

OPTIMUM CLEANUP SCENARIOS FOR VOC'S AND PERCHLORATE IN  
BALDWIN PARK OPERABLE UNIT, SAN GABRIEL VALLEY,  
SOUTHERN CALIFORNIA

by

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Christian Sam

## **Dedication**

To:

My beloved family and friends who have helped me in the strive for success!

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## **Abstract**

The Baldwin Park Operable Unit (BPOU) in the San Gabriel Valley basin, California has been identified by the USEPA as a groundwater contaminant superfund site. This research employs optimized hydraulic gradient control to cost-effectively remove the contaminant plumes. Perchlorate (PCR) and Volatile Organic Compounds (VOCs), predominantly tetrachloroethene (PCE) and trichloroethene (TCE) are still prevalent in the Baldwin Park Operable Unit (BPOU). This study proposes a methodology that optimizes available groundwater and surface water resources and blocks, traps and reduces the contaminant concentration below the maximum contaminant level (MCL) or notification level (NL).

The current strategy by USEPA and other agencies involved in the cleanup of BPOU is “pump and treat.” Pump and treat has not been effective as high levels of PCE, TCE and PCR are still prevalent in BPOU after approximately 14 years of cleanup. Blending groundwater from wells not impacted by the contamination with groundwater from wells impacted by the contamination is a supplemental strategy employed by the water purveyors to circumvent the contamination problem. This research provides an overall systematic strategy to clean up the entire aquifer at BPOU.

The MODFLOW, MODPATH and RT3D modules of Ground Water Vistas (ground water modeling program) are used to generate the ground water flow model, particle tracking model and contaminant transport models. Further migration of the contaminants is limited. Contaminants are rapidly trapped and removed by a set of experimentally

designed injection and production wells. The procedure utilizes sequential simulation with optimization to optimally and rapidly remove the contaminants using a hydraulic gradient scheme. The hydraulic gradient scheme was run to test future scenarios using assumed wet and dry cycles from the hydrologic base period.

An economic analysis was performed to compare the cost of the existing system with the cost of the strategy proposed in this study. The strategy proposed in this study resulted in a cost effective solution.

# **Chapter 1**

## **Introduction**

### **1.1 Problem Statement**

In 1999, the United States Environmental Protection Agency (USEPA) determined that volatile organic compounds (VOCs) have been released into groundwater in the San Gabriel Valley basin and that a substantial threat of release to groundwater still existed (USEPA, 1999). High concentrations of Volatile Organic Compounds (VOC's) exist in the Baldwin Park Operable Unit (BPOU) of the San Gabriel Valley basin. The term "Operable Unit" (OU) defines a discrete action that is an incremental step toward a comprehensive site remedy (USEPA, 1999). Operable units address certain geographic areas, specific site problems, initial phases of a remedy, or a set of actions over time (USEPA, 1999). Tetrachloroethene (PCE) and trichloroethene (TCE) form the bulk of the VOC's found in the region. PCE and TCE are solvents used for degreasing and cleaning. Perchlorate (PCR) is an inorganic component of solid-fuel rockets. PCR exists in large quantities in the Baldwin Park Operable Unit of the San Gabriel Valley Basin. This study focuses on PCE, TCE and PCR in the Baldwin Park Operable Unit (BPOU).

The contaminant plume affects potable groundwater in several Operable Units in San Gabriel basin. The contamination spreads as groundwater is pumped. This is a result of high demand for groundwater supply and scarcity of surface water sources. Overdraft occurs when groundwater production exceeds natural and artificial replenishment. The contaminant plume not only threatens current groundwater production but also prevents

future use of valuable groundwater resources.

## **1.2 History of the Contamination**

The contaminants TCE, PCE, other VOC's and PCR are a result of manufacturing and industrial spills in El Monte, South El Monte, Azusa and Baldwin Park in the 1980s.

Since the groundwater flow is south west, most of the contaminants from El Monte have migrated to South El Monte and contaminants from Azusa migrated south to Baldwin Park.

Groundwater contamination by volatile organic compounds (VOCs) in the San Gabriel Valley basin was first detected in 1979 when Aerojet Electrosystems in Azusa sampled nearby wells in the Valley County Water District. Subsequently, the California Department of Health Services (CDHS) initiated a well sampling program to assess the extent of contamination. By 1984, high levels of VOCs were detected in 59 wells in the San Gabriel Valley basin. Hundreds of individual facilities could be contributing to the contamination in the basin through improper handling and disposal practices. The area of contamination parallels the San Gabriel River to the east. The watershed is drained by the San Gabriel River and Rio Hondo River that are tributaries of the Los Angeles River.

The basin's groundwater provides approximately 90 percent of the domestic water supply for over 1,000,000 people who live in the Valley. Over 400 water supply wells are used in the basin to extract groundwater for industrial, business, agricultural, and domestic uses. Forty-five different water suppliers operate in the basin and provide drinking

water to more than 1,000,000 people. In 1992, USEPA completed construction of a water treatment plant for the Richwood Mutual Water Company to assist them in providing water that meets drinking water standards (Main San Gabriel Basin Watermaster, 2006)

### **1.3 Perchlorate**

Perchlorate is an inorganic contamination that exists in the form of salts. The most common type is ammonium perchlorate. Perchlorate occurs naturally and is also a by-product of industrial or manufacturing spills. Perchlorate is used industrially as solid propellants for rockets, missiles and fireworks. It is also used in the production of matches, flares, pyrotechnics, ordnance and explosives.

Perchlorate was initially discovered in California in 1997 and subsequent groundwater monitoring revealed that the contaminant was widespread in drinking water (CDHS, 2006). Currently no federal or state drinking water standards (Maximum Contaminant Level - MCL) exist for Perchlorate. However the California Department of Health Services (CDHS) has an advisory notification level of 6 µg/L. The CDHS recommends consumer notification for contaminant levels slightly above this standard and source removal for levels far exceeding this standard.

### **1.3.1 Health Effects of Perchlorate**

Perchlorate is deemed to cause interference with iodide uptake by the thyroid gland that result in decreased production of thyroid hormones. The thyroid hormones are needed for prenatal and postnatal growth and development, as well as for normal metabolism and mental function in the adult (CDHS, 2006).

### **1.4 Research Contributions**

This research will further the understanding of contaminant migration and current state of pollution in the Baldwin Park Operable Unit. The models developed here utilize state of the art software and the most up-to-date engineering principles and theories.

The current remedy at the superfund sites in San Gabriel Valley basin is large-scale "pump and treat" (i.e., groundwater extraction and treatment). "Pump and Treat" has not been very effective as high concentrations of PCE, TCE and PCR still exist in the Baldwin Park Operable Unit.

The methodology presented in this research provides an optimal method to rapidly remove the contaminants from the aquifers. A comprehensive transient contaminant transport model is developed using the RT3D module of Groundwater Vistas (groundwater modeling program). Further migration of the contaminants is limited. Contaminants are rapidly trapped and removed by a set of experimentally designed injection and production wells. The procedure utilizes sequential simulation with optimization to optimally and rapidly remove the contaminants.

## 1.5 Research Objectives

Multi-objectives govern the optimal allocation of groundwater and surface water from sources to users to disposal sites. The original objectives of this research were developed by Dr. Dennis E. Williams, owner, Geoscience Incorporated and Research Professor of Civil and Environmental Engineering at the University of Southern California (USC). Dr. Williams consulted Stetson Engineers, the engineers for the Main San Gabriel Valley Watermaster (Watermaster). The objectives of this research are:

- determine the origin of the contaminants; PCE, TCE and PCR;
- determine where the contaminants are going (predict the fate and transport);
- develop a groundwater flow model using MODFLOW, a particle tracking model using MODPATH, a solute transport model using RT3D. Then “layer” on the optimization code to simulate several cleanup scenarios;
- use the groundwater flow model, particle tracking, solute transport model and optimization models to determine the optimum cleanup schemes for both VOC’s and perchlorate (PCR).

This study developed a contaminant transport model and an optimization model that managed the optimal allocation of surface and groundwater supplies under the constraint of minimizing PCE, TCE and PCR contaminant plumes in the Baldwin Park Operable Unit in San Gabriel Valley basin. An economic analysis was also performed to compare the cost of operating the existing system with the cost of operating the strategy proposed in this study.

## **1.6 Management Model**

The management objectives considered in the optimization problem include: Meeting the projected water demands; maximizing groundwater use; maximizing reclaimed water use; minimizing imported water use; minimizing overdraft; minimizing PCE and TCE contaminant plume and minimizing total operation and maintenance cost.

This study analyzes simulation with optimization as a viable management tool for water allocation under the prevailing condition of extracting the contaminant plume. The solutions to the cost optimization model provide managers with a set of policies that determine optimal groundwater pumping rates and schedules and the allocation of groundwater from sources to users to disposal sites whilst restraining the contaminant plume to a desired concentration level. A linear optimization scheme is developed to optimally remove the contaminant plume at the lowest cost.

## **1.7 Study Area**

Baldwin Park Operable Unit (BPOU) is approximately 1 mile wide and 8 miles long. BPOU lies in the central portion of the San Gabriel Valley Basin (Figure 1.1), approximately 25 miles from the Pacific Ocean, in eastern Los Angeles County. BPOU lies south of the San Gabriel Mountains, east of the 605 Freeway, north of the 10 Freeway and west of Azusa Avenue (Figure 1.2). BPOU is fully developed and has a mixture of residential, commercial and industrial facilities. The region has large parcels of open land with active and inactive gravel pits and the Santa Fe Flood control basin. Figure 1.3 shows Hydrologic Boundaries of the San Gabriel Basin. Figure 1.4 shows a

satellite image of BPOU. Figure 1.5 shows the 3D Surface Map and Figure 1.6 shows the Shaded Relief Map of BPOU.

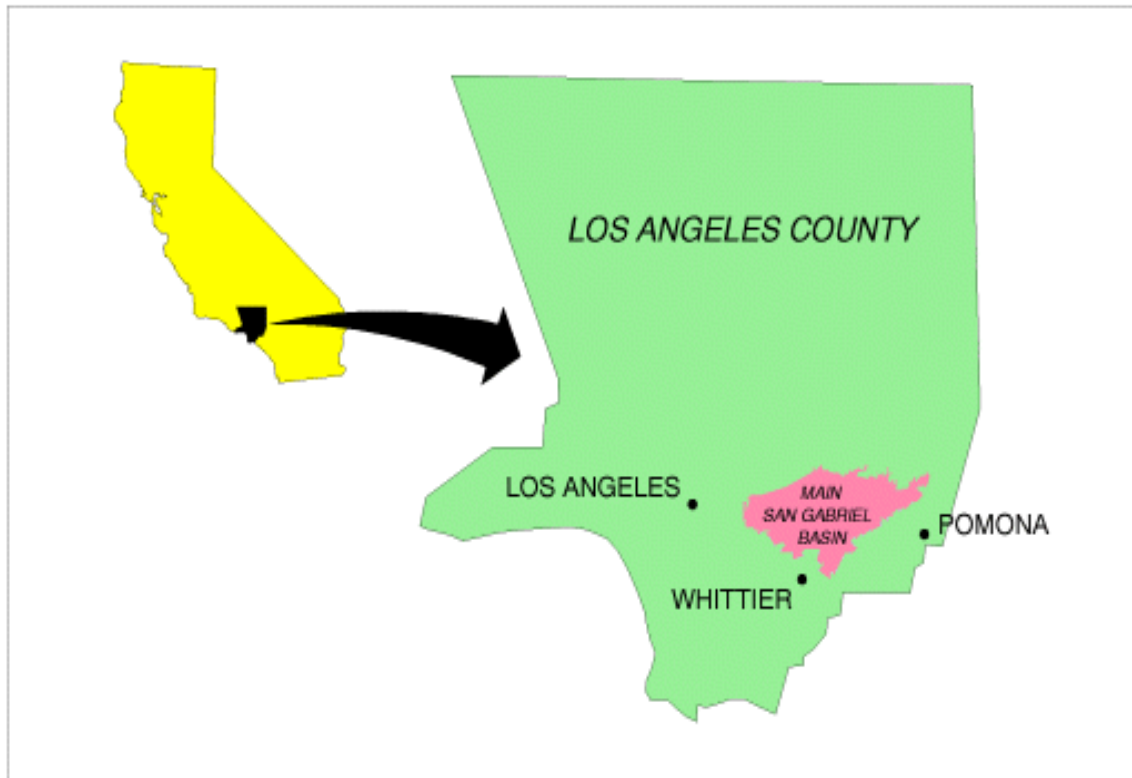


Figure 1.1 Main San Gabriel Basin

*Courtesy of Main San Gabriel Basin Watermaster*

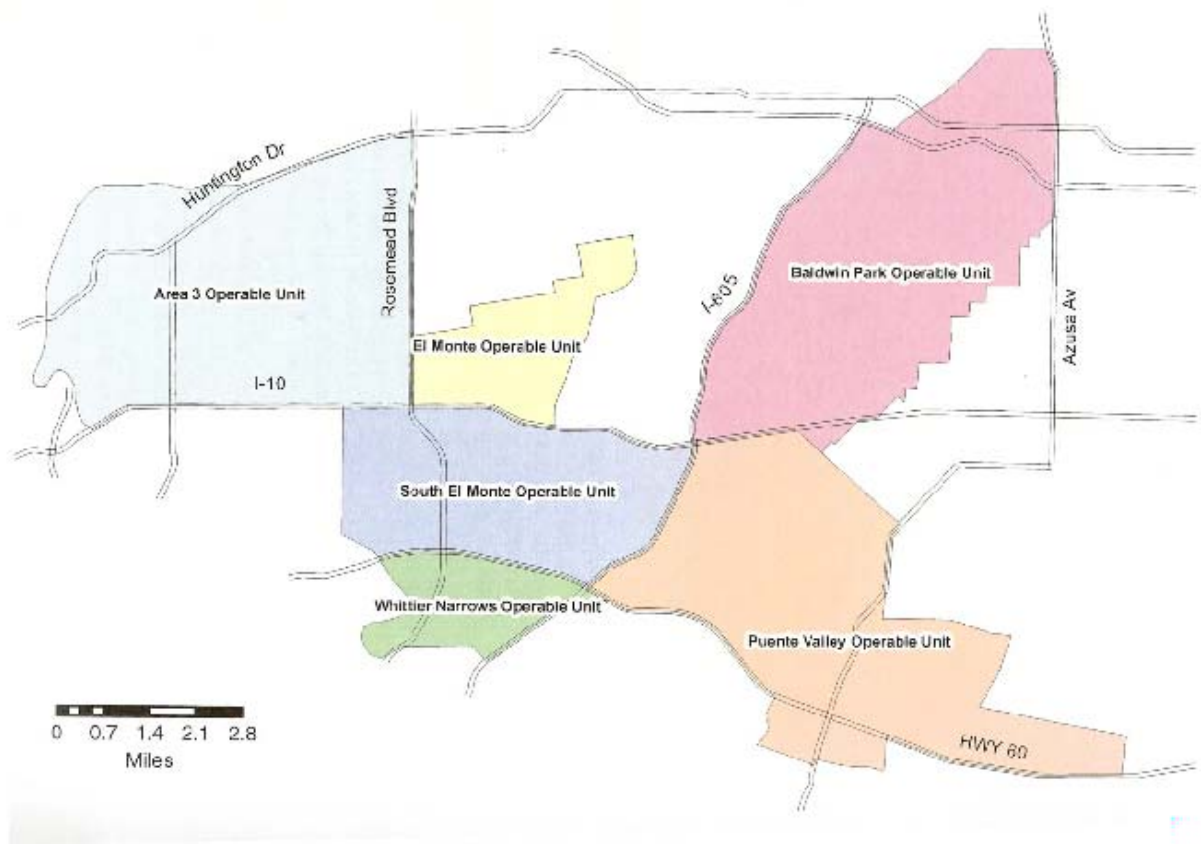


Figure 1.2 San Gabriel Basin Operable Units  
(Watermaster, Five Year Water Quality Plan (2005/2006))

