacity of wells in the lower Petaluma Valley was reported to be typically less than 5 gpm/foot of drawdown.
- Accompanying maps show the western portion of the Site is within or immediately adjacent to an area of poor quality water, characterized by mixed sodium and

magnesium chloride (Figure 21 of the report). The Site is also mapped as being in an area where total dissolved solids in groundwater exceed 500 milligrams per liter (mg/l) and hardness exceeds 200 mg/l (Figure 22 of the report).

Ford provides updated information on the major water-bearing formations in the basin. Most notably, he accepts the assignment by others of rocks exposed along the Sonoma Mountains southeast of Lakeville to the Petaluma Formation. As discussed previously, Cardwell (1958) attributed these rocks to the Merced formation. This is significant because the Petaluma formation is noted for its low well yield. The unconsolidated materials in the Petaluma Valley are reported to include the following materials: the older alluvium described by Cardwell (1958); Holocene age alluvial fans along the eastern side of the valley that range in thickness from 50 to 200 feet; Holocene age younger alluvium that formed from floodplain deposits of major streams and is 30 to 150 feet thick; active stream channel deposits; and Bay Mud deposits.

The older alluvium is described as consisting of lenticular beds of compacted silty clay, silt, sand, and gravel, and does not produce large yields of groundwater to wells. Wells completed in the alluvial fan deposits are reported to yield adequate quantities of groundwater for domestic purposes. Well yields in the younger alluvium are reported to be highly variable.

The Bay Mud deposits occur in the tidal areas of the lower Petaluma Valley. These deposits are composed of organic clay, silt and fine sand. Peat and decomposing tules also may be present. The thickness of these deposits is up to several hundred feet; and they are underlain by the older alluvium. Bay Mud deposits are not a reliable source of potable groundwater: yields are low and water quality is poor.

3.4 DEPARTMENT OF WATER RESOURCES BULLETIN 118, 1980

In Bulletin 118-80, "Ground Water Basins in California", DWR (1980) updated Bulletin 118 in response to the Legislature's direction under Water Code Section 12924, which instructed DWR to consider political boundaries wherever practical in defining groundwater basins. As a result of this work, thirty-seven basin boundaries were changed from those in Bulletin 118 (DWR, 1975). The Sonoma County Basin was revised as follows: the older and younger alluvium and volcanics of Sonoma County were joined together into a single unit. The basin was terminated at the Marin County line based on local comments. The Petaluma Valley groundwater basin in Sonoma County, part of the San Francisco Bay Hydrologic Study Area, showed no evidence of overdraft.

3.5 DEPARTMENT OF WATER RESOURCES BULLETIN 118-4, VOLUME 3, 1982

This cooperative study, by DWR and the SCWA, was designed to augment the earlier countywide investigation by Ford (1975). The results of the study were published in five volumes: the Ford (1975) study was published as DWR Bulletin 118-4, Volume 1; Volume 2 is concerned with the Santa Rosa Plain; Volume 3 describes groundwater conditions in the Petaluma Valley; Volume 4 concentrates on Sonoma Valley, and Volume 5 pertains to the Alexander Valley and Healdsburg areas.

Volume 3 updates and summarizes information from the previous USGS and DWR studies. Average annual groundwater recharge to the groundwater basin, primarily in the Merced formation and alluvial fan deposits, as well as some Sonoma Volcanics, was estimated to be 40,000 acre-feet per year. The total storage capacity of the Petaluma Valley groundwater basin was estimated to be 1,697,000 acre-feet. The thickness of the water-yielding materials in the basin was estimated to be 0 to 660 feet. The total amount of groundwater in storage in 1980 was estimated to be 1,420,000 acre-feet, including water affected by seawater intrusion. Seawater intrusion in the southern portion of the Valley was estimated to have affected an estimated 71,000 acre-feet. The report includes a groundwater elevation contour map, but groundwater elevations were not contoured for the southern Petaluma Valley.

An updated description of the geologic units, and a new generalized geologic cross section across the valley just south of Lakeville provide an updated picture of the regional hydrostratigraphic setting in the lower Petaluma Valley. The portion of the geologic map accompanying the report that includes the Site is reproduced herein as **Figure 3**, and a copy of the geologic cross section south of Lakeville is included as **Figure 4**. The updated general hydrostratigraphy underlying the Site vicinity includes the following deposits (listed in order of increasing depth and not including Recent stream channel deposits):

- Bay Mud, consisting of mud rich in organic matter, silty mud, silt, and fine sand, occurs at the surface on the valley floor.
- Alluvial fan deposits, consisting of unconsolidated fine sand, silt, and silty clay, coarse sand and gravel, underlie the Site along Lakeville Highway. (This combines the previously described younger alluvium, alluvial fan deposits along the eastern part of the valley, and older alluvium into one unit.)

- The Petaluma formation, consisting of consolidated clay and shale with minor amounts of sandstone, is shown underlying the alluvial fan deposits at depth and cropping out east of Lakeville Highway near the Site.
- The Franciscan complex of Cretaceous age, a mélange consisting of chert, sandstone, shale, greenstone and serpentinite, presumably occurs below the drilled depth of wells in the valley fill deposits, and crops out near the summit of the Sonoma Mountains.

The report indicates that seawater intrusion and brackish connate water affect the few water-yielding zones in the Bay Mud deposits in the southern portion of the valley. In addition, seawater intrusion has affected aquifers in alluvial fan deposits near Petaluma as a result of pumping in the late 1950's and early 1960's. The Bay Mud and alluvial deposits are generally affected to shallow depths on the order of 100 feet. Connate water in the Petaluma formation is also of poor quality in some locations. Based on limited data, the extent of seawater intrusion appears to have changed little between 1962 (when municipal pumping declined and Russian River water delivery began) and 1982; however, the report identified the potential for renewed seawater intrusion near Petaluma if groundwater pumping increases to historical levels without mitigating measures. Groundwater in wells in the southern part of Petaluma Valley is reported to be of generally poor quality. Although good quality water is produced from the alluvial fan deposits at the base of the hills that border the valleys, the quantity is limited.

3.6 U.S. GEOLOGICAL SURVEY GROUND WATER ATLAS OF THE UNITED STATES – SEGMENT 1 CALIFORNIA NEVADA, 1995

The Ground Water Atlas of the United States (USGS, 1995) provides a summary of the groundwater resources in the major groundwater basins of the 50 States, Puerto Rico, and the U.S. Virgin Islands. The Atlas describes the location, extent, and geologic and hydrologic characteristics of all the important aquifers in the United States. California and Nevada compose Segment 1 of the Ground Water Atlas. The Petaluma Basin is described under the heading North San Francisco Bay Area Valleys within the section Costal Basins Aquifers. Much of the background discussion presented in **Section 2** was collected from the summary of information prepared by the authors of the Atlas.

3.7 DEPARTMENT OF WATER RESOURCES BULLETIN 118, 2003

In 1999, the Legislature approved funding and directed DWR to update the inventory of groundwater basins contained in Bulletin 118 (1975) and Bulletin 118-80 (1980). In 2001, the Legislature passed AB 599, requiring the State Water Resources Control Board to establish a comprehensive monitoring program to assess groundwater quality in each groundwater basin in the State and to increase coordination among agencies that collect groundwater contamination information. In 2002, the Legislature passed SB 1938, which contains new requirements for local agency groundwater management plans to be eligible for public funds for groundwater projects.

Petaluma Valley is (still) referred to as basin number 2-1 within the San Francisco Bay Hydrologic Region 2. The report notes that areas of high total dissolved solids (TDS) and chloride concentrations are typically found in the region's groundwater basins that are situated close to the San Francisco Bay, such as the northern Santa Clara, southern Sonoma, Petaluma, and Napa valleys (DWR, 2003).

3.8 SONOMA COUNTY GENERAL PLAN 2020 – WATER RESOURCES ELEMENT

An update to the Sonoma County General Plan 2020 Water Resources Element is currently in progress. A draft of the Water Resources Element was obtained from the Sonoma County website (www.sonoma-county.org/prmd/gp2020/draft1/water/pdf) and contains the following pertinent policies:

- Where studies or monitoring find that saltwater intrusion has occurred, support analysis of how the intrusion is related to groundwater extraction and develop a groundwater management plan or other appropriate measures to avoid further intrusion and reverse past intrusion.
- In the marshlands and agricultural areas south of Sonoma and Petaluma, require all environmental assessments and discretionary approvals to analyze and avoid an increase in saltwater intrusion into groundwater.

3.9 DWR WELL RECORDS

Completion reports for wells within approximately 2 ½ miles of the center of the Site were obtained from DWR to assess water use and lithology in the Site vicinity. Records were provided by DWR for 32 wells in the Site area. Information regarding an additional 25 wells was obtained from Cardwell (1958), DWR (1982) and the DWR website. Summary information regarding these wells is included in **Table 1**. The locations of these wells are shown on **Figure 5**. It should be noted that the current status of these wells and whether they are being actively used is not known. In addition, there may be other wells in the Site vicinity for which records were not obtained.

A total of 57 wells were identified, ranging in depth from 12 to 736 feet, with an average depth of 243 feet. Some of the well completion records include bail down test data, and the specific capacities of the wells calculated from this data range from 0.03 to 30 gpm/foot of drawdown, with an average of 1.9 gpm/foot.

Driller's logs are available for a little over half of the wells. In most cases, the logs indicate that the wells were completed in sedimentary sequences consisting primarily of clay with lesser amounts of sand and gravel. Clay-rich sequences consistent with the Bay Mud were present in the upper portions of some of the wells. It was not possible to reliably distinguish alluvial sediments from the underlying Petaluma formation based on the driller's logs. Some of the wells near Lakeville Highway are reported to bottom out in rock with descriptions consistent with the Franciscan Assemblage at depths of several hundred feet. This would suggest that those wells penetrated the Petaluma Formation; however, we lack the means to confirm this interpretation. Two wells in the hills east of Lakeville Highway are reported to be completed in fractured bedrock.

3.10 INFORMATION OBTAINED FROM HYDROSCIENCE

Hydroscience installed a water production well at the Site (well North #2) and performed aquifer tests on this well and also a well that had previously been installed at the Site (well North #1) and a well located approximately 1 mile southeast of the Site (well South #1). In addition, HydroScience measured water levels in these wells and in other wells at the Site and a cluster of four wells located approximately 0.8 mile east of the Site. The locations of these wells are shown on **Figure 5**. The findings of HydroScience's work were not published in a report; however, HydroScience provided two memoranda regarding their work (HydroScience, 2003 and 2005a), spreadsheets containing water level and discharge measurements taken during the pumping tests, and a series of construction field records for installation of well North #2.

Information available regarding the wells identified by HydroScience is summarized below. The results of water level measurements in these wells are discussed in **Section 4.3** and the results of aquifer tests are discussed in **Section 4.4**.

- *North #1:* This well was installed prior to 2003. Well construction details and a lithologic log were not available; however, a borehole geophysical log (spontaneous potential (SP) and short and long normal resistivity) extending to a depth of 332 feet was provided. The geophysical log suggests the well is completed in interbedded fine gained deposits and intermittent beds containing a significant coarser grained fraction below a depth of 180 feet. The SP log reportedly reflects a lower TDS content than the

SP log for well South #1. A pump test data form for this well indicates the pump was set at 413 feet bgs during the test, indicating the well must be at least that deep.

- *North #2:* This well was installed under the supervision of HydroScience in 2003. A lithologic log and geophysical log were not available for this well; however, a well completion diagram was provided and additional information regarding well completion was provided in a memorandum (HydroScience, 2003) and the field construction records provided by HydroScience. The borehole for the well was drilled using the mud rotary method to a depth of 650 feet, where "bedrock" was encountered. The borehole penetrated Bay Mud to a depth of approximately 150 feet, which was reportedly underlain by alternating layers of clay and alluvial gravel deposits. The well was subsequently installed to a depth of 650 feet and screened at intervals between 160 feet and 630 feet bgs.

- *Observation Well:* An observation well was reportedly drilled approximately 100 feet east of North #2 in 2003 under the supervision of HydroScience (HydroScience, 2005b). A lithologic log and completion details for the well were not available; however, the following information was obtained from HydroScience (2005b). The lithologic log for this well reportedly indicates that Bay Mud is present to a depth of 46 feet and is underlain by alluvium. The well was reportedly completed to a depth of 60 feet bgs (HSE, 2005a).

- *South #1:* Well South #2 was installed prior to 2003 approximately 1 mile southeast of the Site. No lithologic log, geophysical log or well completion details were available; however, details were discussed in HydroScience's 2003 memo. HydroScience indicated that the geophysical log for this well indicates the presence of water bearing strata starting at a depth of 180 feet, with significant zones present between 300 and 380 feet. The SP log for this well indicates a higher TDS content than North #1.

- *Four Existing Wells:* Four existing wells were reported by HydroScience as being located in a cluster astride Highway 37 approximately 0.8 mile east of the Site. All four of these wells were reportedly completed in the alluvial aquifer at depths ranging from 207 to 278 feet. Well construction details or logs were not available.

4 SITE EVALUATION

4.1 GEOLOGY AND HYDROGEOLOGY

The following account of the geology and hydrogeology of the Site is interpreted mainly from the information sources noted in **Section 3**, supplemented by other data as referenced.

The Site is underlain by three hydrostratigraphic units that are of importance to this study: the Bay Mud, alluvial deposits, and the Petaluma formation. The actual depth and lateral extent of each unit beneath the Site is unknown and cannot be determined from the available data.

Bay Mud deposits (Holocene). The Bay Mud, which consists of estuarine and marsh deposits composed of mud, silty mud, silt, and small amounts of sand, all rich in organic matter, underlies the southern Petaluma Valley (DWR, 1982). Artificial fill covers the Bay Mud in many areas. Sea water was trapped in the Bay Mud as it was deposited, and water pumped from this formation is generally brackish. The thickness of the Bay Mud deposits between the Site and San Pablo Bay ranges from approximately 50 to at least 239 feet based on published information (Ford, 1975) and well logs obtained from DWR. However, Ford (1975) indicated that the Bay Mud deposits may be up to "several hundred feet thick." These deposits have a low permeability and the few water yielding zones are generally assumed to lack horizontal and vertical continuity, although the data are limited. For example, DWR (1982) reported that no hydrographs were available for the few shallow wells in the lower Petaluma Valley that pump from the Bay Mud deposits. Due to the low permeability of these deposits, the hydraulic connection between the Bay Mud and the underlying alluvial fan deposits is expected to be relatively weak; however, it has been recognized that significant pumping from the underlying deposits could induce leakage of brackish water from the Bay Mud into the alluvial fan deposits. The thickness of Bay Mud encountered at the Site varies from 0 feet near the Lakeville Highway to approximately 150 feet at the western extent of the Site at well North #2.

Alluvial deposits (Late Pleistocene and Holocene). Alluvial deposits occur at the surface along the eastern edge of the southern Petaluma Valley in the vicinity of the Lakeville Site, as shown on the geologic map (**Figure 3**) and extend beneath the Bay Mud as shown on the geologic cross section (**Figure 4**). These deposits consist of poorly sorted heterogeneous mixtures of clay, silt, sand and some gravel. The proportion of sand and gravel is expected to be greater closer to the mountains; even so, review of local driller's logs indicates that fine grained deposits are still abundant in these areas. The thickness of the younger and older alluvium is reported to be as much as 200 and 300 feet in the southern Petaluma Valley, respectively (Cardwell, 1958); however, the alluvium may be difficult to distinguish from the underlying Petaluma formation in driller's logs (**Section 3.9**). In addition, the published literature contains conflicting information regarding the thickness of the alluvium in the vicinity of the Site and the depth at which the underlying Petaluma formation may be encountered. For these reasons, the true thickness of alluvium at the Site cannot be determined without further study.

Yields from the alluvial aquifer are variable, but it is generally thought to be capable of supporting domestic uses and small irrigation projects. Although the alluvial deposits are quite heterogeneous, they are regarded as a single aquifer unit for the purposes of this study. Groundwater within the alluvial aquifer may occur under unconfined conditions near the heads of alluvial fan deposits, but semi-confined and possibly confined conditions are expected to occur in distal areas where finer grained deposits are more prominent or where the formation is overlain by the Bay Mud.

Water quality data indicate seawater intrusion has occurred locally into alluvial deposits near and possibly adjacent to the Site (DWR, 1982). Although groundwater from the alluvial deposits, especially near the valley margins and sometimes at depth beneath the Bay Mud, is often of good quality, it is of limited extent (Ford, 1975; DWR, 1982).

Petaluma Formation (Pliocene). The Petaluma formation is exposed in the mountains to the east of the Site (**Figure 3**). As shown on the geologic cross section (**Figure 4**), it also underlies the alluvium beneath the Site. It consists chiefly of shallow marine to brackish-water deposits and continental deposits of clay, shale and sandstone, with lesser amounts of conglomerate and nodular limestone (DWR, 1982). Near Lakeville, a 1,059 feet thick section was measured which contained 70 percent clay, shale, and clayey or shaley beds (DWR, 1982). As mentioned above, the contact between the alluvium and the underlying Petaluma formation may be difficult to distinguish from drill cuttings; therefore, the depth at which the Petaluma formation occurs beneath the Site is not known. It seems likely that some of the deeper wells in the Site's vicinity may include screened zones in the Petaluma formation.

Because of its large percentage of clay, the Petaluma formation is generally of low permeability; however, wells drawing from coarser grained units can yield moderate flow rates. Hydraulic heads are often variable in wells tapping the Petaluma formation, indicating that the water-bearing coarser gained lenses can be hydraulically isolated by the abundance of clay. In these water-bearing lenses, groundwater can occur under confined and semi-confined conditions.

Groundwater quality in the Petaluma formation varies with the degree of continuity of its included aquifers. At the southern end of the Petaluma Valley, and underlying or near the Site, a zone of poor quality groundwater has been identified at depths from approximately 150 to 700 feet bgs that extends from approximately 1 mile south of Lakeville to at least the intersection of Highway 37 and Lakeville Road (DWR, 1982). This poor quality water has been variously interpreted as connate water from sea water trapped in the fine-grained sediments of the Petaluma formation at the time of deposition (DWR, 1982), or the result of seawater intrusion into the alluvial aquifers (Cardwell, 1958; Ford, 1975).

4.2 GROUNDWATER USE IN THE PETALUMA VALLEY BASIN

The Petaluma Valley Basin has been subject to intensive development for domestic use (mostly in rural areas), and moderate development for stock watering, municipal, irrigation and industrial use (DWR, 1975a). There are no municipal water supply wells in the lower Petaluma Valley. The closest municipal wells serve the City of Petaluma, approximately 9 miles north of the Site (USGS, 1982). Historically, the City of Petaluma relied extensively on groundwater for its water supply, and water levels fell from the mid-1950s through the early 1960s causing water quality problems due to intrusion of brackish water from the tidal portions of the Petaluma River. Imported water from the Russian River became available in 1962, and by 1980, Petaluma met 15 percent of its water demand with groundwater. At that time, the total annual groundwater pumpage in the basin was estimated as 7,800 acre-feet.

The California Water Code requires the DWR to publish an update of the California Water Plan every five years. This is accomplished through the Bulletin 160 series, which includes an evaluation of agricultural, urban and environmental water uses throughout the state during the study period as well as projected future uses. The most recent update to the California Water Plan was issued in Bulletin 160 Update 2005. WorleyParsons Komex contacted DWR staff to obtain spreadsheets summarizing recent and projected water use data used in this update (DWR, 2006). The spreadsheets included data for the South Sonoma Detailed Analysis Unit (DAU), which includes the Petaluma Valley, the Sonoma Valley, and surrounding mountain ranges. Based on data compiled by the DWR in these spreadsheets, the municipal and industrial groundwater pumping in the South Sonoma DAU in 1999 was 6,000 acre-feet, while the agricultural groundwater pumping was 16,000 acre-feet, for a total of 22,000 acre feet. By 2020, municipal and industrial groundwater use is expected to decrease to 400 AFY, while agricultural groundwater use is expected to decrease to 5,500 AFY, for a total of 5,900 AFY.

The spreadsheets obtained from DWR do not break down groundwater use between the Petaluma Valley and other areas, but it is instructive to note that the estimated groundwater use in the Petaluma Valley basin represents approximately 35 percent of the estimated groundwater use in the South Sonoma DAU in 1999. In the last several decades, the City of Petaluma has relied on surface water deliveries from SCWA, and has used its groundwater wells only for emergency backup purposes (Dyett & Bhatia, 2006), so the municipal and industrial groundwater pumping reported in the DWR spreadsheets is comprised primarily of groundwater pumping by other municipal water agencies in the DAU. Projected groundwater demand by the City of Petaluma is discussed further below. Agricultural groundwater demand in the DAU is projected to decrease by 63 percent from 1999 levels by 2020, indicating that a significant reduction in agricultural groundwater pumping in the Petaluma Valley basin may be expected during this time period. Based on conversations with DWR staff, this is due largely to expanding urbanization.

As mentioned above, the City of Petaluma has relied on groundwater only for emergency backup purposes for the last several decades. Reportedly, local groundwater, when used, is drawn from wells completed at depths over 400 feet (City of Petaluma, 2006). Based on projected urban growth, the City's water demand is expected to increase from 11,000 AFY during 2002 to 15,650 AFY by 2025 (Dyett & Bhatia, 2006). Future water demand is expected to exceed the City's allotment under the Restructured Agreement for Water Supply with SCWA as early as 2007 during peak demand periods (maximum allotment of 21.8 mgd) and as early as 2010 for annual demand (capped at 13,400 AFY). The City intends to meet the projected shortfall in demand through increased reliance on recycled water, expansion of water conservation efforts and modest reliance on groundwater. Through 2023, the City plans to continue relying on groundwater emergency backup only; however, beginning in 2024, the City projects it may use up to 186 AFY of groundwater for supply during peak demand periods.

The SCWA plans to expend its allotted deliveries to its customers through implementation of the Water Supply and Transmission System Project (WSTSP). In 2003, SCWA suffered a significant setback in its proposed expansion plans, when the First District Court of Appeals ruled against SCWA in the case *Friends of the Eel River et al. v. SCWA and Pacific Gas and Electric Company*. The lawsuit had been filed by Friends of the Eel River in 1999, and challenged SCWA's WSTSP, which sought to increase SCWA's annual diversion of water from the Russian River by 33 percent (Friends of the Eel River, 2003a). The Sonoma County Superior Court ruled against Friends of the Eel River in August 2000, but the decision was reversed by the First District Court of Appeals in its 16 May 2003 ruling (*Friends of the Eel River v. Sonoma County Water Agency*, 108 Cal. App. 4th 859). The ruling concluded that the EIR submitted by the SCWA for the WSTSP was inadequate. The effect of the ruling was to postpone the project, while SCWA prepared a supplemental EIR (SCWA, 2004b). Recently, SCWA has announced that an entirely new EIR will be prepared for a modified project known as the Water Supply, Transmission, and Reliability Project (Winzler & Kelly, 2005). This EIR is currently expected to be completed in October 2007 (Dyett & Bhatia, 2006). In the meantime, the Restructured Agreement for Water Supply states that SCWA will not be liable in the event it is not able to meet its customer's entitlements due to drought, environmental laws or regulations, or other causes beyond SCWA's control, and prioritizes delivery of available water if the entitlements cannot be met.

The Site area is within the Lakeville Storage Unit with a low density of wells per square mile section (five to 15) in the vicinity of the Site (DWR, 1975b). Well construction records were requested from the DWR for water supply wells completed in the vicinity of the Site. **Figure 5** shows the locations of wells on file with DWR within approximately 1 ½ miles of the Site, in addition to wells identified in the general vicinity in previous USGS (Cardwell, 1958) and DWR (1982) studies. **Table 1** lists pertinent data for these 57 wells, including well use and depth information where available. To our knowledge, historical pumping records are not available for these wells. It should be noted that the current status of these wells and whether they are being actively used is not known. In addition, there may be other wells in the Site vicinity for which records were not obtained.

The wells range in depth from 12 to 736 feet, with an average depth of 243 feet. Most of the wells are located along the base of the mountains or on the valley floor, and are completed in alluvial fan deposits underlying the Bay Mud. Various deeper wells, including some wells along the mountain front and those within the Sonoma Mountains, are likely completed in the Petaluma formation. Note that depth to the contact between the alluvial fan deposits and the Petaluma formation varies with location, and cannot be readily distinguished based on the available data (see **Section 3.9** for a discussion of information from driller's logs). Some shallow wells on the alluvial plain are likely completed in the Bay Mud.

Reported well uses include domestic (35), stock watering/dairy (six), irrigation (two), monitoring (two) and "other" (one). In addition, the use of eight of the identified wells is not reported, and three of these wells are reported as being abandoned. Reported yields varied from 1 to 70 gpm based on short term well pumping tests performed at or around the time the well was completed, and specific capacities may be characterized as low to moderate (**Section 3.9**).

4.3 GROUNDWATER LEVELS

Information on groundwater levels is limited for the Site area. Many wells have associated water level data, but in most cases this consists of a single measurement taken at the time of well installation. In most cases it is unknown whether these water level data represent "water strike" levels during well drilling, or water levels measured after well installation, and in the latter case whether the levels are static or represent pumping conditions. Such data may not be representative of true conditions and must be used with caution.

HydroScience measured water levels in three wells at the Site: North #1, North #2 and an old agricultural well (possibly well number 53 or 54 on **Figure** 5). In addition, HydroScience measured water levels in well South #1, located approximately 1 mile southeast of the Site and four existing wells located approximately 0.8 mile east of the Site (HydroScience, 2003). A summary of HydroScience's water level measurements is presented below.

Well Designation	Well Depth (feet)	Measured Water Level (feet bgs)
North #1	>413	Flowing
North #2	650	Flowing
South #1	380	Flowing
"Existing Well"	207	0
"Existing Well"	278	8
"Existing Well"	250	15
"Existing Well"	250	15

Cardwell (1958) noted that groundwater levels in southern parts of Petaluma Valley respond to tidal loading, depending on the proximity of tidal channels and the degree of hydraulic connection between the aquifer materials tapped by the well and the tidal channels. In some wells, the response due to the tides masked the seasonal fluctuations; in other wells, typically deeper wells and wells farther removed from the tidal sloughs, seasonal fluctuations dominated the water level changes. Wells influenced by the tides would be expected to show a diurnal pattern, however regional water level data are not typically available on a daily basis. Cardwell

(1958) reported observations of tidal response in well 03N06W05A001M in January 1950 where the change in tide stage over a 15-hour period was 3.81 feet and the water level change observed in the well was 0.23 feet. This well is 83 feet deep, perforated in a gravel stratum (between 77 and 83 feet) within the Bay Mud formation, and is located only about 100 feet from a tidal slough on the west side of the Petaluma River (near well 12 on **Figure 5**).

Only a handful of wells in the vicinity of the Site are routinely monitored; these five wells are shown on **Figure 5** and include well numbers 9, 14, 18, 41, and 56 (see **Table 1** for the corresponding State Well Number). These wells are included in DWR's monitoring network and the data are available online. However, the available data are intermittent. These data are discussed in the following paragraphs.

Well 03N06W11L001M (map number 14) is located near where the Petaluma River meets the San Pablo Bay. DWR (1982) reported the depth of this well to be 520 feet (**Table 1**), but no descriptions of the lithology or well completion were discovered. The regional geologic setting, the depth of the well, and its hydrograph suggest that this well is completed in the alluvial fan deposits underlying a thick (approximately 200 feet) section of Bay Mud deposits. Water level measurements are available from 1990-95. The average depth to water in the well over this period was approximately 2 feet bgs.

Well 03N06W01Q001M (map number 9) is located between the Site and San Pablo Bay. USGS (1958) reported the depth of this well to be 225 feet (**Table 1**), but no descriptions of the lithology or well completion were discovered. The regional geologic setting, the depth of the well, and its hydrograph suggest that this well is completed in the alluvial fan deposits underlying Bay Mud deposits. Water level measurements are available from 1950 through 1999 although data for a few years in the late 1980's and early 1990's are missing. The average depth to water in the well over this period was approximately 0.5 feet bgs.

Well 04N06W36N001M (map number 56) is located near the southeast corner of the Site where the Bay Mud is absent. We were able to obtain a Water Well Drillers Report from DWR, which indicates that the depth of this well is 50 feet (**Table 1**), and the well is apparently completed in the alluvial fan deposits. Water level measurements are available from December 1989 through April 1994. The average depth to water in the well over this period was approximately 19 feet bgs.

Well 04N06W27B001M (map number 41) is located about 1 mile to the north- northwest of the Site off Old Lakeville Road. We were able to obtain a Water Well Drillers Report from DWR, which indicates that the depth of this well is 150 feet (**Table 1**), and the well is apparently completed in the Petaluma formation – based on its location relative to the contact between the alluvial fan deposits and the Petaluma formation (see Blake et al., 2000 and Graymer et al., 2002). Water level measurements are available from October 1980 through March 1998. The average depth to water in the well over this period was approximately 29 feet bgs.

Well 04N06W21A001M (map number 18) is located about 2 miles to the north- northwest of the Site. DWR (1982) reported the depth of this well to be 259 feet (**Table 1**), but no descriptions of the lithology or well completion were discovered. Based on its location, the well is apparently completed in the Petaluma formation. Water level measurements are available from December 1989 through October 2000. Average depth to water in the well over this period was approximately 59 feet bgs.

Hydrographs of the wells with historical water level measurements by DWR are shown on **Figure 6** for wells completed in the alluvial aquifer and **Figure 7** for wells completed in the Petaluma Formation.

It is difficult to correlate the water level records for the three wells completed in the alluvial aquifer (**Figure 6**) because (1) the periods of record differ, (2) the wells are completed at varying depths, and (3) the wells are located remote from one another. Although two of the wells are located relatively close to the bay, they are completed at different depths; the third well is located near the base of the mountains at a shallow depth. There are only two water level measurements for well 03N06W01Q001M in the period which covers the available data for wells 03N06W11L001M and 04N06W36N001M. And while measured water level data in wells 03N06W11L001M and 04N06W36N001M are available for a common period (1989-1995) and the wells are completed in the same aquifer, one is located at the apex of the fan deposits and the other is located at the distal end; therefore, these wells would be expected to respond differently to the different hydrologic phenomena in their locality (e.g., proximity to recharge and discharge zones).

Despite the limitations noted above, the following observations may be made. Water levels in the alluvial wells are all relatively close to sea level and generally fluctuate within a relatively narrow range of a few feet. Both seasonal and climatic trends are apparent. Comparison with the cumulative departure from average annual precipitation plot, also shown on **Figure 6** for convenience, indicates that each well responds to periods of below and above average precipitation. This observation is consistent with the interpretation that these wells are completed in the alluvial aquifer, which is recharged along the mountain fronts from runoff due to precipitation in the Sonoma Mountains. Also, during dry periods increased groundwater pumping may have occurred in the area, contributing to this trend. Over time, groundwater levels in well 03N06W01Q001M appear to be increasing; however, this increase is very slight (at most 3 feet between 1950 and 2000) and therefore may not be significant.