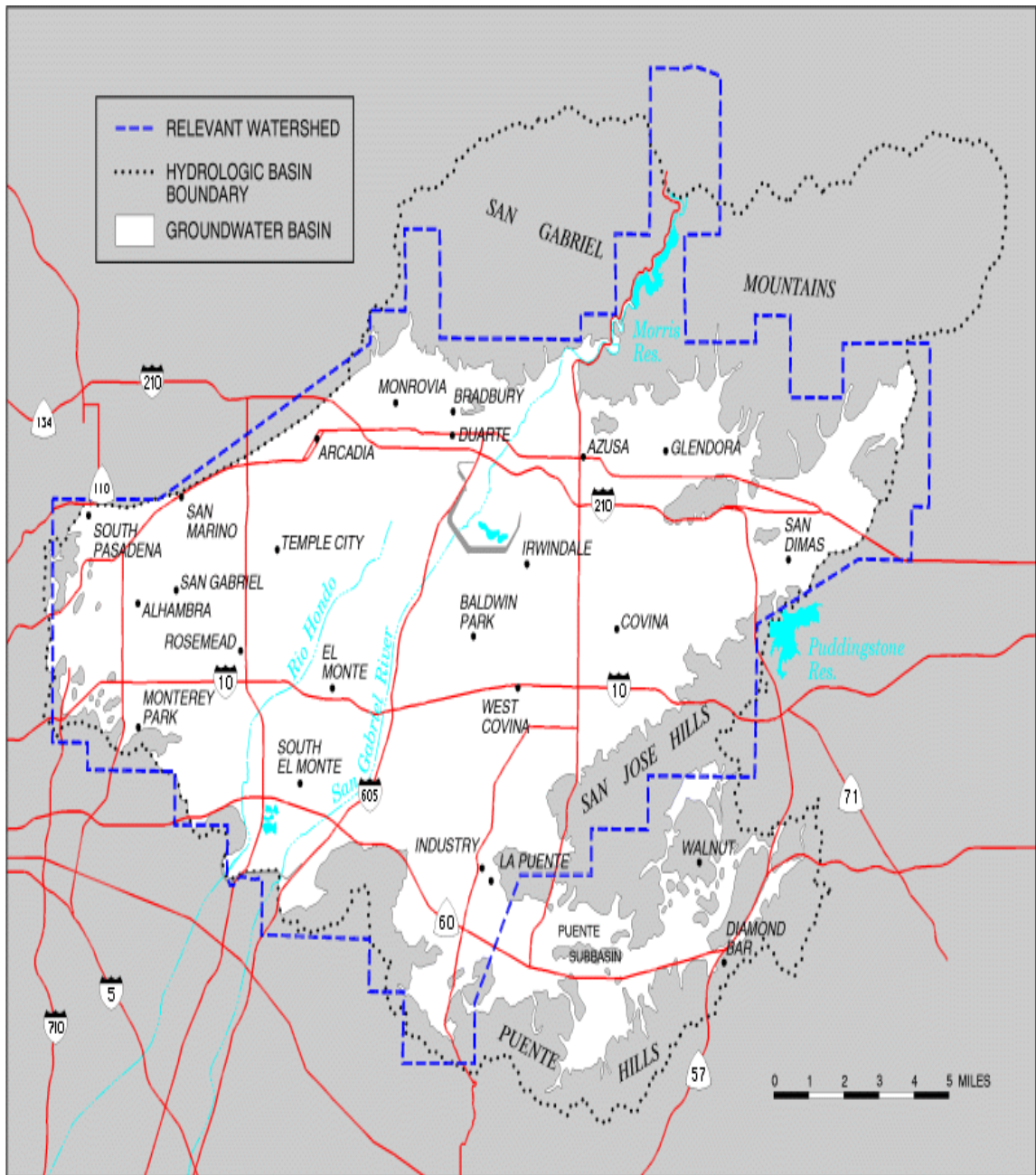


Figure 1.3 San Gabriel Basin Showing Hydrologic Boundaries

Courtesy of Main San Gabriel Basin Watermaster

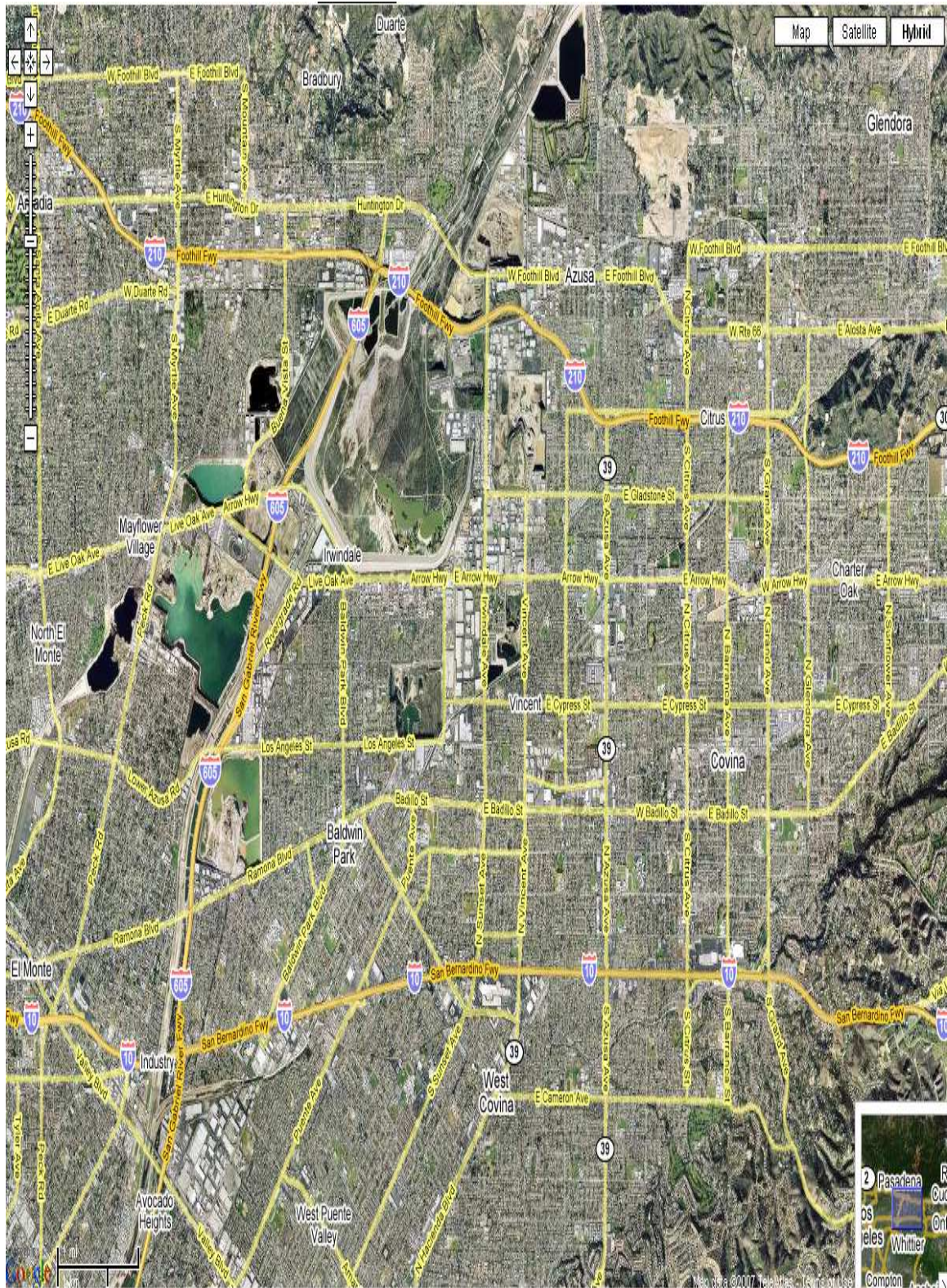


Figure 1.4 Satellite Image of BPOU (courtesy of [www.google.com](http://www.google.com))

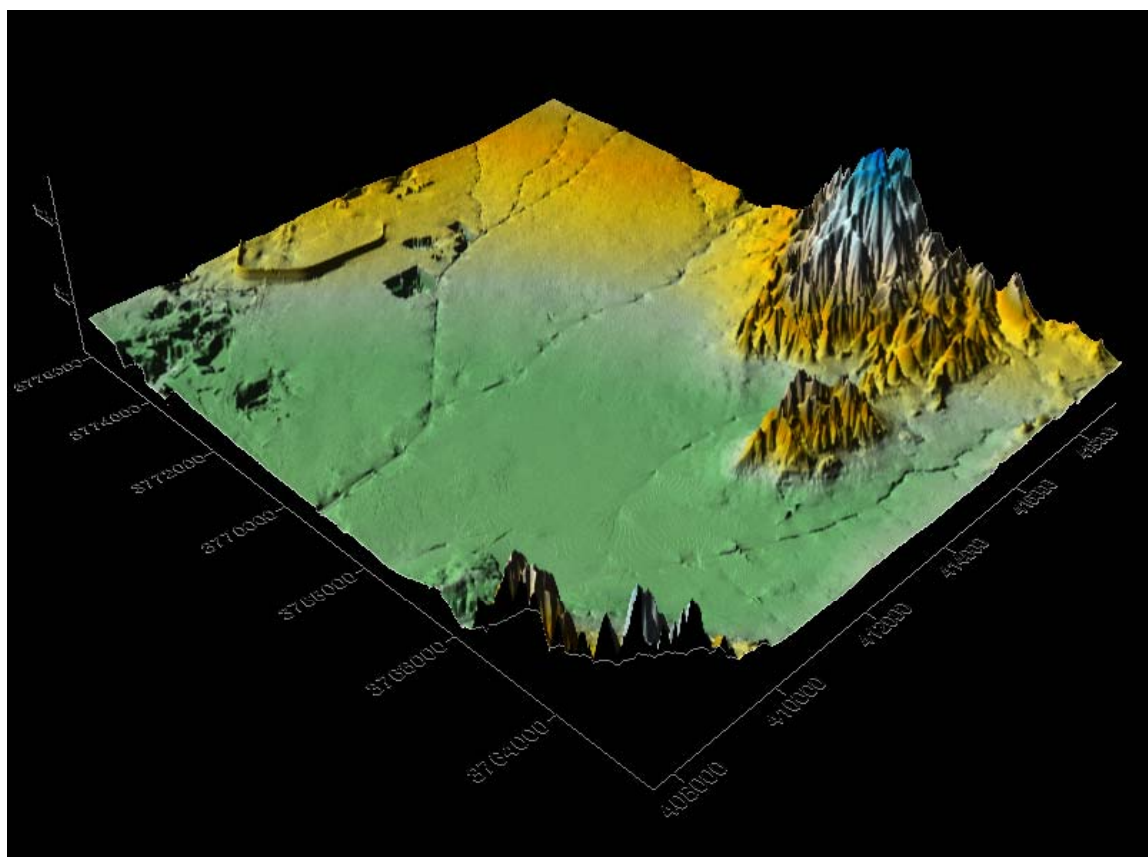


Figure 1.5  
3D Surface Map of BPOU

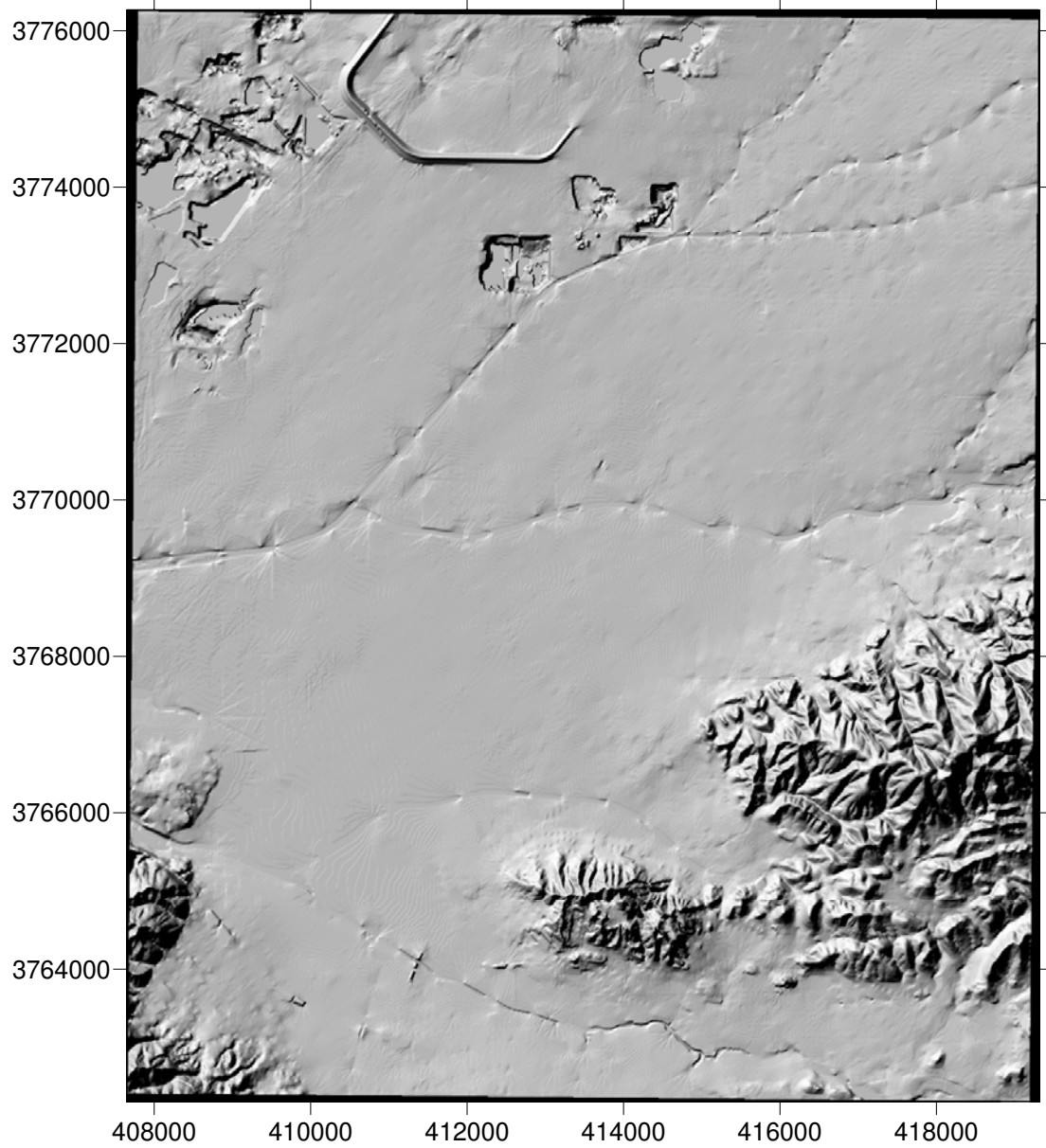


Figure 1.6

Shaded Relief Map of BPOU

## **1.8 Sources of Data**

Several sources of data were utilized in developing the flow, particle tracking and contaminant transport models. Hydrogeologic data for BPOU were obtained from the following sources:

### **1.8.1 Geologic Data**

- Aquifer Systems – California Department of Water Resources (1966)
- Well Lithology - USEPA San Gabriel Valley Database 2008
- Ground Elevation - DEM - [www.gisdatadepot.com](http://www.gisdatadepot.com)
- Bottom Elevation- USEPA San Gabriel Valley Database 2008

### **1.8.2 Water Data**

- Semi-Annual Water levels (to establish transient general head boundaries)- Main San Gabriel basin Water master- Annual Reports
- Initial Water levels- USEPA San Gabriel Valley Database 2008
- Water Level Target Heads- USEPA San Gabriel Valley Database 2008
- Pumping Wells and Pumping Rates- USEPA San Gabriel Valley Database 2008
- Monitoring Wells and Water Quality Data- USEPA San Gabriel Valley Database 2008
- Aquifer properties – Hydraulic conductivity, specific storage, effective porosity- USEPA San Gabriel Valley Database 2008

## 1.9 Contaminants

Multiple commingled plumes of groundwater contamination exist in the Baldwin Park Operable Unit spanning over a mile wide and eight miles long. The groundwater depth varies from approximately 150 to 350 feet. The groundwater contamination extends from the water table to more than 1,000 feet below ground surface. The most prevalent contaminants in the groundwater are trichloroethene (TCE), tetrachloroethene (PCE), carbon tetrachloride, perchlorate, and N-nitrosodimethylamine (NDMA). TCE, PCE, and carbon tetrachloride are solvents used for degreasing and cleaning. Perchlorate is used in solid propellant for rockets, missiles, and fireworks; and NDMA is associated with liquid-fuel rockets. Other VOCs including the chemical 1, 4-dioxane, which has been used as a stabilizer in chlorinated solvents exist. The peak PCE contaminant concentration detected in groundwater in the Baldwin Park Operable Unit is 38,000  $\mu\text{g/L}$ , exceeding 7500 times the maximum contaminant level (MCL) allowed by Federal and State law.

## 1.10 Developing the Hydraulic Gradient Control Scheme

The development of the optimized hydraulic gradient control scheme proposed in this study consists of a two-step process:

*Step 1: Generate Hydraulic and Contaminant Flow Field:* Initially, groundwater flow is simulated. Then the particle-tracking program is run to simulate contaminant flow paths. RT3D generates contaminant plume contours. This procedure establishes base line contaminant contours to delineate the existing condition and aids in developing the

hydraulic control scheme.

Preliminary strategic locations of wells that are most effective in cleaning up the plume are determined in an experimental design approach. Wells that are most effective in cleaning up the plume are selected by several simulation and optimization runs to determine their optimal locations and flow rates. The iterative coupling of simulation with optimization procedure guarantees a successful hydraulic gradient control scheme (Figure 1.7).

Step 2: *Optimization* - rates for recharge and extraction are optimized using various constraints using output from the simulation model. Simulation models depict the chemical and the physical behaviors of the system. The hydraulic gradient control scheme initiates inward gradient that prevents further intrusion, stabilizes and removes the contaminant plume. Hydraulic gradients are achieved by varying recharge and pumping rates using groundwater simulation, particle tracking and contaminant transport modeling. The annual cost of operation and maintenance is minimized while satisfying the set of the constraints. Plume cleanup is achieved when the PCE and TCE contaminant concentration is less than 5  $\mu\text{g/L}$  and the PCR plume is less than 6  $\mu\text{g/L}$ .



Figure 1.7 Flow Chart of Iterative Simulation with Optimization

## 1.11 Coupling Simulation with Optimization

Optimal management of groundwater resources requires the most efficient allocation of water supply and demand under given constraints to ensure optimum benefits to users.

Simulation with optimization models have been effectively used in groundwater remediation. Models may minimize annual cost of operation and maintenance, impact to the environment or maximize net benefits to users.

Simulation of groundwater flow is a very important tool used in hydrogeologic investigations to study the interactions between surface water and groundwater. As a

management tool, simulation models predict the long and short term impacts of Groundwater recharge and extraction and subsurface inflow and outflow. Simulation models are highly useful in the study of contaminant transport. Water managers use simulation models to analyze and assess their strategies and alternatives. When coupled with optimization, water managers can find optimal solutions to their operating policies. Hence, researchers and water managers highly depend on simulation coupled with optimization to seek optimal alternatives.

Optimization methods have been extensively used for decision-making. Optimization is used to maximize resource allocation or minimize adverse effects or costs to the user and environment. The real life problem is transformed into a mathematical statement. Physical, institutional, operational and legislative constraints are formulated to satisfy management objectives. Optimization yields the optimal solution to the given management objective subject to the given constraints.

This research minimizes the annual maintenance and operation cost of contaminant plume stabilization and cleanup while meeting operational demand. The polluted plume is isolated from the rest of otherwise hydraulically connected system by producing a hydraulic regime that directs groundwater flow toward the core of the plume. The contaminant removal methodology developed in this study is predicated on the experimental design of a hydraulic control scheme based on variation of pumping and recharge flow rates.

## **1.12 PCE and TCE Superfund Sites**

The Baldwin Park Operable Unit has been identified by the USEPA as a volatile organic compound (VOC) superfund site. Clean up efforts are currently underway. PCE and TCE are halogenated aliphatic volatile organic compounds (VOCs) and have been widely used as an ingredient in industrial cleaning solutions and as a universal degreasing solvent due to its unique properties and solvent effects. TCE (tetrachloroethene) and TCA (trichloroethane) are the most frequently detected VOCs in groundwater in the United States (Fischer et al., 1987). Approximately 20% of 215 wells sampled in a New Jersey study contained PCE and TCE and other VOCs above the ppb detection limit (Fusillo et. al, 1985). The presence of PCE and TCE has led to the closure of water supply wells on Long Island, N.Y. and in Massachusetts (Josephson, 1983). Detectable levels of at least one of 18 VOCs, including TCE, were reported in 15.9% of 63 water wells sampled in Nebraska, a State having a low population density and industrial base (Goodenkauf and Atkinson, 1986).

### **1.12.1 PCE and TCE Health Effects**

The groundwater in Baldwin Park Operable Unit is contaminated with various VOC's, predominantly PCE and TCE. Inhaling vapors from the contaminated groundwater exposes people to hazardous substances. PCE and TCE are not directly carcinogenic. They are thought to become a human health hazard only after processing in the human liver (Bartseh et al., 1979). However, processing in the human liver is not the only way in which PCE and TCE may become a health hazard. Reductive dehalogenation of PCE

and TCE through natural or induced mechanisms may result in production of vinyl chloride (VC) which, in contrast to TCE, is a known carcinogen (Federal Register, 1984).

### **1.13 Dissertation Overview**

Each chapter contains the pertinent tables and figures. Except otherwise stated, the figures are not to scale and the north arrow points to the top of the page. Chapter 2 discusses current cleanup efforts in the Baldwin Park Operable Unit (BPOU). The methodology employed in this research is recommended as a strategic solution to basin wide contamination in the Baldwin Park Operable Unit. Historic, current, research output and Watermaster projected contaminant concentrations are plotted.

Chapter 3 discusses general literature review of groundwater simulation, contaminant transport and management studies focused on contaminant plume containment and removal. Groundwater flow and contaminant transport simulation models and plume cleanup using water resources management and optimization techniques are emphasized.

Chapter 4 discusses the geological and hydrological features of BPOU. The geologic settings, geologic units and model layers are discussed. Groundwater elevations, flow direction and seepage velocities are highlighted as well as recharge and discharge.

Chapter 5 discusses the methodologies adopted in this study. Computer software utilized in this study and the extent of their application described. Boundary conditions and governing equations are established for groundwater flow simulation, particle tracking

and contaminant transport. An optimal hydraulic gradient control scheme is developed and the procedure outlined. The simulation and optimization models are mathematically formulated. The key software packages for the groundwater simulation model are discussed. The multi-objective optimization model and its subsequent transformation into a single-objective management model are outlined. The management model constraints are also highlighted.

Chapter 6 describes the Baldwin Park Operable Unit case study used to test the methodology adopted in this research. Groundwater flow simulation, particle tracking and contaminant transport models are formulated and calibrated to match site conditions. Boundary conditions and assumptions for the case study are outlined. Optimization procedures are discussed. The selection procedure for potential decontamination and hydraulic gradient control wells is also highlighted.

Chapter 7 presents the results of the groundwater flow simulation, particle tracking, contaminant transport and optimization models for the Baldwin Park Operable Unit case study. Model scenarios and applicability are discussed. An economic analysis was performed to compare the cost of the existing system with the cost of the strategy proposed in this study.

The last chapter summarizes this study and discusses the results. It also presents the conclusions of this study, describes the attainment of research goals and recommends strategies for implementation.

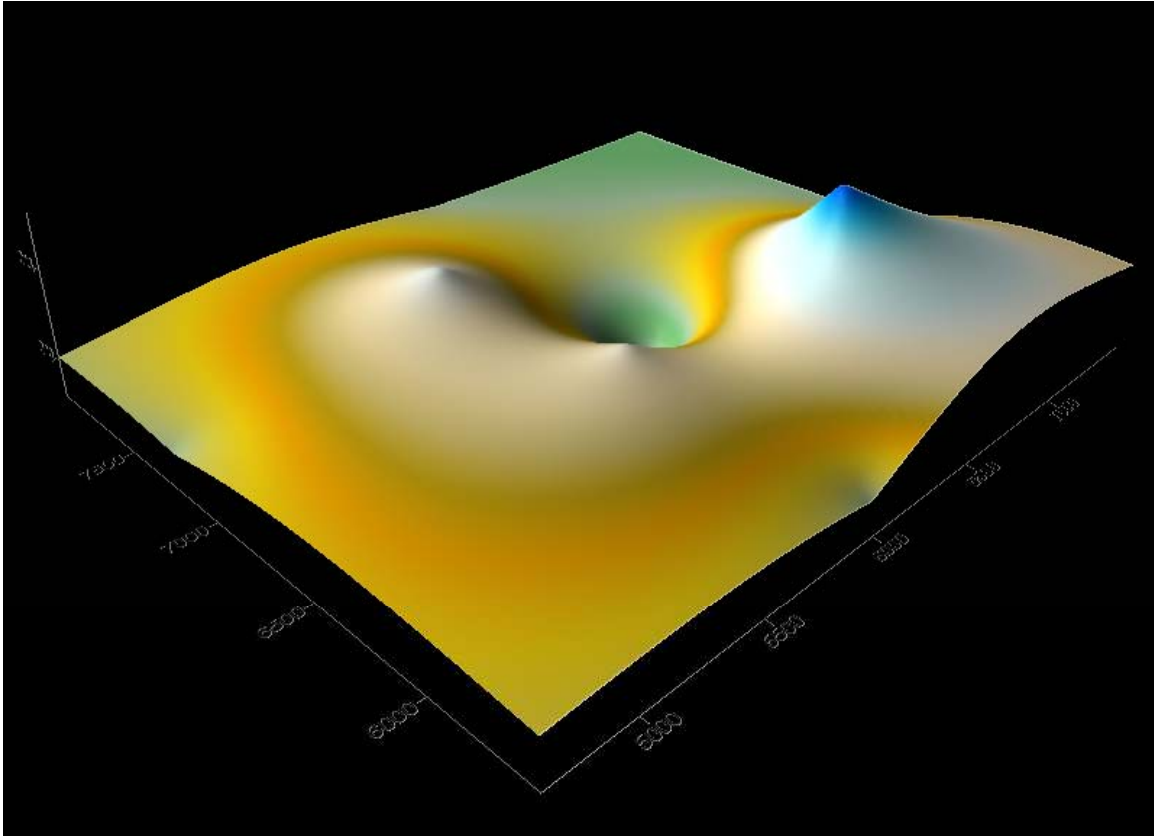
## **Chapter 2**

### **Current Cleanup Efforts**

#### **2.1 Cleanup Strategy**

This chapter elaborates on current cleanup efforts in the Baldwin Park Operable Unit (BPOU). This research highlights the fact that current contamination levels of PCE, TCE and PCR are still high and establishes the need for the current regulatory agencies overseeing the cleanup to adopt the methodology developed in this research to effectively cleanup the contaminants in BPOU. Figure 1.1 shows BPOU in the San Gabriel Basin.

This study involves a practical and systematic development of a hydraulic gradient control scheme in which locations and rates of wells are varied subject to given constraints to effectively stabilize, capture and cleanup the contaminant plume. The optimized hydraulic gradient control methodology developed in this study physically contains, shrinks and extracts the contaminant plume below the MCL. The methodology utilized in this study is based on iterative transient groundwater simulation, particle tracking, contaminant transport and optimization. The methodology employed in this research namely; iterative transient groundwater simulation, particle tracking, contaminant transport and a hydraulic gradient scheme cleans up the contaminant plume. Optimization ensures that the plume is cleaned using optimum well locations and flow rates. The figure below depicts a 3-dimensional view of the hydraulic gradient scheme developed in this research.



**Figure 2.1**  
**Hydraulic Gradient Control Scheme**

## **2.2 Agencies Effecting Cleanup**

### United States Environmental protection Agency (USEPA)

The United States Environmental Protection Agency (USEPA) is the lead agency overseeing the clean up of the contaminants at the San Gabriel Valley Basin superfund sites in southern California. Other key agencies corroborating with the USEPA on the clean up include: The San Gabriel Basin Watermaster, the California Department of Health Services (DHS) and the San Gabriel Basin Water Authority. The current strategy by all the agencies involved in the cleanup is “pump and treat.” Groundwater pumped at

production wells is treated by treatment facilities on-site. Blending groundwater from wells not impacted by the contamination with groundwater from wells impacted by the contamination is a supplemental strategy employed by the water purveyors to circumvent the contamination problem. This research provides an overall systematic strategy to clean up the entire aquifers at the study location namely, the Baldwin Park Operable Unit (BPOU).

The United States Environmental Protection Agency (USEPA) is the federal and lead agency overseeing the cleanup of the contaminants at the San Gabriel Valley Basin superfund sites in southern California. Other agencies at the state and local levels are involved in the cleanup.

#### San Gabriel Basin Watermaster

In, 1973, the Los Angeles County Superior Court created the San Gabriel Basin Watermaster to resolve water supply issues among water users in the San Gabriel Valley. In the late 1970s and 1980s, several contaminants were discovered in the San Gabriel basin. The contamination was a result of disposal and leaks from local industrial and manufacturing facilities. Also nitrates from agricultural activities infiltrated into the groundwater.

In 1989, local water agencies adopted a joint resolution that mandated the San Gabriel Basin Watermaster to coordinate local activities for preserving and restoring groundwater quality in the basin. The joint resolution initiated a cleanup plan.

In 1991, the Los Angeles County Superior Court granted Watermaster the authority to control groundwater quality by controlling the amount of groundwater pumped. Since then, the San Gabriel Basin Watermaster's role has evolved to include managing water quality. Watermaster develops a Five Year Water Quality Plan, updates it annually and submits it to the California Regional Water Quality Control Board, Los Angeles Region. This plan is available for public review by November 1, each year.

Currently, the San Gabriel Basin Watermaster coordinates groundwater activities so that both water supply and water quality are enhanced and protected. The Watermaster sets limit on the amount of groundwater that can be extracted from the basin (safe yield). The 2006/2007 safe yield is set at 240,000 acre -ft/year. Safe yield is required to prevent subsidence and ensure that pumping does not lead to further degradation of water quality in the basin. Other objectives of the Watermaster include:

- Monitoring groundwater supply and quality;
- Projecting future groundwater supply and quality;
- Review and coordinate cleanup projects;
- Provide technical assistance to other agencies;
- Address and provide a clean up plan for new contaminants discovered in the basin;
- Develop water supply and water quality cleanup plans consistent with the USEPA plans for the basin;

- Coordinate and manage the permitting, design, construction and performance of the Baldwin park operable unit (BPOU) cleanup and water supply plan (Five - Year Water Quality and Supply Plan, San Gabriel Basin Watermaster, 2006/2007).

#### San Gabriel Basin Water Quality Authority

The San Gabriel Basin Water Quality Authority is the key agency appointed to develop a comprehensive water quality plan to clean up the groundwater pollution in the San Gabriel basin. The plan addresses:

- Contamination in all the operable units
- Projects that combine cleanup with water supply
- Seeks funding from responsible parties, federal, state and local agencies to effect the clean up.

The primary remedial approach is pump and treat contaminated groundwater whilst providing adequate groundwater supply to its users.

### **2.3 Baldwin Park Operable Unit Cleanup History**

The Baldwin Park Operable Unit (BPOU) in the San Gabriel Valley is one of the largest Superfund cleanups in the United States. High concentration of VOCs and Perchlorate exceeding 100 times the maximum contaminants levels still exist. Peak concentrations of 38000 µg/L for PCE and 4000 µg/L for TCE have been detected at this site.

The BPOU encompasses a seven-mile long, one mile wide area of contamination that stretches from north of the 210 freeway to south of the 10 freeway in Baldwin Park. The VOC's and Perchlorate contamination resulted predominantly from improper disposal from industrial and manufacturing facilities in Azusa. The contaminants are spreading in the south westerly direction.

The record of decision (cleanup plan) issued by the USEPA in 1999 called for pumping and treating the contaminants to limit further migration of the contaminants. The record of decision recommended adding the treated water to the potable water supply instead of using the treated water for recharge or disposing to storm drains.

The previously constructed treatment facilities were hampered by the discovery of new contaminants Perchlorate and NDMA. The treatment facilities had to be shut down because they were specifically designed to treat VOC's. The shutdown caused a ripple effect facilitating the spread of contaminants to previously unaffected areas downstream.

In 2002 after several years of negotiations, the San Gabriel Watermaster brokered a deal in which eight of the BPOU Responsible Parties (called Cooperating Respondents or CRs) and seven water agencies signed the BPOU project agreement. Under this agreement, the CRs pay the cost to construct and operate the USEPA planned BPOU cleanup and treatment facilities whilst Watermaster provides project management services. Local water purveyors own and operate the treatment facilities and supply the treated water in their water supply systems. Also, the San Gabriel Basin Water Quality

Authority has obtained external funds to help construct additional treatment facilities, extraction wells and pipelines.

The treatment facilities operated in the current BPOU Cleanup Project are: Valley County Water District's (VCWD) Lante Plant (7,800 gpm), San Gabriel Valley Water Company's (SGVWC) Plant B5 (7,800 gpm) and Plant B6 (7,800 gpm), and La Puente Valley County Water District's (LPVCWD) site (2,500 gpm). These facilities have a combined extraction and treatment capacity of up to 25,900 gpm. The status of these treatment facilities are also tabulated in Table 2.1.

Table 2.1 BPOU Water Treatment Projects

| Project        | Capacity (gpm) | Location     | Customers                                                  | Status                                                   |
|----------------|----------------|--------------|------------------------------------------------------------|----------------------------------------------------------|
| LPVCWD         | 2,500          | Baldwin Park | La Puente and Industry                                     | Completed 2000                                           |
| SGVWC Plant B6 | 7,800          | Baldwin Park | Baldwin Park, La Puente, Basset and Lower Hacienda Heights | Completed 2005                                           |
| SGVWC Plant B5 | 7,800          | Industry     | Baldwin Park, La Puente, Basset and Lower Hacienda Heights | Completed 2007                                           |
| VCWD           | 7,800          | Irwindale    | Baldwin Park, La Puente, Basset and Lower Hacienda Heights | Completed 2005.<br>Supplemental Treatment Completed 2007 |

The VCWD project located in northern BPOU comprises three extraction wells, including two new wells pumping up to 7,800 gpm (average annual rate of 7,000 gpm) and an in-line treatment facility. The VCWD treatment facility has facilities that separately treats VOCs, perchlorate, NDMA and 1, 4-dioxane. The VCWD will serve up to 6,000 gpm of treated water through a pipeline to Suburban Water Systems (SWS) to offset production lost due to contamination of some of its wells.

The VCWD was permitted by the California Department of Health Services in Fall 2005 and is currently operational. However, the discovery of a new contaminant, 1, 2, 3-TCP has delayed full implementation. Currently, the treated water is discharged to a storm channel and supplemental treatment that cleans 1, 2, 3-TCP is being installed. It is anticipated that the treatment facility will provide treated potable water by mid-2007.

The remaining three treatment projects, SGVWC Plant B5 and Plant B6 and LPVCWD are located in the southern BPOU. The LPVCWD was permitted by CDHS has been operational since March 2001. Treated water exceeding LPVCWD's demand is diverted to SWS.

The SGVWC B5 project comprises one new extraction well with two existing wells providing up to 7,800 gpm to an on-site treatment facility. The SGVWC is currently in construction and will be operational by spring 2007. The facility will treat contaminated water for VOCs, perchlorate, NDMA and 1, 4-dioxane. After the permit from CDHS is issued, the facility will provide portable water to the City of Industry customers up to

1,200gpm and still continue to serve SGVWC customers.

The SGVWC B6 project has been permitted by CDHS and has been in operation since July 2005. The project comprises four new off-site extraction wells and a central treatment facility located at the site. The treatment facility removes all currently discovered contaminants namely VOCs, perchlorate, NDMA and 1, 4-dioxane. The SGVWC has treated approximately 28,000 acre feet of water and removed 2,500 pounds of contaminants.

In addition to the four treatment projects described above, several water purveyors have constructed treatment facilities in the less contaminated or non-contaminated zones. Blending groundwater from the non-contaminated wells with treated water from the contaminated wells is an alternative treatment approach utilized by several water purveyors in the BPOU to satisfy water supply demand. Watermaster has recently (2005/06 fiscal year) approved the construction of a new well at the California Domestic Water Company (CDWC) wellfield. The new well will enable CDWC to produce blended water that satisfies all regulatory water quality standards and avoids the construction and operation of expensive new treatment facilities. In addition, blending reduces reliance on expensive imported water and reduces contamination. Table 2.2 list the treatment technologies used to treat the pumped water at the treatment facilities.