A much greater difference in water levels (approximately 60 feet) is apparent for the two wells completed in the Petaluma formation (**Figure 7**). In addition, water level fluctuations are much greater for the wells completed in this formation. Water level fluctuations range from approximately 16 feet for well 04N06W21A001M to over 110 feet for well 04N06W27B001M. This is consistent with the interpretation that the clayey character of the Petaluma formation can result in hydraulic isolation of individual water-bearing zones. Both seasonal and climatic water level trends are apparent in well 04N06W27B001M. During dry periods, the amplitude of seasonal water level changes appears to increase substantially and water levels decrease, potentially due to increased pumping. Climatic trends are less apparent for well 04N06W21A001M; however, a dip in water levels in 1990 may coincide with below average precipitation. Both wells exhibit a slight declining trend in water levels from 1990 or 1991 to near present time that appears unrelated to climatic influences and may be related to regional groundwater pumping.

4.4 HYDROGEOLOGIC CHARACTERISTICS OF THE AQUIFERS

Hydrogeologic parameters of the aquifers beneath the Site are available from three sources:

1. Specific capacities for wells near the Site reported by HydroScience;
2. Data from pumping (bail down) tests of wells in the Site vicinity for which data were obtained from DWR; and
3. Analysis of pumping test data provided by HydroScience for two wells at the Site (North #1 and #2) and one well located approximately 1 mile southeast of the Site (South #1).

The following table summarizes specific capacities reported by HydroScience for wells in the Site vicinity.

Well Designation	Well Depth (feet)	Formation	Estimated Specific Capacity (gpm/foot)
North #1	>413	Alluvium/Petaluma Formation	0.3
North #2	650	Alluvium/Petaluma Formation	Not Estimated
South #1[1]	380	Bay Mud / Alluvium	3
"Existing Well"	207	Alluvium	0.8
"Existing Well"	278	Alluvium	1.0
"Existing Well"	250	Alluvium	0.3
"Existing Well"	250	Alluvium	0.1

1. This well was reported by HydroScience as being completed in the Bay Mud (HydroScience, 2003); however, based on information regarding the four "Existing Wells" and driller's logs for other wells located in close proximity to San Pablo Bay, it seems reasonable to assume that at the alluvial aquifer underlying the Bay Mud has been penetrated.

The specific capacities reported by HydroScience may be characterized as consistent with low to moderate yields, which is consistent with the published data and information available from DWR records. Pump (bail down) test data were obtained form DWR for 24 wells. The calculated specific capacities at the time of installation ranged from 0.03 to 30 gpm/foot of drawdown, with an average of 1.85 gpm/foot. The maximum value was for a single well, and the next highest value was 3 gpm/foot. When the highest value is ignored as anomalous, the range is 0.03 to 3 gpm/foot with an average of 0.6 gpm/foot.

Data for pumping tests performed on wells North #1, North #2 and South #1 in 2003 were provided by HydroScience. Pumping tests performed included a constant discharge test (with recovery) on North #1, a step drawdown test and constant discharge test (with recovery) on North #2, and a constant discharge test (with recovery) on South #1. Drawdown was measured in the pumping wells only. The data provided by HydroScience, including drawdown and discharge measurements for the pumping wells, are included in **Appendix A**.

We analyzed the time-drawdown data using the Cooper-Jacob method (Driscoll, 1986) and the results are summarized in the table below. Figures showing the corresponding semi-log time-drawdown graphs are presented with the field data in **Appendix A**. In the table below, the reliability/quality of the pumping test results is ranked based on the stability of pumping rates, degree of recovery from previous pumping at the start of the test, apparent well efficiency, potential boundary effects, and potential interference drawdown effects. Based on our review, the constant discharge test data for North #1 and the constant discharge and recovery data for South #1 represent the most reliable data sets for evaluation of aquifer parameters (however South #1 is one mile off-Site and is thus not representative of conditions at the Site). The step-drawdown transmisssivity value for North #2 (45.5 square foot per day (ft^2/day)) is essentially the same as the constant discharge values for North #1 (about 50 ft^2/day).

Test Well	Test Type	Discharge Rate (gpm)	Transmissivity (ft^2/day)	Confidence/ Data Quality
North #1	48-Hour Constant Discharge	90 (reduced to 75 gpm after 105 minutes)	55.3 (47.1) [1]	Good
North #1	Recovery	76	37.4	Fair
North #2	Step Drawdown	100	45.5	Fair
North #2	48-Hour Constant Discharge	125	26.5	Poor
South #1	48-Hour Constant Discharge	200	779	Excellent
South #1	Recovery	200	1049	Excellent

1. The parenthetical value was calculated for the lower discharge rate.

We note that the constant discharge tests for North #1 and South #1 started on the same day. As a result, the North #1 well apparently detected interference drawdown from the South #1 test about 2,100 minutes after pumping began at North #1. If this interpretation is correct, the recovery data from North #1 are less reliable than the other aquifer test data. It is unusual for the recovery analysis in a pumping well to give a smaller transmissivity than the drawdown analysis. An alternate (less likely) interpretation of the slope break at 2,100 minutes in North #1 is that a barrier boundary such as a bedrock contact was encountered.

Based on the analysis of the pumping test data for North #1 and #2, the estimated transmissivity of the wells completed in the alluvial aquifer/Petaluma formation at the Site is approximately 50 ft^2/day. This is a relatively low value and corresponds with a hydraulic conductivity of approximately 0.25 feet/day, assuming the wells are screened in 200 ft of permeable alluvium. The inferred hydraulic conductivity of approximately 0.25 feet/day is consistent with screened intervals dominated by fine grained (silt and clay) sediments. The transmissivity (and hydraulic conductivity) calculated for South #1 is more than one order of magnitude higher, which is consistent with a system dominated by moderately permeable sediments, such as silty sand.

In conclusion, the best available data for estimating aquifer parameters the Site are the constant discharge transmisssivity value for North #1 and the step-drawdown value for North #2. Thus the analytical-element model described in **Section 5** was developed using a transmissivity value of 50 ft^2/d, based on results of these two tests. The hydraulic conductivity value inferred from this transmissivity value was subsequently adjusted during model development, as described in **Section 5**.

5 POTENTIAL IMPACTS OF USING GROUNDWATER TO SUPPLY THE PROPOSED DEVELOPMENT

5.1 PROPOSED DEVELOPMENT AND WATER SUPPLY REQUIREMENTS

The proposed development includes the establishment of a hotel and casino on the Site with a water demand that will be met through groundwater pumping at a rate of 200 gpm (HydroScience, 2005b). The actual rate of pumping at the Site may vary depending on varying water demand, and 200 gpm represents a long-term average pumping rate. HydroScience (verbal communication) proposes using existing wells North #1 and North #2, possibly augmented by additional wells, if needed, to support the required pumping rate. As discussed in **Section 4**, the Site is underlain by three water-bearing deposits. The Bay Mud is characterized by low permeability and poor water quality, and therefore is not considered suitable as a water supply for the project. The alluvial aquifer and underlying Petaluma formation are expected to yield higher quantities of better quality groundwater than the Bay Mud, and are therefore proposed to be developed for the project's water supply.

5.2 DRAWDOWN MODEL DEVELOPMENT

The following factors will affect the drawdown impacts resulting from development of the proposed water supply at the Site:

- the design of the pumping well(s) and the rate of groundwater extraction;
- the hydrogeologic characteristics of the aquifer that is pumped;
- the geometry and boundary conditions of the aquifer that is pumped;
- the rate and location of recharge; and
- the rate of groundwater underflow down the axis of the valley at the northern end of the model boundary.

Evaluating drawdown impacts for the proposed project is complicated by fact that the Site is located in a relatively complex area hydrogeologically, and relatively little information is available to allow accurate evaluation of certain factors that influence drawdown impacts. The water supply well design and water demand are understood, and data are available to estimate the hydrogeologic properties of the aquifer to be used for water supply with adequate confidence for a reconnaissance-level evaluation (**Appendix A**), but information regarding recharge rates, groundwater underflow at the northern model boundary, groundwater leakance rates through the Bay Mud and other potential confining deposits, and the amount of interaction between groundwater and surface water in the Petaluma River and San Pablo Bay was not available. The evaluation is further complicated by effects resulting from the presence of "no-flow" boundaries at bedrock contacts along both flanks of the valley, and the fact that the hydraulic relationship between the alluvial aquifer and the Petaluma formation has not been clearly delineated. Further, the limited yield of the North #1 well compared with the South #1 well (**Section 4.4**) suggests it is possible the North #1 well either has poor efficiency (and thus the transmissivity of the aquifer has been underestimated) or that its transmissivity could be more representative of the Petaluma Formation than the alluvial aquifer.

Because of these data limitations, reliable prediction of drawdown impacts from the development is not possible without further investigation. If the transmissivity of the alluvial aquifer is greater than calculated based on the North #1 well pumping test, then drawdown will be less, and the drawdown cone associated with pumping the proposed Site wells will extend for a greater distance from the Site. Ultimately, the cone of depression would expand until it captures enough areal and mountain front recharge and leakance from surface water bodies (San Pablo Bay and the Petaluma River) to equal the groundwater pumping rate. Alternatively, if the North #1 pumping test accurately reflects the transmissivity of the alluvial aquifer and surface water leakance rates are higher, pumping at the Site will induce a relatively deep cone of depression between the edge of the valley and the Petaluma River, and brackish surface water will be drawn into the aquifer.

To gain further perspective on these potential impacts, we have developed an analytical element model to simulate the latter case, which would result in greater drawdown and greater potential for groundwater degradation due to intrusion of brackish water. The uncertainties associated with this model are then discussed in a qualitative sense. Performing a modeling analysis of drawdown aids in developing an understanding of (1) the general range of impacts that can be reasonably expected, (2) the major assumptions that exist in the evaluation, and (3) the resulting inherent uncertainties in the model's results. What the modeling cannot do in this context is to predict the exact drawdown at any given location.

To evaluate potential drawdown impacts, the WhAEM2000 code was used (Kraemer, Haitjema, and Kelson, 2004). This code was developed by the U.S. Environmental Protection Agency (EPA) Office of Research and Development and is available on EPA's website. WhAEM is an analytical element modeling code developed for wellhead protection simulations. It has the ability to simulate the effects of multiple pumping wells, no-flow boundaries, and "line sinks" which can be used to simulate rivers and other surface water bodies. The model calculates what the steady-state (long-term stabilized) water level elevations would be in the model domain under the assumed pumping, aquifer and boundary conditions. More information about these assumptions and their implications are discussed in **Sections 5.2.1 and 5.2.2.**

5.2.1 MODEL INPUTS

Pumping was assumed to take place from North #1 and North #2 at a combined rate of 200 gpm (to represent the average long-term sustained pumping rate for the development). The depth of the aquifer was assumed to be 500 feet. A no-flow boundary was set to the east of the Site, along the assumed contact between the alluvial aquifer and the Petaluma formation. The Petaluma River and San Pablo Bay were modeled as line sink boundaries (*i.e.*, the groundwater table was connected to a surface water body through a "riverbed layer" having a specified resistance to infiltration flow). The following table summarizes the parameters incorporated into the analytical element model for groundwater extraction at the Site.

Parameter	Modeled Value	Remarks
Pumping Rates	North #1 = 90 gpm North #2 =110 gpm	The pumping rates were selected to meet the water demand for the project (HydroScience 2005b) and were adjusted during model calibration.
Base Elevation	-500 feet amsl	The modeled base of the aquifer is at approximately the mid-point in depth between North #1 and North #2.
Aquifer thickness	200 feet	The effective thickness of the aquifer from which pumping takes place is set as 200 feet, based on the screened intervals of the pumping wells. This is in the range of what is expected at the Site.
Hydraulic Conductivity	0.5 feet/day	The hydraulic conductivity for the aquifer used in the model is estimated by dividing the transmissivity estimated from the analysis of the North #1 and #2 pumping tests (50 ft^2/d) (**Section 4.4**) by the aquifer thickness (200 ft). The resulting value of 0.25 ft/d was multiplied by 2.0 during model development. The reason for this and the resulting implications are discussed in the text of this report, below.

Parameter	Modeled Value	Remarks
Boundary Head (Line Sink Boundary)	0 feet amsl	In the model, groundwater elevations at the Petaluma River and San Pablo Bay are held constant at sea level. The model simulates a "riverbed layer" between the surface water body and the aquifer below. The riverbed layer has a specified resistance to infiltration flows.
Boundary Resistance	50 days	These parameters simulate the river bed sediments and Bay Mud below the Petaluma River and San Pablo Bay that insulate these surface water bodies from the underlying alluvial aquifer, and determine the rate at which surface water from these sources would percolate into the subsurface when adjacent water levels are lowered.
Boundary Width	200 feet	
Boundary depth (thickness)	1 foot	

The assumed model boundary conditions and parameter inputs and associated implications are discussed below.

Boundary Conditions: To model the groundwater flow system in the vicinity of the Site, a no-flow boundary was placed to the east near the mapped location of the contact between the alluvial fan deposits and the Petaluma formation (**Figure 3** and **Figure 8**). It is possible that the lower portions of the production wells at the Site draw water from the Petaluma formation, in which case the no-flow boundary might be better placed at the contact between the Petaluma formation and the Franciscan Assemblage, approximately ½ mile to 1 mile further to the east. Moving the no-flow boundary further to the east would extend the prediction of drawdown impacts to include areas where the Petaluma formation crops out at the ground surface east of Lakeville Highway; however, it is not likely to significantly influence the magnitude of steady state drawdown calculated near the Site. For this reason, although the drawdown contours generated by the model are terminated near the alluvium-Petaluma formation contact, similar drawdown impacts are possible in areas further to the east.

A no-flow boundary may also be inferred to exist at the contact between alluvial fan deposits and the Franciscan Assemblage on the western flank of Petaluma Valley (**Figure 3**); however, the Petaluma River is present between this contact and the Site and may also form a boundary to groundwater flow. Under non-pumping conditions, the Petaluma River may form a boundary if groundwater discharges to the river. As drawdown occurs, the river could form a boundary if recharge induced from the river as water levels drop is sufficient to maintain groundwater levels near the ground surface at the river, or creates a groundwater ridge or divide beneath the river. On the other hand, if little or no recharge occurs through the river bed and underlying Bay Mud, the cone of depression would propagate beneath the river until it reaches the no-flow boundary on the other side of the valley.

Scenarios between these two endpoints are also possible (and may be likely), where the cone of depression propagates beneath the river at depth but the river forms a groundwater divide in the shallow subsurface. Similarly, the cone of depression that results from pumping at the Site may be expected to migrate for some distance beneath San Pablo Bay. The degree to which the cone of depression would migrate beneath the river and the bay depends on the rate of recharge induced from these surface water bodies, which in turn depends on the permeability of the underlying sediments and whether any coarser-grained pathways exist along which vertical groundwater flow could occur. The rate of recharge that can be induced from the river and Bay by subsurface pumping at the Site cannot be estimated based on the available data; however, since seawater intrusion likely occurred in the past, it is probable that at least some recharge would be induced.

For the purposes of this evaluation, we have modeled the Bay and the River as line-sink boundaries and we have assumed that constant-head wells can be used to simulated the effects of underflow from the north. Areal recharge is neglected, see **Section 5.2.2.** Under a scenario where the rate of induced recharge is less than the pumping rate, the cone of depression would continue to expand until it covers an area with equivalent recharge from other sources. The result would be a broader cone of depression. A more sophisticated model, and knowledge regarding recharge rates and distribution, as well as the rate of groundwater flow coming down the valley, would be needed to model such a scenario in a meaningful way.

Pumping Rates: The two proposed wells were modeled at a combined pumping rate of 200 gpm (the long-term average sustained pumping rate), divided between the wells in accordance with the depths and well capacities estimated by HydroScience (2003). If the two modeled wells cannot support the required water demand for the project, additional wells could be added but would not significantly change the distribution of drawdown in the area.

Base Elevation and Aquifer Thickness: The aquifer base elevation was selected to be between the depths of the two wells and such that the modeled aquifer would not dewater during the simulation. Changing the base elevation can change whether a modeled pumping well goes dry in a simulation, but would not change the modeled drawdown in the vicinity of a well that does not go dry. The aquifer thickness was selected to be representative of the total thickness of water-yielding sediments penetrated by wells at the Site and their screened intervals, such that the aquifer thickness multiplied by the aquifer hydraulic conductivity is consistent with the estimated transmissivity of the wells.

Hydraulic Conductivity: The hydraulic conductivity of the sediments tapped by the on-Site wells is estimated to be approximately 0.25 foot/day; however, when this value was used in the model, the wells dewatered. For this reason, a hydraulic conductivity of 0.5 foot/day was used in the model. One clear implication is that the sediments penetrated by the wells may not be transmissive enough to yield water to the existing wells at a sufficient rate to meet the water demand for the proposed development. However, doubling the hydraulic conductivity may be within the reasonable range of certainty for aquifer parameter estimation, especially when analyzing a pumping test performed near a no-flow boundary using only data from the pumping well. In addition, doubling of the hydraulic conductivity allows evaluation of drawdown impacts that would occur if the water demand can be met by existing wells or by additional wells installed at the Site. As discussed in **Section 5.2**, the extent of the cone of depression is dependant on the transmissivity of the valley aquifer, and the cone of depression will be shallower and will propagate for a greater distance if the actual transmissivity of the aquifer is greater than calculated from the North #1 pumping test.

Boundary Head for Line Sink Boundary: The boundary head for the line sink boundaries at the river and the Bay has been set at sea level to represent the scenario discussed under the boundary conditions section above.

Other Line Sink Boundary Parameters: The remaining parameters applied to the constant head boundaries, including boundary resistance, boundary thickness and boundary width, relate to the permeability of the sediments beneath these surface water bodies that insulate them from the underlying alluvial aquifer. As discussed previously, information regarding the permeability of these sediments, and the rate of recharge from the Bay and river, is not known. The values of these parameters were set such that some significant retardation of groundwater percolation from the river and Bay would occur, but that the rate would be slow enough such that the cone of depression that forms near the Site would be similar to drawdowns observed during pumping tests at the Site. Higher rates of recharge would result in shallower drawdowns, and lower rates of recharge would result in greater drawdowns (and realistically, migration of the cone of depression beneath the boundary).

5.2.2 ADDITIONAL MODEL ASSUMPTIONS

The following additional assumptions are also incorporated into the analytical element model developed for the Site:

1. The aquifer being pumped is homogeneous and isotropic. Although the alluvial aquifer and Petaluma formation are neither homogeneous nor isotropic, we are assuming for purposes of this analysis that the aquifer's parameters can be represented by the transmissivity values obtained from the aquifer test described in **Section 4.4** and the parameters described in **Section 5.2.1.** These are assumed to represent average values for the heterogeneous and anisotropic aquifer. Note that quantitative data on the variations in hydraulic properties are not available. We have inferred that these variations exist based on the lithologic and geophysical well logs. Nevertheless, this is a standard assumption in this type of model.
2. The aquifer is uniform in thickness. The lateral extent of drawdown from the proposed pumping well(s) depends on the lateral and vertical extent and interconnectivity of the sandy units within the aquifers tapped by the well. As stated above, the available information indicates that these units likely vary in depth and lateral extent, and that some permeable beds in the Petaluma formation may be hydraulically isolated from other water bearing sediments. Nevertheless, this is a standard assumption in this type of model.
3. The aquifer receives no areal recharge. Significant areal recharge (such as from infiltration of stormwater runoff through the alluvial fans underlying the base of the Sonoma Mountains) will influence the shape and extent of the cone of depression.
4. The pumping well is screened in, and receives water from, the full thickness of the aquifer.
5. The modeled pumping well is 100 percent efficient and simulates drawdown in the first few minutes of recovery from constant discharge pumping.
6. Laminar flow exists throughout the well and aquifer.
7. The water table or potentiometric surface has no slope.
8. The proposed well will pump at a constant rate (*i.e.*, without seasonal or weekly variations). This assumption is suitable for making long-term predictions of drawdown under steady state conditions.

5.3 EVALUATION OF GROUNDWATER-LEVEL IMPACTS IN THE SITE VICINITY FROM PROPOSED PUMPING WELLS

As discussed in **Section 5.2**, reliable prediction of drawdown impacts from the development is not possible without further investigation. To provide perspective on the possible range of drawdown impacts, steady state drawdowns were calculated using the analytical element model for the Site under the inputs and assumptions described in **Section 5.2.1** and **Section**

5.2.2, and the results are presented on **Figure 8**. As shown on **Figure 8**, pumping at the Site could result in drawdowns measuring 10's of feet between the Site and the river and over 100 feet near the Site. Conversely, if the transmissivity of the valley aquifer is greater than calculated based on the pumping test performed on well North #1 and/or the leakance rate induced from surface water bodies is less than assumed in the model, then the cone of depression would be shallower and would extend for a greater distance form the Site. For example, if the transmissivity of the valley aquifer were closer to that derived from the well South #1 pumping test recovery data and surface water leakance and areal recharge are neglected, drawdown measuring 10 feet or more would be expected to extend across the Petaluma River, upvalley and beneath San Pablo Bay, possibly for several miles. In addition, under such circumstances drawdown near the Site would be well under 100 feet.

5.4 INTERFERENCE DRAWDOWN IMPACTS IN OFF-SITE WELLS

5.4.1 TYPES OF IMPACTS AND CONTRIBUTING FACTORS

Information was obtained from DWR for wells located within about 2 ½ miles from the Site. Additional information for wells in the area was obtained from reports compiled by USGS (Cardwell, 1958) and DWR (DWR, 1982) and the DWR website. Information regarding these wells is summarized in **Table 1**, and covers the area for which drawdown impacts are predicted by the analytical element drawdown model prepared for the Site. Drawdowns have not been estimated in **Table 1** for specific wells on **Figure 8** because the model calculations are approximate and represent one scenario in a continuum of possible outcomes. Nevertheless, it is clear that drawdowns of the magnitude shown on **Figure 8** are reasonably possible and cannot be ruled out. Such drawdowns would significantly impact wells in the vicinity of the Site. Depending on the degree of interconnectivity of the water-yielding strata penetrated by the production wells at the Site and water yielding beds in the Petaluma formation, similar drawdowns could occur in areas of Petaluma formation outcrop east of Lakeville Highway. Drawdown impacts may also be experienced in the area west of the Petaluma River, though the magnitude of these drawdowns is dependant on various factors that are not known and cannot be predicted at this time. However, as discussed in **Section 5.3**, drawdowns exceeding 10 feet could conceivably extend for several miles beyond the river and cannot be ruled out.

The project-related drawdown at any affected well (interference drawdown) will result in a decreased saturated thickness available to be pumped at that well. In the most extreme case, this could result in drawdown of the water level in a well to a depth below the screen of the well (*i.e.,* the affected well goes dry as a result of project pumping). At the other extreme, the effect of project pumping may be so small that the project-related drawdown is insignificant relative to short term or seasonal fluctuations, or the drawdown could represent an insignificant impact to the well user. The following possible significant impacts could occur:

1. The interference drawdown results in the water level in the aquifer being drawn down below the screen of the well (*i.e.*, the well goes dry as discussed above).

2. The interference drawdown results in the water level in the aquifer being drawn down to a point where the remaining saturated thickness is too small for the affected well to provide an adequate water supply for the intended use, or the pumping water level is too close the intake level of the pump, exposing it to potential damage.

3. The interference drawdown results in the water level in the well during pumping (the well's pumping water level) being drawn near the intake of the pump, requiring lowering of the pump intake in order for the well to remain operational. This is essentially a variation of case 2, but there is space below the pump allowing an adequate flow rate to be restored by lowering the pump. Energy costs would be expected to increase after the pump is lowered.

4. The interference drawdown results in a decrease in saturated thickness such that the well and pump can continue to operate and produce the required amount of water, but pumping must occur at either greater frequency/duration and must lift water for a greater height, using more energy, therefore resulting in greater operational and maintenance costs. This is a condition that can develop prior to the onset of case 1, 2 or 3.

The hydrogeologic factors that determine which of the above impacts will occur are the saturated thickness of the well before interference drawdown and the amount of interference drawdown that is applied (which varies with the impacted well's location and distance from the project well). The impact from interference drawdown has the potential to be more severe if it represents a higher percentage of the well's initial saturated thickness prior to the onset of interference drawdown. For example, a 10-foot drop in water level has a greater potential to cause Impacts 1 or 2 in a shallower well; whereas, the same drop in water level in a deeper well might result in less serious, but potentially still significant, impacts such as 3 or 4. In general, small variations in saturated thickness are not considered significant when assessing transmissivity values from the interpretation of aquifer test drawdown data (Jacob, 1950). However, in assessing the impacts of interference drawdown to neighboring pumping wells, a small change in saturated thickness (*e.g.*, 3 feet or more) could still cause a significant increase in electrical costs.

The impacts resulting from interference drawdown are also dependant on several factors that may vary from well to well, even if the wells have the same saturated thickness and applied interference drawdown. These well-specific factors include the following:

- Local variations in the transmissivity of the saturated sediments in which the well is completed (*i.e.*, their ability to yield water to the well with a given amount of drawdown in the aquifer);

- The condition and efficiency of the well (*i.e.*, the water level in the well bore compared to the water level in the aquifer just outside the well, which can be significantly lower if the well is in poor condition or poorly designed);

- The well's pump specifications, including its rating curve, the depth at which the pump intake is set, and the resulting pumping water level in the well during operation;

- The well's screened interval, which usually, but not always, extends to the bottom of a well; and

- The minimum required water production rate of the well.

The factors listed above affect the amount of water a well can produce, the amount of drawdown in the aquifer needed to produce that water, and the pumping water level inside the well while it is operating, which may be lower than the water level in the aquifer. As such, information regarding these factors is important when assessing impacts to individual wells; however, it is not readily available for wells in the Site vicinity. Well-specific impacts are most appropriately evaluated and addressed during the mitigation phase of the project (**Section 6.7**).

Table 1 lists 57 wells identified from DWR records and published reports in the Site vicinity. The reported depths of these wells range from 12 to 736 feet. All of these wells except well 12 (which is completed across the river from the Site) and wells 26, 27, and 28 (completed in fractured bedrock) may be expected to experience drawdown impacts of at least 10 and possibly over 100 feet. Depending on the degree of influence of the Petaluma River and the transmissivity of the valley aquifer, it is possible that well 12 will also experience drawdown impacts exceeding 10 feet. It is clear that some of these wells will potentially go dry as a result of pumping, whereas others will experience diminished yield, and yet others will experience increased electrical costs. Wells completed at shallow depths, and located near the Site boundary, and having low efficiencies, are most at risk for interference drawdown impacts from the proposed pumping wells.

5.4.2 IMPACTS ON OPERATING COST OF NEARBY SHALLOW AND DEEPER WELLS

Interference drawdown changes the operational characteristics of the pump operating within an existing water well. The additional interference drawdown effectively results in an increase in pump head (the distance the pump must lift the water), which in turn decreases the pump discharge rate, and changes the pump power requirements. The well will have to be pumped for a longer time each day as a result. Thus, more power will be required to pump the same total volume of water. The extent to which a well might be impacted by increased electrical costs may be dependant upon several factors, including the following:

- The amount of drawdown associated with the proposed pumping and the distance from the proposed pumping wells to the off-site well of concern and/or (*i.e.*, the amount of interference drawdown;
- Aquifer characteristics;
- Depth of the off-site well;
- Pumping water level in the well prior to the onset of interference drawdown;
- Pump specifications;
- Well condition and efficiency; and
- Nature of pumping (rate and duration/frequency).

Because information regarding the well-specific factors above (except well depth) is not readily available for wells near the Site, several operational scenarios and their associated changes in pumping power requirements were examined in order to add perspective on the range of impacts that might be anticipated. These included:

- A range of interference drawdown to represent varying drawdown scenarios and distance between the pumping wells at the Site and the off-site well;
- Three pumping rates to generally represent well uses for residential, and irrigation /industrial purposes;
- A range of pump depths to represent typical well depths in the area; and
- A range of pumping water levels and the addition of potential interference drawdown; and

- The assumption of appropriate pumps installed in the wells to produce the designated flow rates under the assumed conditions.

For each scenario, our engineer selected a pump that would be appropriate to supply water at the approximate rate specified given the well depth and water level. Thus, for purposes of this analysis, wells with different pumping water levels were assumed to contain different pumps, in order to maintain a reasonable match in each case between the well's pump, water level, and flow rate. The changes in electrical consumption to pump 1 acre-foot of water were then evaluated for that pump when the different levels of interference drawdown were applied. Additional details regarding our methodology are presented in **Appendix B**.

By attempting to model a range of conditions, we hoped to bracket the real world pumps and ensure that their operating conditions lie within the feasible space of this analysis. While this analysis is not exact and may not be representative of all actual installed pump types and conditions, it does offer some insight as to how much additional power might be required to pump 1 acre-foot of water if additional water table drawdown occurs. Once the drawdown impacts form Site pumping are more precisely known and if site-specific information regarding water wells and pumps becomes available in the future, this analysis could be adapted to examine power requirement impacts for those specific pumps during the mitigation phase of the project.

Evaluations representing three different well and pump configurations under two different interference drawdown conditions, were made based upon the following ranges of values and boundary conditions:

- Three pumping rates: 20 gpm, 100 gpm and 200 gpm;
- Two pump installation depths: 200 feet bgs and 600 feet bgs;
- Three initial pumping water levels: 150 feet bgs, 400 feet bgs and 500 feet bgs; and
- Two interference drawdown depths: 10 feet and 100 feet.

The evaluations were combined to produce the following matrix for which the additional incremental power (in kilowatt-hours [kW-hours]) required to pump 1 acre-foot of water was evaluated per the procedures outlined in **Appendix B**.

Additional Power Consumption Caused by Interference Drawdown Under Representative Well Configurations for the Rohnert Park Site Vicinity

Pump Discharge Rate (gpm)		20
Pump Installation Depth (feet bgs)		250
Pumping Water Level (feet bgs)		150
	Interference Drawdown (feet)	Additional Power Consumption (kW-hours/acre-foot)
	10	9.7
	100	207.9
Pump Discharge Rate (gpm)		100
Pump Installation Depth (feet bgs)		600
Pumping Water Level (feet bgs)		400
	Interference Drawdown (feet)	Additional Power Consumption (kW-hours/acre-foot)
	10	3.6
	100	135.9
Pump Discharge Rate (gpm)		200
Pump Installation Depth (feet bgs)		600
Pumping Water Level (feet bgs)		500
	Interference Drawdown (feet)	Additional Power Consumption (kW-hours/acre-foot)
	10	8.9
	100	77.4

The following conclusions may be drawn from the above results:

- As interference drawdown increases, the additional power required to pump 1 acre-foot of water also increases.
- As the depth to the pumping water level increases, the additional power required to pump 1 acre-foot of water when interference drawdown is applied also increases.
- Wells operated at lower flow rates (20 gpm) may experience a greater increase in the power required to pump an acre-foot of water when interference drawdown occurs than higher capacity wells. Conversely, at higher flow rates (*e.g.*, 200 gpm), interference drawdown causes less of an increase in power to pump 1 acre-foot of water than for the 20 gpm flow rate.