Table 2.2 Treatment Technologies for Contaminants in BPOU

| Chemical                      | Source                                          | Treatment Technology                    |
|-------------------------------|-------------------------------------------------|-----------------------------------------|
| Perchloroethylene (PCE)       | Industrial solvents for cleaning and degreasing | Air stripping with off-gas treatment    |
| Trichloroethene (TCE)         | Industrial solvents for cleaning and degreasing | Air stripping with off-gas treatment    |
| Carbon Tetrachloride          | Industrial solvents for cleaning and degreasing | Air stripping with off-gas treatment    |
| Perchlorate (PCR)             | Solid rocket fuel                               | Ion exchange                            |
| Nitrate                       | Fertilizer                                      | Ion exchange                            |
| N-nitrosodimethylamine (NDMA) | Liquid rocket fuel                              | Ultraviolet light and hydrogen peroxide |
| 1,4-dioxane                   | Stabilizer for chlorinated solvents             | Ultraviolet light and hydrogen peroxide |

## 2.4 Potentially Responsible Parties (PRPs)

The USEPA identified several Potentially Responsible Parties (PRPs) but only eight of the (PRPs) are paying for most of the cleanup. The eight PRPs are:

- Aerojet - General Corporation
- Azusa land Reclamation Company, Incorporated
- Fairchild Holding Corporation
- Hartwell Corporation
- Oil and Solvent Process Company
- Reichhold Incorporated and
- Wynn Oil Company

The federal government provides supplemental funding through superfund legislation for San Gabriel Valley.

## **1.5 Contractors**

Several consultants and contractors are involved in the BPOU cleanup including:

- CH2 M Hill is the consultant for USEPA for San Gabriel Valley.
- Stetson Engineers, Incorporated of Covina, CA , serve as the engineer for Watermaster and construction manager for the remediation work
- Calgon Carbon Corporation of Pittsburgh, PA supply perchlorate removal systems
- Trojan Technologies Incorporated of Tucson, AZ, supply NDMA and 1,4-dioxane removal systems
- R. C. Foster of Corona, CA installs the treatment systems
- Valverde Construction, Incorporated of Santa Fe Springs, CA, installs the pipelines.

## **1.6 Precipitation**

The long term average rainfall in the San Gabriel Valley Basin is 18.52 inches. In 2004-05 fiscal year, an average of 46 inches of rainfall fell on the San Gabriel Valley Basin. In 2005-06, the San Gabriel Valley Basin received an average of 16 inches of rain. Figure 2.2 shows average annual rainfall for the San Gabriel Basin from 1974 to 2006. The precipitation data is used to develop the transient groundwater flow and contaminant transport model. The precipitation data is used to develop the hydrologic base period and hydrologic model.

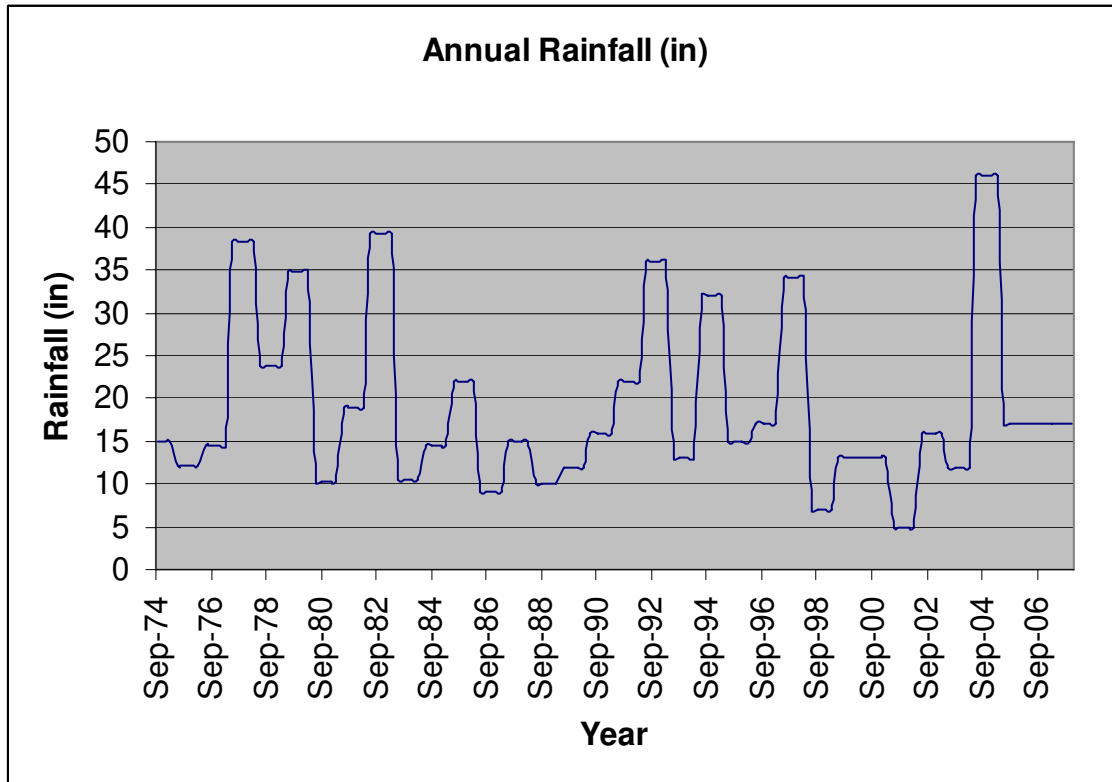


Figure 2.2 Average Annual Rainfall (in) San Gabriel Basin

## 2.7 Hydrograph – Baldwin Park Key Well

The Key Well located in Baldwin Park is the benchmark for determining groundwater levels for the entire basin. Figure 2.3 shows Key Well groundwater elevations from 1986 to 2006. The Key Well water level fell to a historic low of 195.5 feet above mean sea level on December 23, 2004. Heavy rainfall in the winter of 2005 recharged the basin and elevated the Key Well water level to 251 feet in early June 2005. Watermaster strives to maintain the Key Well water level between 200 feet and 250 feet above mean

sea level through proficient basin management practices. Moderate rainfall in 2005-06, continued water capture, and significant untreated imported water deliveries for groundwater recharge helped maintain a high water elevation of 250.6 feet at the Baldwin Park Key Well on June 30, 2006. Hydrographs at several wells are used to develop the transient groundwater flow and contaminant transport model.

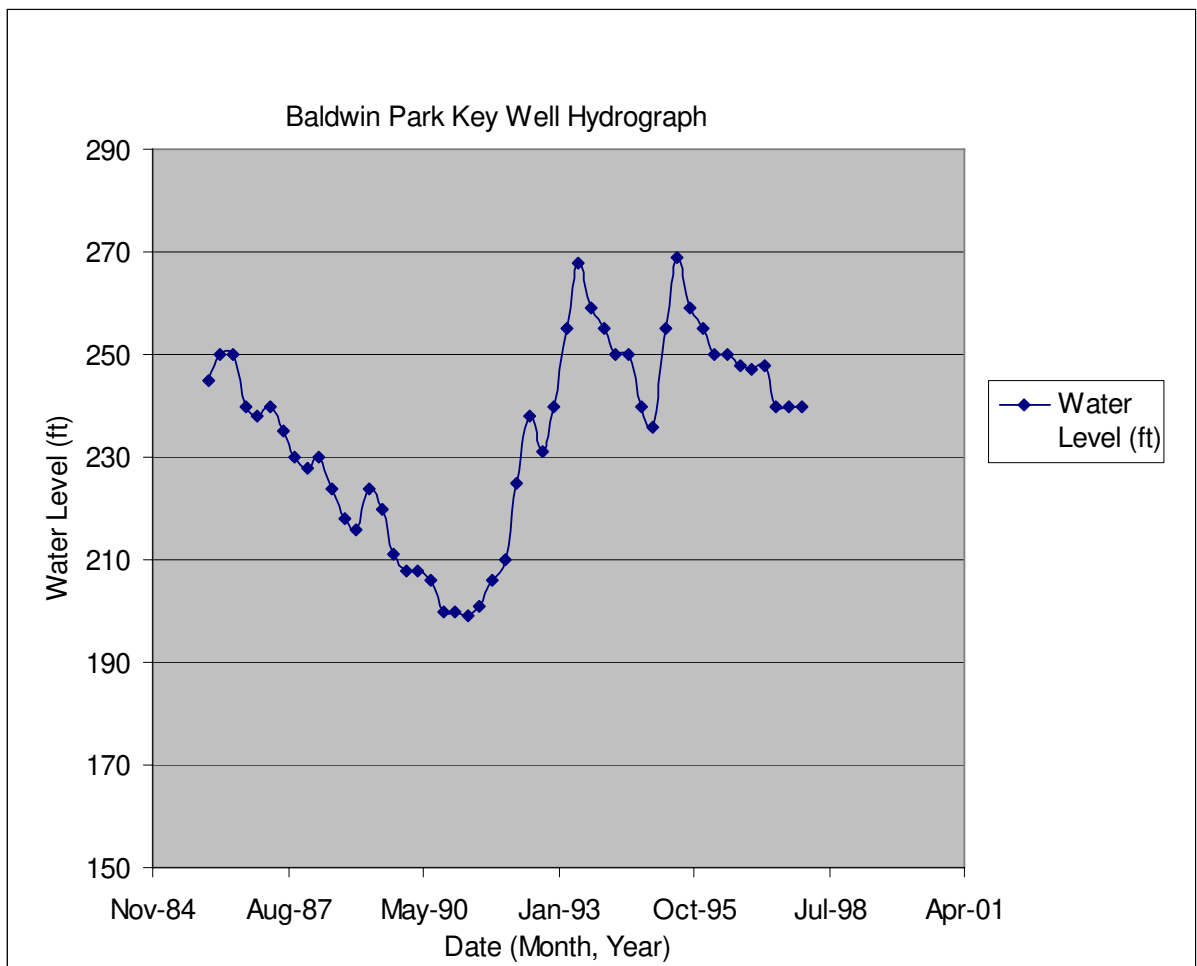


Figure 2.3 Baldwin Park Key Well Hydrograph (ft)

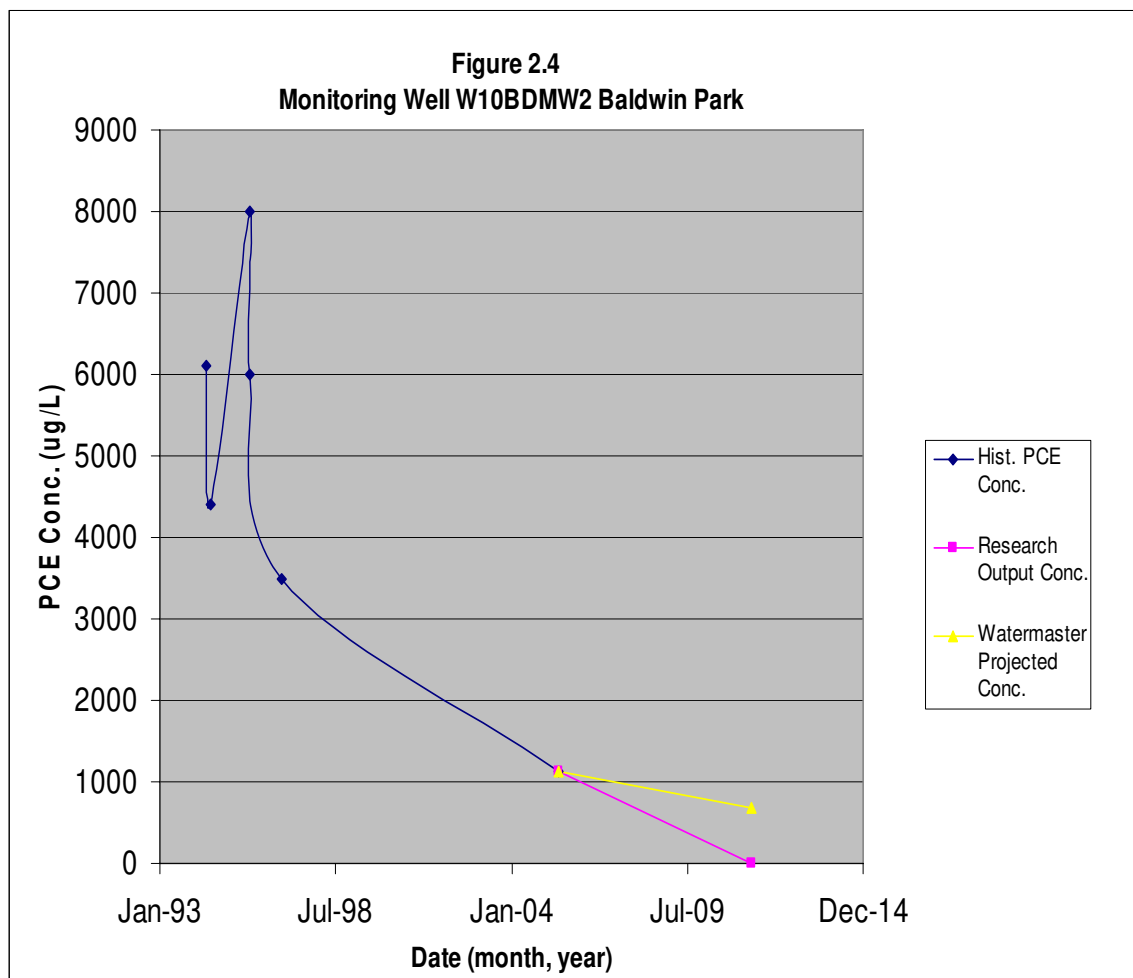
## 2.8 Research Strategy

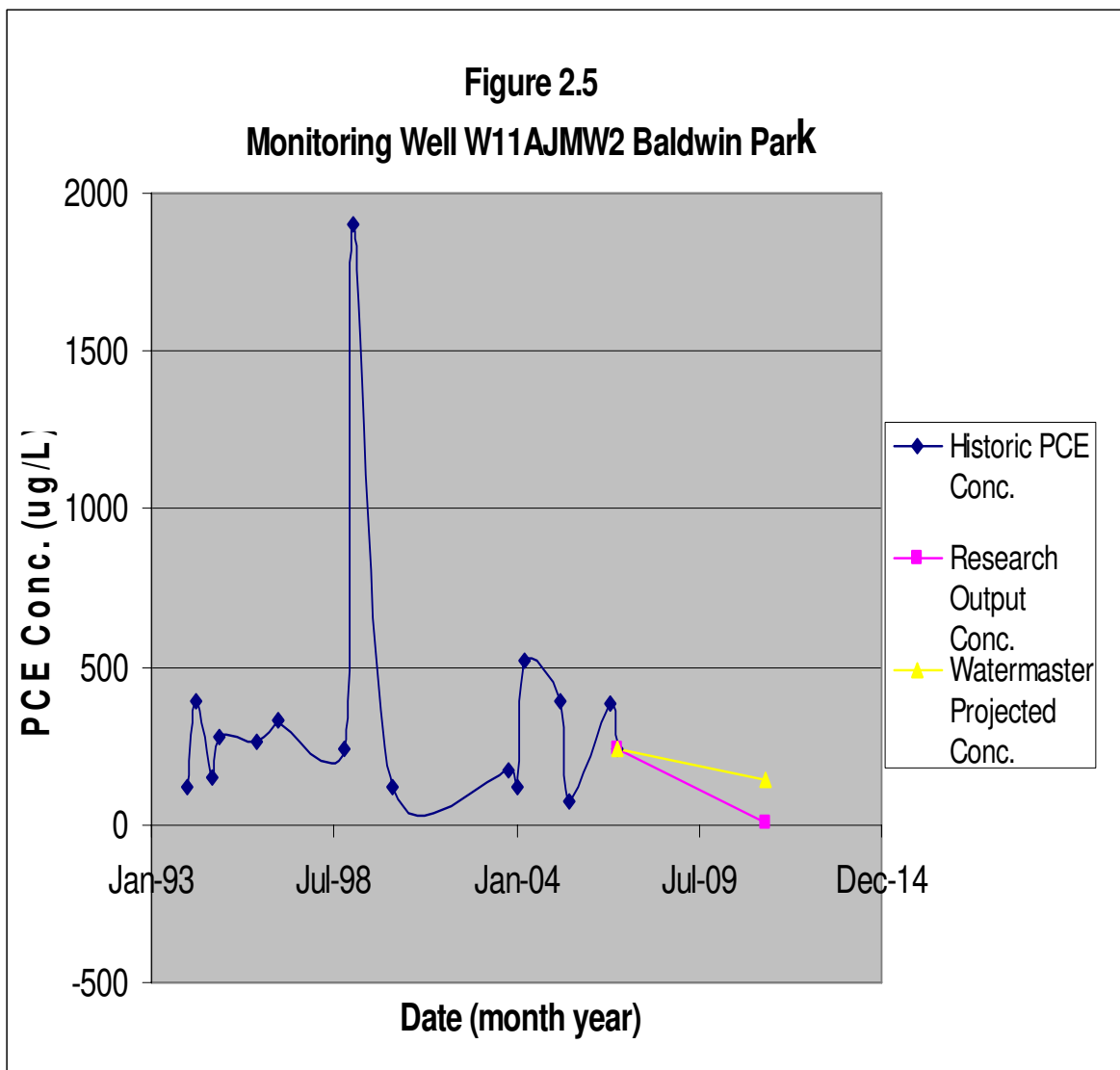
This research provides a viable basin-wide methodology to rapidly and effectively remove contaminant pollution in BPOU. The current methodology employed by regulatory agencies performing the clean up is not effective. High levels of PCE, TCE, other VOC's and perchlorate still exist in BPOU. Currently, a pump and treat methodology is employed to spot clean critical areas of contamination in BPOU. The current focus of the regulatory agencies overseeing the cleanup is to treat and supply groundwater that meets the MCL or NL of contaminants. Their primary goal lacks an overall strategy to clean up the aquifer. The graphs shown in Figure 2.4 to Figure 2.13 show that high levels of VOCs and Perchlorate still exist after 18 years of clean up in BPOU. Thus a methodology is needed to effectively cleanup the plume in the basin. This study develops a methodological process that sequentially contains and removes the contaminant plume. The methodology employed in this research involves transient groundwater and contaminant transport simulations, ingenious analysis of the contaminant flow and transport, application of hydraulic gradients and optimization. Sequential simulation with optimization methodologically reduces the contaminant levels below MCL. The graphs also show output concentrations from the methodology proposed in this study. The graphs of the contaminant histories for PCR, PCE and TCE reveal that:

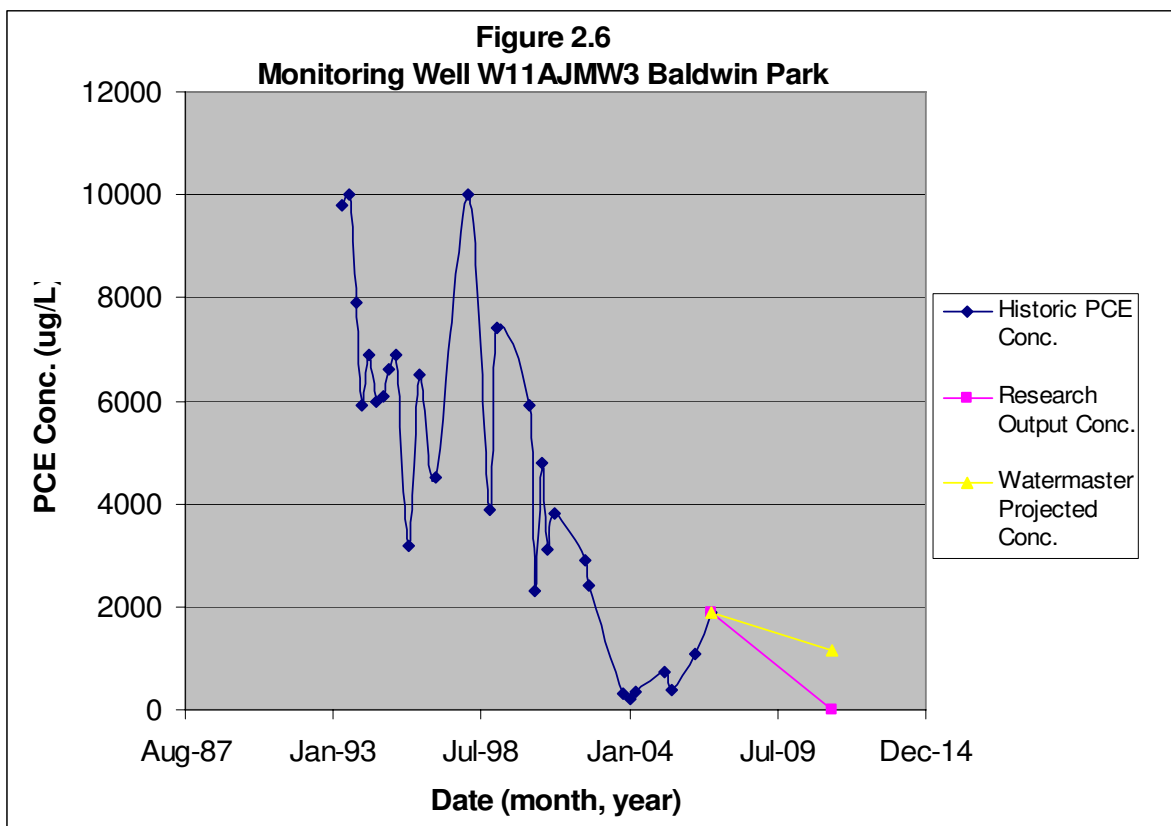
- Most wells in BPOU still show high levels of contamination.
- Observed concentrations vary exhibiting seasonal change in concentration flux
- USEPA's current method of "pump and treat" for large scale contamination

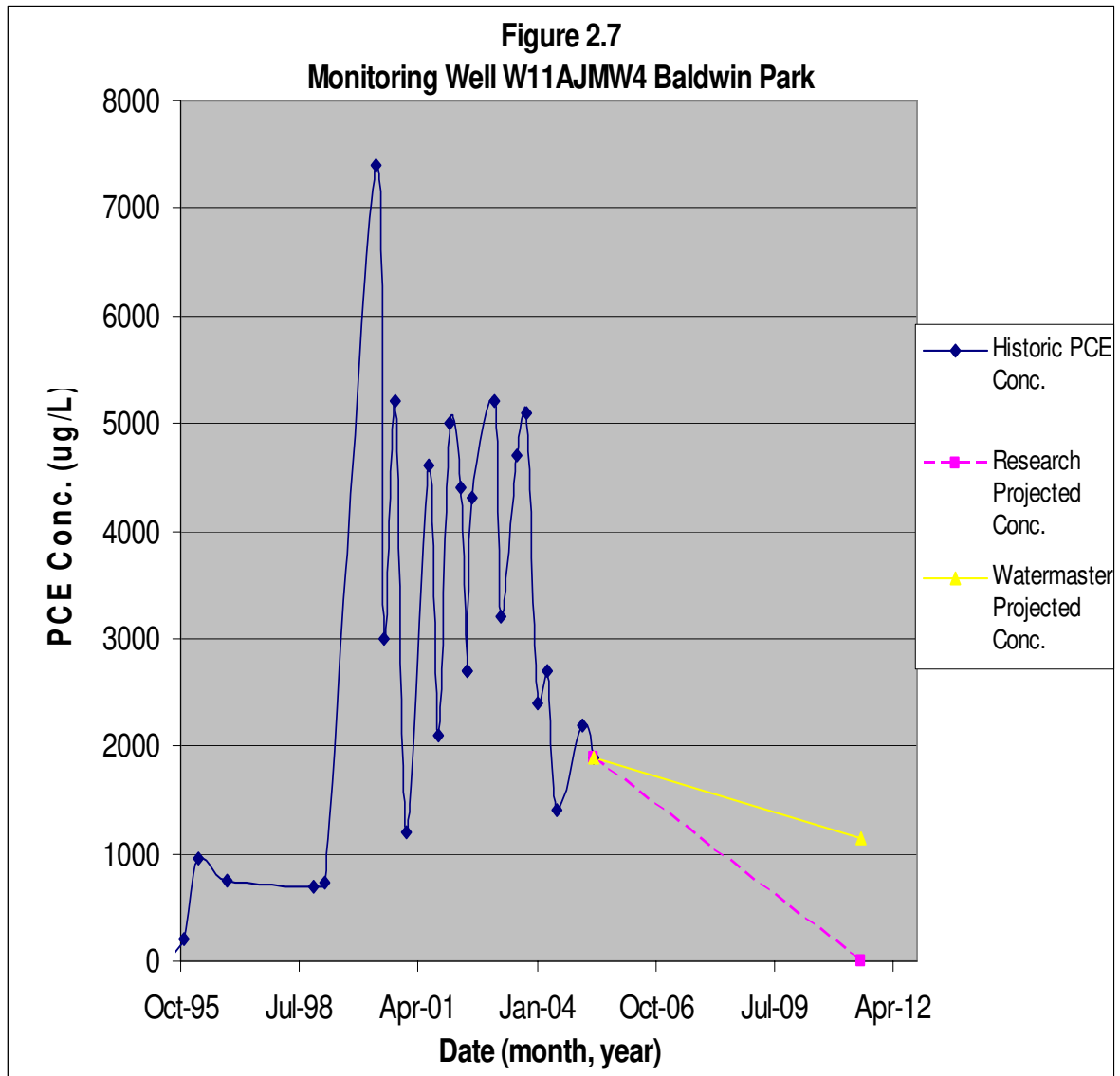
cleanup such as in BPOU is ineffective for overall basin cleanup.

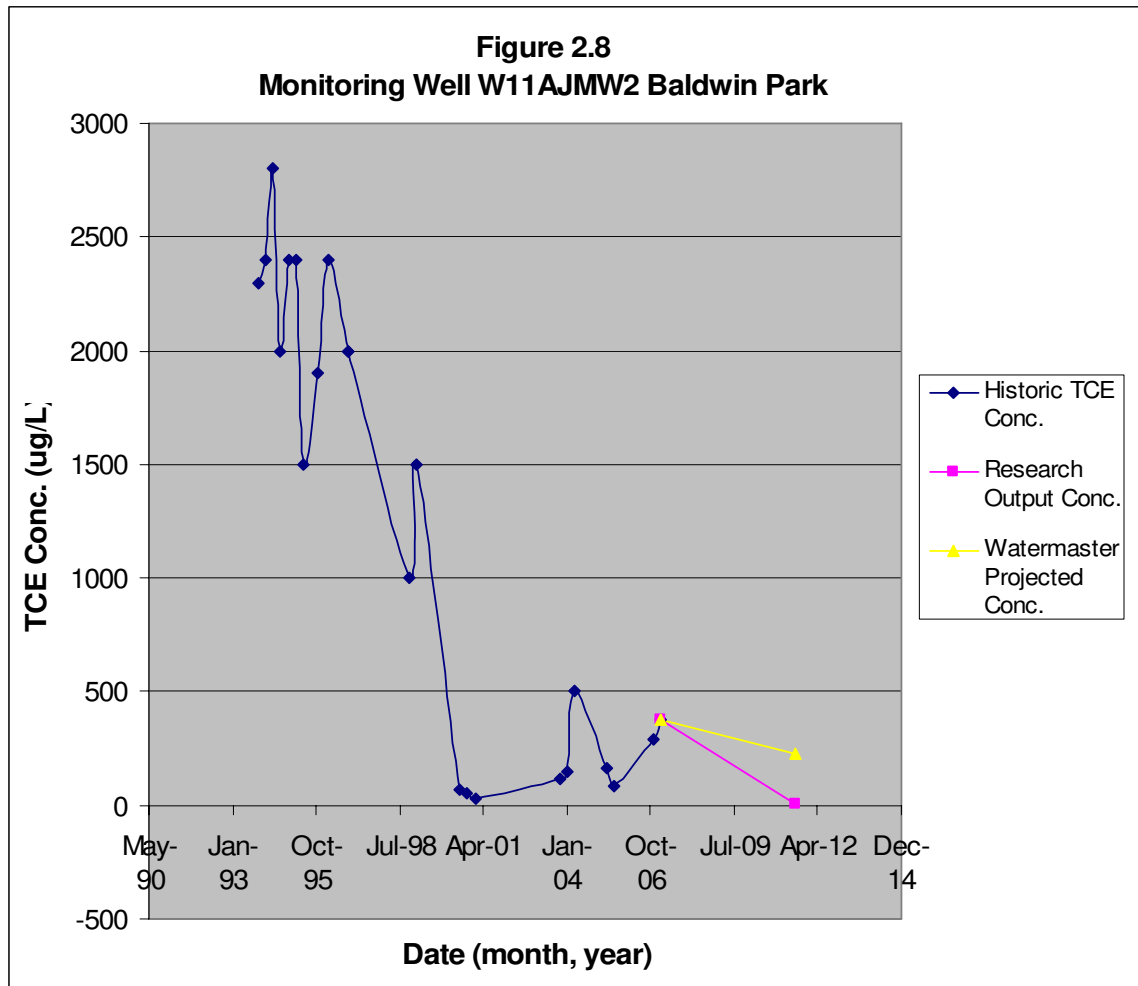
Watermaster projected current contaminant concentration values to predict an approximate 40% decrease in contaminant concentrations by the year 2011. Watermaster predicts that the VOC contaminant plume will decrease by approximately 40% by 2010-11 after full implementation of the BPOU remedy (all supplemental treatment facilities operational). Figures 2.4-2.10 show the VOC contaminant plume based on model output in this research. The methodology employed in this research will clean the VOC contamination below MCL by 2010-11.