- The difference between the power consumption increases for different pump and drawdown scenarios can be significant, reflecting the importance of using well- and pump-specific information in assessing impacts to wells during the mitigation phase of the project.

Notwithstanding the increase in unit power consumption rates, the overall cost increase resulting from interference drawdown will be greater for higher capacity wells than for lower capacity wells. This is because lower capacity wells are typically associated with residential use, and the annual water volume pumped by a residential user is comparatively small. According to the American Water Works Association (AWWA), the average household in the United Stated uses approximately 1/3 acre-foot of water per year. Another rule of thumb that is commonly used is that 1 acre-foot of water can supply two households for one year. Therefore, the additional electrical cost to domestic well users will probably not be significant. For perspective, assuming an electricity cost of $0.15 per kW-hour, the additional yearly cost to operate the pumps modeled above would range form about $1 to $16 per year. Conversely, water wells pumping at higher rates are typically associated with agricultural or industrial users, with water requirements in the hundreds or thousands of acre-feet per year. (There are no known municipal water supply wells near the Site.) Even though less additional power is generally required per acre-foot of water when interference drawdown occurs in higher capacity wells, the need to pump many acre-feet of water per year results in a larger overall annual cost increase. For perspective, if an industrial or agricultural water user pumps 320 AF of water from a well in a year (which corresponds to a sustained pumping rate of about 200 gpm), the additional annual power requirement for the scenarios modeled above will be about 2,850 kW-hours with 10 feet of interference drawdown and 25,000 kW-hours with 100 feet of interference drawdown. At $0.15 per kW-hour, the additional cost impact to that user would be approximately $430 and $3,750 per year, respectively. Note that this represents a cost increase of about 1 and 11 percent, respectively, over the baseline pumping cost for the modeled pumps.

5.5 EVALUATION OF OTHER IMPACTS RELATED TO DRAWDOWN

As discussed in several hydrogeologic references reviewed during this study (Cardwell, 1958, Ford, 1975 and DWR, 1982), significant pumping of freshwater aquifers in the lower Petaluma Valley has the potential to cause seawater intrusion. The risk of inducing seawater intrusion and the extent and rate of migration of salinity impacts depends largely on the degree of connectedness between the Petaluma River and San Pablo Bay and the aquifers that will be pumped for the proposed project. In addition, connate brackish water in the Bay Mud and

Petaluma formation could be induced to migrate into the alluvial aquifers as a result of pumping.

The degree of connectedness, and whether any preferential pathways exist for migration of saline water (*e.g*, along local coarser grained deposits, especially if exposed by dredging activities along shipping channels) is not known at this time. Nevertheless, based on the reported hydrostratigraphy of the area, the detection of salinity impacts in the alluvial aquifers near the Site, and documentation of historical pumping-induced seawater intrusion in Petaluma, it is reasonable to assume that some seawater intrusion will occur. For perspective, it is instructive to note that the rate of seawater intrusion will depend upon the degree to which the assumptions regarding induced recharge from the Petaluma River and the Bay discussed in **Section 5.2.1** hold true.

Seawater intrusion, if induced by pumping for the proposed development, would impact off-Site wells between the Site and the Petaluma River or the Bay. The consequences of seawater intrusion include well, pump and pipe corrosion, rendering water objectionable or unusable, or creating the need for water treatment prior to use. In addition, seawater intrusion could trigger regulatory requirements to cease pumping and possibly to restore affected groundwater.

Other drawdown-related impacts could include the compaction of depressurized or dewatered clay deposits and resulting subsidence. The risk of subsidence depends on whether unconsolidated or semi-consolidated clay deposits are depressurized or dewatered, which is a function of the interconnectedness between subsurface strata, the rate of lateral groundwater inflow and the rate of recharge in both the pumped alluvial strata as well as the overlying Bay Mud. Evaluation of the potential for subsidence is beyond the scope of this report.

5.6 CUMULATIVE IMPACTS

The Petaluma Valley Basin has been subject to intensive development for domestic use (mostly in rural areas), and moderate development for stock watering, municipal, irrigation and industrial use (DWR, 1975a). The lower Petaluma Valley is characterized by a low density of wells (DWR, 1975b) and no municipal water supply wells. The closest municipal wells serve the City of Petaluma, approximately 9 miles north of the Site (USGS, 1982). Groundwater pumping by the City of Petaluma caused water levels to fall from the mid-1950s through the early 1960s, resulting in intrusion of brackish water. The City subsequently reduced its reliance on groundwater, and by 1980, met 15 percent of its water demand with groundwater. At that time, the total annual groundwater pumpage in the basin was estimated as 7,800 acre-feet, and DWR indicated that there was no evidence of overdraft in the basin (DWR, 1980). In the last several decades, Petaluma has used its groundwater wells only for emergency backup purposes and intends to continue doing so through 2023 (Dyett & Bhatia, 2006). Beginning in 2024, the City projects it may use up to 186 AFY of groundwater for supply during peak demand periods. Information obtained from DWR indicates that combined municipal, industrial and agricultural groundwater pumping in the South Sonoma DAU (which includes both Petaluma and Sonoma Valleys) is expected to decrease about 73 percent below 1999 levels by the year 2020 (DWR, 2006).

Water levels in DWR monitoring wells completed in the alluvial aquifer in the southern Petaluma Valley are generally close to the elevation of the valley floor and fluctuate within a relatively narrow range of a few feet (**Figure 6**), and wells North #1, North #2 and South #1 were reported to be flowing (water levels above the land surface) in 2003 (**Section 4.3**). Much greater water level fluctuations are recorded for two DWR monitoring wells completed in the Petaluma Formation (approximately 16 feet to over 110 feet, **Figure 7**). Seasonal and climatic fluctuations are apparent in hydrographs both for wells completed in the alluvial aquifer and the Petaluma formation, and are probably accentuated by increased local pumping during dry periods. A slight declining trend in water levels is apparent in the hydrographs of the wells completed in the Petaluma formation that may be related to regional water pumping.

Basinwide, the proposed pumping rate for the project (200 gpm or 323 AFY) is about 4 percent of the estimated groundwater pumping in 1980, which did not result in observed conditions indicative of overdraft. Based on the available data, it is likely that basinwide groundwater demand will remain substantially below 1980 levels, even with the addition of project pumping. As such, the project is not likely to contribute to a basinwide groundwater decline; however, given the hydrogeologic setting of the Site, it is more likely to result in more localized effects in the southern Petaluma Valley basin, where it will represent a much higher percentage of local groundwater pumping. The current rate of groundwater pumping by existing groundwater users in the southern Petaluma Valley Basin is not known, and evaluation of existing and potential future impacts from local pumping would require further investigation. As such, the cumulative contribution of Site pumping to local groundwater drawdown and potentially related impacts such as seawater intrusion cannot be evaluated at this time.

5.7 POTENTIAL MITIGATION MEASURES

5.7.1 GROUNDWATER PUMPING TEST

The actual drawdown impacts from using groundwater to supply the proposed projects can be verified and more accurately assessed if a pumping test is conducted at the Site to simulate project pumping and drawdown is measured in one or more observation wells. During the pumping test, water levels in at least two nearby shallow and two nearby deeper wells should be monitored. If adequate monitoring wells are not available, we recommend that monitoring wells be installed. The results of the pumping test could be used to update the analytical element model prepared for this study with refined Site-specific data.

5.7.2 GROUNDWATER MONITORING

The actual drawdown impacts from using groundwater to supply the proposed projects can only be accurately assessed with the implementation of a properly designed monitoring program, and information regarding existing groundwater users and baseline trends is needed to assess local cumulative impacts. Such a program would allow documentation of the actual distance-drawdown relationship in the vicinity of the Site, local ambient groundwater level trends and the potential influence of interference drawdown from other water users in the area. This information in turn can be used to guide the application and evaluate the effectiveness of the hydrogeological mitigation measures that are being considered as part of the project, and can form the basis for assessment of impacts to well owners and the aquifer in the Site Vicinity.

A groundwater level monitoring program could include existing wells and/or new wells installed for the project. We recommend that a monitoring program be designed based on an evaluation of completion data and lithologic logs for existing wells that may be available for that purpose. The monitoring program should include at least the following:

- An inventory of nearby groundwater pumpers and actual extraction rates, to the extent this information is available;
- Two wells completed at depths shallower than 50 feet, one near the Petaluma River and one near San Pablo Bay;
- One well completed between 200 and 500 feet near the Petaluma River; and
- One well completed between 50 and 150 feet and one well completed between 200 and 500 feet close to the Site, preferably within ½ mile.

If existing wells are used, they should not be used for water production within one month of being measured. The monitoring wells should not be located near wells that are being actively pumped. We recommend that water level measurements begin at least one year prior to project development to develop sufficient baseline data, and that both spring and fall measurements be taken. During each monitoring event, these wells should also be sampled for total dissolved solids, chloride and conductivity to assess potential impacts to water quality.

Data from groundwater level monitoring that is conducted by DWR can be used to assess the ongoing regional groundwater level trend in the Petaluma Valley basin and establish a regional baseline.

5.7.3 POTENTIAL ON-SITE HYDROGEOLOGIC MITIGATION MEASURES

Drawdown impacts will be mitigated to some extent by the disposal of highly-treated wastewater in sprayfields at the Site, and could be mitigated further by constructing reservoirs in the drainages in the upland portions of the Site to enhance seasonal recharge. As discussed below, the effectiveness of these measures to reduce drawdown impacts is directly proportional to the rate of new recharge compared with the pumping rate. In addition, wells could be drilled in the upland portion of the Site to reduce the amount of water that is pumped from the alluvial aquifer system, or consideration could be given to drilling a well laterally into the Tolay Fault (**Figure 3**).

5.7.4 POTENTIAL OFF-SITE HYDROGEOLOGIC MITIGATION MEASURES

In addition to mitigation measures that are being considered as part of Site development, participation in off-Site artificial recharge projects is currently being considered. This could be accomplished by purchasing rights to wells that are currently pumping and then not using them (*in lieu* recharge) or underwriting local water conservation programs.

5.7.5 POTENTIAL MITIGATION MEASURES FOR IMPACTS TO NEARBY WELLS

The actual amount of interference drawdown associated with the project will be estimated from the proposed pumping test and groundwater level monitoring program (**Sections 5.7.1 and 5.7.2**). We recommend that these data be used in the proposed mitigation program to distinguish the portion of impacts to nearby wells that is project related vs. the portion that is attributable to interference drawdown from other nearby high-capacity wells. At least one year of baseline data and one year of data after project pumping begins should be collected prior to implementation of the mitigation/cost reimbursement program outlined below.

- Well Usability (Impacts 1 and 2) – The tribe would reimburse the owners of wells that become unusable within 3 years[1] of the onset of project pumping for a portion of the prevailing, customary cost for well replacement, rehabilitation or deepening. The mitigation method for which reimbursement is made would be the lowest-cost customary and reasonable method to restore the lost well capacity. The percentage of the cost reimbursed by the tribe would depend upon the degree to which the impact is caused by project pumping vs. pumping by other nearby operators of high capacity wells. Reimbursement would be for replacement in-kind; that is, for a well of similar construction, but deepened so as to restore the lost well capacity. A depreciation allowance would be subtracted from the reimbursement amount for wells or pumps that have condition issues. In order to be eligible, the well owner would need to provide the tribe with documentation of the well location and construction (diameter, depth, screened interval, pump type, etc.), and that the well was constructed and usable before project pumping was initiated.

- Groundwater level falling near or below pump intake (Impact 3) – Whether a pump intake requires lowering depends on a number of well-specific factors that are not known at this time. The tribe would reimburse the owners of wells with pumps that require lowering within 3 years[3] of the onset of project pumping for a portion of the prevailing, customary cost for this service. The percentage of the cost reimbursed by the tribe would take into consideration the degree to which the impact is caused by project pumping vs. pumping by other nearby operators of high capacity wells, and the degree to which a well's capacity may have been reduced in the absence of project pumping due to shallow placement of the pump intake. Replacement discharge piping would not be reimbursed, and replacement of pumps would not be reimbursed unless the pump was damaged due to project-related interference drawdown. In order to be eligible, the well owner would need to provide the tribe with documentation of the well location and construction, including pump intake depth, and that the well was constructed and usable before project pumping was initiated. The tribe must be made aware of the cost reimbursement claim prior to lowering of the pump intake, so that the need for possible well deepening, replacement or rehabilitation can be assessed. At the tribe's discretion, compensation may be paid toward well deepening, replacement or rehabilitation in lieu of toward lowering the pump intake.

[1] Three years was selected as a reasonable amount of time for interference drawdown impacts to manifest themselves and for well owners to become aware of them and submit a claim. This time period could be extended if the proposed pumping test suggests it will take a longer period of time for water levels to stabilize.

- Increased Electrical and Maintenance Cost (Impact 4) – The tribe would reimburse well owners pumping more than 100 AFY for their additional annual electrical costs at the prevailing electrical rate based on the following formula[2]:

$$\text{KWhr/year} = \frac{\text{(gallons Pumped/year) x (feet of interference drawdown)}}{1621629}$$

  In order to qualify for reimbursement, the well owner must provide proof of the actual annual volume of water pumped and/or the electrical usage associated with the pumping. As an alternative to annual payments, a one-time lump sum payment of a mutually agreeable amount could be made.

- No reimbursement would be made available for wells installed after operation of the project wells commences.

- For any of the above impacts, the tribe may choose at its discretion to provide the well owner with a connection to a local public or private water supply system *in lieu* of the above mitigation measures, at reduced cost in proportion to the extent the impact was caused by project pumping.

The known owners of identified wells within 3 miles of the proposed project pumping well(s) would be notified of the mitigation program outlined above before project pumping begins. We recommend that the tribe contract with a third party such as the County of Sonoma to oversee this mitigation program.

Monitoring could be conducted to provide early warning of potential seawater intrusion or land subsidence. Seawater intrusion could be mitigated to some extent by injecting treated wastewater between the production wells for the Site and the Petaluma River or the Bay.

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[2] This formula is derived from combining the following two formulas:

KW input = ([Pump brake horsepower] x 0.7457) / (motor efficiency)

Pump brake horsepower = ([gpm] x [feet of water] x [specific gravity]) / (3960 x [pump efficiency])

Where:
specific gravity = 1;
typical motor efficiency = 85%; and
typical pump efficiency = 60%

6 CONCLUSIONS

- The Site is undeveloped and located at the foot of the Sonoma Mountains in the lower Petaluma Valley near San Pablo Bay. Petaluma River, a tidal slough, is located approximately 2 miles west of the Site. Elevations across the Site range from 40 feet above mean seal level (amsl) to near sea level on the valley floor on the west side of the Site.

- The principal water-bearing deposits in the Site vicinity are Quaternary to Recent alluvial deposits and the Pliocene Petaluma formation, which consists largely of claystones with some sand and gravel deposits. Consolidated Mesozoic basement rocks that underlie the entire area have little permeability and form the boundaries of the groundwater flow system. In the southern Petaluma Valley, these deposits are overlain by low-permeability estuarine deposits of the Bay Mud.

- Groundwater is unconfined in the shallow alluvial deposits exposed near the valley margins. The groundwater is confined or semiconfined in deeper parts of the basin. The continental deposits of sand and gravel are interbedded with fine-grained silt and clay, which locally may act as confining layers. Recharge is primarily through infiltration of runoff from precipitation in the surrounding mountains: infiltration occurs along streambeds and in the permeable sediments of the valley floor at the basin margins. Groundwater flow in the aquifers is generally parallel to the long axis of the basin or away from the flanking ridges to discharge areas along the tidal sloughs and San Pablo Bay.

- Yields from the alluvial aquifer are variable, but it is generally thought to be capable of supporting domestic uses and small irrigation projects. Although the alluvial deposits are quite heterogeneous, they are regarded as a single aquifer unit for the purposes of this study. Groundwater within the alluvial aquifer may occur under unconfined conditions near the heads of alluvial fan deposits, but semi-confined and possibly confined conditions are expected to occur in distal areas where finer grained deposits are more prominent or where the formation is overlain by the Bay Mud. Groundwater within the Petaluma formation generally occurs under confined or semi-confined conditions.

- Water quality data indicate seawater intrusion has occurred locally into alluvial deposits near and possibly adjacent to the Site. Although groundwater from the alluvial deposits, especially near the valley margins and sometimes at depth beneath the Bay Mud, is often of good quality, it is of limited extent. At the southern end of the Petaluma Valley, and underlying or near the Site, a zone of poor quality groundwater has been identified at depths from approximately 150 to 700 feet. This poor quality water has been variously interpreted as connate water from sea water trapped in the fine-grained sediments of the Petaluma formation at the time of deposition, or the result of seawater intrusion into the alluvial aquifers. Several references discuss seawater intrusion as impacting aquifers in the southern Petaluma Valley in the Vicinity of the Site, and indicate that increased groundwater withdrawal in the area could result in additional seawater intrusion.

- Fifty-seven wells have been identified within about 1 ½ miles of the Site from available records. The wells range in depth from 12 to 736 feet, with an average depth of 243 feet. Most of the wells are located along the base of the mountains or on the valley floor, and are completed in alluvial fan deposits underlying the Bay Mud. Various deeper wells, including some wells along the mountain front and those within the Sonoma Mountains, are likely completed in the Petaluma formation. Reported well uses include domestic (35), stock watering/dairy (six), irrigation (two), monitoring (two) and "other" (one). The use of eight of the identified wells is not reported, and three of these wells are reported as being abandoned. There are no municipal water supply wells in the lower Petaluma Valley.

- HydroScience identified two wells as being located at the Site. North #1 is located near the Sonoma Mountains on the east side of the Site and is 413 feet deep. North #2 is located on the west side of the Site and is 650 feet deep. Both wells are screened in the alluvial aquifer and possibly in the underlying Petaluma formation. HydroScience reported the Bay Mud as being 150 feet thick at North #2 and bedrock being encountered at 650 feet. Water levels in both wells were reported to be artesian in 2003.

- Review of hydrographs for three nearby wells completed in the alluvial aquifer indicates that water levels are close to sea level and generally fluctuate within a relatively narrow range of a few feet. Both seasonal and climatic trends are apparent. Comparison with cumulative departure from average annual precipitation indicates that each well responds to periods of below and above average precipitation.

- A much greater difference in water levels (approximately 60 feet) is apparent for two wells completed in the Petaluma formation. In addition, water level fluctuations are much greater for these wells – fluctuations range from approximately 16 feet to over 110 feet. This is consistent with the interpretation that the clayey character of the Petaluma formation can result in hydraulic isolation of individual water-bearing zones. Nevertheless, both seasonal and climatic water level trends are apparent. A slight declining trend in water levels is apparent in the hydrographs of both of these wells that does not appear related to climatic influences and may be related to regional water pumping.

- Specific capacities for two wells at the Site and four wells in the Site vicinity are reported by HydroScience to range from 0.3 to 3 gallons per minute (gpm)/foot of drawdown. This is in the range of values reported for 24 wells tested in the Site vicinity, which is 0.03 to 30 gpm/foot. Analysis of pumping test data provided by HydroScience indicates a transmissivity of approximately 0.5 square foot per day for North #1. This corresponds with a hydraulic conductivity of approximately 0.25 foot/day, which is typical of strata dominated by fine grained sediments (*i.e.*, silt and clay).

- Due to the relatively complex hydrogeologic setting of the Site and the limited data available, it is not possible to quantitatively estimate drawdown impacts from the proposed pumping. To evaluate one scenario of potential drawdown impacts, an analytical element model was prepared using the U.S. Environmental Protection Agency's WhAEM2000 code. The model was set up using a simplified set of assumed boundary conditions to simulate the influence of the relatively long and narrow configuration of the valley and recharge that may be induced from the Petaluma River and San Pablo Bay. Hydrogeologic parameters for the analytical model were derived from the available data at the time of this study.

- The hydraulic conductivity of the sediments tapped by the on-Site wells is estimated to be approximately 0.25 foot/day; however, when this value was used in the model, the wells dewatered. For this reason, a hydraulic conductivity of 0.5 foot/day was used in the model. One clear implication is that the sediments penetrated by the wells may not be transmissive enough to yield water to the existing wells at a sufficient rate to meet the water demand for the proposed development. However, doubling the hydraulic conductivity may be within the reasonable range of certainty for aquifer parameter estimation, especially when analyzing a pumping test performed near a no-flow boundary using only data from the pumping well. In addition, doubling of the hydraulic conductivity allows evaluation of drawdown impacts that would occur if the water demand can be met by existing wells or by additional wells installed at the Site.

- Based on the model results, the evaluated scenario indicates that pumping at the Site could result in drawdowns measuring 10's of feet between the Site and the river and over 100 feet near the Site. An alternate scenario could also be realistic would assume that the amount of leakance that can be induced form Petaluma River and San Pablo Bay is more limited, and the hydraulic conductivity of the alluvial valley aquifer is much greater than calculated based on the North #1 well pumping test. Under such a scenario, the drawdown near the Site would be less, but drawdowns in the range of 10 feet or so might be expected to extend up to several miles beyond the Petaluma River and beneath San Pablo Bay.

- These estimates are approximate; however, it is clear that drawdowns of the magnitude shown on **Figure** 8 would significantly impact wells in the vicinity of the Site. **Table** 1 lists 57 wells identified from DWR records and published reports in the Site vicinity. The reported depths of these wells range from 12 to 736 feet. All of these wells except well 12 (which is completed across the river from the Site) and wells 26, 27, and 28 (completed in fractured bedrock) may be expected to experience drawdown impacts of at least 10 and in some cases over 100 feet. However, as noted above, it is also possible that drawdown near the Site will be less and that significant drawdown will be experienced beyond the Petaluma River from the Site.

- Wells in the Site vicinity are predicted to experience some drawdown impacts (interference drawdown) and a resulting proportional decrease in well yield or efficiency, pumping cost and pump life. In the absence of well-specific data regarding transmissivity, use, condition and efficiency, these impacts may be assumed to be generally proportional to the amount of interference drawdown and the remaining saturated thickness of the well after interference drawdown.

- The most serious impact that could be experienced by a nearby groundwater user would be having their well go dry or rendered unusable because the remaining saturated thickness after drawdown is too small to support pumping at the required rate. The wells most at potential risk for this impact are expected to be primarily shallow wells near the Site. Deeper wells or wells located at increased distance form the Site are at lower risk of being dewatered or rendered unusable.

- In some wells, if water levels fall to a point where the well is in danger of going dry or becoming unusable, the pump intakes can be lowered to extend the life of the well. Without more specific information regarding well construction and pump depth, it is not possible to estimate how many wells may be at risk of experiencing this impact. However, pump intakes for shallow or domestic wells are generally set near the well bottoms and cannot be lowered; whereas, pump intakes for deeper municipal, industrial or agricultural wells are sometimes set at a relatively shallow depth and could require lowering if the wells are located near the Site.

- Interference drawdown will cause an increase in the electrical cost to pump a unit volume of groundwater from a well. This cost increase is not expected to be significant for domestic wells because of the relatively low volume of groundwater pumped by a typical household, but could be significant (ranging from several hundred to several thousand dollars) for higher capacity agricultural or industrial wells near the Site. For the pumps modeled, the increased costs for higher capacity pumping represented approximately a 1 to 11 percent increase in overall pumping costs.

- As discussed in several hydrogeologic references reviewed during this study, significant pumping of freshwater aquifers in the lower Petaluma Valley has the potential to cause seawater intrusion. The risk of inducing seawater intrusion and the extent and rate of migration of salinity impacts depends largely on the degree of connectedness between the Petaluma River and San Pablo Bay and the aquifers that will be pumped for the proposed project, which is not known at this time. Nevertheless, based on the reported hydrostratigraphy of the area, the detection of salinity impacts in the alluvial aquifers near the Site, and documentation of historical pumping-induced seawater intrusion in Petaluma, it is reasonable to assume that some seawater intrusion will occur.

- Seawater intrusion, if induced by pumping for the proposed development, would impact off-Site wells between the Site and the Petaluma River or the Bay. The consequences of seawater intrusion include well, pump and pipe corrosion, rendering water objectionable or unusable, or creating the need for water treatment prior to use. In addition, seawater intrusion could trigger regulatory requirements to cease pumping and possibly to restore affected groundwater.

- Other drawdown-related impacts could include the compaction of depressurized or dewatered clay deposits and resulting subsidence. The risk of subsidence depends on whether unconsolidated or semi-consolidated clay deposits are depressurized or dewatered, which is a function of the interconnectedness between subsurface strata, the rate of lateral groundwater inflow and the rate of recharge in both the pumped alluvial strata as well as the overlying Bay Mud. Evaluation of the potential for subsidence is beyond the scope of this report.

- A more sophisticated model, based on better knowledge of the hydrostratigraphy, groundwater flow, recharge and degree of interconnection between surface water and groundwater in the Site vicinity, could be used to predict drawdown impacts and related impacts due to seawater intrusion and subsidence with greater certainty. If greater certainty is required, we recommend that field work including subsurface exploration, additional aquifer testing and surface reconnaissance be performed to support a numerical modeling study.

7 CLOSURE/LIMITATIONS

This report has been prepared for the exclusive use of Analytical Environmental Services, Inc. as it pertains to the assessment of the effects of the use of groundwater to supply the proposed Graton Rancheria hotel/casino development at an alternative Site near Lakeville, California. Our services have been performed using that degree of care and skill ordinarily exercised under similar circumstances by reputable, qualified environmental consultants practicing in this or similar locations. No other warranty, either express or implied, is made as to the professional advice included in this report. These services were performed consistent with our agreement with our client.

Opinions and recommendations contained in this report apply to conditions existing when services were performed and are intended only for the client, purposes, locations, time frames, and project parameters indicated. We do not warrant the accuracy of information supplied by others or the use of segregated portions of this report. Our evaluations were performed on the basis of information that was reasonably available within the time frame and constraints of the project, and may not include all of the data that are available. Reasonable extrapolations and interpretations were made; however, these are not a substitute for Site-specific data. Such data can be used to more accurately predict well yields, drawdown impacts, and potential seawater intrusion and subsidence impacts, and can be used to optimize water supply design for these factors.

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Table 1 - Sumary of Reported Informaton Regarding Water Wells in the Site Vicinity

CLIENT: AES
PROJECT No.: N0410C
LOCATION: Graton Rancheria Hotel and Casino, Lakeville Site
DESCRIPTION: Groundwater Study

DATE: 3-Dec-06
BY: Alan Blakemore
REVISION: Mike Tietze

Map Well Number	Drilled Date	Well Depth (feet)	Screened Interval (feet bgs)	Use	Information Source
1	1950	200	Unknown	Domestic	USGS (1958)
2	Unknown	50	Unknown	Unknown	USGS (1958)
3	1947	165	Unknown	Domestic	USGS (1958)
4	Unknown	435	Unknown	Domestic	USGS (1958)
5	1972	239	59-239	Domestic	DWR files
6	1975	180	160-180	Domestic	DWR files
7	1965	30	15-27	Domestic	DWR files; USGS (1958)
8	Unknown	15.5	Unknown	Domestic	USGS (1958)
9	Unknown	225	Unknown	Domestic	USGS (1958)
10	Unknown	Unknown	Unknown	Stock	USGS (1958)
11	Unknown	36	Unknown	Domestic	USGS (1958)
12	1951	162	Unknown	Domestic	DWR files
13	1924	250	Unknown	Domestic	USGS (1958)
14	Unknown	520	Unknown	Unknown	DWR (1982)
15	Unknown	200	Unknown	Domestic	USGS (1958)
16	1972	215	183-214	Domestic	DWR files
17	2005	510	250-510	Irrigation	DWR files
18	Unknown	259	Unknown	Unknown	DWR ONLINE
19	1948	184	Unknown	Domestic	USGS (1958)
20	1949	464	Unknown	Domestic	USGS (1958)
21	1939	318	Unknown	Abandoned	USGS (1958)
22	1976	382	142-382	Domestic	DWR files
23	1983	464	429-450	Domestic	DWR files
24	1995	106.5	91.5-106.5	Monitoring	DWR files
25	Unknown	21	Unknown	Domestic	USGS (1958)
26	1985	320	Unkown	Domestic	DWR files
27	1987	594	170-590	Domestic	DWR files
28	1987	265	78-265	Domestic	DWR files
29	1979	350	120-350	Domestic	DWR files
30	1986	260	100-260	Domestic	DWR files
31	1987	154	94-154	Domestic	DWR files
32	1987	214	94-214	Domestic	DWR files
33	1987	125	64-124	Domestic	DWR files
34	1987	276	75-275	Domestic	DWR files
35	1955	200	38-195	Domestic	DWR files

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resources & energy

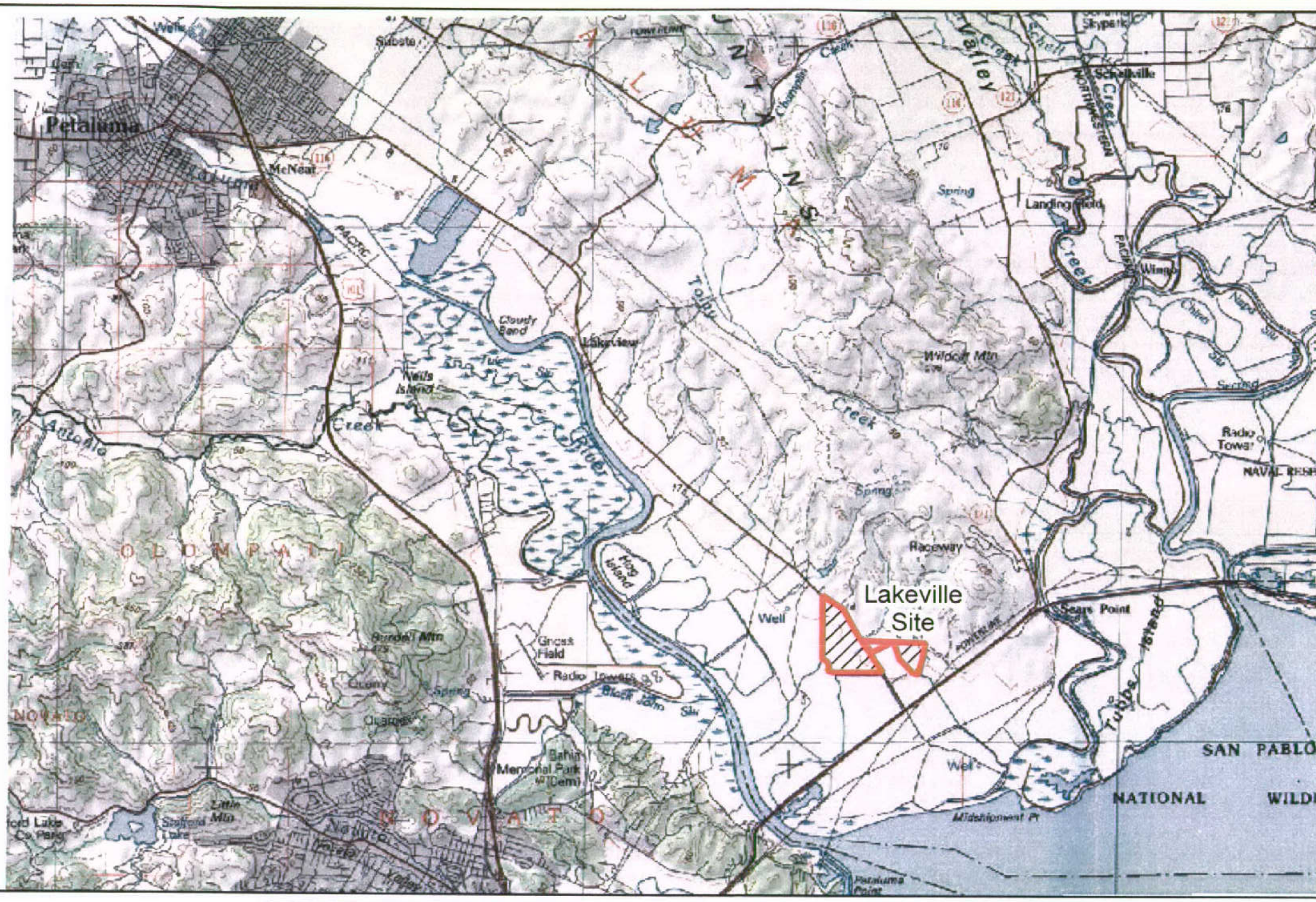
Table 1 - Sumary of Reported Informaton Regarding Water Wells in the Site Vicinity