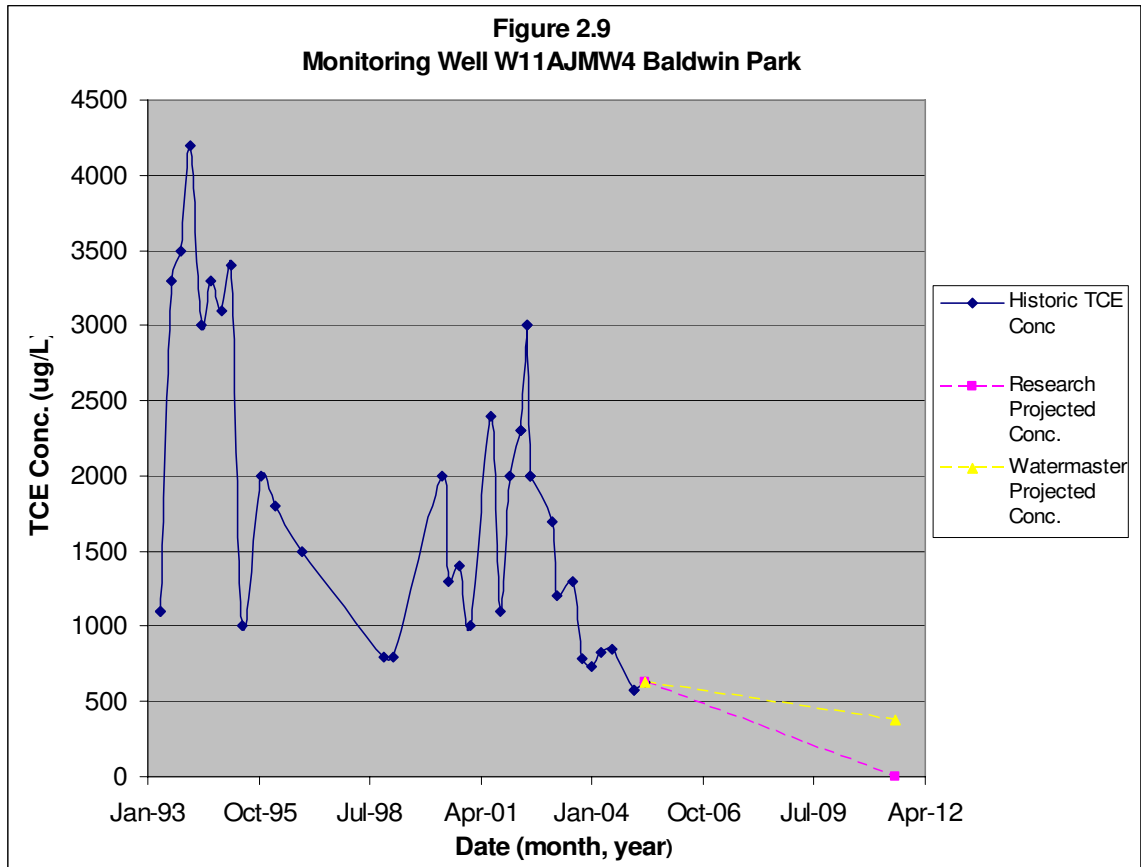


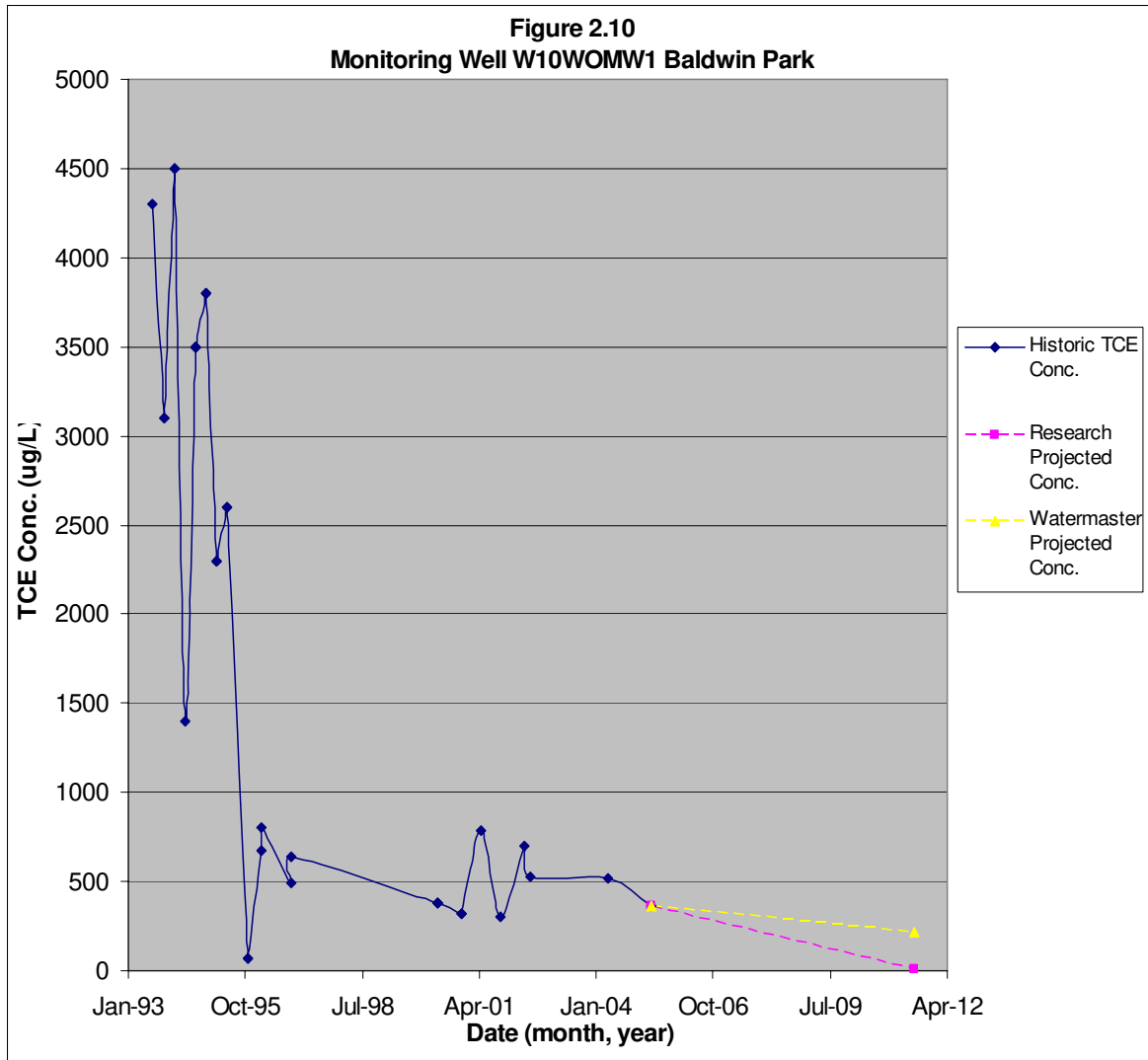








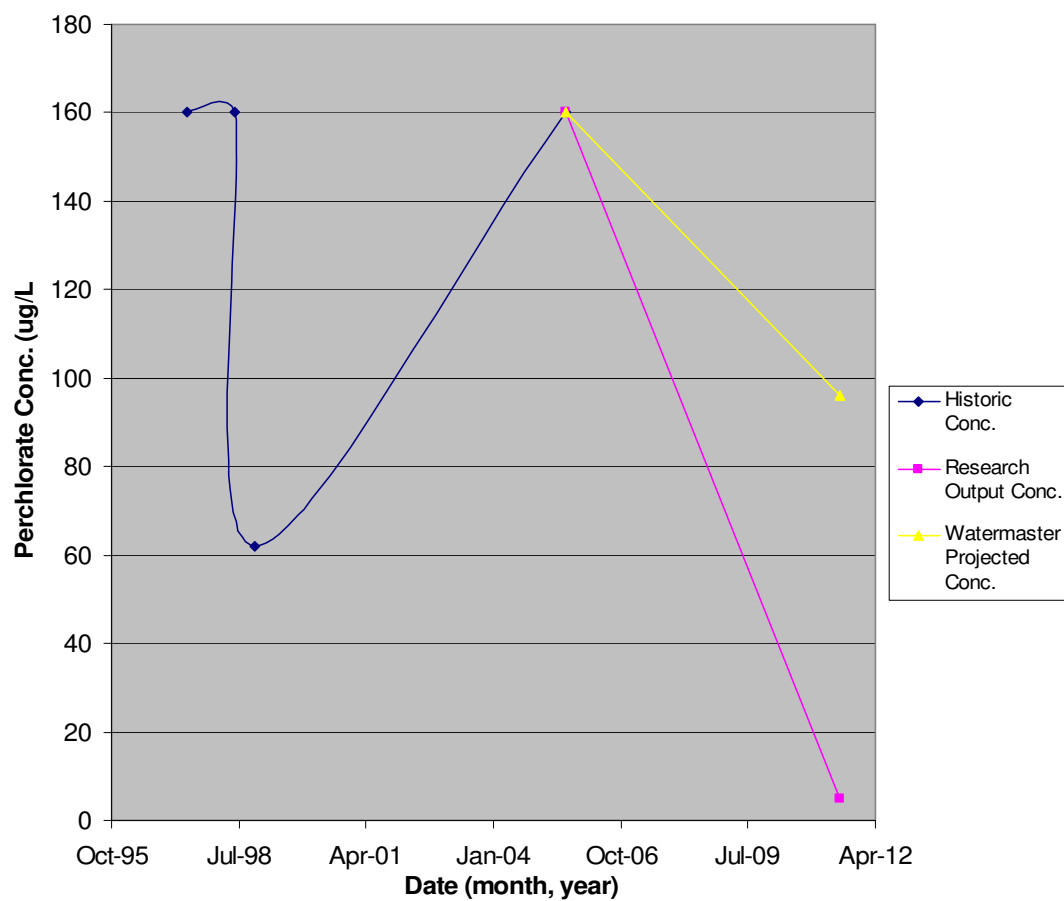


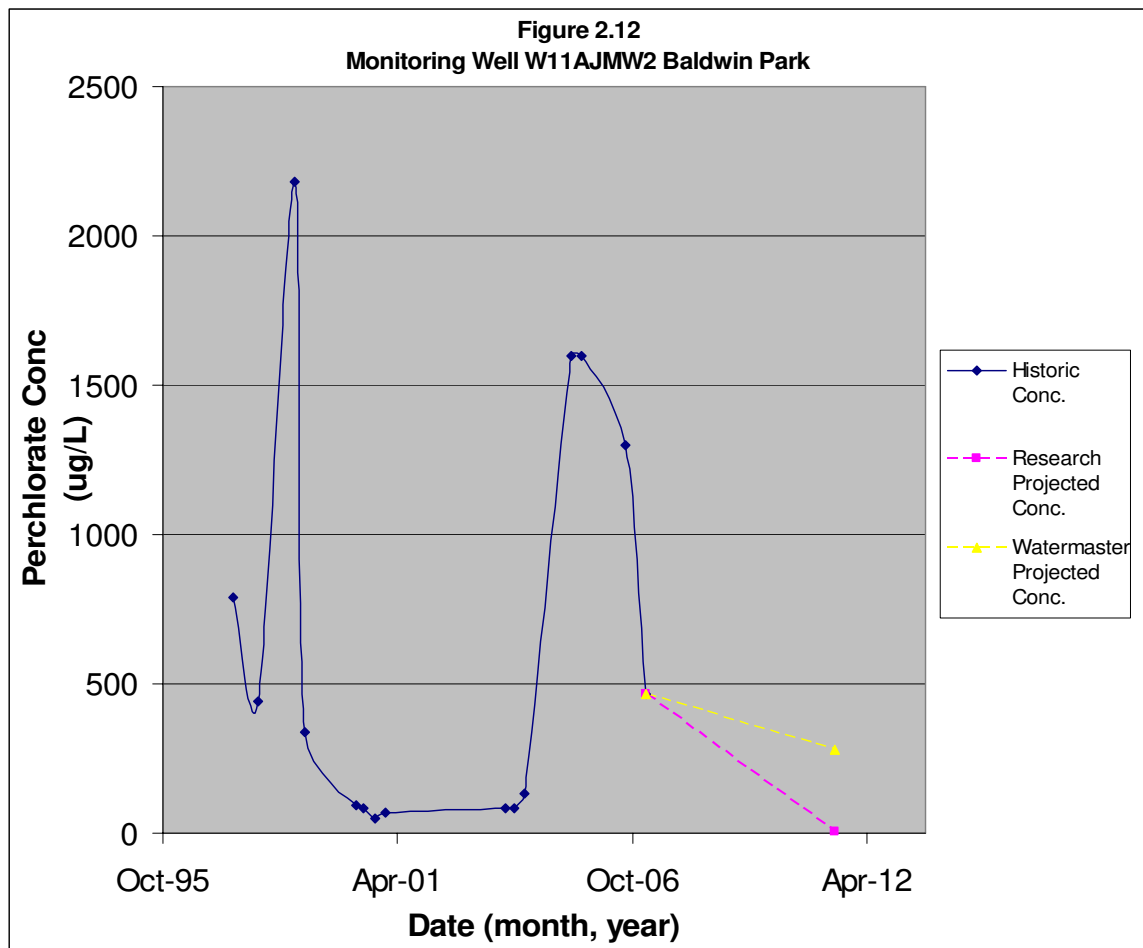


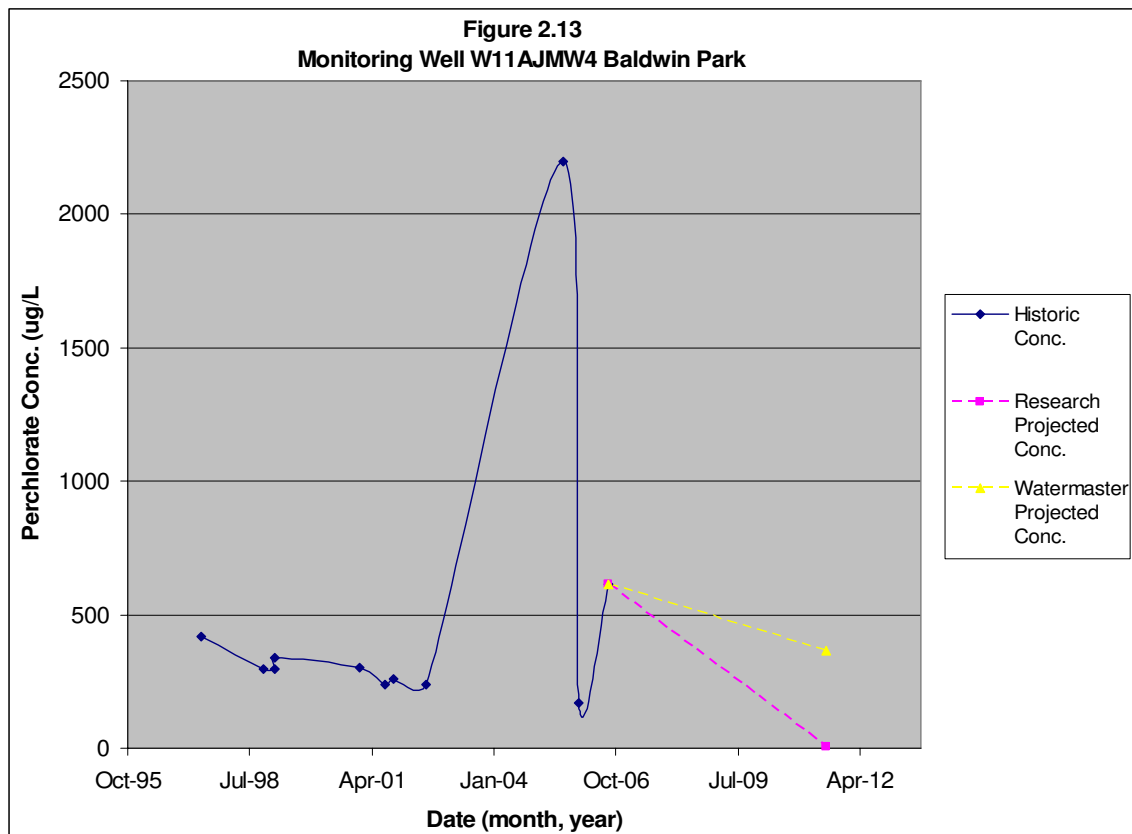
Similarly, Watermaster predicts that the Perchlorate contaminant plume will decrease by approximately 40% by 2010-11 after full implementation of the BPOU remedy (all supplemental treatment facilities operational). Figures 2.11-2.13 show the predicted perchlorate contaminant plume based on model output in this research. The methodology employed in this research will clean the perchlorate contamination below Notification Level (NL) by 2010-11.

Historic PCE, TCE and PCR data obtained from USEPA database (USEPA, 2007) were used to construct the graphs. Research output concentrations were obtained from RT3D output computed in this study for BPOU. Watermaster projected concentration reduction of approximately 40% by 2010-2011 was obtained from the Five Year Plan Water Quality Plan (2005-06) by San Gabriel Valley Watermaster.

**Figure 2.11**  
**Monitoring Well W11AJMW1 Baldwin Park**







## **Chapter 3**

### **Literature Review**

Advances in computing have led to more accurate groundwater flow simulation and contaminant transport models. These models are used in a wide range of applications. They are used to predict system responses to external stresses and as analytic, interpretive and predictive tools for analyzing groundwater and contaminant transport systems. This study employs groundwater flow simulation, particle tracking, contaminant transport and linear optimization to manage the groundwater and surface water supplies under the prevailing condition of shrinking the PCE and TCE contaminant plume below the maximum contaminant level (MCL) of 5 µg/L and PCR below the California Department of Health Services (CDHS) advisory notification level of 6 µg/L.

Previous studies were performed by the United States Environmental Protection Agency (USEPA) – Record of Decisions (1994, 1999, 2002) and San Gabriel Basin Watermaster (2005-2006) annual report. A comprehensive geologic study of the San Gabriel Basin was performed by California Department of Water Resources (1966).

Several different water resources articles and journals were reviewed. In recent years, notable advances have taken place in groundwater research especially in the field of simulation with optimization. The methodology employed in this study was thoroughly researched. The literature review included studies of contaminant plume migration, groundwater flow and contaminant transport as well as management optimization techniques. Specifically, water resources planning and management models including

hydraulic gradient control schemes for groundwater reclamation sites were reviewed.

Hydraulic gradient control schemes that utilize pumping and recharge flow rates, velocity control and head parameters were studied in detail.

### **3.1 Groundwater Contamination**

Groundwater degrades when excessive pumping or withdrawal is not balanced by artificial recharge or natural replenishment. PCE and TCE are halogenated aliphatic volatile organic compounds (VOCs) and have been widely used as ingredients in industrial cleaning solutions and as universal degreasing solvents due to their unique properties and solvent effects. Perchlorate (PCR) is an inorganic contamination that exists in the form of salts. Perchlorate is used industrially as solid propellants for rockets, missiles and fireworks. Perchlorate occurs naturally and is also a by-product of industrial or manufacturing spills. It is also used in the production of matches, flares, pyrotechnics, ordnance and explosives. PCE, TCE and PCR are found in groundwater due to industrial or manufacturing spills or waste. The spills then form a contaminant plume in groundwater. PCE and TCE migrate at the advective velocity of water with retardation factored in the plume movement. PCR is a conservative tracer and moves at bulk velocity of groundwater. Pumping shrinks the contaminant plume. The contaminant plume is redirected toward the pumping depression when an efficient hydraulic gradient scheme is developed.

Groundwater contaminant movement has been simulated through the use of distributed parameter models that utilize groundwater flow equation and the advection-dispersion

equation (Bear, 1979). Groundwater contaminant transport generally involves advection, diffusion, dispersion, adsorption, biological, decay and chemical reactions (Bedient et al, 1994). Literature reviews have been limited to advective transport whereby the contaminant plume moves approximately with the groundwater seepage velocity whilst incorporating retardation. When advective transport dominates, dispersion is assumed small and therefore negligible. The stability of a finite difference scheme should be evaluated in choosing the proper finite difference scheme for flow and contaminant transport.

### **3.2 Groundwater Management Models**

Groundwater should be efficiently managed to meet supply, storage and quality demands. State-of-the art advances in computer and numerical simulations have greatly improved analytic capabilities. Management should proficiently utilize advanced groundwater flow simulation, mass transport and optimization techniques. The water manager can address multi-objective management problems that are linear, non-linear, convex or robust. Continuous variables, discrete variables or mixed integer programming problems can be formulated. Ahfeld and Mulligan (2000) developed linear, binary and non-linear optimization formulations that can be coupled with groundwater simulation models.

Several aquifer reclamation methods have been documented in literature. Tiedman and Gorelick (1993) developed a nonlinear chance-constrained model groundwater management model that identified minimum pumping strategies to contain a vinyl chloride plume at a site in southwest Michigan. Reichard et al (2003) evaluated the

geohydrology and geochemistry of groundwater in the West Coast Basin, Los Angeles County, California. They linked a transient simulation model to the optimization model to identify the least cost strategies for improving hydraulic control of seawater intrusion in the West Coast Basin by means of increased injection (barrier wells) and (or) in-lieu delivery of surface water. Farrar et al. (2006) prepared a geohydrologic characterization, water chemistry and groundwater flow simulation of the Sonoma Valley Area, Sonoma County, California. They quantified historical changes in the groundwater system, prepared a groundwater simulation model and interpreted surface geophysical data that defined the geometry of the groundwater system. Lehr and Nielsen (1982) presented three categories for remedial alternatives; (a) physical containment where physical systems such as slurry trench, cut off walls, grout curtains will be put to prevent the flow of contaminated groundwater, (b) in situ plume interception and aquifer rehabilitation where action involving injection and extraction are supplemented by chemical treatment, and (c) withdrawal followed by treatment and use. There has been extensive research in the management of groundwater systems using simulation and optimization models. Gorelick (1983) classified groundwater management into two types of models; hydraulic management models and policy evaluation and allocation models. The distinguishing factor between these two is the basis on which management makes decisions. The first model focuses on managing groundwater decision variables such as hydraulic heads, pumping and recharge rates and location of wells. Aquifer hydraulics is the primary focus but economic considerations may be incorporated. The second model focuses on the economics of water allocation and policy evaluation. This model examines complex economic interactions that reflect operational or institutional decisions on water

allocation and the resulting economic impact. The key objective in both models is cost. Both models utilized groundwater flow and contaminant transport simulations.

In the field of groundwater remediation, several multi-objective management models have been developed. Ahfeld and Heidari (1994) applied an optimal hydraulic gradient control scheme to a groundwater system in coastal New Jersey. Their hydraulic gradient control scheme developed a capture zone that contracted the contaminant plume in both the horizontal and vertical directions. They imposed constraints on pairs of horizontally and vertically separated points on the perimeter of the plume. Decision makers can utilize their optimization model to determine feasible solutions and analyze trade offs caused by relaxing constraints necessary to obtain feasible operating strategies. Atwood and Gorelick (1985) developed a contaminant removal system that assumed extraction well location and rate in the interior of the plume. They employed linear response matrices that selected pump locations and rates outside the contaminated region while satisfying constraints that guaranteed plume velocities point inward. Lefkoff and Gorelick (1985) in extension to a previous study, added a feature of selecting pumping rate at the interior extraction well to their model.

Coupling simulation with optimization may be accomplished by formulating the objective and constraints as functions of decision variables and simulation model output (Ahfeld and Mulligan, 2000). Simulation with optimization finds wide application in groundwater management models. Linking the simulation model to the optimization model is achieved by making the simulation model a constraint in the management

model. In 1983, Gorelick coupled simulation with optimization using: (a) embedding approach where approximations of the equations are treated as constraints in the set of linear or nonlinear optimization. This method guarantees that the solution to the optimization problem determines the entire state of the system and the decision variables; (b) response matrix method in which the groundwater simulation model outputs unit responses that describe the pulse stimulus from changes in hydraulic heads at specified locations. The responses form an influence coefficient matrix. This method only applies to linear systems. The responses are additive and are superimposed. Superposition may also be utilized for transient groundwater flow provided the governing equations and boundary conditions are linear. An optimization problem is nonlinear if the objective function or constraints are nonlinear functions of the decision variables or the state variables.

### **3.3 Model Selection**

A flow model is a three - dimensional depiction of flow in an aquifer system.

Formulating an accurate model requires a good understanding of the physical system properties, constraints, boundary conditions and existing data. Strong formulation of the objective function and constraints is required to clearly depict the management model and obtain a feasible solution. Ahfeld and Mulligan (2000) stated that improper formulations can lead to infeasible solutions or solutions that are physically meaningless. The stability of partial differential equations and approximation errors are important to model selection. The error depends on the type of numerical method used, time and spatial discretization. Anderson & Woesner, (1992) reiterated that a good model maintains a

balance between round off and truncation errors and utilizes an appropriate time step. Sun (1995) postulated model selection criteria. He stressed the need for careful evaluation of the purpose of model, the quantity and quality of available data and degree of computational effort. Tradeoffs may exist between different models. A pure advective transport model requires less computational effort than a method of characteristics (MOC) model but the accuracy of the pure advective transport model is relatively compromised. Model selection requires prior knowledge of the goals of the model, required level of sophistication, level of confidence in the available field data and clear understanding of the output. Distributed parameters such as porosity, hydraulic conductivity, dispersion coefficient and specific yield are required to develop a distributed parameter model.

Transient groundwater management models have also been developed. Ahfeld and Mulligan (2000) studied transient models that coupled optimization with simulation. Chang et al (1992) used dynamic programming to determine the benefits of transient groundwater pumping policies when a decision is made at each time step of the simulation. Georgakakos and Vlatza (1991) investigated optimal transient groundwater management strategies for stochastic models.

### **3.4 Hydraulic Gradient Control**

Fowler et. al. (2004) developed mathematical hydraulic capture methods for remediation attempt to control the direction of groundwater movement in order to manipulate the migration of a contaminant plume. They presented numerical results for two hydraulic

capture models, flow based hydraulic control and transport based concentration control using the implicit filtering algorithm. Lefkoff and Gorelick (1985) developed a hydraulic gradient control system based on the minimum velocity required to move a pollutant particle from a point on periphery of the contaminant plume to the extraction well. The computed velocities were used to calculate the target hydraulic head gradients. Their model generated coefficients of the response matrix using unit-response functions.

This study involves a practical and systematic development of a hydraulic gradient control scheme in which locations and rates of wells are varied subject to given constraints to effectively stabilize, capture and clean up the contaminant plume. The optimized hydraulic gradient control methodology developed in this study physically contains, shrinks and cleans up the contaminant plume below the MCL. The methodology utilized in this study is based on iterative transient groundwater simulation, particle tracking, contaminant transport and optimization.

The hydraulic gradient control methodology proposed in this study is made up of recharge and pumping wells contained within the contaminant plume. Recharge and pumping wells are meticulously placed to induce hydraulic gradient toward the core of the plume. Flow reversal stabilizes the contaminant plume, creates inward gradient and effects plume clean up.

Potential recharge and extraction wells are selected from existing or proposed well locations. The contaminant plume transport is simulated to approximate plume

boundary. Cost effective optimal contaminant removal can be achieved using existing wells. The locations of the wells that most effectively clean the contaminant plume are selected. Recharge and extraction wells are efficiently located in order to effectively clean up the plume. Management should employ a high level of engineering ingenuity by carefully studying the flow and contaminant transport regimes in selecting well locations. Wells can be switched from recharge to extraction at the end of the cleanup of the contaminant plume.

# **Geohydrology**

## **4.1 Geologic Settings**

The Baldwin Park Operable Unit (BPOU) lies south of the San Gabriel Mountains, east of the 605 Freeway, north of the 10 Freeway and west of Azusa Avenue. The Baldwin Park area is fully developed and has a mixture of residential, commercial and industrial facilities. The region has large parcels of open land with active and inactive gravel pits and the Santa Fe Flood control basin.

Major hydrogeologic structures include the topographic highs (i.e., San Gabriel Mountains and southern hills) and topographic lows (i.e., Whittier Narrows).

Groundwater flow in the basin is potentially impacted by four major faults: the Sierra Madre Fault System, the Raymond Fault, the Lone Hill-Way Hill Fault, and the Workman Hill Fault.

## **4.2 Geologic Units**

BPOU lies approximately 10 miles south of the San Gabriel Mountains. Alluvial deposits are predominant in the region and are unconsolidated to partially unconsolidated non-marine sediments of Recent and Pleistocene age. These sediments were deposited by fluvial and geomorphic processes resulting from San Gabriel River and its tributaries. Massive gravel pits exist in the northern and central portions of Baldwin Park. Lithologic evaluation of well logs reveal that gravel depths exceed 500 ft in the north mixed with 10 to 30 feet of clay in the south. The layers of clay are interbedded but discontinuous and

are not sufficiently long to qualify as a model layer. Therefore the aquifer has been analyzed as an unconfined aquifer in all previous studies. Alluvial sediments range from a few hundred feet in the north to approximately 2000 ft in the south.

The basin is underlain by two distinct sources of sediment in the basin; the coarse-grained crystalline rocks of the San Gabriel Mountains and the finer-grained sedimentary rocks of the hills to the southeast and southwest. Alluvial sediment derived from the northern San Gabriel Mountains is generally coarser than that from the hills to the south. As such, the hydraulic conductivity of the alluvium generally increases with proximity to the northern San Gabriel Mountains. Sediment distribution is directly related to the position relative to river and tributary courses. Coarse-grained sediments are predominant in the San Gabriel River proximity. The San Gabriel Basin geology is characterized by interfingering lenses of alluvial deposits (e.g., cobbles, gravel, silt, and clay) that exhibit a high degree of variability in sediment type, both vertically and laterally. Figure 4.1 shows a sample well drill log plotted from the data in the USEPA database (2008). Lithologic layer depths are in feet.

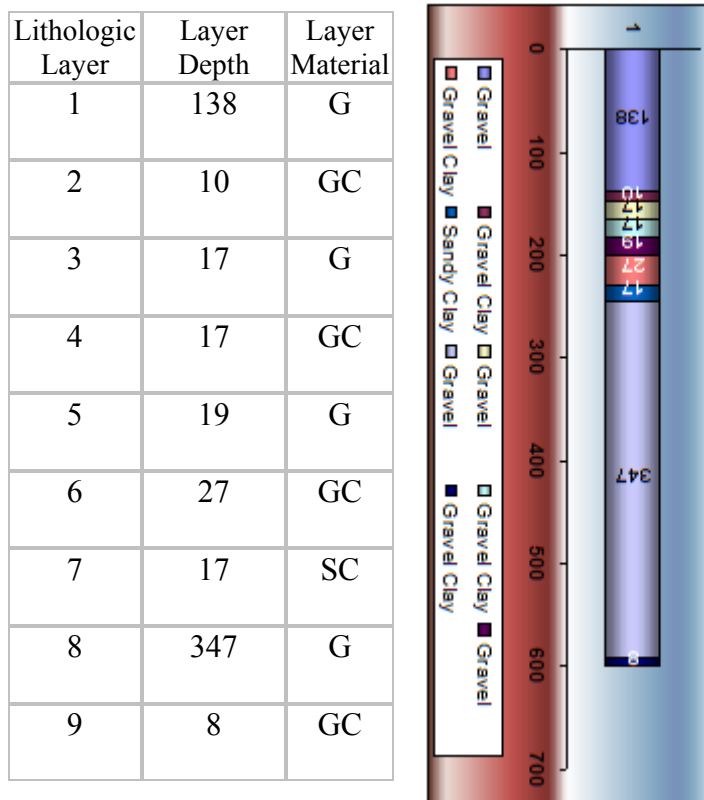


Figure 4.1 Well Lithology

(Well No. 01900035, State Well NO. 1S/10W-17N01)

The area north of the Route 10 freeway is predominantly sand, gravel, gravel sand, sandy gravel interbedded with sandy clay and gravel clay. The area south of Route 10 is interbedded with layers of clay and silt.

### 4.3 Aquifer Systems

Two aquifer systems (quaternary and tertiary) exist in the San Gabriel Valley. Table 4.1 shows the aquifer systems (per DWR 1966) in San Gabriel Valley. The Baldwin Park Operable Unit lies in the Quaternary Aquifer system which is made up of recent, upper

Pleistocene and lower Pleistocene series. The recent series is made of recent alluvium (0-100) ft. The upper Pleistocene is made of older alluvium (0-4000) ft. The lower Pleistocene is made of the San Pedro formation (0-2000) ft.

Table 4.1 Aquifer Systems

| <b>System</b> | <b>Series</b>     | <b>Formation</b>            | <b>Lithology</b>                                  | <b>Maximum Thickness (feet)</b> |
|---------------|-------------------|-----------------------------|---------------------------------------------------|---------------------------------|
| Quaternary    | Recent            | Recent Alluvium             | Sand, gravel, silt                                | 0-100 +                         |
|               | Upper Pleistocene | Older Alluvium              | Sand, gravel, silt, shale, clay                   | 0-4000±                         |
|               | Lower Pleistocene | San Pedro Formation         | Marine/non-marine:sand, gravel, silt, shale, clay | 0-2000+                         |
| Tertiary      | Upper Pliocene    | Upper Member-Pico Formation | Marine/non-marine:sand, gravel, silt, shale, clay | 300±                            |

### 3.4 Model Layers

The unconfined aquifer was divided into 3 layers. Layer 1 extends from the ground surface to 266.67 ft below Mean Sea level (MSL). Layer 2 extends from 266.67 below MSL to 1233.33 ft below MSL. Layer 3 extends from 1233.33 ft below MSL to the base

of the aquifer. Cross sections from several lithologic wells are used to determine the composition of the layers (Table 4.2). Cross Sections ABC and CD shown on the plan view in Figure 4.2 are shown in profile view in Figures 4.3 and 4.4.

Table 4.2 Lithologic Layers

| WELL_ID  | LITH_NUM | THICKNESS | THICK_UNIT | MATERIAL |
|----------|----------|-----------|------------|----------|
| 01900028 | 1        | 376       | FT         | G        |
| 01900029 | 1        | 470       | FT         | G        |
| 01900029 | 2        | 40        | FT         | SC       |
| 01900029 | 3        | 105       | FT         | G        |
| 01900031 | 1        | 119       | FT         | G        |
| 01900031 | 2        | 60        | FT         | G        |
| 01900031 | 3        | 316       | FT         | G        |
| 01900031 | 4        | 104       | FT         | G        |
| 01900035 | 1        | 138       | FT         | G        |
| 01900035 | 2        | 10        | FT         | GC       |
| 01900035 | 3        | 17        | FT         | G        |
| 01900035 | 4        | 17        | FT         | GC       |
| 01900035 | 5        | 19        | FT         | G        |
| 01900035 | 6        | 27        | FT         | GC       |
| 01900035 | 7        | 17        | FT         | SC       |
| 01900035 | 8        | 347       | FT         | G        |
| 01900035 | 9        | 8         | FT         | GC       |

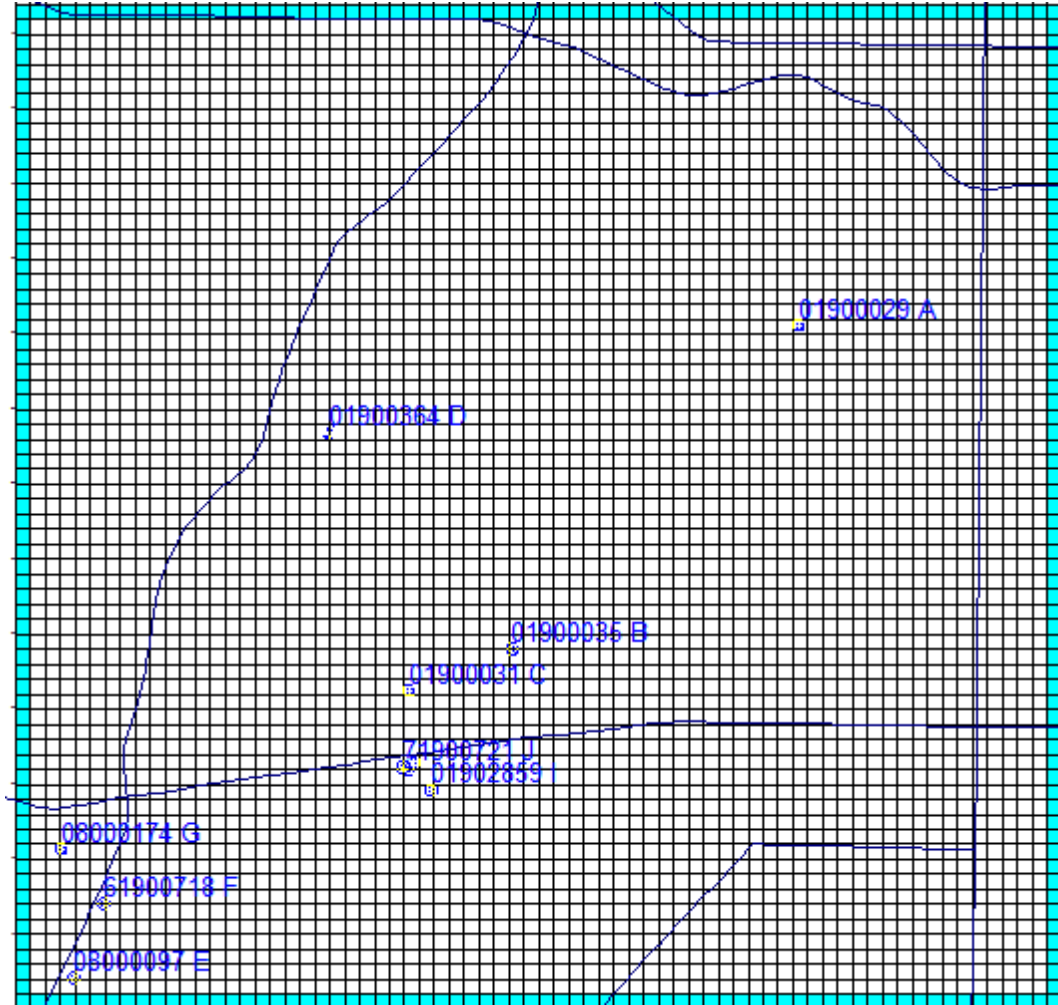
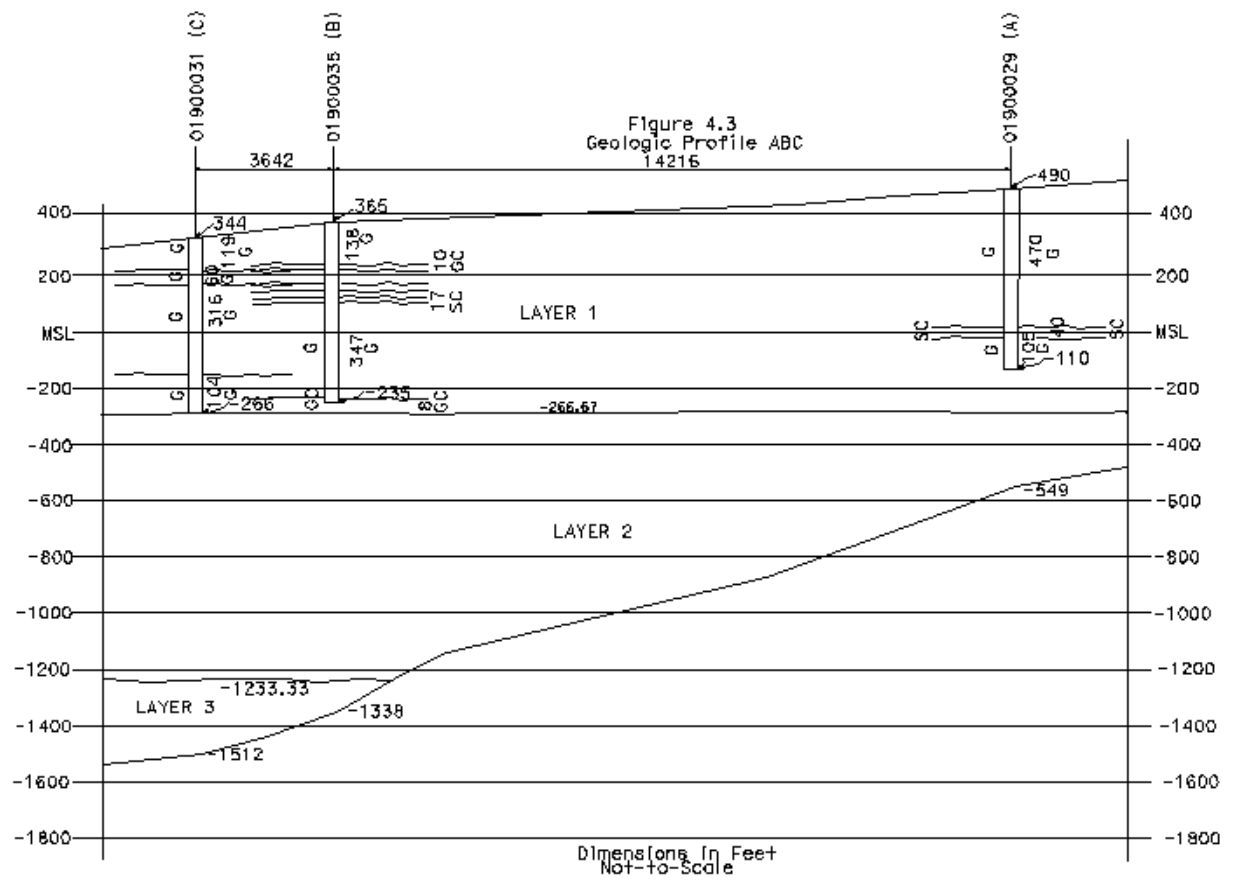
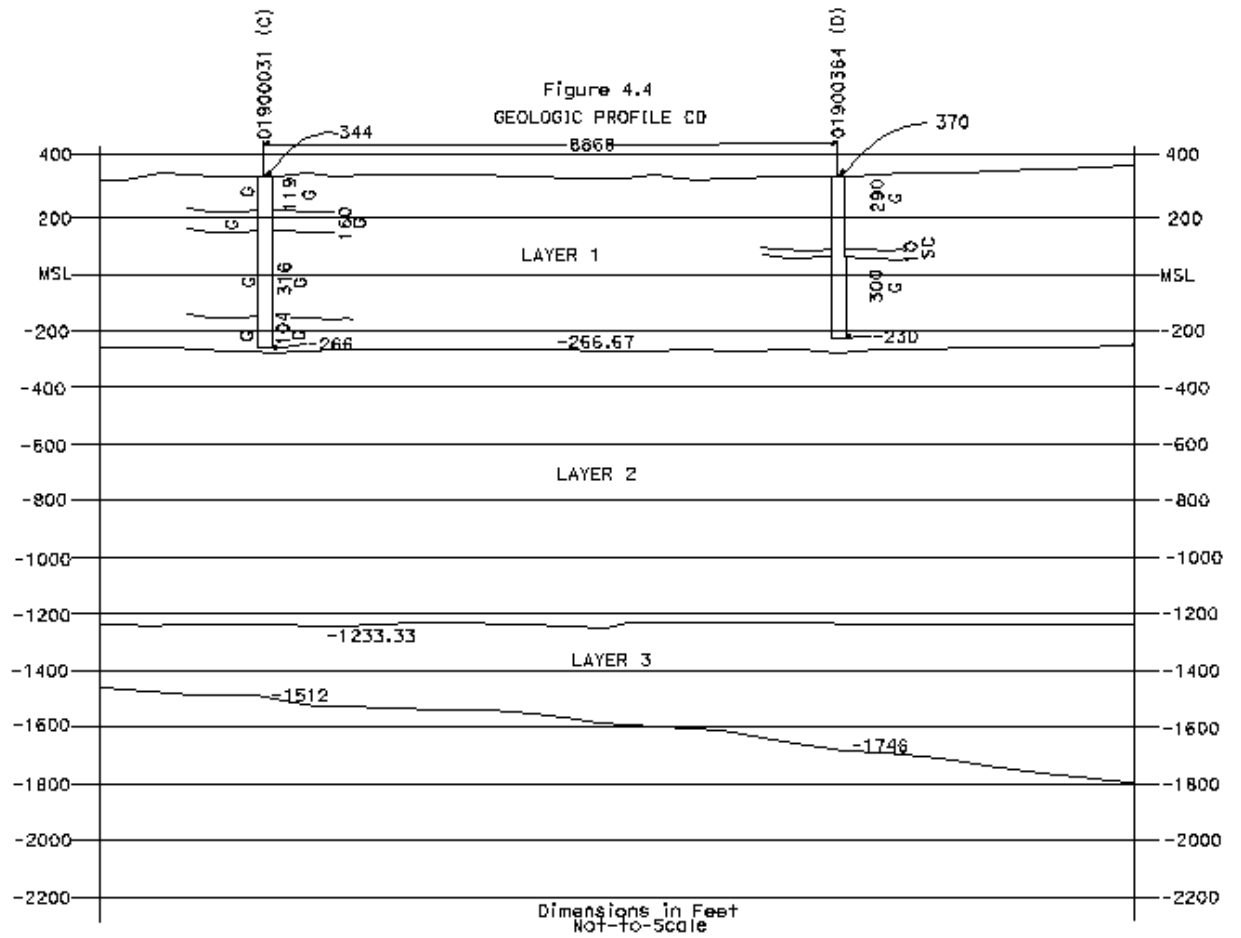