CLIENT: AES
PROJECT No.: N0410C
LOCATION: Graton Rancheria Hotel and Casino, Lakeville Site
DESCRIPTION: Groundwater Study

DATE: 3-Dec-06
BY: Alan Blakemore
REVISION: Mike Tietze

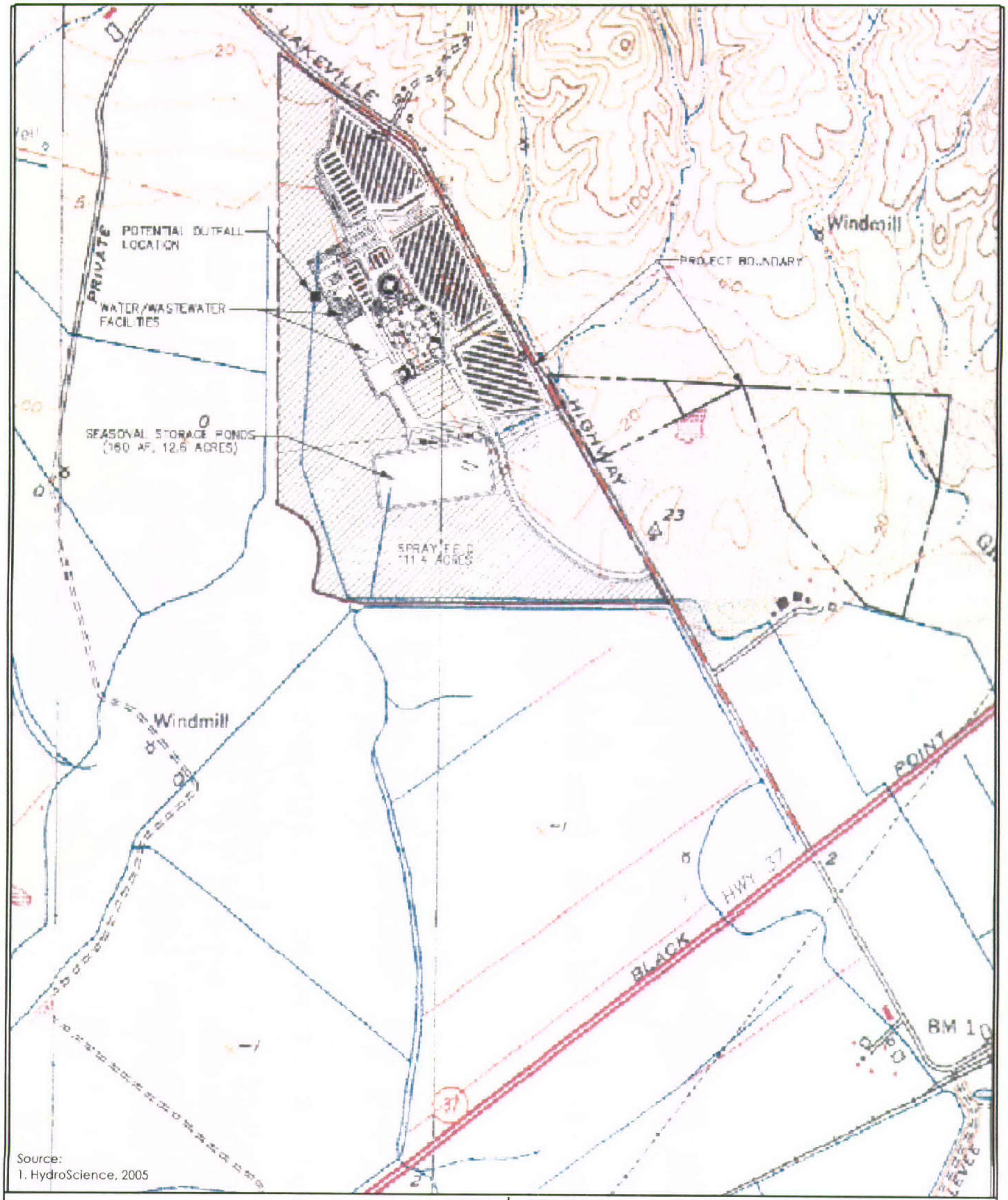
Map Well Number	Drilled Date	Well Depth (feet)	Screened Interval (feet bgs)	Use	Information Source
36	1976	505	90-505	Domestic	DWR files
37	Unknown	325	Unknown	Domestic	USGS (1958)
38	1940	325	Unknown	Other	DWR files; USGS (1958)
39	1987	123	62-122	Domestic	DWR files
40	1988	207	180-200	Domestic	DWR files
41	1978	150	50-150	Domestic	DWR files
42	1977	628	74-628	Dairy	DWR files
43	1945	500	98-500	Unknown	DWR files
44	Unknown	Unknown	Unknown	Unknown	USGS (1958)
45	1941	222	145-222	Unknown	DWR files; USGS (1958)
46	1934	736	Unknown	Unknown	DWR files
47	1939	390	Unknown	Abandoned	DWR files
48	Unknown	35	Unknown	Abandoned	USGS (1958)
49	1948	175	Unknown	Irrigation	USGS (1958)
50	Unknown	92	Unknown	Unknown	USGS (1958)
51	1930	250	Unknown	Stock	USGS (1958)
52	1978	282	182-282	Domestic	DWR files
53	1947	165	Unknown	Stock	USGS (1958)
54	Unknown	12	Unknown	Stock	USGS (1958)
55	1995	30	20-30	Monitoring	DWR files
56	1975	50	30-50	Domestic	DWR files
57	1940	12	Unknown	Stock	USGS (1958)

Notes:
bgs = Below Ground Surface
DWR Files = Well completion records provided by DWR



Notes:
1 - Basemap data from ESRI DVD Dataset
2 - Projection in NAD 83 CA State Plane Zone II

LAKEVILLE HYDROGEOLOGIC STUDY	WorleyParsons Komex resources & energy		
REGIONAL MAP	SWL	DJ	10/2005
	N0410		1



LAKEVILLE HYDROGEOLOGIC STUDY

WorleyParsons Komex
resources & energy

0 0.5 Miles

SITE VICINITY MAP

SWL	DJ	10/2005
NO410		**2**

ROCK UNITS

QUATERNARY	Pleistocene–Holocene	Qal	ALLUVIUM – sand, silt, clay, and gravel; variable specific yield (5–15%)
		Qbm	BAY MUD – mud, silty mud, silt, and sand, rich in organic matter; very low specific yield (<3%)
		Qf	ALLUVIAL FAN DEPOSITS – fine sand, silt, silty clay, coarse sand and gravel, with gravel more abundant near fan heads; moderate to high specific yield (8–17%)
TERTIARY	Pliocene	Tsv / Tsi	SONOMA VOLCANICS – volcanic flows, agglomerates, and tuffs; Tsi indicates presence of volcanic sediments; highly variable specific yield (0–15%)
		Tm	MERCED FORMATION – continental and marine coarse-to fine-grained sandstone with minor amounts of clay; high specific yield (10–20%)
		Tp	PETALUMA FORMATION – clay and shale with minor amounts of sandstone; low specific yield (3–7%)
		Ttv	TOLAY VOLCANICS – volcanic flows, tuffs, breccias, and agglomerates; unknown specific yield; assumed to be very low (<3%)
JURA-CRETACEOUS		KJf	FRANCISCAN COMPLEX – includes chert, sandstone, shale, greenstone; includes fault-bounded, rounded serpentinite bodies; apparent specific yield is very low (<3%)

Notes:
1 - Basemap data from ESRI DVD Dataset
2 - Projection in NAD 83 CA State Plane Zone II

LAKEVILLE HYDROGEOLOGIC STUDY

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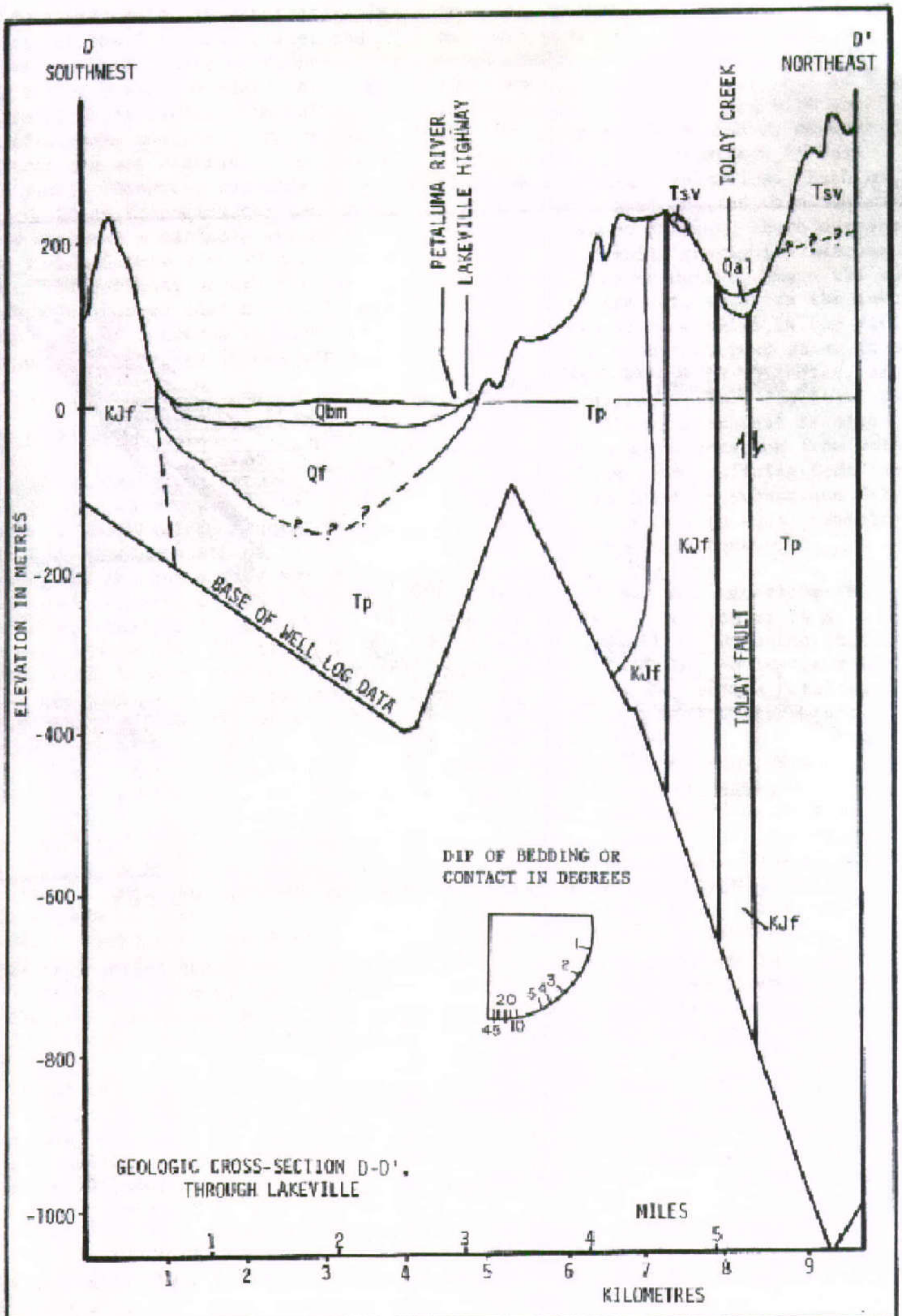
GEOLOGIC MAP

SWL	DJ	10/2005
N0410		3

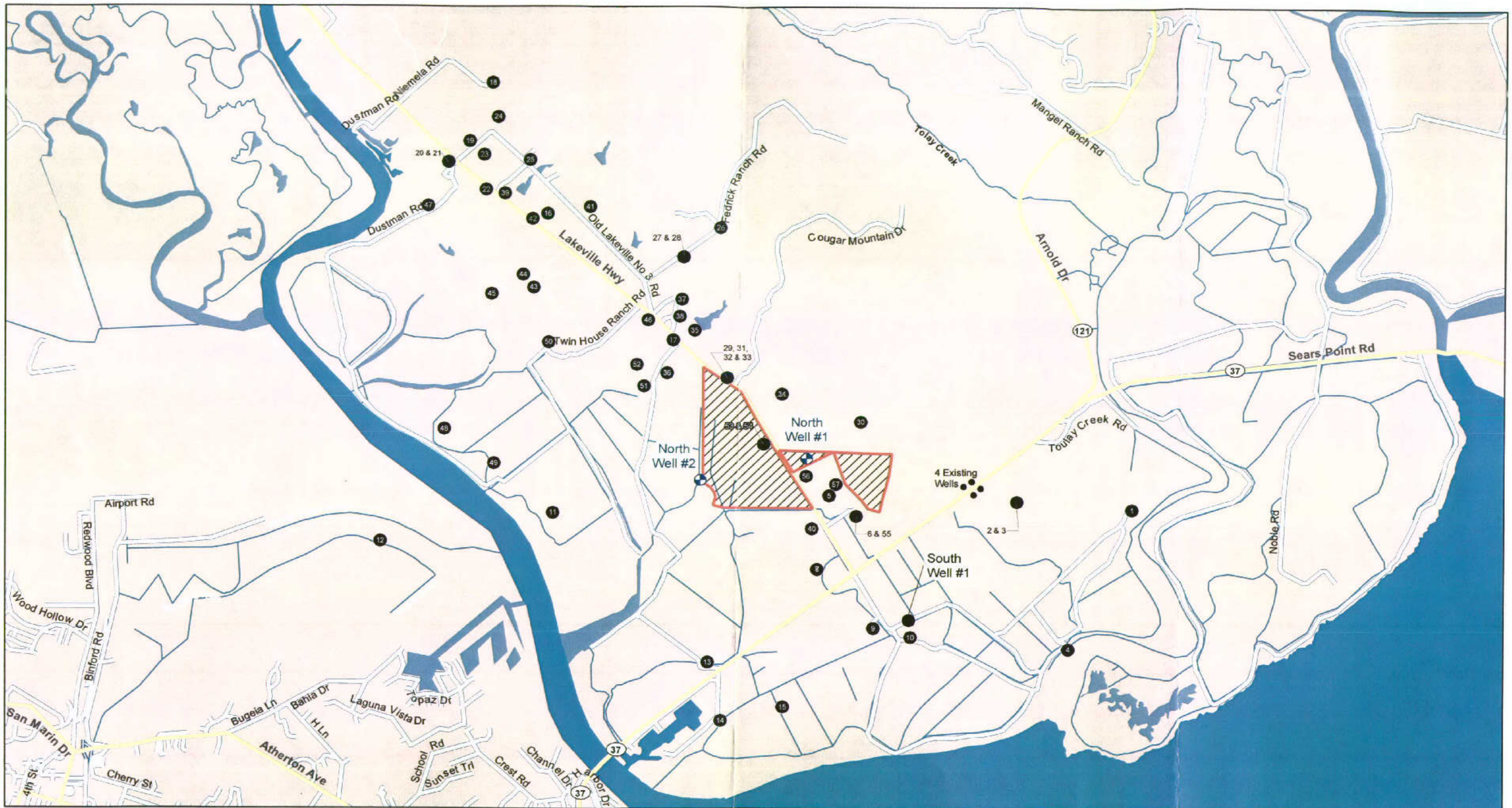


Source:
1. DWR, 1982

LAKEVILLE HYDROGEOLOGIC STUDY

WorleyParsons Komex
resources & energy

GEOLOGIC CROSS SECTION	SWL	DJ	10/2005
	NO410		**4**



Notes:
1 - Basemap data from ESRI DVD Dataset
2 - Projection in NAD 83 CA State Plane Zone II
3 - Location of all wells is approximate and was not field verified. See Table 1 for well construction details.

LAKEVILLE HYDROGEOLOGIC STUDY

WorleyParsons Komex
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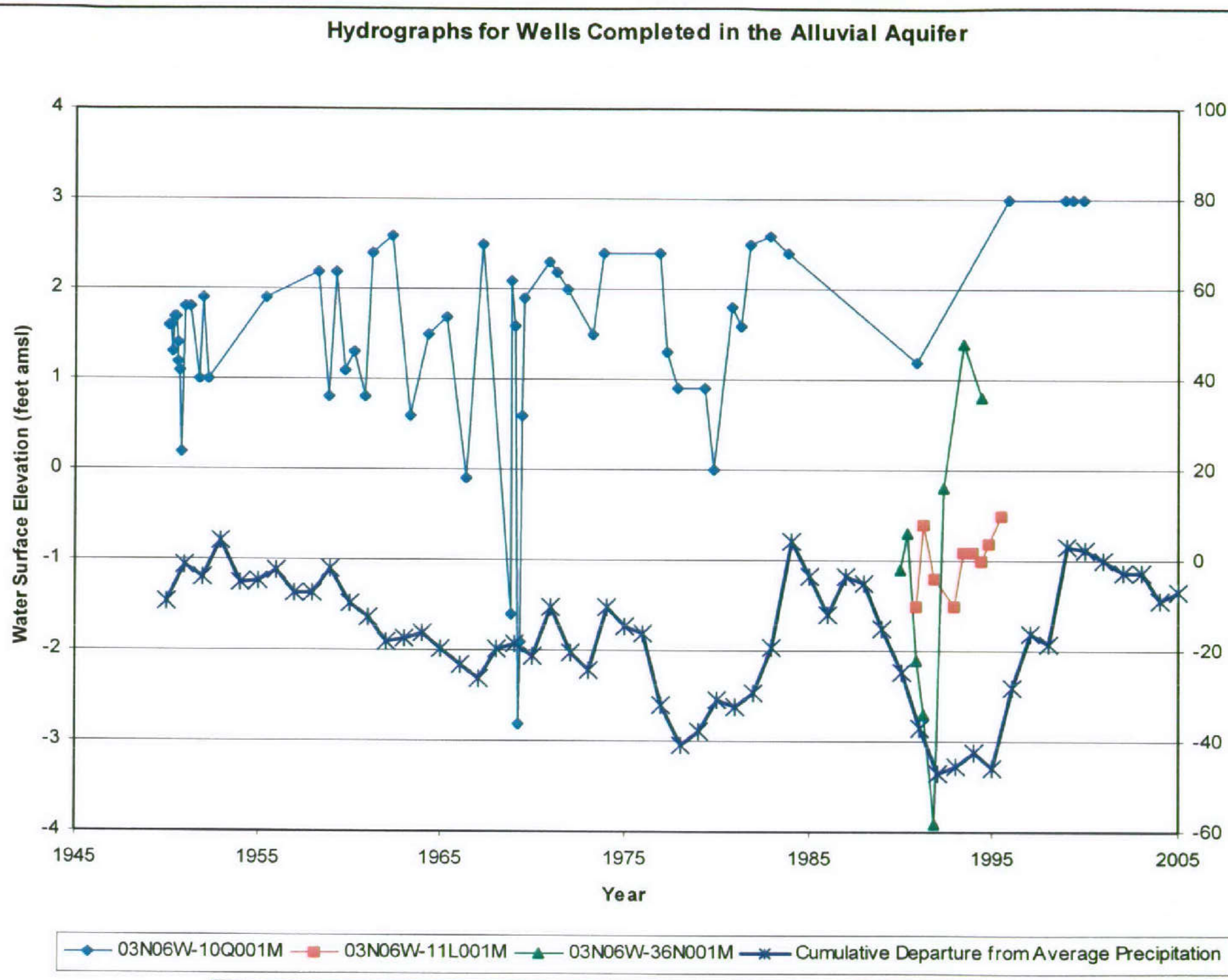
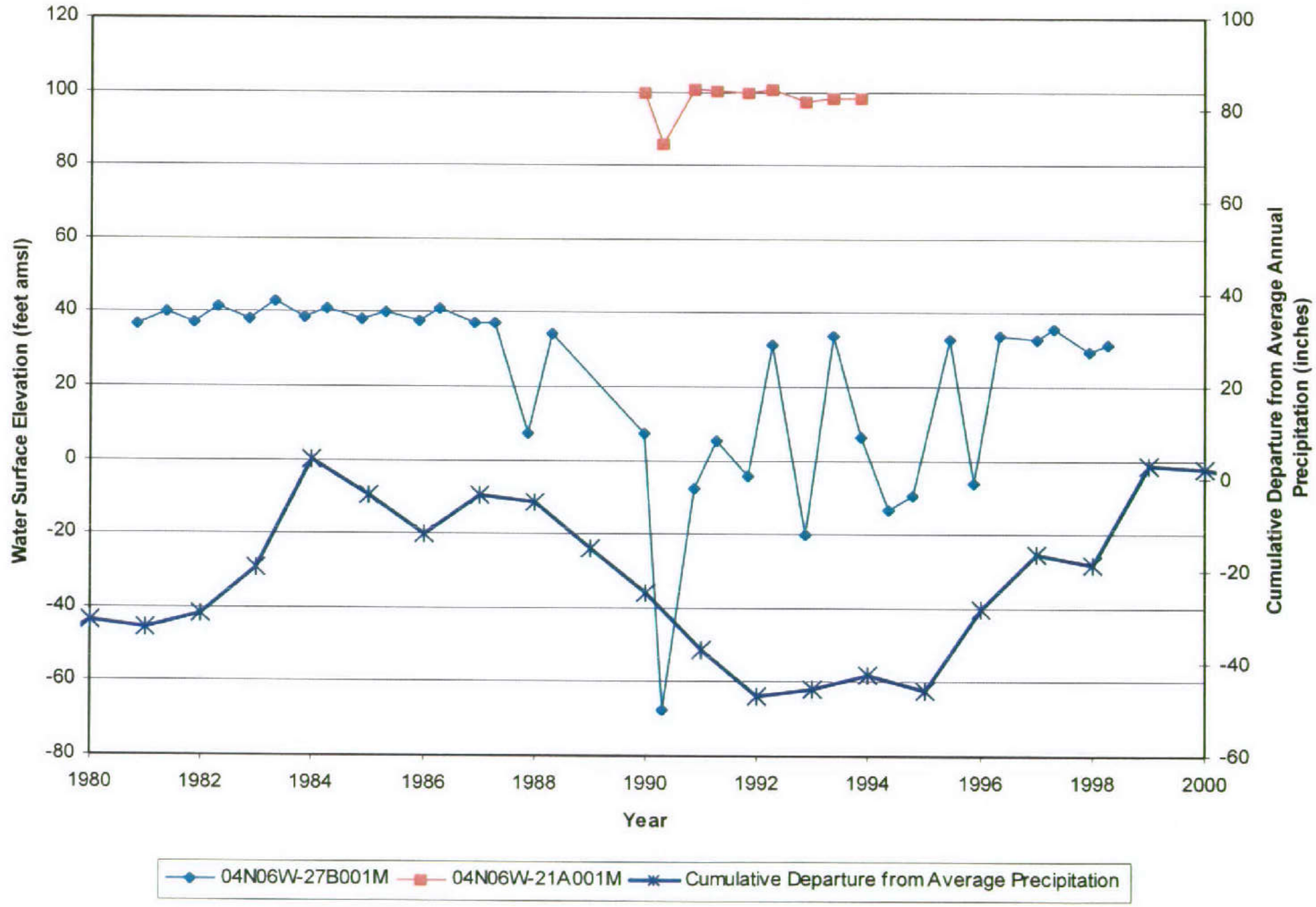
REPORTED GROUNDWATER WELLS IN THE SITE VICINITY

SWL	DJ	10/2005
N0410		5

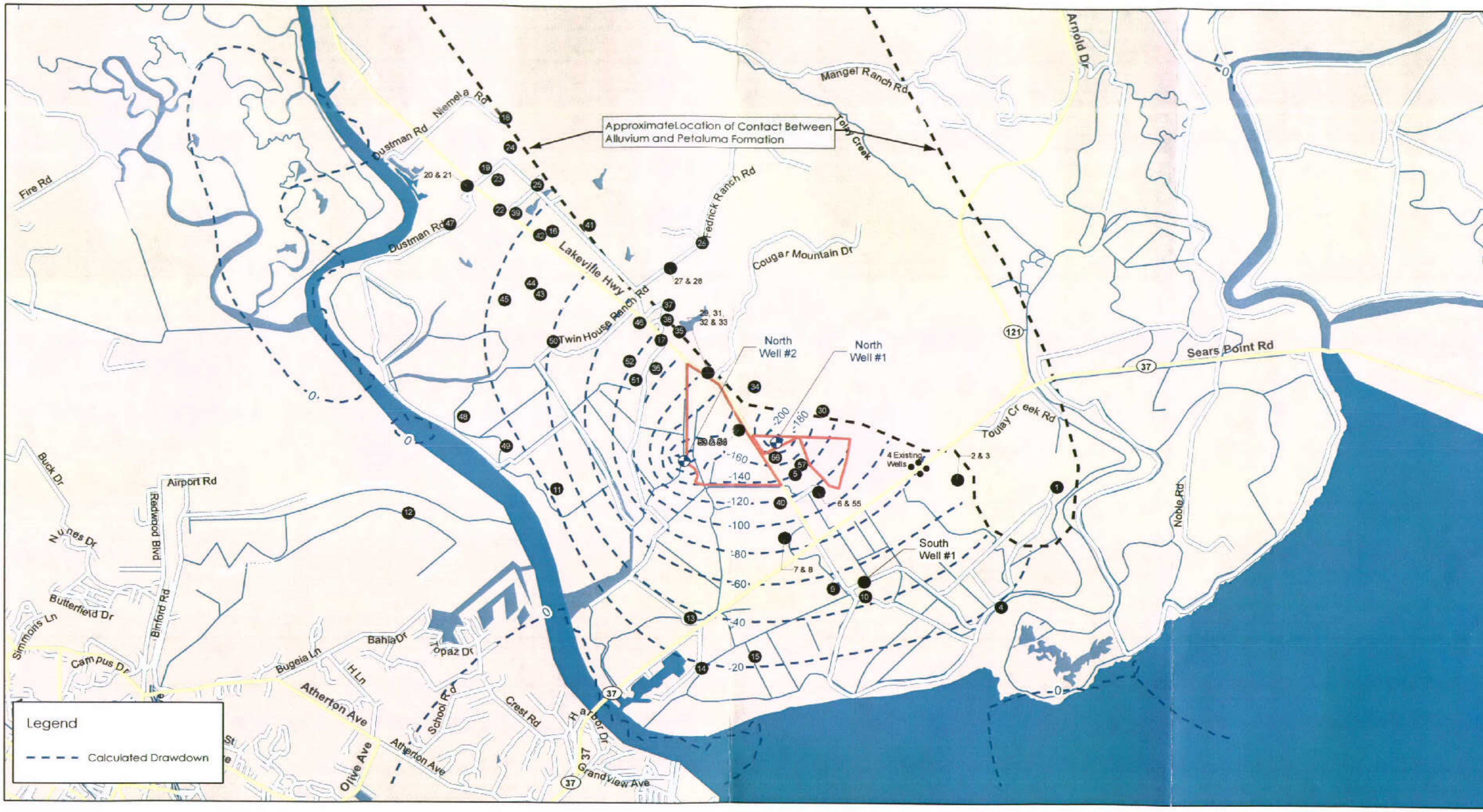


LAKEVILLE HYDROGEOLOGIC STUDY	WorleyParsons Komex resources & energy			
ALLUVIAL AQUIFER WELL HYDROGRAPHS	SWL	DJ	10/2005	
	N0410		6	

Hydrographs for Wells Completed in the Petaluma Formation Aquifer



LAKEVILLE HYDROGEOLOGIC STUDY	WorleyParsons Komex resources & energy		
PETALUMA FORMATION WELL HYDROGRAPHS	SWL	DJ	10/2005
	N0410	7	



Notes:

1 - Basemap data from ESRI DVD Dataset
2 - Projection in NAD 83 CA State Plane Zone II
3 - Location of all wells is approximate and was not field verified. See Table 1 for well construction details
4 - Contour lines shown on this figure depict analytical modeling calculations only and are not representative of predicted drawdown magnitudes or groundwater surface elevations. See text for explanation

LAKEVILLE HYDROGEOLOGIC STUDY

WorleyParsons Komex

resources & energy

ANALYTICAL ELEMENT MODELING RESULTS

SWL	DJ	10/2005
N0410		**8**

APPENDIX A

DATA FROM PUMPING TESTS PERFORMED BY HYDROSCIENCE

HydroScience Engineers, Inc.

TEST DATA SHEET

☑ Pumping Well ☐ Observation Well

Owner: SC Sonoma Development, LLC Well Name: North Well No. 1 Well Location: Lakeville Road & Hwy. 37 - Sonoma County

SWL Depth: * ft., bgs Reference Point Elevation: 1.25 ft., ags From: Top of flange Test Date: 08/19/03

* Artisian (~1.75 ft. ags.) Test Pump @ ~ 413ft bgs

STEP DRAWDOWN ☐ CONSTANT RATE ☑ RECOVERY ☐

Time of Day	Time (minutes)	Depth to Water (ft)	Drawdown (ft)	Flow Rate (gpm)	Comments
1:45 PM	0	*		90	Totalizer = 19.6236; McCrometer No. 01-03645-3"
1:46 PM	1	33		90	AG Well No. 2 is artisian
1:47 PM	2	43 1/2		90	
1:48 PM	3	51 1/2		90	
1:49 PM	4	58		90	
1:50 PM	5	62		90	
1:55 PM	10	78 1/2		90	
2:00 PM	15	88		90	
2:05 PM	20	95 1/2		90	
2:10 PM	25	101 1/2		90	
2:15 PM	30	106 1/2		90	
2:20 PM	35	110		90	
2:25 PM	40	114		90	
2:30 PM	45	116		90	
2:35 PM	50	118 1/2		90	
2:40 PM	55	121 1/2		90	Opening valve
2:45 PM	60	123 1/2		90	Opening valve
3:00 PM	75	131 1/2		90	
3:15 PM	90	137 1/2		90	
3:30 PM	105	144		90	

HydroScience Engineers, Inc.

TEST DATA SHEET

☑ Pumping Well ☐ Observation Well

Owner: SC Sonoma Development, LLC Well Name: North Well No. 1 Well Location: Lakeville Road & Hwy. 37 - Sonoma County

SWL Depth: * ft., bgs Reference Point Elevation: 1.25 ft., ags From: Top of flange Test Date: 08/19/03

STEP DRAWDOWN ☐ CONSTANT RATE ☑ RECOVERY ☐

Time of Day	Time (minutes)	Depth to Water (ft)	Drawdown (ft)	Flow Rate (gpm)	Comments
3:45 AM	120	149 1/4		75	3:45pm changed Q=75 because can't control pump @ Q=90. Valve NWP open.
4:00 AM	135	137 3/4		75	
4:15 AM	150	137 3/4		75	
4:30 AM	165	139 1/4		75	
4:45 AM	180	140 3/4		75	
5:15 AM	210	143 3/4		75	
5:45 AM	240	146 1/2		75	
6:15 AM	270	149		75	
6:45 AM	300	151 1/4		75	
7:15 AM	330	153 3/4		75	
7:45 AM	360	155 3/4		75	
8:45 AM	420	159 3/4		75	
9:45 AM	480	162 3/4		75	
10:45 AM	540	165 1/2		75	
11:45 AM	600	167 1/2		75	
12:45 PM	660	170		75	
1:45 PM	720	171 3/4		75	
3:45 PM	840	176 1/2		75	opened valve Q dropped slightly below Q = 75
5:45 PM	960	182		75	
7:45 PM	1080	185 1/4		75	

HydroScience Engineers, Inc.

TEST DATA SHEET

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SWL Depth: * ft., bgs Reference Point Elevation: 1.25 ft., ags From: Top of flange Test Date: 08/19/03

STEP DRAWDOWN ☐ CONSTANT RATE ☑ RECOVERY ☐

Time of Day	Time (minutes)	Depth to Water (ft)	Drawdown (ft)	Flow Rate (gpm)	Comments
9:45 AM	1200	189		75	8:07am open valve; 8:40am opened valve a little more.
11:45 AM	1320	191		75	
1:45 PM	1440	192		75	
3:45 PM	1560	193		75	
5:45 PM	1680	194		75	
7:45 PM	1800	185.5		75	
9:45 PM	1920	197.5		75	
11:45 PM	2040	199		75	
1:45 AM	2160	202.25		75	
3:45 AM	2280	205.75		75	
5:45 AM	2400	208		75	3:10am opened valve
7:45 AM	2520	210		75	8:30am opened valve
9:45 AM	2640	214.25		75	
11:45 AM	2760	215		75	
1:45 PM	2880	215.25		75	Totalizer = 20.2826

HydroScience Engineers, Inc.

TEST DATA SHEET

☑ Pumping Well ☐ Observation Well

Owner: SC Sonoma Development, LLC Well Name: North Well No. 1 Well Location: Lakeville Road & Hwy. 37 - Sonoma County

SWL Depth: * ft., bgs Reference Point Elevation: 1.25 ft., ags From: Top of flange Test Date: 08/19/03

STEP DRAWDOWN ☐ CONSTANT RATE ☐ RECOVERY ☑

Time of Day	Time (minutes)	Depth to Water (ft)	Drawdown (ft)	Flow Rate (gpm)	Comments
1:48 PM	0	215 1/4			
1:49 PM	1	183			
1:50 PM	2	170			
1:51 PM	3	161 1/2			
1:52 PM	4	155 3/4			
1:53 PM	5	151 1/4			
1:58 PM	10	137			
2:03 PM	15	125			
2:08 PM	20	118			
2:13 PM	25	113 3/4			
2:18 PM	30	106 1/4			
2:23 PM	35	102			
2:28 PM	40	97 1/2			
2:33 PM	45	94			
2:38 PM	50	91			
2:43 PM	55	88			
2:48 PM	60	85			
3:03 PM	75	78			
3:18 PM	90	72 1/4			
3:33 PM	105	67 1/4			
3:48 PM	120	63 1/4			

HydroScience Engineers, Inc.

TEST DATA SHEET

☑ Pumping Well ☐ Observation Well

Owner: SC Sonoma Development, LLC Well Name: North Well No. 1 Well Location: Lakeville Road & Hwy. 37 - Sonoma County

SWL Depth: * ft., bgs Reference Point Elevation: 1.25 ft., ags from: Top of Well Flange Test Date: 08/08/03

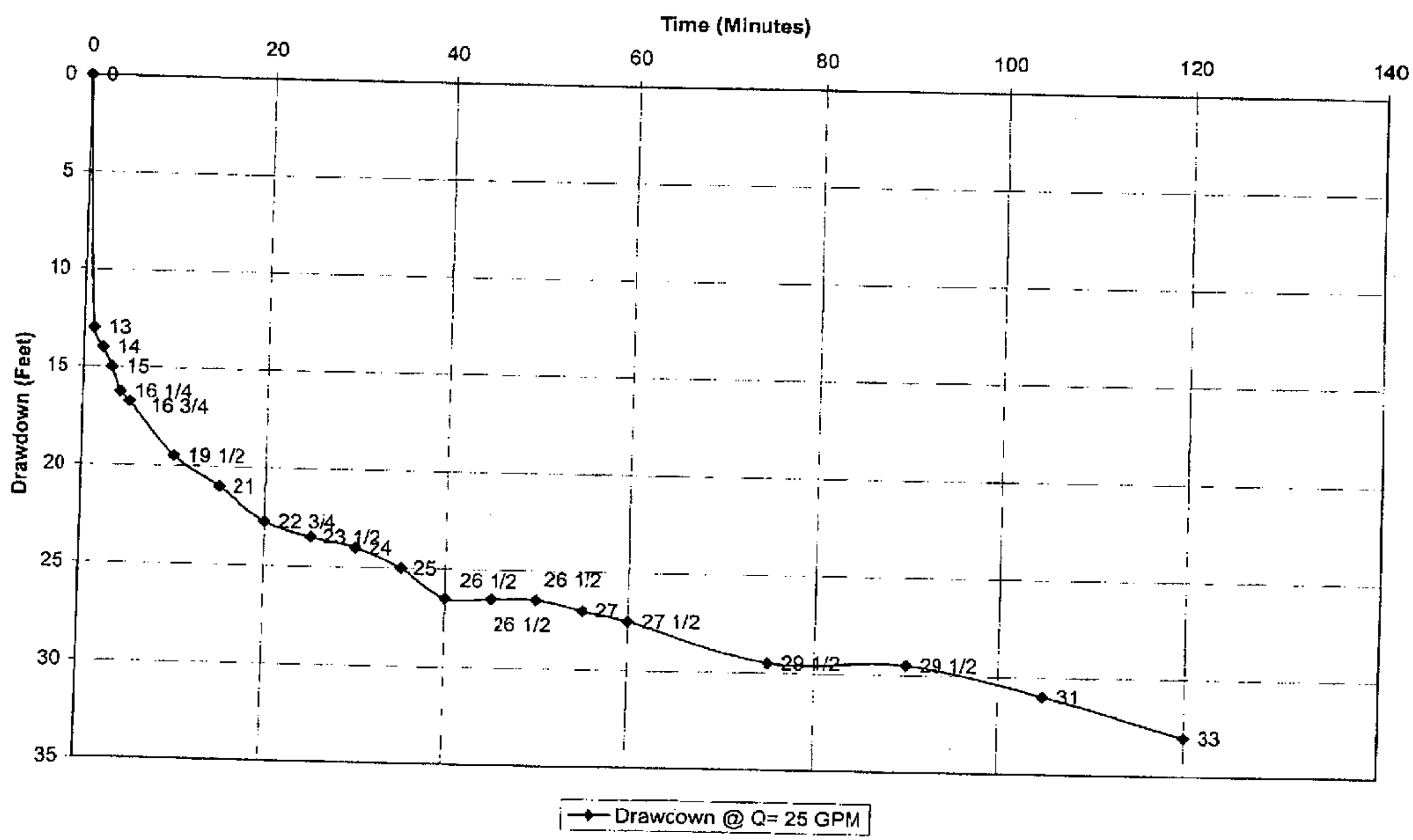
* = Artesian

Test Pump located @ ~ 413 ft. bgs.

STEP DRAWDOWN ☑ CONSTANT RATE ☐ RECOVERY ☐

Time of Day	Time (minutes)	Depth to Water (ft)	Drawdown (ft)	Flow Rate (gpm)	Comments
7:20 AM	0	*	0	25	Totalizer = 112.0486; Assume SWL ~ 6 ft ags
7:21 AM	1	9 1/2	13	25	Agricultural Wel No.2 is artesian at beginning of test
7:22 AM	2	10 1/2	14	25	
7:23 AM	3	11 1/2	15	25	
7:24 AM	4	12 3/4	16 1/4	25	
7:25 AM	5	13 1/4	16 3/4	25	
7:30 AM	10	16	19 1/2	25	
7:35 AM	15	17 1/2	21	25	
7:40 AM	20	19 1/4	22 3/4	25	
7:45 AM	25	20	23 1/2	25	
7:50 AM	30	20 1/2	24	25	Agriculture Well No. 2 is not longer artesian
7:55 AM	35	21 1/2	25	25	
8:00 AM	40	23	26 1/2	25	
8:05 AM	45	23	26 1/2	25	
8:10 AM	50	23	26 1/2	25	
8:15 AM	55	23 1/2	27	25	
8:20 AM	60	24	27 1/2	25	
8:35 AM	75	26	29 1/2	25	
8:50 AM	90	26	29 1/2	25	
9:05 AM	105	27 1/2	31	25	
9:20 AM	120	29 1/2	33	25	

North Well No 1 Step Drawdown Test



HydroScience Engineers, Inc.

TEST DATA SHEET

☑ Pumping Well ☐ Observation Well

Owner: SC Sonoma Development, LLC Well Name: North Well No. 1 Well Location: Lakeville Road & Hwy. 37 - Sonoma County

SWL Depth: * ft., bgs Reference Point Elevation: 1.25 ft., ags from: Top of Well Flange Test Date: 08/08/03

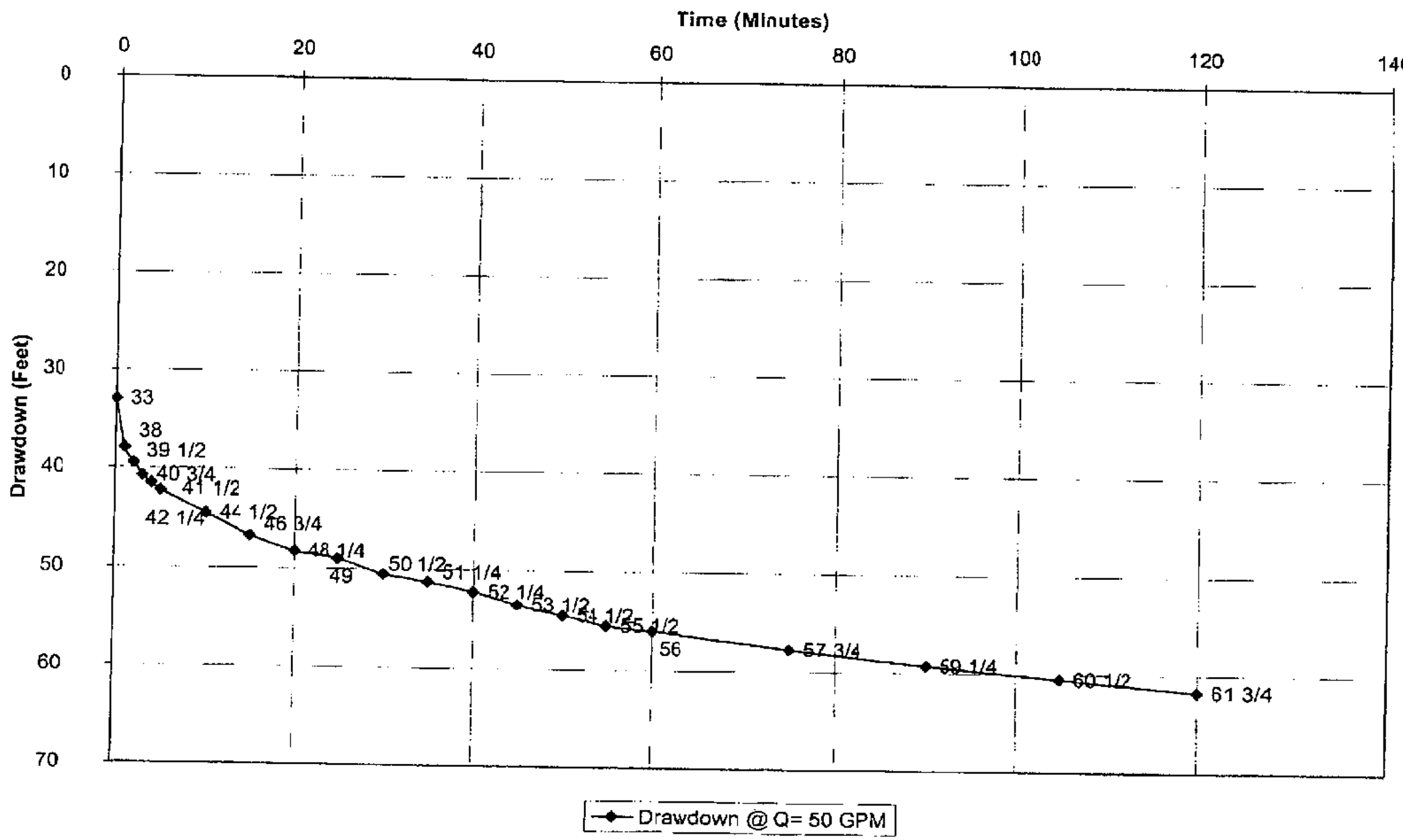
* = Artesian at 7:20am

Pump @ 413 ft. bgs.

STEP DRAWDOWN ☑ CONSTANT RATE ☐ RECOVERY ☐

Time of Day	Time (minutes)	Depth to Water (ft)	Drawdown (ft)	Flow Rate (gpm)	Comments
9:20 AM	0	29 1/2	33	50	
9:21 AM	1	34 1/2	38	50	
9:22 AM	2	36	39 1/2	50	
9:23 AM	3	37 1/4	40 3/4	50	
9:24 AM	4	38	41 1/2	50	
9:25 AM	5	38 3/4	42 1/4	50	
9:30 AM	10	41	44 1/2	50	
9:35 AM	15	43 1/4	46 3/4	50	
9:40 AM	20	44 3/4	46 1/4	50	
9:45 AM	25	45 1/2	49	50	
9:50 AM	30	47	50 1/2	50	
9:55 AM	35	47 3/4	51 1/4	50	
10:00 AM	40	48 3/4	52 1/4	50	
10:05 AM	45	50	53 1/2	50	
10:10 AM	50	51	54 1/2	50	
10:15 AM	55	52	55 1/2	50	
10:20 AM	60	52 1/2	56	50	
10:35 AM	75	54 1/4	57 3/4	50	
10:50 AM	90	55 3/4	59 1/4	50	
11:05 AM	105	57	60 1/2	50	
11:20 AM	120	58 1/4	61 3/4	50	

North Well No.1 Step Drawdown Test



HydroScience Engineers, Inc.

TEST DATA SHEET

☑ Pumping Well ☐ Observation Well

Owner: SC Sonoma Development, LLC Well Name: North Well No. 1 Well Location: Lakeville Road & Hwy. 37 - Sonoma County

SWL Depth: * ft., bgs Reference Point Elevation: 1.25 ft., ags from: Top of Well Flange Test Date: 08/08/03

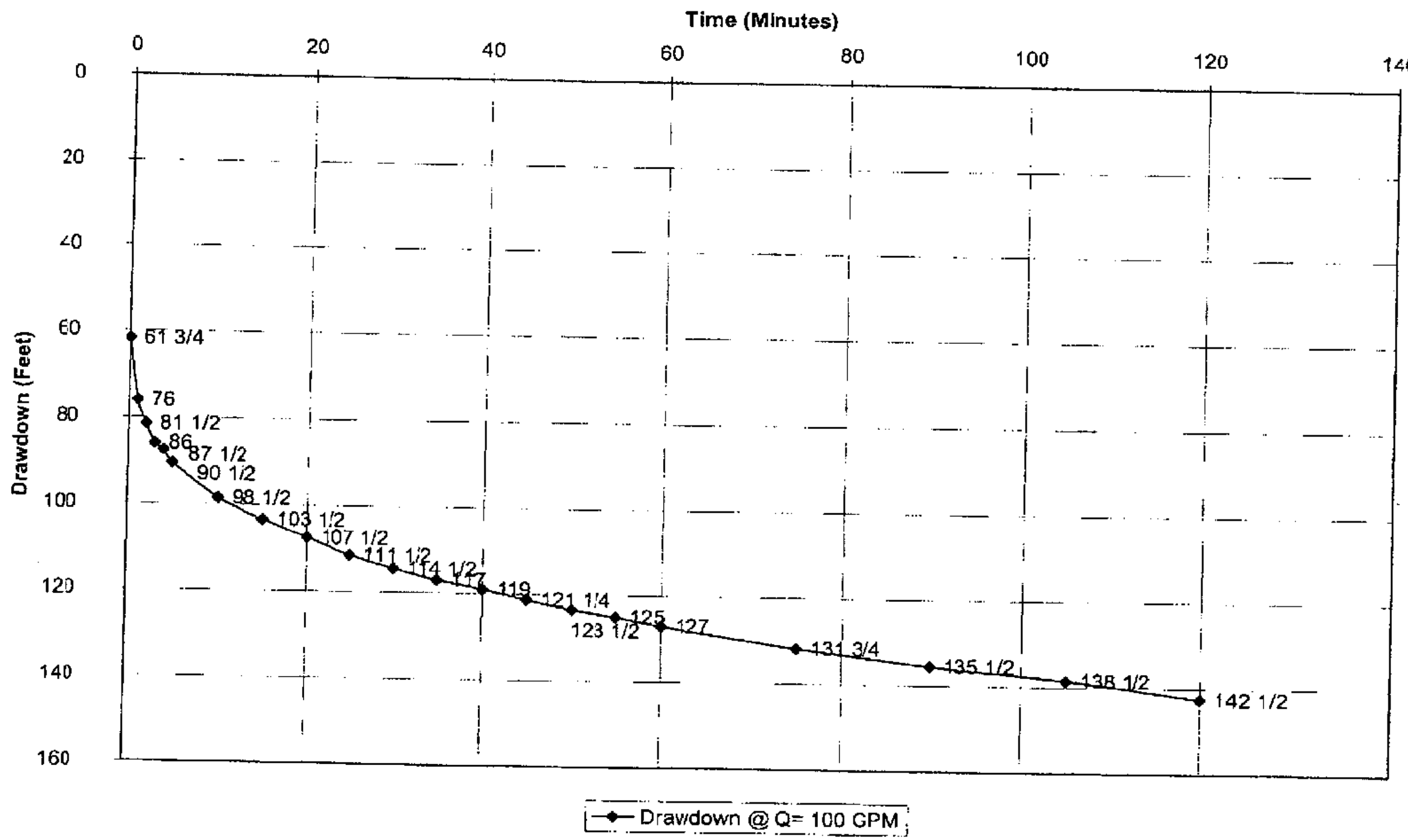
* = Artesian at 7:20am

Pump @ 413 ft. bgs.

STEP DRAWDOWN ☑ CONSTANT RATE ☐ RECOVERY ☐

Time of Day	Time (minutes)	Depth to Water (ft)	Drawdown (ft)	Flow Rate (gpm)	Comments
11:20 AM	0	58 1/4	61 3/4	100	
11:21 AM	1	72 1/2	76	100	
11:22 AM	2	78	81 1/2	100	
11:23 AM	3	82 1/2	86	100	
11:24 AM	4	84	87 1/2	100	
11:25 AM	5	87	90 1/2	100	
11:30 AM	10	95	98 1/2	100	
11:35 AM	15	100	103 1/2	100	
11:40 AM	20	104	107 1/2	100	
11:45 AM	25	108	111 1/2	100	
11:50 AM	30	111	114 1/2	100	
11:55 AM	35	113 1/2	117	100	
12:00 PM	40	115 1/2	119	100	
12:05 PM	45	117 3/4	121 1/4	100	
12:10 PM	50	120	123 1/2	100	
12:15 PM	55	121 1/2	125	100	
12:20 PM	60	123 1/2	127	100	
12:35 PM	75	128 1/4	131 3/4	100	
12:50 PM	90	132	135 1/2	100	
1:05 PM	105	135	138 1/2	100	
1:20 PM	120	139	142 1/2	100	

North Well No.1 Step Drawdown Test



HydroScience Engineers, Inc.

TEST DATA SHEET

☑ Pumping Well ☐ Observation Well

Owner: SC Sonoma Development, LLC Well Name: North Well No. 1 Well Location: Lakeville Road & Hwy. 37 - Sonoma County

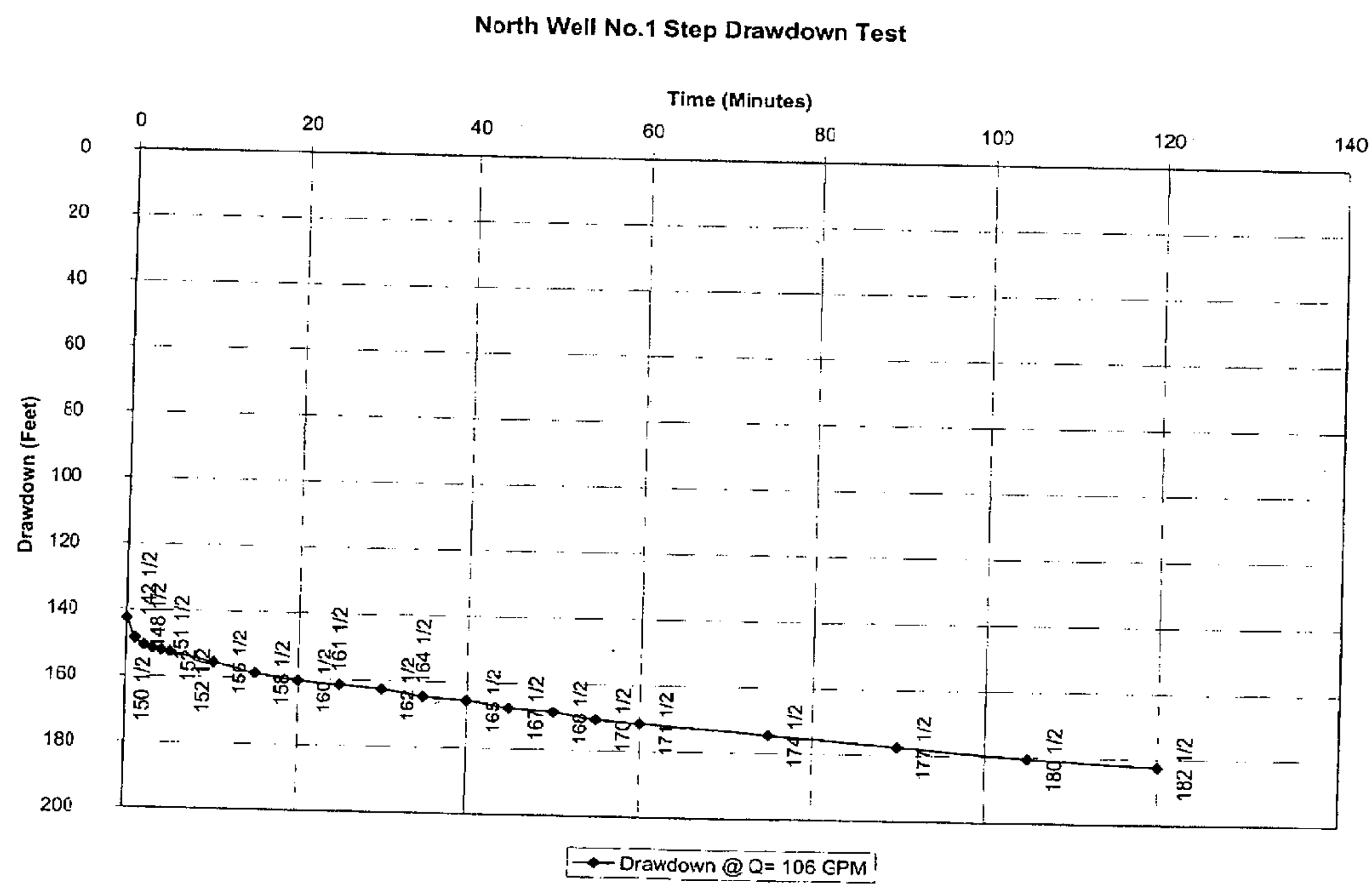
SWL Depth: * ft., bgs Reference Point Elevation: 1.25 ft., ags From: Top of Well Flange Test Date: 08/08/03

* = Artesian at 7:20am

Pump @ 413 ft. bgs.

STEP DRAWDOWN ☑ **CONSTANT RATE** ☐ **RECOVERY** ☐

Time of Day	Time (minutes)	Depth to Water (ft)	Drawdown (ft)	Flow Rate (gpm)	Comments
1:20 PM	0	139	142 1/2	110	Discharge valve wide open. Maximum pump discharge for a 4-inch dia. pump
1:21 PM	1	145	148 1/2	110	
1:22 PM	2	147	150 1/2	110	
1:23 PM	3	148	151 1/2	110	
1:24 PM	4	148 1/2	152	108	Discharge valve still wide open. Can not control pump discharge
1:25 PM	5	149	152 1/2	108	
1:30 PM	10	152	155 1/2	108	
1:35 PM	15	155	158 1/2	108	
1:40 PM	20	157	160 1/2	107	Discharge valve still wide open. Can not control pump discharge
1:45 PM	25	158	161 1/2	107	
1:50 PM	30	159	162 1/2	107	
1:55 PM	35	161	164 1/2	107	
2:00 PM	40	162	165 1/2	107	
2:05 PM	45	164	167 1/2	106	Discharge valve still wide open. Can not control pump discharge
2:10 PM	50	165	168 1/2	106	
2:15 PM	55	167	170 1/2	106	
2:20 PM	60	168	171 1/2	106	
2:35 PM	75	171	174 1/2	106	
2:50 PM	90	174	177 1/2	105	Discharge valve still wide open. Can not control pump discharge
3:05 PM	105	177	180 1/2	107	Discharge valve still wide open. Can not control pump discharge
3:20 PM	120	179	182 1/2	107	



North Well No.1 Step Drawdown Test
Time (Minutes)
Drawdown (Feet)
Drawdown @ Q= 106 GPM
0
20
40
60
80
100
120
140
160
180
200
142 1/2
148 1/2
150 1/2
151 1/2
152 1/2
156 1/2
158 1/2
160 1/2
161 1/2
162 1/2
164 1/2
165 1/2
167 1/2
168 1/2
170 1/2
171 1/2
174 1/2
177 1/2
180 1/2
182 1/2

HydroScience Engineers, Inc.

TEST DATA SHEET

☑ Pumping Well ☐ Observation Well

Owner: SC Sonoma Development, LLC Well Name: North Well No. 1 Well Location: Lakeville Road & Hwy. 37 - Sonoma County

SWL Depth: ___*___ ft., bgs Reference Point Elevation: ___1.25___ ft., ags From: Top of Well Flange Test Date: ___08/08/03___

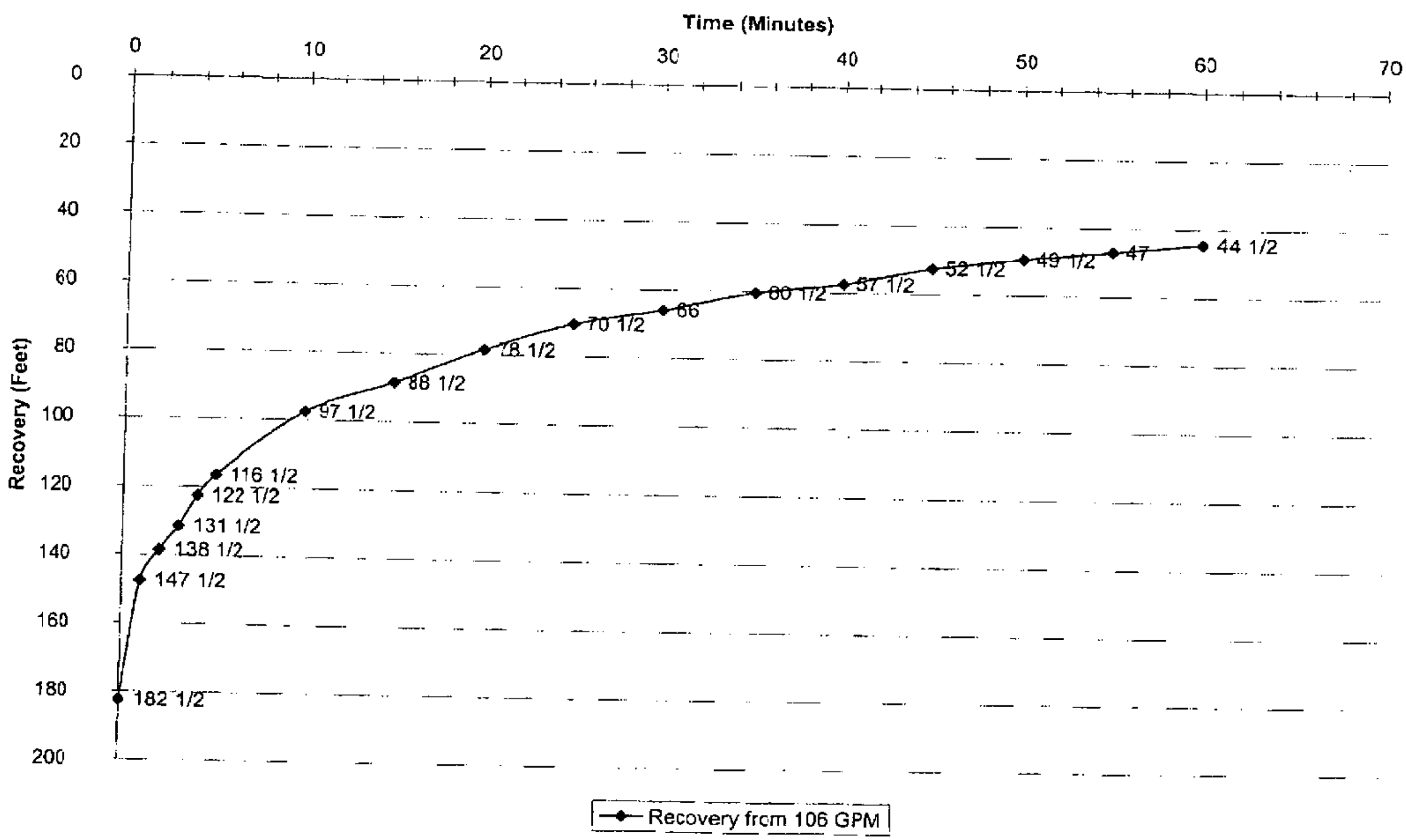
*** = Artisian at 7:20am**

Pump @ 413 ft. bgs.

STEP DRAWDOWN ☐ **CONSTANT RATE** ☐ **RECOVERY** ☑

Time of Day	Time (minutes)	Depth to Water (ft)	Drawdown (ft)	Flow Rate (gpm)	Comments
3:20 PM	0	179	182 1/2		Starting the recovery test
3:21 PM	1	144	147 1/2		
3:22 PM	2	135	138 1/2		
3:23 PM	3	128	131 1/2		
3:24 PM	4	119	122 1/2		
3:25 PM	5	113	116 1/2		
3:30 PM	10	94	97 1/2		
3:35 PM	15	85	88 1/2		
3:40 PM	20	75	78 1/2		
3:45 PM	25	67	70 1/2		
3:50 PM	30	62 1/2	66		
3:55 PM	35	57	60 1/2		
4:00 PM	40	54	57 1/2		
4:05 PM	45	49	52 1/2		
4:10 PM	50	46	49 1/2		
4:15 PM	55	43 1/2	47		
4:20 PM	60	41	44 1/2		End recovery test

North Well No.1 Recovery Test



HydroScience Engineers, Inc.

TEST DATA SHEET

☑ Pumping Well ☐ Observation Well

Owner: SC Sonoma Development, LLC Well Name: North Well No. 2 Well Location: Lakeville Road & Hwy. 37 - Sonoma County

SWL Depth: 10.75 ft., bgs Reference Point Elevation: 2.0 ft., ags From: Top of pump head Test Date: 07/23/03

STEP DRAWDOWN ☐ CONSTANT RATE ☑ RECOVERY ☐

Time of Day	Time (minutes)	Depth to Water (ft)	Drawdown (ft)	Flow Rate (gpm)	Comments
8:15 AM	0	10 3/4		125	Totalizer = 257.8037 Acre - ft.
8:16 AM	1	35		125	Throttling pump
8:17 AM	2	55		125	Throttling pump
8:18 AM	3	77		125	Throttling pump
8:19 AM	4	92		125	Throttling pump
8:20 AM	5	105		125	Throttling pump
8:25 AM	10	153 1/2		125	Throttling pump
8:30 AM	15	185 1/2		125	Throttling pump
8:35 AM	20	209 1/2		125	Throttling pump
8:40 AM	25	228 1/2		125	Throttling pump
8:45 AM	30	244		125	Throttling pump
8:50 AM	35	254		125	Mud & san is showing up in sand tester
8:55 AM	40	258 1/2		125	Throttling pump
9:00 AM	45	263		125	Throttling pump
9:05 AM	50	270 1/2		125	Starts sand tester at 9:10AM
9:10 AM	55	278 1/2		125	
9:15 AM	60	280 1/2		125	Sand tester = 1 1/2 mL of bentonite & rust chips
9:30 AM	75	291		125	
9:45 AM	90	296 1/2		125	
10:00 AM	105	303 1/2		125	

HydroScience Engineers, Inc.

TEST DATA SHEET

☑ Pumping Well ☐ Observation Well

Owner: SC Sonoma Development, LLC Well Name: North Well No. 2 Well Location: Lakeville Road & Hwy. 37 - Sonoma County

SWL Depth: 10.75 ft., bgs Reference Point Elevation: 2.0 ft., ags From: Top of pump head Test Date: 07/23 & 24/03

STEP DRAWDOWN ☐ CONSTANT RATE ☑ RECOVERY ☐

Time of Day	Time (minutes)	Depth to Water (ft)	Drawdown (ft)	Flow Rate (gpm)	Comments
10:15 AM	120	309 1/2		125	
10:30 AM	135	316		125	
10:45 AM	150	319 1/2		125	
11:00 AM	165	320 1/2		125	
11:15 AM	180	320 3/4		125	
11:45 AM	210	320 3/4		125	
12:15 PM	240	324 1/4		125	
12:45 PM	270	323 3/4		125	
1:15 AM	300	329 1/2		125	
1:45 AM	330	331		125	
2:15 AM	360	332		125	
3:15 AM	420	335		125	Throttling pump
4:15 AM	480	352 3/4		125	Another well on? Drawdown dropping!!
5:15 AM	540	376		125	Water is cloudy; First 3 screens are uncovered
6:15 AM	600	394 3/4		125	
7:15 AM	660	393 1/2		125	Sim Blake on-site 7:30 PM
8:15 AM	720	393 1/4		125	9:15 392 3/4
10:15 AM	840	394		125	
12:15 PM	960	397 1/4		125	
2:15 AM	1080	397 1/2		125	

HydroScience Engineers, Inc.

TEST DATA SHEET

☑ Pumping Well ☐ Observation Well

Owner: SC Sonoma Development, LLC Well Name: North Well No. 2 Well Location: Lakeville Road & Hwy. 37 - Sonoma County

SWL Depth: 10.75 ft., bgs Reference Point Elevation: 2.0 ft., ags From: Top of pump head Test Date: 07/24 & 25/03

STEP DRAWDOWN ☐ CONSTANT RATE ☑ RECOVERY ☐

Time of Day	Time (minutes)	Depth to Water (ft)	Drawdown (ft)	Flow Rate (gpm)	Comments
4:15 AM	1200	397 1/2		125	Slight pump ad.
6:15 AM	1320	442 1/2		125	Either 1 another pump on or 2 Deduction of upper aquifer cloudy
8:15 AM	1440	342 1/2		120*	*Q = 100 gpm @ 7:00 am WL = 503 Pump adj - see attached
10:15 AM	1560	409		125	Q = 125 gpm @ 8:00 am WL = 309
12:15 PM	1680	272 1/2		100**	** Q = 100 gpm @ 11:00am WL = 500 Tot. = 258.394 WQS at 12:15
2:15 PM	1800	268 1/2		100	
4:15 PM	1920	269 1/2		100	
6:15 PM	2040	270		100	
8:'5 PM	2160	270		125	
10:15 PM	2280	270		125	
12:15 AM	2400	268 1/4		125	
2:15 AM	2520	268 1/2		125	Sim getting sleepy
4:15 AM	2640	270		125	
6:15 AM	2760	272 1/2		125	
8:15 AM	2880	269 3/4		125	Totalizer = 258.7590

HydroScience Engineers, Inc.

TEST DATA SHEET

☑ Pumping Well ☐ Observation Well

Owner: SC Sonoma Development, LLC Well Name: North Well No. 2 Well Location: Lakeville Road & Hwy. 37 - Sonoma County

SWL Depth: 10.75 ft., bgs Reference Point Elevation: 2.0 ft., ags From: Top of pump head Test Date: 07/24

STEP DRAWDOWN ☐ CONSTANT RATE ☑ RECOVERY ☐

Time of Day	Time (minutes)	Depth to Water (ft)	Drawdown (ft)	Flow Rate (gpm)	Comments
6:30 AM		472		100	
6:40 AM		494		125	
6:50 AM					
6:52 AM					Pump broke suction, cloudy water
6:53 AM		502			Adj. Temporarily throttled back, retried 125 gpm 496' clear water
7:00 AM		502 1/2		105	
7:35 AM		372		100	Throttling back pump to Q = 100 gpm
7:45 AM		343		100	WT has recovered fast!
7:50 AM		331 1/2		100	
8:00 AM		309		100	
8:07 AM		329		125	Throttle pump to Q = 125 gpm
8:11 AM		338		125	
8:30 AM		358		125	
9:00 AM		375		125	
9:30 AM		379 1/2		125	
10:00 AM		396		125	
10:15 AM		409		125	
10:30 AM		436		125	
10:42 AM		464		125	
10:45 AM		477		125	
11:00 AM		500		100	Reduce flow rate to Q = 100 gpm
11:07 AM		450		100	
11:15 AM		421		100	
11:30 AM		346 1/2		100	
12:00 PM		280		100	
1:41 AM		260		100	
2:15 AM		268 1/2		100	
3:15 AM		269 1/4		100	

HydroScience Engineers, Inc.

TEST DATA SHEET

☑ Pumping Well ☐ Observation Well

Owner: SC Sonoma Development, LLC Well Name: North Well No. 2 Well Location: Lakeville Road & Hwy. 37 - Sonoma County

SWL Depth: 10.75 ft., bgs Reference Point Elevation: 2.0 ft., ags From: Top of pump head Test Date: 07/25/03

STEP DRAWDOWN ☐ CONSTANT RATE ☐ RECOVERY ☑

Time of Day	Time (minutes)	Depth to Water (ft)	Drawdown (ft)	Flow Rate (gpm)	Comments
8:15 AM	0	269 3/4			
8:17 AM	2	172			
8:18 AM	3	157			
8:19 AM	4	147 1/2			
8:20 AM	5	138			
8:25 AM	10	102 1/2			
8:30 AM	15	82			
8:35 AM	20	70			
8:40 AM	25	62 1/2			
8:45 AM	30	58			
8:50 AM	35	55 1/4			
8:55 AM	40	53 1/4			
9:00 AM	45	52			
9:05 AM	50	50			
9:10 AM	55	49 1/2			
9:15 AM	60	48 1/2			
9:30 AM	75	46 3/4			
9:45 AM	90	45			
10:00 AM	105	44			
10:15 AM	120	42 1/2			

HydroScience Engineers, Inc.

TEST DATA SHEET

☑ Pumping Well ☐ Observation Well

Owner: SC Sonoma Development, LLC Well Name: North Well No. 2 Well Location: Lakeville Road & Hwy. 37 - Sonoma County

SWL Depth: 10.75 ft., bgs Reference Point Elevation: 2.0 ft., ags From: Top of pump head Test Date: 07/25/03

STEP DRAWDOWN ☐ CONSTANT RATE ☐ RECOVERY ☑

Time of Day	Time (minutes)	Depth to Water (ft)	Drawdown (ft)	Flow Rate (gpm)	Comments
10:30 AM	135	42			
10:45 AM	150	41			
11:00 AM	165	40			
11:15 AM	180	39			
12:30 PM	255	37 1/4			End of test

HydroScience Engineers, Inc.

TEST DATA SHEET

☑ Pumping Well ☐ Observation Well

Owner: SC Sonoma Development, LLC Well Name: North Well No. 2 Well Location: Lakeville Road & Hwy. 37 - Sonoma County

SWL Depth: 5.50 @ 8:00 pm ft., bgs Reference Point Elevation: 2 ft., ags From: Top of Pump's head Test Date: 07/22/03

STEP DRAWDOWN ☑ CONSTANT RATE ☐ RECOVERY ☐

Time of Day	Time (minutes)	Depth to Water (ft)	Drawdown (ft)	Flow Rate (gpm)	Comments
8:45 AM	0	46 1/2		100	257.635.0 A-F = Totalizer
8:46 AM	1	65		100	
8:47 AM	2	78		100	
8:48 AM	3	87		100	
8:49 AM	4	95		100	
8:50 AM	5	103 1/2		100	
8:55 AM	10	127 1/2		100	
9:00 AM	15	143 1/2		100	
9:05 AM	20	154		100	
9:10 AM	25	160		100	
9:15 AM	30	166 1/2		100	
9:20 AM	35	170 1/2		100	
9:25 AM	40	174 1/4		100	
9:30 AM	45	179 3/4		100	
9:35 AM	50	181 3/4		100	
9:40 AM	55	185 1/4		100	
9:45 AM	60	189 1/2		100	
10:00 AM	75	197 3/4		100	
10:15 AM	90	207		100	
10:30 AM	105	208 1/4		100	
10:45 AM	120	208 1/4		100	
11:00 AM	135	213		100	Throttle moved @ 10:45
11:15 AM	150	216		100	
11:30 AM	165	214 1/2		100	
11:45 AM	180	214 1/2		100	

HydroScience Engineers, Inc.

TEST DATA SHEET

☑ Pumping Well ☐ Observation Well

Owner: SC Sonoma Development, LLC Well Name: North Well No. 2 Well Location: Lakeville Road & Hwy. 37 - Sonoma County

SWL Depth: 5.50 @ 8:00 pm ft., bgs Reference Point Elevation: 2 ft., ags From: Top of Pump's head Test Date: 07/22/03

STEP DRAWDOWN ☑ **CONSTANT RATE** ☐ **RECOVERY** ☐

Time of Day	Time (minutes)	Depth to Water (ft)	Drawdown (ft)	Flow Rate (gpm)	Comments
11:45 AM	0	217		150	Throtte
11:46 AM	1	228		150	Throtte
11:47 AM	2	236		150	Throtte
11:48 AM	3	243		150	Throtte
11:49 AM	4	247 1/2		150	Throtte
11:50 AM	5	255		150	
11:55 AM	10	274		150	
12:00 PM	15	289 1/4		150	
12:05 PM	20	307 1/2		150	Throtte @ 12:08 PM
12:10 PM	25	317 1/4		150	
12:15 PM	30	325		150	
12:20 PM	35	333		150	Throttle @ 12:23 PM
12:25 PM	40	344 1/2		150	
12:30 PM	45	350		150	Throttle @ 12:31 PM
12:35 PM	50	363		150	
12:40 PM	55	366 3/4		150	
12:55 PM	60	374 3/4		150	
1:00 PM	75	363 1/2		150	Sample #1 at 12:55PM: cloudy
1:15 PM	90	319		150	pump air; meter erratic
1:30 PM	105	304 1/2		150	Sample #2 at 1:20PM: water is clear; pumping air
1:45 PM	120	303		150	pumping air: meter swinging wide
2:00 PM	135	312 1/2		150	pumping air: meter erratic & swing wide
2:15 PM	150	312 3/4		150	pump air, meter swing wide: sample #3
2:30 PM	165	311 1/4		150	pump air; meter swing wide (50 gal./min.)
2:45 PM	180	305 1/2		150	

HydroScience Engineers, Inc.

TEST DATA SHEET

☑ Pumping Well ☐ Observation Well

Owner: SC Sonoma Development, LLC Well Name: North Well No. 2 Well Location: Lakeville Road & Hwy. 37 - Sonoma County

SWL Depth: 5.50 @ 8:00 pm ft., bgs Reference Point Elevation: 2 ft., ags From: Top of Pump's head Test Date: 07/22/03

STEP DRAWDOWN ☑ CONSTANT RATE ☐ RECOVERY ☐

Time of Day	Time (minutes)	Depth to Water (ft)	Drawdown (ft)	Flow Rate (gpm)	Comments
2:45 PM	0	313		200	Throttle
2:46 PM	1	313 1/2		200	Throttle
2:47 PM	2	323		200	Throttle
2:48 PM	3	327 1/2		200	meter is swinging wide (~50 gpm)
2:49 PM	4	331		200	meter is swinging wide (~50 gpm)
2:50 PM	5	334		200	meter is swinging wide (~50 gpm)
2:55 PM	10	343 1/2		200	meter is swinging wide (~50 gpm)
3:00 PM	15	345 1/2		200	meter is swinging wide (~50 gpm)
3:05 PM	20	357		200	meter is swinging wide (~50 gpm)
3:10 PM	25	359 1/2		200	meter is swinging wide (~50 gpm)
3:15 PM	30	363		200	meter is swinging wide (~50 gpm)
3:20 PM	35	364 1/2		200	meter is swingirg wide (~50 gpm)
3:25 PM	40	365 3/4		200	End Totalizer - 257.803.7
3:30 PM	45	364		200	Throttle @ 12:31 PM
3:35 PM	50	362		200	
3:40 PM	55	351		200	
3:45 PM	60	337 1/2		200	Meter is swinging wider (~75 gpm)
4:00 PM	75	328 1/4		200	
4:15 PM	90	363		200	372 @ 4:18 meter swinging less (~50 gpm) 390 @ 4:22
4:30 PM	105	362		175	392 @ 4:39 400 @ 4:44 but 150 gpm only
4:45 PM	120	402		175	*can not hold 200 gpm

HydroScience Engineers, Inc.

TEST DATA SHEET

☑ Pumping Well ☐ Observation Well

Owner: SC Sonoma Development, LLC Well Name: North Well No. 2 Well Location: Lakeville Road & Hwy. 37 - Sonoma County

SWL Depth: 5.50 @ 8:00 pm ft., bgs Reference Point Elevation: 2 ft., ags From: Top of Pump's head Test Date: 07/22/03

STEP DRAWDOWN ☑ **CONSTANT RATE** ☐ **RECOVERY** ☐

Time of Day	Time (minutes)	Depth to Water (ft)	Drawdown (ft)	Flow Rate (gpm)	Comments
4:45 PM	0				
4:46 PM	1	340			
4:47 PM	2	254			
4:48 PM	3	235			
4:50 PM	4	215			
4:51 PM	5	198 1/2			
4:52 PM	6	173 1/2			
4:53 PM	7	159			
4:54 PM	8	145 1/4			
4:55 PM	9	133			
4:56 PM	10	122 3/4			
4:57 PM	11	112 1/2			
4:58 PM	12	104			
4:59 PM	13	96			
5:00 PM	14	89 3/4			
5:02 PM	16	77 3/4			
5:04 PM	18	68 3/4			
5:06 PM	20	60 1/2			
5:08 PM	22	55 3/4			
5:10 PM	24	51			
5:12 PM	26	47 1/2			
5:14 PM	28	44 1/4			
5:16 PM	30	42			
5:18 PM	32	39 1/2			
5:20 PM	34	38			
5:25 PM	39	34 3/4			
5:30 PM	44	34 1/2			
5:35 PM	49	31			
5:40 PM	54	29 1/4			
5:45 PM	59	28			

HydroScience Engineers, Inc.

TEST DATA SHEET

☑ Pumping Well ☐ Observation Well

Owner: SC Sonoma Development, LLC Well Name: South Well No. 1 Well Location: Lakeville Road & Hwy. 37 - Sonoma County

SWL Depth: * ft., bgs Reference Point Elevation: 1.25 ft., ags From: Top of flange Test Date: 08/19/03

* Artisian (~ 2.0 ft. ags.) Pump location @ ~ 300 bgs

STEP DRAWDOWN ☐ CONSTANT RATE ☑ RECOVERY ☐

Time of Day	Time (minutes)	Depth to Water (ft)	Drawdown (ft)	Flow Rate (gpm)	Comments
7:25 AM	0	*		200	Totalizer = 112.1925 AC ft.
7:26 AM	1	40		200	Water Specialty meter N°. 940835-3"
7:27 AM	2	50		200	
7:28 AM	3	54		200	
7:29 AM	4	55		200	
7:30 AM	5	56 1/2		200	
7:35 AM	10	67 1/2		200	
7:40 AM	15	66 1/2		200	
7:45 AM	20	67 1/4		200	
7:50 AM	25	68 1/2		200	
7:55 AM	30	69 3/4		200	
8:00 AM	35	70		200	
8:05 AM	40	70 1/2		200	
8:10 AM	45	70 3/4		200	Water is cloudy; a lot of bentonite!
8:15 AM	50	71 3/4		200	
8:20 AM	55	72 1/4		200	
8:25 AM	60	72 1/4		200	
8:40 AM	75	72 3/4		200	
8:55 AM	90	73 3/4		200	Water is cloudy
9:10 AM	105	74 1/4		200	

HydroScience Engineers, Inc.

TEST DATA SHEET

☑ Pumping Well ☐ Observation Well

Owner: SC Sonoma Development, LLC Well Name: South Well No. 1 Well Location: Lakeville Road & Hwy. 37 - Sonoma County

SWL Depth: * ft., bgs Reference Point Elevaton: 1.25 ft., ags From: Top of flange Test Date: 08/19/03

* Artisian (~ 2.0 ft. ags.)

Pump location @ ~ 300 bgs

STEP DRAWDOWN ☐ CONSTANT RATE ☑ RECOVERY ☐

Time of Day	Time (minutes)	Depth to Water (ft)	Drawdown (ft)	Flow Rate (gpm)	Comments
9:25 AM	120	74 1/2		200	
9:40 AM	135	74 1/2		200	
9:55 AM	150	76 1/2		200	Water is cloudy
10:10 AM	165	76 1/2		200	
10:25 AM	180	76 3/4		200	Water is cloudy
10:55 AM	210	76 1/2		200	
11:25 AM	240	76 3/4		200	
11:55 AM	270	76 1/2		200	Water is clear now
12:25 PM	300	76 3/4		200	
12:55 PM	330	77 1/4		200	
1:25 AM	360	78		200	
2:25 AM	420	79		200	
3:25 AM	480	79 1/2		200	
4:25 AM	540	80		200	Water is clear now
5:25 AM	600	80 1/2		200	
6:25 AM	660	80 1/2		200	Water clear
7:25 AM	720	81 1/4		200	
9:25 AM	840	81 1/2		200	
11:25 AM	960	82 1/4		200	
1:25 AM	1080	82 1/2		200	

HydroScience Engineers, Inc.

TEST DATA SHEET

☑ Pumping Well ☐ Observation Well

Owner: SC Sonoma Development, LLC Well Name: South Well No. 1 Well Location: Lakeville Road & Hwy. 37 - Sonoma County

SWL Depth: * ft., bgs Reference Point Elevation: 1.25 ft., ags From: Top of flange Test Date: 08/20 & 21/03

* Artisian (~ 2.0 ft. ags.) Pump location @ ~ 300 bgs

STEP DRAWDOWN ☐ CONSTANT RATE ☑ RECOVERY ☐

Time of Day	Time (minutes)	Depth to Water (ft)	Drawdown (ft)	Flow Rate (gpm)	Comments
3:25 AM	1200	82		200	
5:25 AM	1320	83 1/4		200	
7:25 AM	1440	83 3/4		200	Moved hose discharge about 70 ft south.
9:25 AM	1560	83 3/4		200	Opened valve slightly to 200 gpm
11:25 AM	1680	85 1/2		200	
1:25 PM	1800	85 1/2		200	
3:25 PM	1920	86		200	
5:25 PM	2040	86 1/4		200	
7:25 PM	2160	86 1/2		200	
9:25 PM	2280	86 3/4		200	
11:25 PM	2400	86 3/4		200	
1:25 AM	2520	87 1/2		200	
3:25 AM	2640	87 3/4		200	
5:25 AM	2760	87 3/4		200	
7:25 AM	2880	88		200	Totalizer = 113.7318 @ 7:50am
8:35 AM		88		200	7:50am start water sampling
					Water has disolved gas bubbles (air)
					Totalizer = 113.7551 @ 8:35am

HydroScience Engineers, Inc.

TEST DATA SHEET

☑ Pumping Well ☐ Observation Well Page 4 of 4

Owner: SC Sonoma Development, LLC Well Name: South Well No. 1 Well Location: Lakeville Road & Hwy. 37 - Sonoma County

SWL Depth: * ft., bgs Reference Point Elevation: 1.25 ft., ags From: Top of flange Test Date: 08/21/03

* Artisian (~ 2.0 ft. ags.) Pump location @ ~ 300 bgs

STEP DRAWDOWN ☐ CONSTANT RATE ☐ RECOVERY ☑

Time of Day	Time (minutes)	Depth to Water (ft)	Drawdown (ft)	Flow Rate (gpm)	Comments
8:35 AM	0	88			Start recovery test.
8:36 AM	1	45 1/2			
8:37 AM	2	35 1/2			
8:38 AM	3	29 3/4			
8:39 AM	4	26 1/2			
8:40 AM	5	24 1/2			
8:45 AM	10	19			
8:50 AM	15	16 1/4			
8:55 AM	20	15 1/4			
9:00 AM	25	14 1/2			
9:05 AM	30	14 3/4			
9:10 AM	35	13			
9:15 AM	40	12 1/4			
9:20 AM	45	12			
9:25 AM	50	12 3/4			
9:30 AM	55	11 3/4			
9:35 AM	60	11 1/2			
9:50 AM	75	11			
10:05 AM	90	10 1/2			
10:20 AM	105	10 1/2			
10:35 AM	120	10			
11:05 AM	150	9 1/2			
11:35 AM	180	9 1/4			

HydroScience Engineers, Inc.

TEST DATA SHEET

☑ Pumping Well ☐ Observation Well

Owner: SC Sonoma Development, LLC Well Name: South Well No. 1 Well Location: Lakeville Road & Hwy. 37 - Sonoma County

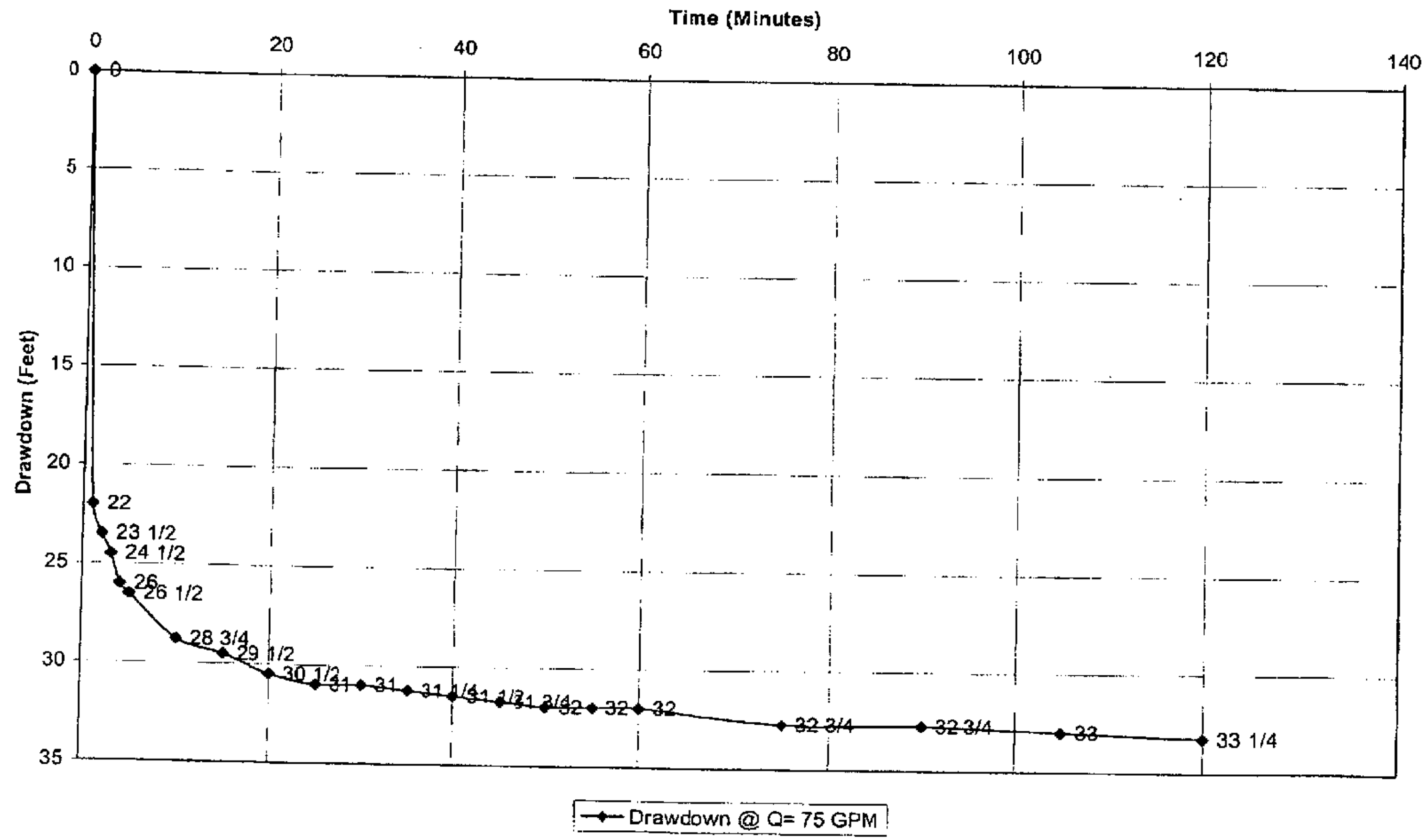
SWL Depth: * ft, bgs Reference Point Elevation: 1.25 ft., ags From: Top of Flange Test Date: 08/13/03

* = Artesian Pump @ 300 ft. bgs

STEP DRAWDOWN ☑ CONSTANT RATE ☐ RECOVERY ☐

Time of Day	Time (minutes)	Depth to Water (ft)	Drawdown (ft)	Flow Rate (gpm)	Comments
9:00 AM	0	*	0	75	Totalizer = 19.2979 Micrometer #01-03645-3. Assume SWL~ 6 FT ags
9:01 AM	1	18 1/2	22	75	Flow meter indicator fluctuates between 70 & 80 gpm
9:02 AM	2	20	23 1/2	75	
9:03 AM	3	21	24 1/2	75	
9:04 AM	4	22 1/2	26	75	
9:05 AM	5	23	26 1/2	75	
9:10 AM	10	25 1/4	28 3/4	75	
9:15 AM	15	26	29 1/2	75	
9:20 AM	20	27	30 1/2	75	
9:25 AM	25	27 1/2	31	75	
9:30 AM	30	27 1/2	31	75	
9:35 AM	35	27 3/4	31 1/4	75	
9:40 AM	40	28	31 1/2	75	9:43 am Contractor indicates that AG Well 2 & North well No. 1 are artesian.
9:45 AM	45	28 1/4	31 3/4	75	
9:50 AM	50	28 1/2	32	75	
9:55 AM	55	28 1/2	32	75	
10:00 AM	60	28 1/2	32	75	
10:15 AM	75	29 1/4	32 3/4	75	
10:30 AM	90	29 1/4	32 3/4	75	
10:45 AM	105	29 1/2	33	75	
11:00 AM	120	29 3/4	33 1/4	75	

South Well No.1 Step Drawdown Test



HydroScience Engineers, Inc.

TEST DATA SHEET

☑ Pumping Well ☐ Observation Well

Owner: SC Sonoma Development, LLC Well Name: South Well No. 1 Well Location: Lakeville Road & Hwy. 37 - Sonoma County

SWL Depth: * ft., bgs Reference Point Elevation: 1.25 ft., ags From: Top of Flange Test Date: 08/13/03

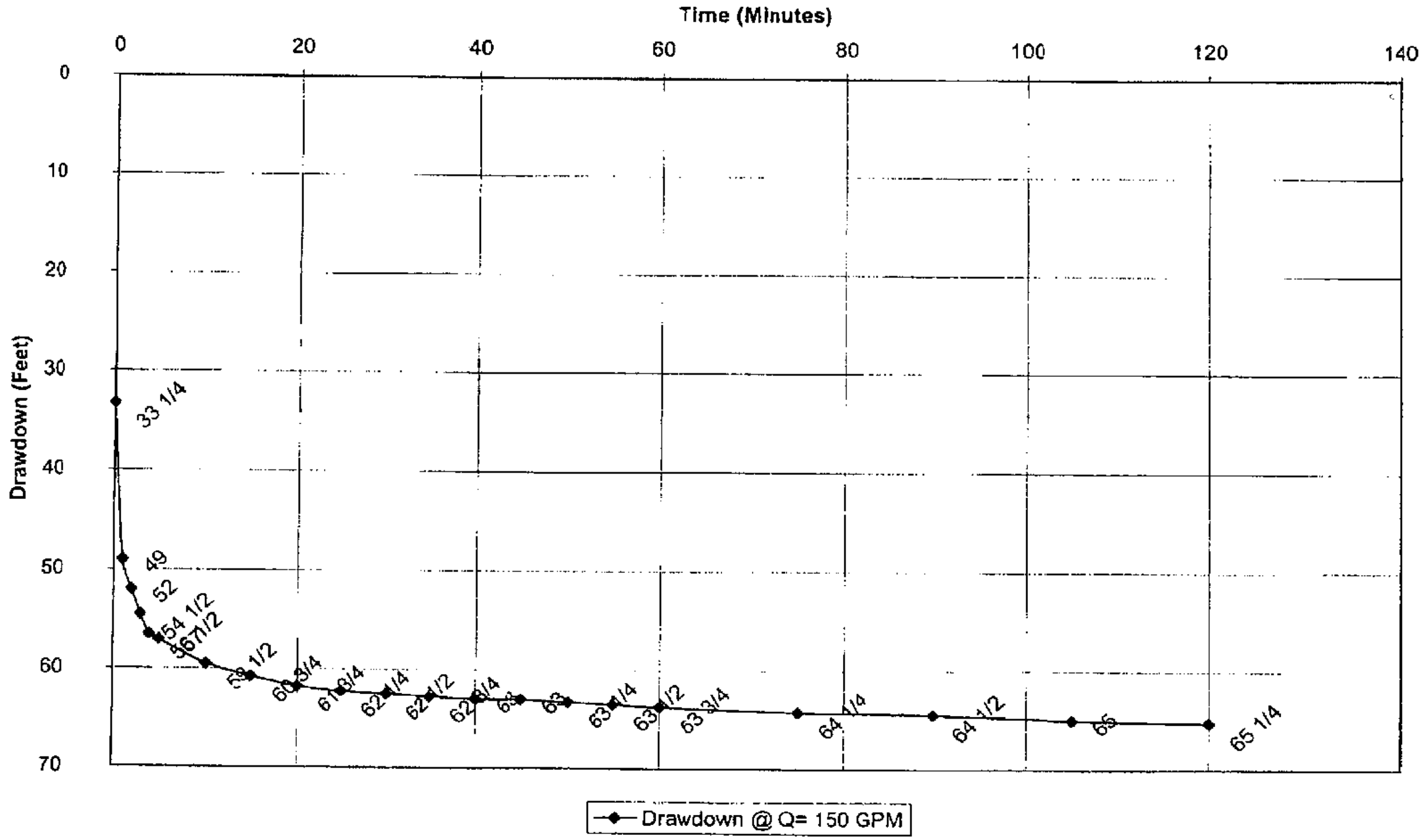
*** = Artesian @ 9:00AM**

Pump @ 300 ft. bgs

STEP DRAWDOWN ☑ **CONSTANT RATE** ☐ **RECOVERY** ☐

Time of Day	Time (minutes)	Depth to Water (ft)	Drawdown (ft)	Flow Rate (gpm)	Comments
11:00 AM	0	29 3/4	33 1/4	150	Flowmeter fluctuates between 145 & 155 gpm. Assume SWL ~ 6 FT ags
11.01 AM	1	45 1/2	49	150	
11:02 AM	2	48 1/2	52	150	
11:03 AM	3	51	54 1/2	150	
11:04 AM	4	53	56 1/2	150	
11:05 AM	5	53 1/2	57	150	
11:10 AM	10	56	59 1/2	150	11:12 AM Contractor's indicates that AG Well & North well No. 1 are still artisian
11:15 AM	15	57 1/4	60 3/4	150	
11:20 AM	20	58 1/4	61 3/4	150	
11:25 AM	25	58 3/4	62 1/4	150	
11:30 AM	30	59	62 1/2	150	
11:35 AM	35	59 1/4	62 3/4	150	
11:40 AM	40	59 1/2	63	150	
11:45 AM	45	59 1/2	63	150	
11:50 AM	50	59 3/4	63 1/4	150	
11:55 AM	55	60	63 1/2	150	
12:00 PM	60	60 1/4	63 3/4	150	
12:15 PM	75	60 3/4	64 1/4	150	
12:30 PM	90	61	64 1/2	150	11:40 AM water is cloudy (Bentonite) and has bronze or iron disulfide chips.
12:45 PM	105	61 1/2	65	150	
1:00 PM	120	61 3/4	65 1/4	150	

South Well No.1 Step Drawdown Test



HydroScience Engineers, Inc.

TEST DATA SHEET

☑ Pumping Well ☐ Observation Well

Owner: SC Sonoma Development, LLC Well Name: South Well No. 1 Well Location: Lakeville Road & Hwy. 37 - Sonoma County

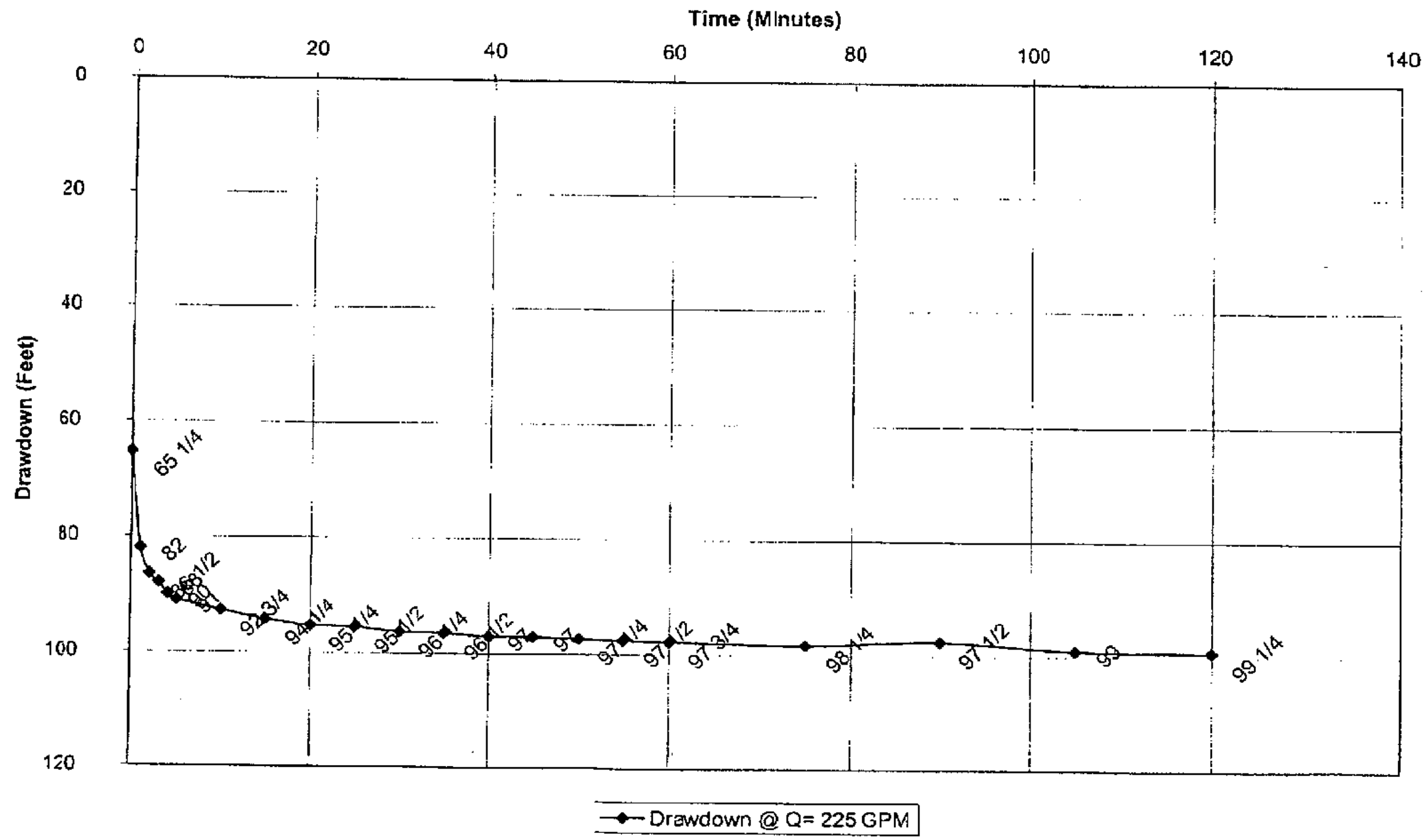
SWL Depth: * ft., bgs Reference Point Elevation: 1.25 ft., ags From: Top of Flange Test Date: 08/13/03

* = Artesian @ 9:00am Pump @ 300 ft. bgs

STEP DRAWDOWN ☑ CONSTANT RATE ☐ RECOVERY ☐

Time of Day	Time (minutes)	Depth to Water (ft)	Drawdown (ft)	Flow Rate (gpm)	Comments
1:00 PM	0	61 3/4	65 1/4	225	
1:01 PM	1	78 1/2	82	225	
1:02 PM	2	83	86 1/2	225	
1:03 PM	3	84 1/2	88	225	
1:04 PM	4	86 1/2	90	225	
1:05 PM	5	87 1/2	91	225	
1:10 PM	10	89 1/4	92 3/4	225	Water is cloudy (Bentonite)
1:15 PM	15	90 3/4	94 1/4	225	
1:20 PM	20	91 3/4	95 1/4	225	
1:25 PM	25	92	95 1/2	225	
1:30 PM	30	92 3/4	96 1/4	225	
1:35 PM	35	93	96 1/2	225	
1:40 PM	40	93 1/2	97	225	
1:45 PM	45	93 1/2	97	225	
1:50 PM	50	93 3/4	97 1/4	225	Water is cloudy (Bentonite). We are developing this well!
1:55 PM	55	94	97 1/2	225	
2:00 PM	60	94 1/4	97 3/4	225	
2:15 PM	75	94 3/4	98 1/4	225	
2:30 PM	90	94	97 1/2	225	
2:45 PM	105	95 1/2	99	225	
3:00 PM	120	95 3/4	99 1/4	225	Totalizer = 19.4627

South Well No.1 Step Drawdown Test



HydroScience Engineers, Inc.

TEST DATA SHEET

☑ Pumping Well ☐ Observation Well

Owner: SC Sonoma Development, LLC Well Name: South Well No. 1 Well Location: Lakeville Road & Hwy. 37 - Sonoma County

SWL Depth: * ft., bgs Reference Point Elevation: 1.25 ft., ags From: Top of Flange Test Date: 08/13/03

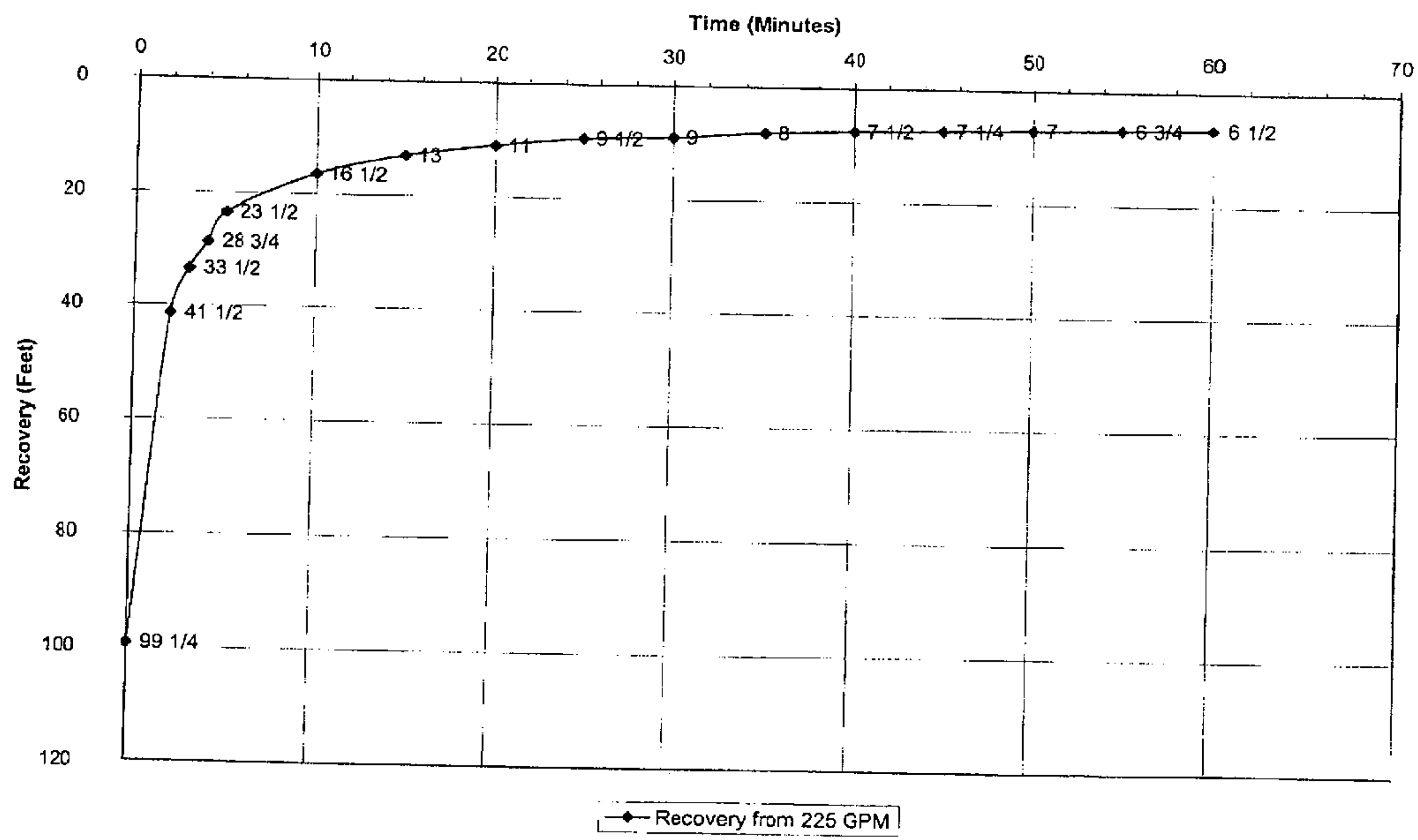
* = Artesian @ 9:00am

Pump @ 300 ft. bgs

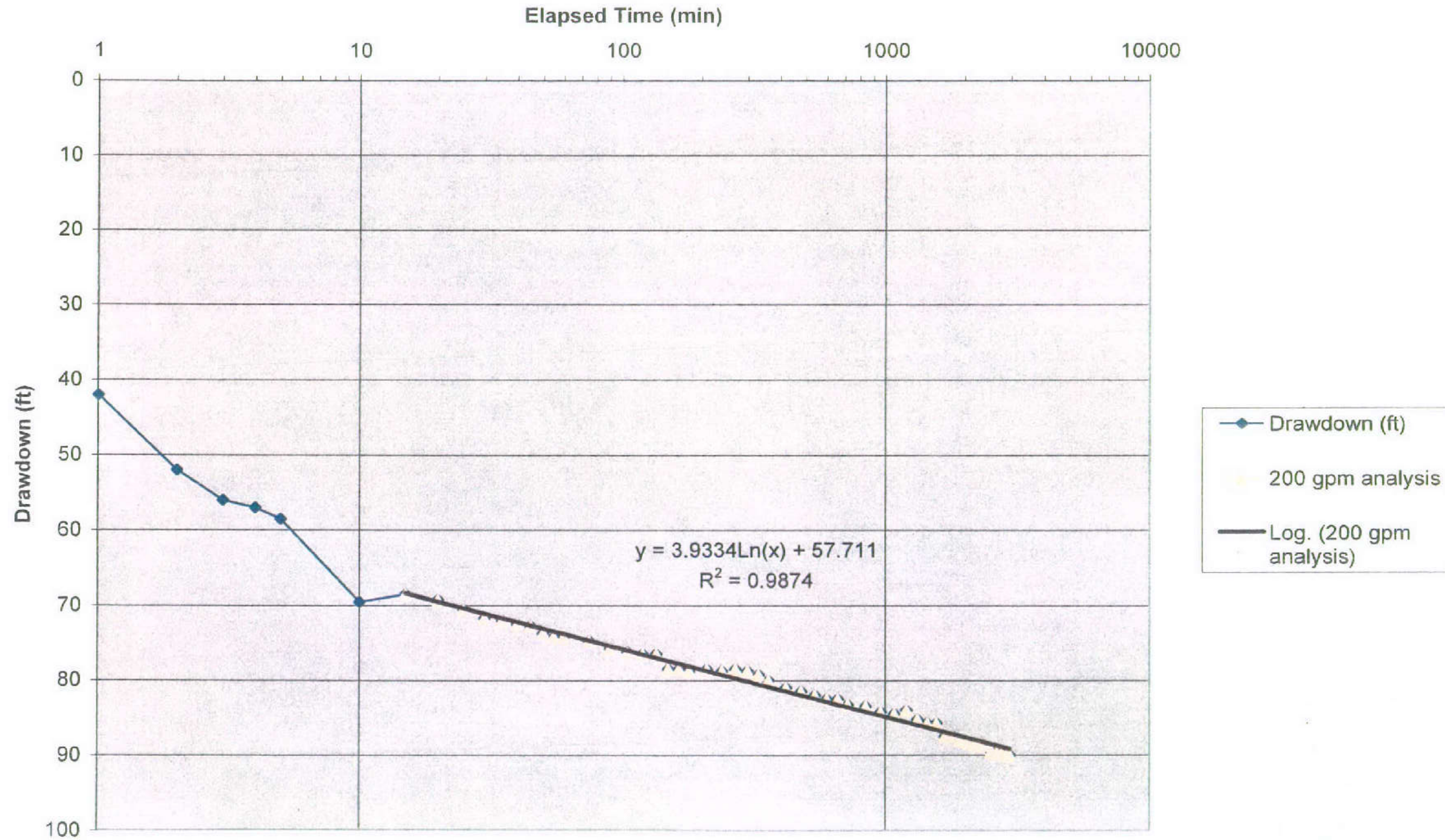
STEP DRAWDOWN ☐ **CONSTANT RATE** ☐ **RECOVERY** ☑

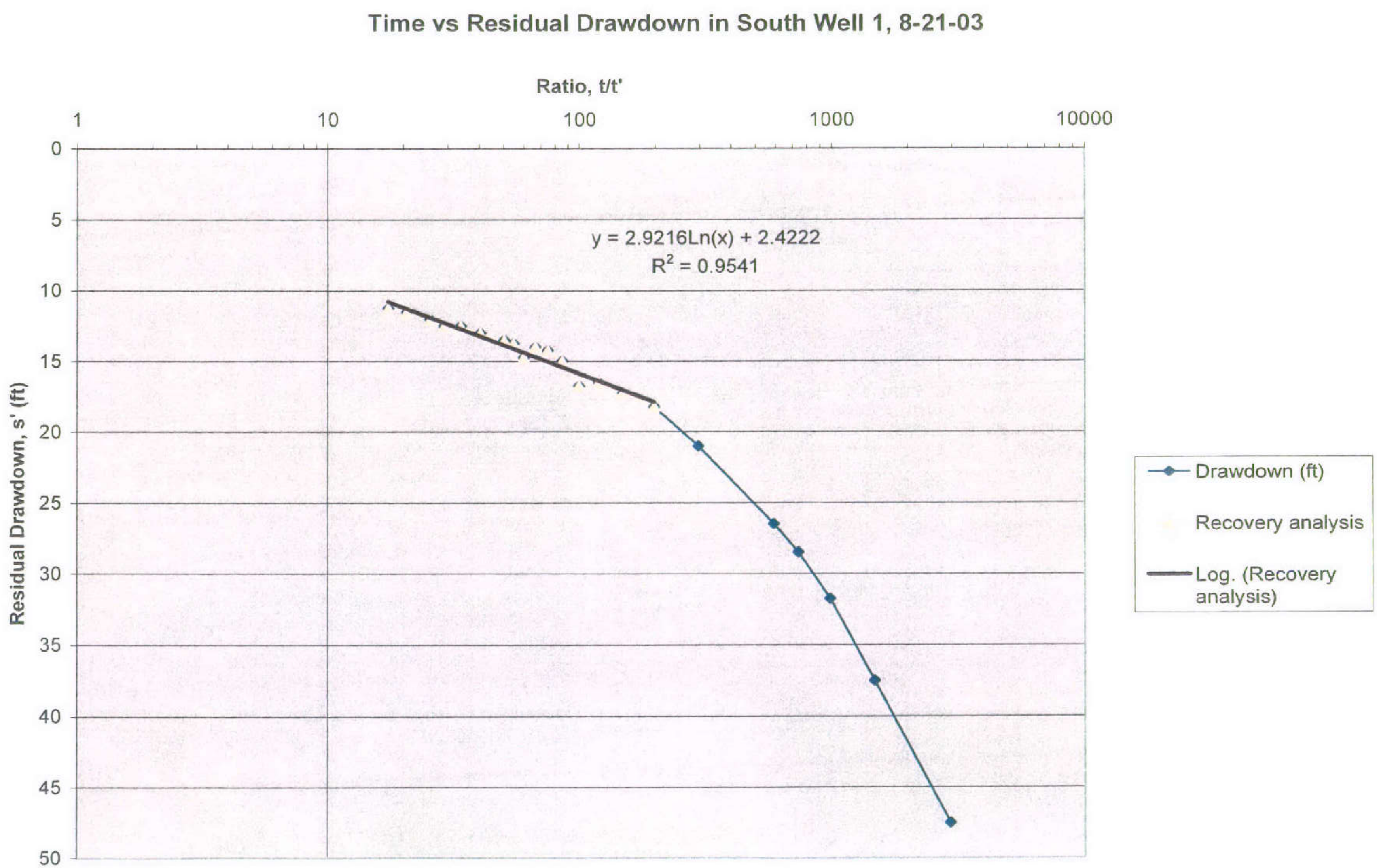
Time of Day	Time (minutes)	Depth to Water (ft)	Drawdown (ft)	Flow Rate (gpm)	Comments
3:00 PM	0	95 3/4	99 1/4		Start recovery test
3:02 PM	2	38	41 1/2		
3:03 PM	3	30	33 1/2		
3:04 PM	4	25 1/4	28 3/4		
3:05 PM	5	20	23 1/2		
3:10 PM	10	13	16 1/2		
3:15 PM	15	9 1/2	13		
3:20 PM	20	7 1/2	11		
3:25 PM	25	6	9 1/2		
3:30 PM	30	5 1/2	9		
3:35 PM	35	4 1/2	8		
3:40 PM	40	4	7 1/2		
3:45 PM	45	3 3/4	7 1/4		
3:50 PM	50	3 1/2	7		
3:55 PM	55	3 1/4	6 3/4		
4:00 PM	60	3	6 1/2		End recovery test

South Well No. 1 Recovery Test

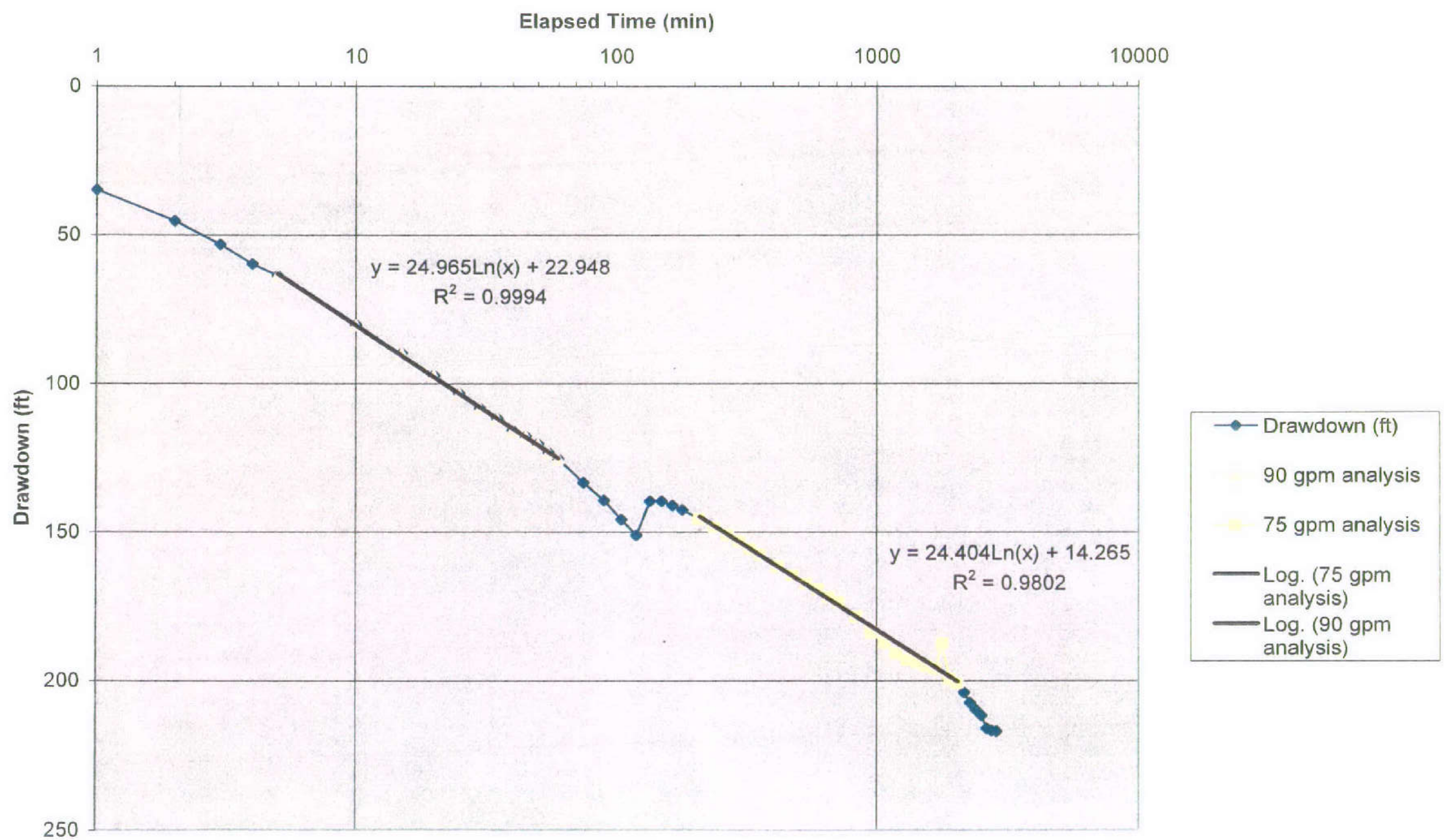


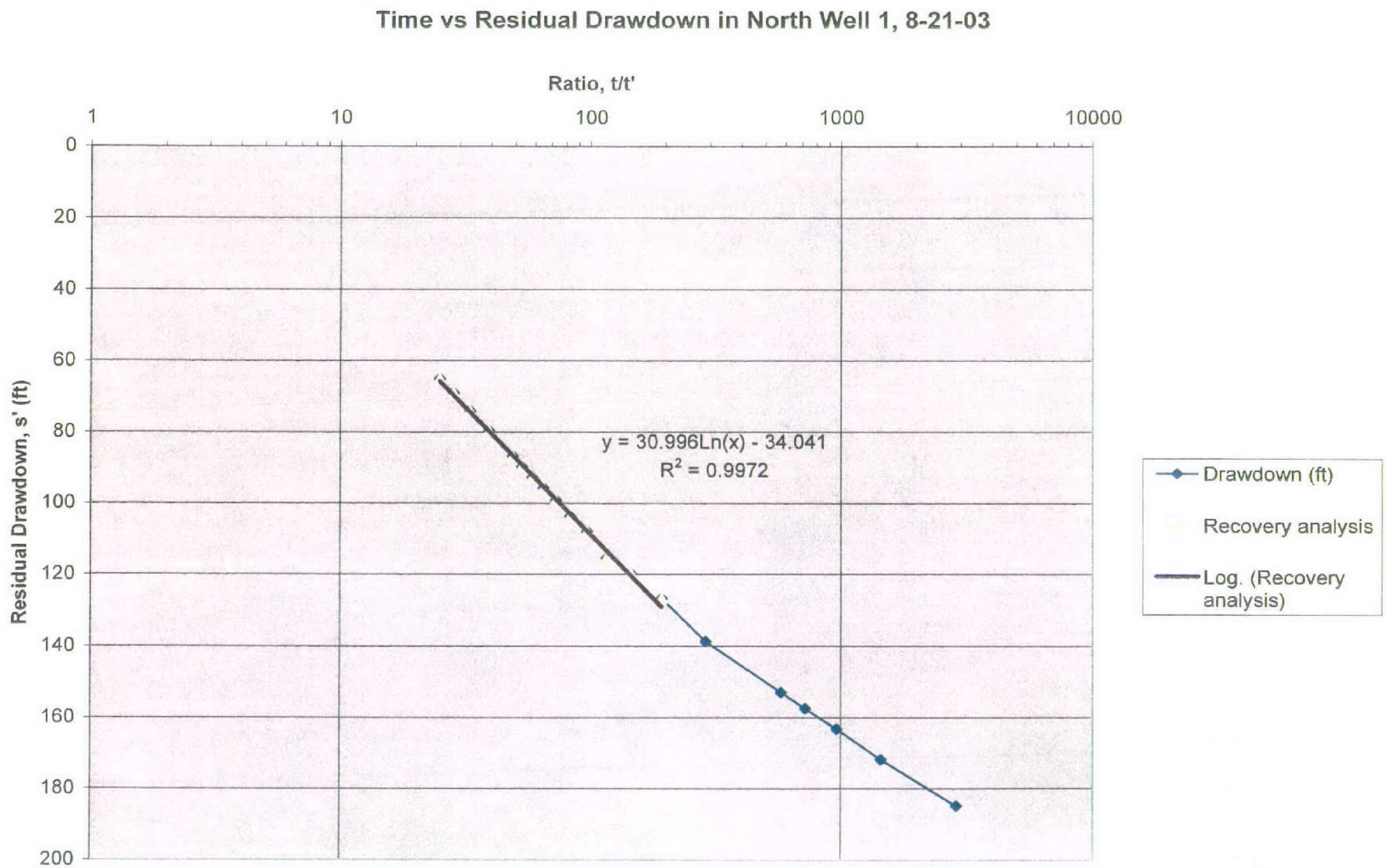
Time vs Drawdown in South Well 1, 8-19-03



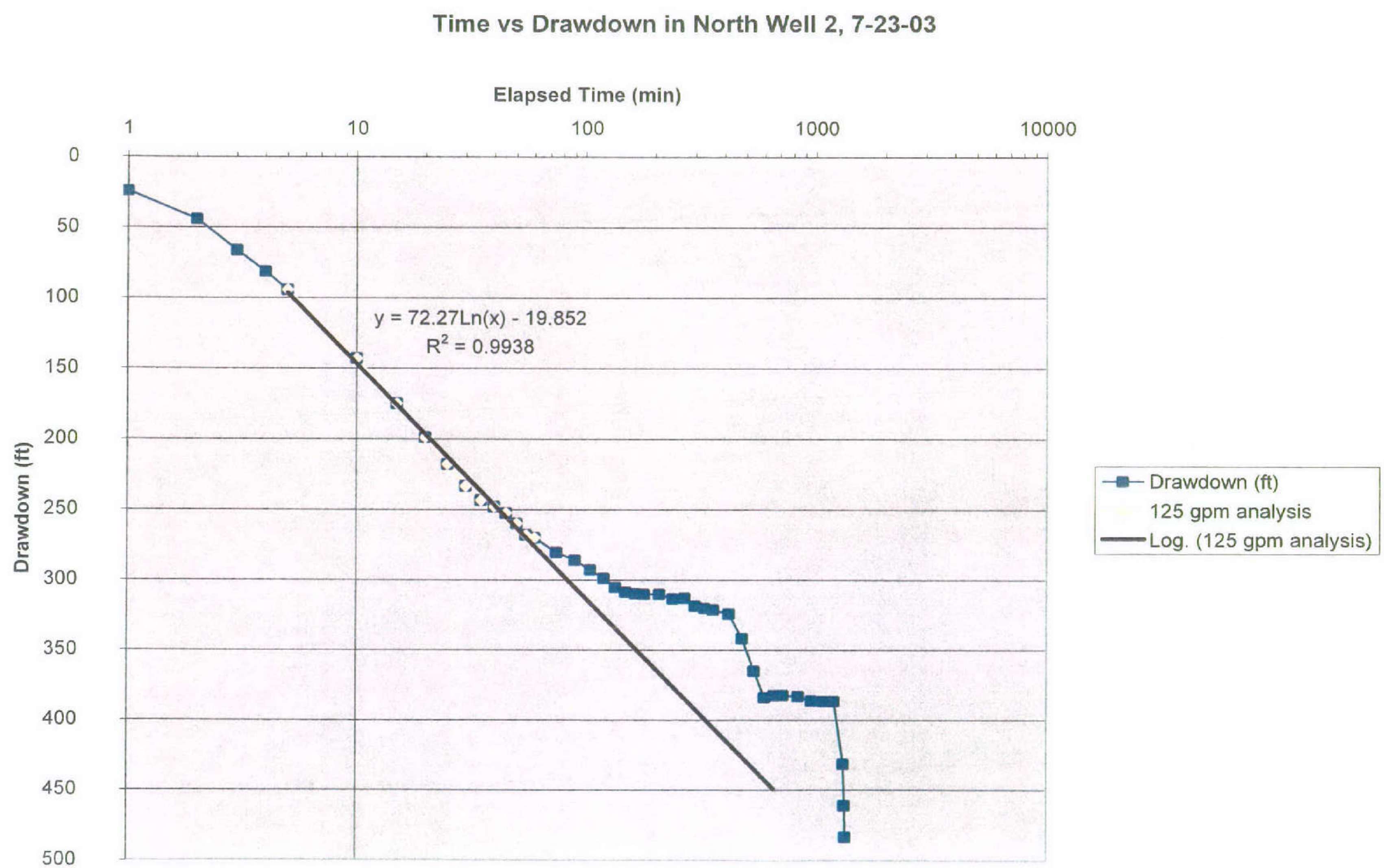

Time vs Residual Drawdown in South Well 1, 8-21-03
Ratio, t/t'
1
10
100
1000
10000
0
5
10
15
20
25
30
35
40
45
50
Residual Drawdown, s' (ft)
y = 2.9216Ln(x) + 2.4222
R² = 0.9541
Drawdown (ft)
Recovery analysis
Log. (Recovery analysis)

Time vs Drawdown in North Well 1, 8-19-03



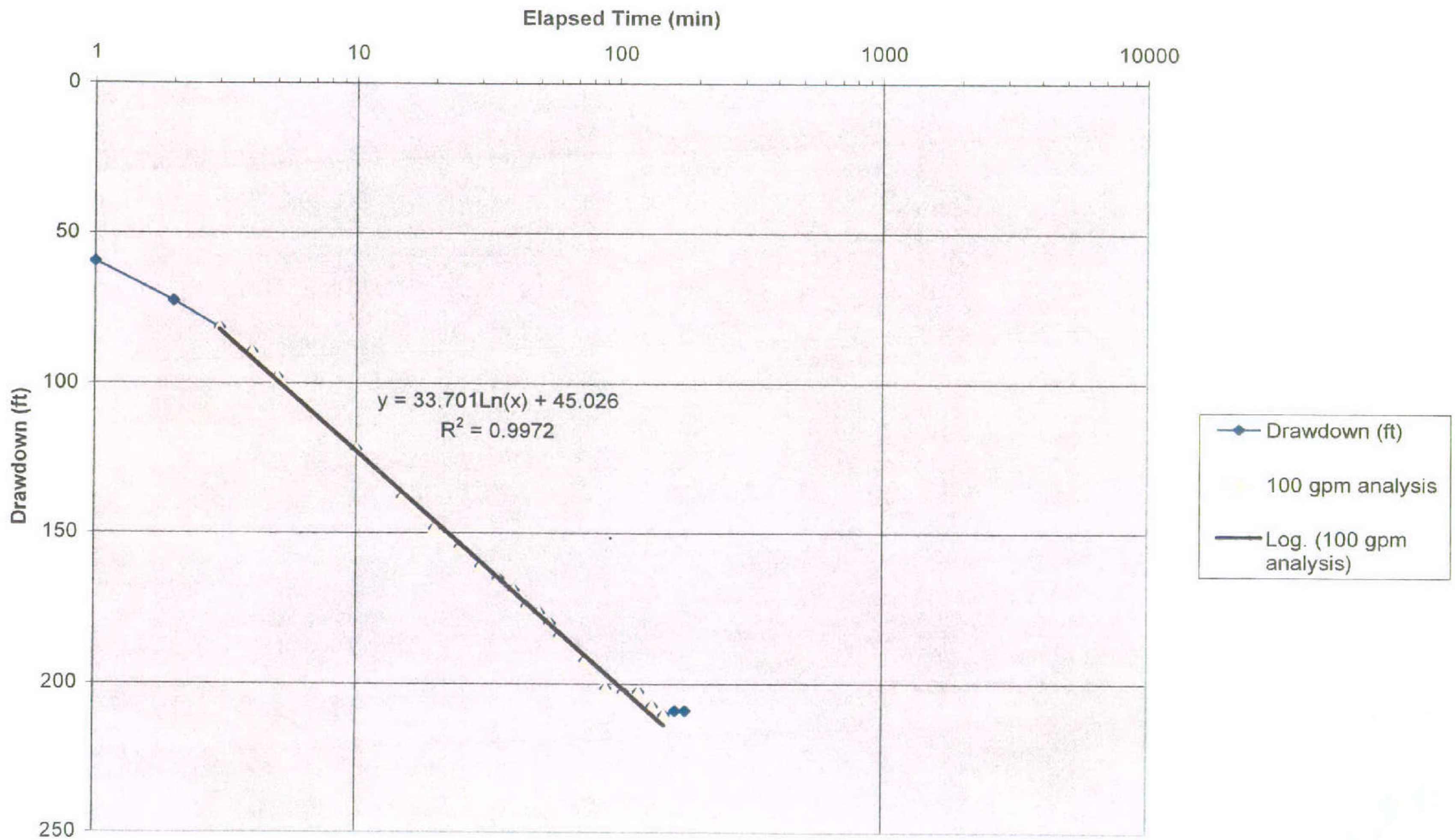


Time vs Residual Drawdown in North Well 1, 8-21-03
Ratio, t/t'
1
10
100
1000
10000
0
20
40
60
80
100
120
140
160
180
200
Residual Drawdown, s' (ft)
y = 30.996Ln(x) - 34.041
R2 = 0.9972
Drawdown (ft)
Recovery analysis
Log. (Recovery analysis)



Time vs Drawdown in North Well 2, 7-23-03
Elapsed Time (min)
1
10
100
1000
10000
0
50
100
150
200
250
300
350
400
450
500
Drawdown (ft)
y = 72.27Ln(x) - 19.852
R2 = 0.9938
Drawdown (ft)
125 gpm analysis
Log. (125 gpm analysis)

Time vs Drawdown in North Well 2, 7-22-03



APPENDIX B

PUMP ELECTRICAL USE EVALUATION

A. LAKEVILLE SITE - PUMP POWER IMPACT ANALYSIS

A.1 Introduction

The purpose of this analysis was to estimate potential impacts to pump power requirements on nearby water wells. These impacts might be caused by water table drawdown resulting from the installation and operation of a new water well that is being considered as part of this development.

Additional water table drawdown in the vicinity of an existing water well changes the operational characteristics of the pump operating within that water well. Additional water table drawdown effectively results in an increase in pump head, which in turn decreases the pump discharge rate, and changes the pump power requirements. On that basis, several operational scenarios were examined in an attempt to quantify, in general terms, the additional power required by nearby water well pumps that might be impacted by additional water table drawdown.

A detailed description of the methodology that was utilized to carry out this analysis, as well as a discussion of the results, is included in the sections that follow.

A.2 Methodology

For the Lakeville Site, three evaluations were made based upon the following ranges of values and boundary conditions:

- Three (3) pumping rates 20 gallons per minute (gpm), 100 gpm, and 200 gpm;
- Two (2) pump installation depths 200 feet below ground surface (bgs), and 600 feet bgs;
- Three (3) initial pumping water levels 150 feet bgs, 400 feet bgs, and 500 feet bgs, and;
- Two (2) interference drawdowns 10 feet, and 100 feet.

As we have no knowledge of the actual pumps and installation conditions for water wells that surround the site, we based our analysis on selecting pumps appropriate for each of the discharge conditions noted above. By attempting to model a range of conditions, we hoped to bracket the real world pumps and ensure that their operating conditions lie within the feasible space of this analysis. While this analysis is not exact and may not be representative of actual installed pump types and conditions, it does offer some insight as to how much additional power might be required to pump one acre-foot of water if additional water table drawdown occurs. If site-specific information regarding water wells and pumps becomes available in the future, this analysis could be adapted to examine power requirement impacts for those specific pumps.

For each case, an appropriate pump was selected that met the baseline condition of zero feet of initial additional drawdown at the pump discharge rates, pump installation depths, and pumping water levels shown above.

Pumps were selected by using the Goulds Pumps website. In particular, pumps were selected using the Design Point Search by inputting flow rate and total head into the selector at the following internet URL:

http://www.pump-flo.com/select/centrifugal/criteria.aspx?DirName=goulds&catName=Goulds%20GL

On the basis of the flow rate and the calculated total head, recommended pump curves were displayed. The most appropriate pump curve for the input conditions was selected and downloaded to start the pump power impact analysis.

Each pump curve was imported into a CAD program so that precise scaling of distances along the Q, H, and P axes could be performed as part of the analysis. Use of the CAD program became critical when determining the effects of additional drawdown on the Q and P of a particular pump. As the incremental changes were typically small for H and P, precise scaling of the changes was required to accurately estimate the overall effect on each system.

Pump curves for each of the pumps selected for this analysis are included at the end of this appendix. A summary table of pump model for each analysis is shown below.

Pump Type & Size	Pump Installation Depth (feet bgs)	Pumping Water Level (feet bgs)	Baseline Pumping Rate (gpm)	Baseline Pump Head (feet)	Baseline Pump Power (h.p.)
VERT-IMMERS, 1SL	250	150	20	161.2	2.06
	600	400	100	408.7	15.99
MULTISTAGE, 1HMS	600	500	200	504.0	35.39

Figure 1 presents a graphical and text summary of the procedure for estimating incremental differences in Q and P for imposed changes in H. The text summary from this figure is also included below.

All of the steps listed in the methodology shown on **Figure 1** were performed within a CAD program for accurate estimation of incremental differences. Please refer to the ten points shown on the figure in conjunction with the following:

- Once a suitable pump has been found that satisfies the discharge and head requirements for a particular case, the discharge (Q), head (H), and power (P) analysis starts at **Point #1** shown on the figure.
- From **Point #1** at the required Q, a vertical line is drawn upward to intersect the pump curve at **Point #2**.
- From **Point #2**, a horizontal line is drawn to the left (to **Point #3**) and the H associated with that particular Q is determined.
- From **Point #1**, a vertical line is drawn downward to intersect the power curve at **Point #4**.
- From Point #4, a horizontal line is drawn to the left (to **Point #5**) and the P associated with that particular Q is determined.
- The Q, H, and P determined by the steps shown above represent the baseline conditions for a particular case.
- Additional drawdown is then imposed, and the incremental effects on the Q, H, and P are then determined. This change in value of the parameters Q, H, and P are used later to estimate the impact to the power requirements to pump one acre-foot of water.

- Additional drawdown is the equivalent of moving higher up the pump curve (higher head).
- From **Point #3**, a vertical line is drawn upwards to **Point #6**. The length of this line is the imposed drawdown and will either be 10.0 feet or 100.0 feet.
- From **Point #6**, a horizontal line is drawn to the right to intersect the pump curve at **Point #7**.
- From **Point #7**, a vertical line is drawn downwards to **Point #8**. Where this line intersects the Q axis determines what the new Q for the pump will be given the additional head (imposed drawdown) for that case. For monotonically decreasing pump curves, as H increases, Q decreases, and conversely, as H decreases, Q increases.
- From **Point #8**, a vertical line is drawn downwards to intersect the power curve at **Point #9**.
- From **Point #9**, a horizontal line is drawn to the left (to **Point #10**) and the new P associated with the new Q is determined.

An example has been prepared for the case of Q = 100 gpm, with the pump installation depth at 600 feet bgs, and the pumping water level at 400 feet bgs.

Baseline

- Q = 100.0 gpm, H = 408.7 feet, P = 15.99 h.p. (**Point #1**, **#3**, and **#5**, respectively).

Impose an additional drawdown of 100.0 feet (**Point #3** to **#6**).

New

- Q = 69.3 gpm, H = 508.7 feet, P = 13.40 h.p. (Point #8, #6, and #10, respectively).

Incremental Difference

- Q = -31.7 gpm, H = +100.0 feet, P = -2.59 h.p.

From this analysis, the additional time and/or power required to pump one acre-foot of water for additional drawdowns of 10.0 and 100.0 feet were calculated.

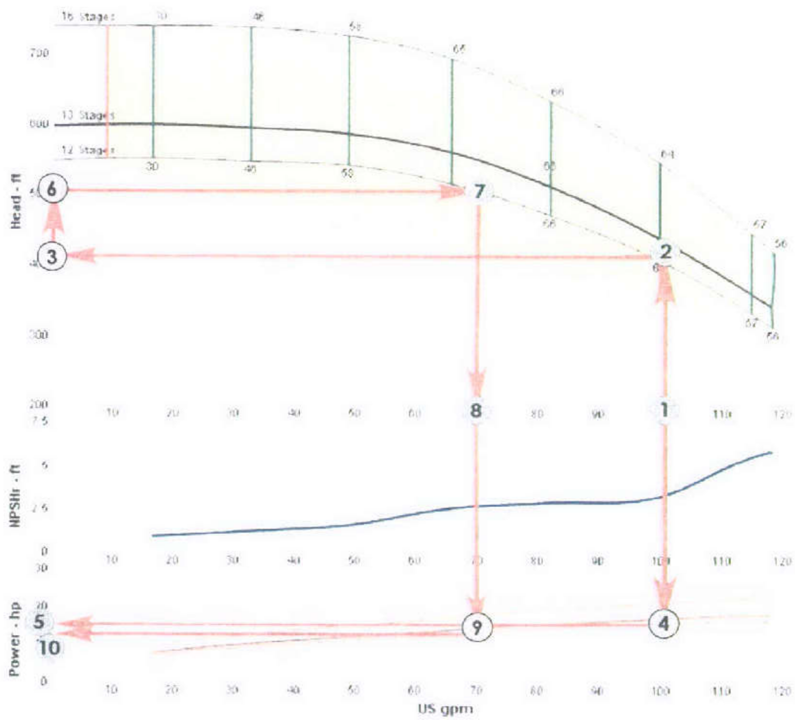
A.3 Results

One acre-foot of water is 325,851 gallons. The differential time to pump that volume of water using the new Q (that resulted from additional drawdown forcing the condition point further up the pump curve), relative to the baseline Q was calculated. This differential time was multiplied by the new power P (in horsepower [h.p]), with the result being incremental power consumption in kilowatt-hours (kW-hours) to pump one acre-foot of water. The incremental power consumption results are shown below.

Pump Discharge Rate (gpm)		**20**
Pump Installation Depth (feet bgs)		**250**
Pumping Water Level (feet bgs)		**150**
Additional Water Table Drawdown (feet)	**0.0**	0.0
	10.0	9.7
	100.0	207.9

Pump Discharge Rate (gpm)		**100**
Pump Installation Depth (feet bgs)		**600**
Pumping Water Level (feet bgs)		**400**
Additional Water Table Drawdown (feet)	**0.0**	0.0
	10.0	3.6
	100.0	135.9

Pump Discharge Rate (gpm)		**200**
Pump Installation Depth (feet bgs)		**600**
Pumping Water Level (feet bgs)		**500**
Additional Water Table Drawdown (feet)	**0.0**	0.0
	10.0	8.9
	100.0	77.4



Methodology Summary

Once a suitable pump has been found that satisfies the discharge and head requirements for a particular case, the discharge (Q), head (H), and power (P) analysis starts at Point #1 shown on the figure.

From Point #1 at the required Q, a vertical line is drawn upward to intersect the pump curve at Point #2.

From Point #2, a horizontal line is drawn to the left (to Point #3) and the H associated with that particular Q is determined.

From Point #1, a vertical line is drawn downward to intersect the power curve at Point #4.

From Point #4, a horizontal line is drawn to the left (to Point #5) and the P associated with that particular Q is determined.

The Q, H, and P determined by the steps shown above represent the baseline conditions for a particular case.

Additional drawdown is then imposed, and the incremental effects on the Q, H, and P are then determined. This change in value of the parameters Q, H, and P are used later to estimate the impact to the power requirements to pump one acre-foot of water.

Additional drawdown is the equivalent of moving higher up the pump curve (higher head).

From Point #3, a vertical line is drawn upwards to Point #6. The length of this line is the imposed drawdown and will either be 10.0 feet or 100.0 feet.

From Point #6, a horizontal line is drawn to the right to intersect the pump curve at Point #7.

From Point #7, a vertical line is drawn downwards to Point #8. Where this line intersects the Q axis determines what the new Q for the pump will be given the additional head (imposed drawdown) for that case. For monotonically decreasing pump curves, as H increases, Q decreases, and conversely, as H decreases, Q increases.

From Point #8, a vertical line is drawn downwards to intersect the power curve at Point #9.

From Point #9, a horizontal line is drawn to the left (to Point #10) and the new P associated with the new Q is determined.

Example

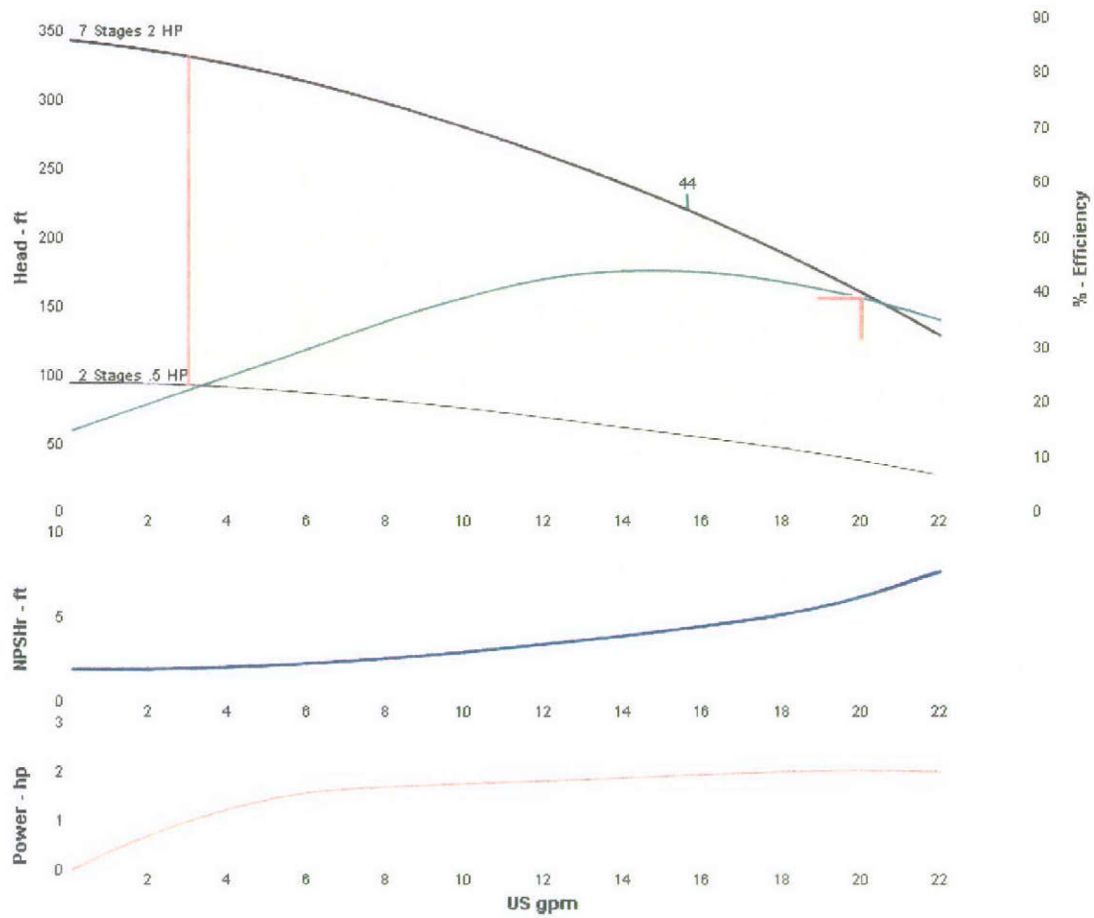
Baseline
Q = 100.0 gpm, H = 408.7 feet, P = 15.99 h.p. (Point #1, #3, and #5, respectively).

Impose an additional drawdown of 100.0 feet (Point #3 to #6).

New
Q = 69.3 gpm, H = 508.7 feet, P = 13.40 h.p. (Point #8, #6, and #10, respectively).

Incremental Difference
Q = -31.7 gpm, H = +100.0 feet, P = -2.59 h.p.

LAKEVILLE SITE	WorleyParsons Komex resources & energy		
METHODOLOGY SUMMARY	DRAWN BY: WM	EDITED BY: WM	DATE: 12/2006
	APPROVED: MT		1



Request for Quote

Submit your selection to Goulds Pumps.

Pump

Type	MULTISTAGE
Size	1SV_2-7
Curve	Goulds SSV
Speed	3450 rpm
Line	7 Stages 2 HP

Redraw Reset

Display Options

- Manufacturer settings
- Custom

Multiple Pumps

Number of pumps 1

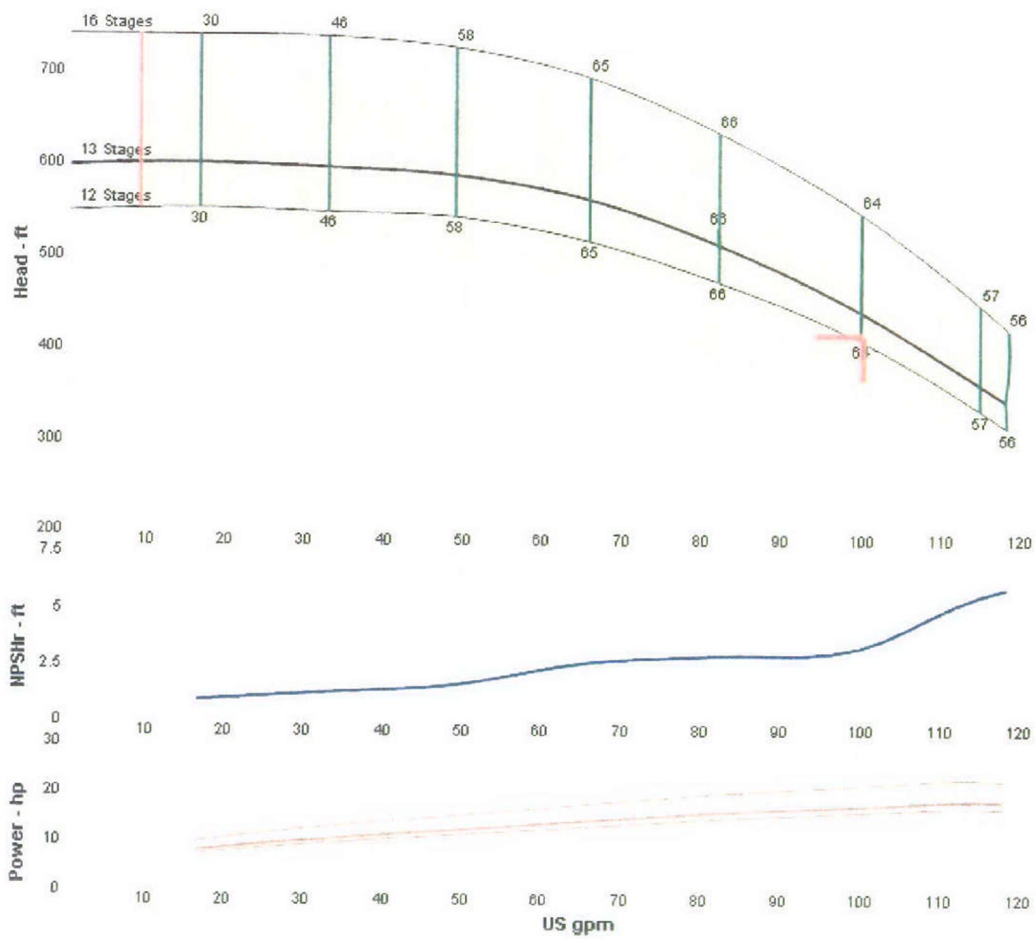
Parallel Series

Multiple Speeds (rpm)

2900 rpm - 3450 rpm

Flow	Head	Eff	BEP	NPSHr	Power	Motor	Frame	Min flow	Sphere
20 US gpm	160 ft	39 %	44 %	6.34 ft	2.06 hp	---	---	3 US gpm	0.0312 in

GOULDS PUMPS



Request for Quote

Submit your selection to Goulds Pumps.

Pump

Type	MULTISTAGE
Size	40.3-12-16
Curve	GP MPVN
Speed	1750 rpm
Line	13 Stages

Redraw Reset

Display Options

- Manufacturer settings
- Custom

Multiple Pumps

Number of pumps 1

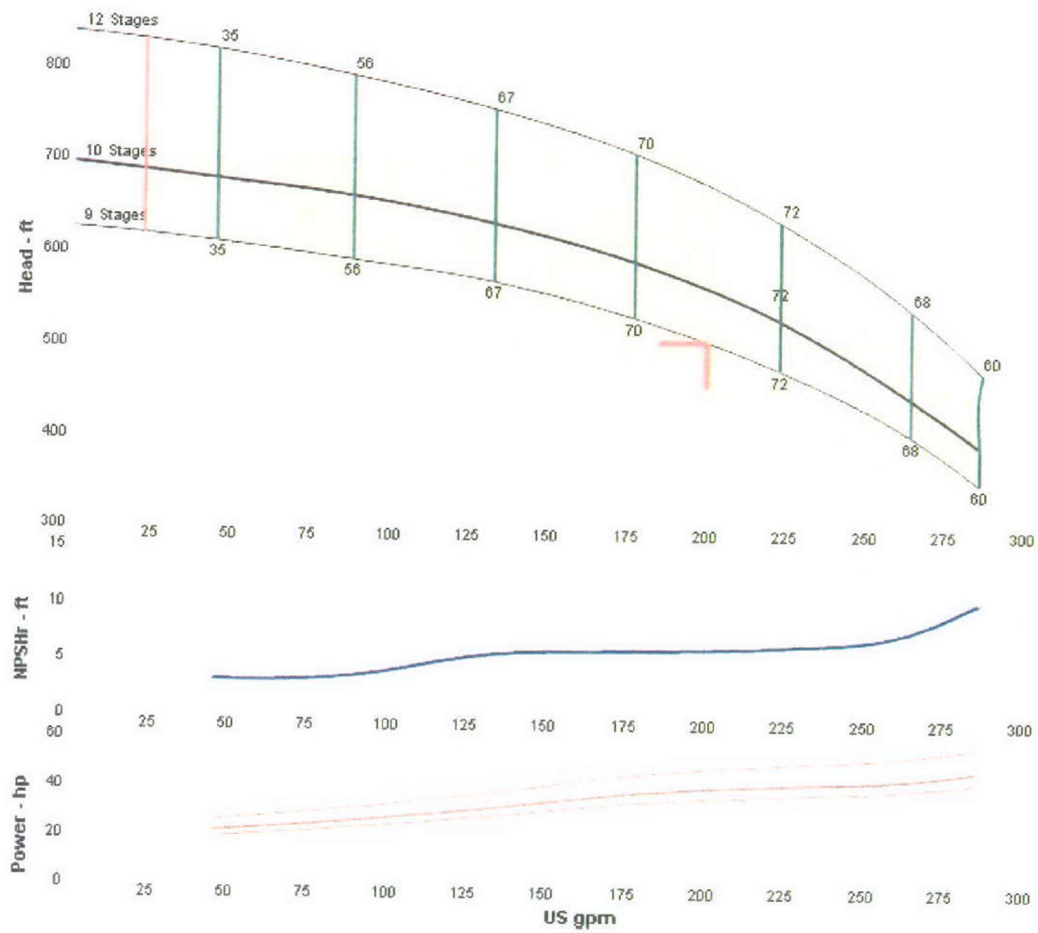
- Parallel
- Series

Multiple Speeds (rpm)

1180 rpm - 1750 rpm

Flow	Head	Eff	BEP	NPSHr	Power	Motor	Frame	Min flow	Sphere
100 US gpm	437 ft	64 %	66 %	3.34 ft	17.3 hp	---	---	8.81 US gpm	---

GOULDS PUMPS



Request for Quote

Submit your selection to Goulds Pumps.

Pump

Type	MULTISTAGE
Size	65.2-9-12
Curve	GP MPVN
Speed	1750 rpm
Line	10 Stages

Redraw Reset

Display Options ?

- Manufacturer settings
- Custom

Multiple Pumps

Number of pumps 1

Parallel Series

Multiple Speeds (rpm)

1180 rpm - 1750 rpm

Flow	Head	Eff	BEP	NPSHr	Power	Motor	Frame	Min flow	Sphere
200 US gpm	557 ft	71 %	72 %	6.07 ft	39.4 hp	---	---	22 US gpm	---