Figure 4.2 Plan View - Cross Sections ABC and CD





#### **4.5 Groundwater Elevations, Flow Directions and Seepage Velocities**

Groundwater elevations vary from 175 ft to 300 ft. Groundwater flow direction is southwest per existing groundwater contours from San Gabriel Valley Watermaster, previous studies and the output (flow model contour map) of this study. Groundwater velocities of between 40 and 120 feet/year were estimated using average gradient and hydraulic conductivity. The slower velocities are in the eastern portion of the sub-basin where flow directions vary considerably.

The groundwater level in the Baldwin Park Key Well is used by the Main San Gabriel Basin Watermaster to monitor changes in groundwater supply for the basin. The Key Well located in Baldwin Park is the benchmark for determining groundwater levels for the entire basin. Figure 2.3 shows Key Well groundwater elevations from 1986 to 2006. The Key Well water level fell to a historic low of 195.5 feet above mean sea level on December 23, 2004. Heavy rainfall in the winter of 2005 recharged the basin and elevated the Key Well water level to 251 feet in early June 2005. Watermaster strives to maintain the Key Well water level between 200 feet and 250 feet above mean sea level through proficient basin management practices. Moderate rainfall in 2005-06, continued water capture, and significant untreated imported water deliveries for groundwater recharge helped maintain a high water elevation of 250.6 feet at the Baldwin Park Key Well on June 30, 2006.

Groundwater levels generally follow topographic slope, with groundwater flow from the edges of the basin toward the center of the basin, then southwest to exit through the Whittier Narrows (DWR 1966) which is a structural and topographic low. Extraction patterns of groundwater can alter this general flow pattern by creating local depressions in the water table.

#### **4.6 Recharge and Discharge**

Recharge of the basin is mainly from direct percolation of precipitation and percolation of stream flow. Annual precipitation in the BPOU is shown in Figure 2.2. Rainfall data was obtained from the Main San Gabriel Basin Watermaster. A small portion of San Gabriel River passes through the basin. The small portion of San Gabriel River was not included in the model because the river is dry 90% of the year. Most of the San Gabriel River is unlined. Thus, runoff in the San Gabriel River percolates into the aquifer. The basin is also recharged by imported water. Groundwater flows southwest and discharges in Whittier narrows. Groundwater is discharged through several pumping wells in BPOU, treated and supplied to customers in the basin.

## **Chapter 5**

### **Proposed Methodology**

This research develops a groundwater flow model, particle-tracking model, contaminant transport model coupled with an optimization model to effectively remove PCE, TCE and PCR in BPOU in the San Gabriel basin. This research extends the previous works of scholars mentioned in the literature review and addresses areas of further research.

In this research, groundwater simulation, particle tracking and contaminant transport models are run and the results are embedded into a linear optimization model. Pumping and recharge rates are decision variables determined in both simulation and optimization models. The methodology proposed in this research, is based on iterative groundwater simulation, particle tracking, contaminant transport and optimization. Results from the groundwater simulation, particle tracking, and contaminant transport models are fed into the optimization model. The output from the optimization model is deemed truly optimal when they satisfy the water quality constraint in the contaminant transport model. The water quality constraint is the maximum contaminant level (MCL) of PCE and TCE or notification level (NL) of PCR.

The research utilizes an existing Groundwater Vistas Version 4 (Rumbaugh et al., 1996) to perform the groundwater simulation. The linear optimization is performed by writing a computer code in LINDO (Schrage, 1997). Contaminant plume migration and cleanup is simulated using a groundwater flow model, particle tracking model, contaminant transport model and an optimization model. The flow model is developed using data

from USEPA database (2008).

## **5.1 Computer Programs**

Groundwater Vistas Version 4 solves the groundwater flow equation and creates the flow model. The groundwater flow model outputs head contours. Groundwater Vistas also has a particle tracking module, MODPATH and a contaminant transport module, RT3D.

LINDO (Schrage, 1997) solves the linear optimization problem. LINDO is well designed to solve large-scale linear programs. The management model is formulated and solved using LINDO.

## **5.2 Conceptual Model**

Model objectives and properties should be clearly defined before a conceptual model can be formulated. Conceptual model formulation of the system helps to clearly depict the physical system. The model formulation includes identifying and defining the physical components of the system such as water sources, users, flow direction, recharge and production wells. The hydrologic budget estimates water flowing in and out of the system as well as the amount retained. The hydrologic budget is prepared using hydrologic information such as precipitation, runoff and measurement of recharge and discharge for select locations. Research data required to develop the flow model include:

- geophysical logs, driller's logs, any published depth to bedrock contours to develop model layers;

- pumping test data, Southern California Edison Test Data to determine aquifer parameters;
- historical water levels to determine groundwater flow direction;
- historical groundwater pumping, artificial recharge, return flows from water uses etc to determine recharge and discharge terms.

Hydrogeologic maps define the water table and potentiometer surfaces of the aquifer, hydraulic conductivity and transmissivity distribution, and storage properties of aquifer. Data for the spatial and temporal distribution of evapotranspiration, groundwater recharge, surface water-groundwater interaction and natural groundwater discharge are obtained from monitoring studies (USEPA, 2008).

In this study, a three-dimensional geometric and hydrogeologic representation of the region was modeled. The aquifer is assumed unconfined per Department of Water Resources (1966). Flow is essentially horizontal in unconfined aquifers per Dupuit's assumption. Figure 5.1 shows the vertical cross section of the conceptual unconfined aquifer.

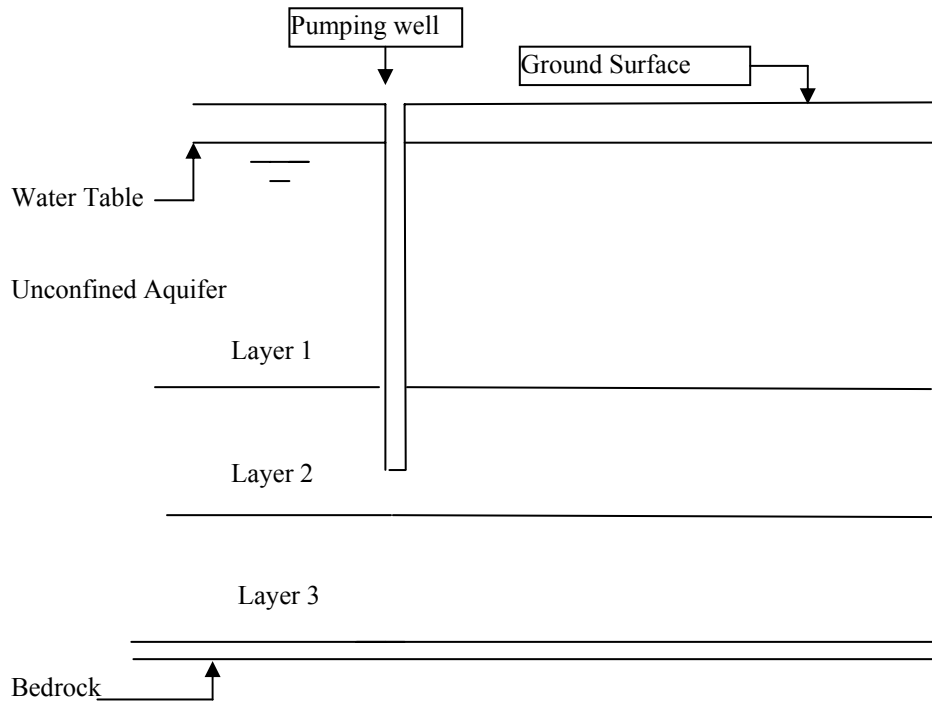


Figure 5.1 – Conceptual Unconfined Aquifer

The conceptual model is defined using basin stratigraphy, geometry and aquifer parameters. Initial and boundary conditions, inflow and outflow sources such as recharge and extraction wells, irrigation return flows, rainfall and subsurface under flow are also input in the flow model.

Aquifer parameters include hydraulic conductivity, transmissivity, specific storage coefficient, effective porosity and depth of unconfined layers. No flow, constant head, general head and flux boundary conditions are defined.

In this study, the PCE and TCE plume is assumed to move at the seepage velocity of groundwater whilst factoring retardation. PCR is a conservative tracer and moves at the seepage velocity of the groundwater. The assumptions outlined below are embedded in

the model.

### **5.3 Model Assumptions**

This study develops a linear optimization model that manages PCE, TCE and PCR in BPOU. The entrapped contaminant plume spreads in the direction of groundwater flow. The contaminant plume can be induced to move inwards to the epicenter of the plume by using a hydraulic gradient scheme that includes injection wells and inner production wells.

The groundwater system properties were obtained from DWR report bulletin 104 and data from USEPA database for San Gabriel Valley (USEPA, 2008). When parameter uncertainty arose, several simplifying assumptions outlined below were used to ensure reasonable parameter estimation.

- The Dupuit Forcheimer assumption applies to flow in the aquifer. Within a layer, the vertical velocity component is assumed very small compared to the horizontal velocity component. Therefore, flow is strictly horizontal with no vertical head gradient. Streamlines are assumed horizontal and are associated with velocities, which are proportional to the slope of the hydraulic gradient and independent of the depth of flow.
- The aquifer is made up of heterogeneous layers with varying thickness.
- Storage changes are instantaneous with changes in head.
- The contaminant plume moves in the direction of groundwater flow at a speed

dictated by the retardation factor. Contaminant flow is simulated using pure advection. Dispersion and chemical reaction are assumed insignificant.

- The heads at the injection wells and production wells are designed to induce effective hydraulic gradients.

## **5.4 Mass Transport Models**

Many types of mathematical models have been used to represent complex aquifer systems. Partial differential equations describe the physical and chemical characteristics of the system including initial and boundary conditions. Mass transport in groundwater flow may be divided into two categories: pure advective model where mass transport is governed by advective groundwater flow and advective-dispersive model where hydrodynamic dispersion equations are used to represent mass transport phenomena in groundwater flow.

Groundwater Vistas has several particle tracking modules; MODPATH, PATH3D etc. Particle tracking delineates the movement of contaminant particles in the flow field tracing flow paths and path lines. MODPATH, a subroutine of MODFLOW computes three dimensional flow paths using output from Groundwater Vistas. MODPATH utilizes head distribution from flow models to track contaminant paths and enhance flow visualization. Thus the accuracy of computed flow paths depends on the accuracy of the head distribution. Interpolation transforms a discrete head distribution to a continuous spatial domain. The contaminant plume migrates in the continuous domain by advection at the seepage velocity of groundwater with retardation factored into the flow.

### 5.4.1 RT3D

RT3D is used for contaminant transport modeling of PCE, TCE and PCR in this study.

The program is a multi-specie reactive transport modeling software included as a module of Groundwater Vistas. RT3D was created by Dr. Prabhakar Clement, Department of Civil Engineering, Auburn University, Auburn, AL. The program is currently customized and marketed by Battelle, Richland, WA.

RT3D has several graphical interfaces and is compatible with several groundwater modeling software. RT3D includes 7 pre-programmed modules including:

1. Tracer Transport
2. Two Species Instantaneous Reaction (BIOPLUME-II type reactions; e.g., Hydrocarbon & Oxygen)
3. Six Species, First-Order, Rate-Limited, BTEX Degradation using Sequential Electron Acceptors (e.g.,  $O_2$ ,  $NO_3^-$ ,  $Fe^{2+}$ ,  $SO_4^{2-}$ ,  $CO_2$ )
4. Rate-Limited Sorption
5. Double Monod Model
6. Sequential First-Order Decay (up to 4 species, e.g., PCE/TCE/DCE/VC)
7. Aerobic/Anaerobic Chlorinated Ethene Dechlorination

The reaction package module 7, Sequential First-Order Decay PCE=>TCE=>DCE=>VC) is used to model the natural attenuation of PCE and TCE. The reaction package module 1, tracer transport is used to model the conservative tracer, PCR.

## 5.5 Numerical Simulation

The numerical model is transformed into a discretized domain of finite difference blocks in the final phase of model conceptualization. Discretizing the flow and contaminant transport model is the initial stage for numerical simulation. The discretization technique determines the accuracy of the numerical scheme. The stability of the numerical scheme should be investigated and ensured.

The conceptual flow model is transformed into partial differential equations and the associated initial and boundary conditions. The formulation of the governing partial differential equation for groundwater flow used in Groundwater Vistas is given in Equation 5.1. The governing equation for constant density groundwater flow in an anisotropic medium can be expressed as:

$$\frac{\partial}{\partial x} \left( K_{xx} \frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y} \left( K_{yy} \frac{\partial h}{\partial y} \right) + \frac{\partial}{\partial z} \left( K_{zz} \frac{\partial h}{\partial z} \right) + Q = S_s \frac{\partial h}{\partial t} \quad (5.1)$$

Where

$K_{xx}, K_{yy}, K_{zz}$  = values of hydraulic conductivity parallel to the x,y, and z coordinate axes (L/T);

$h$  = is the potentiometric head (L);

$Q$  = is a volumetric flux per unit volume and represents sources or sinks (1/T) with  $Q < 0.0$  for discharge or extraction and  $Q > 0.0$  for recharge;

$S_s$  = is the specific storage of the porous material (1/L);

$x,y,z$  = spatial coordinates (L); and

$t$  = time (T);

Equation (5.1) in conjunction with specified initial and boundary conditions depict the mathematical model of groundwater flow in a heterogeneous and anisotropic medium with the principal axes of hydraulic conductivity aligned with the coordinate directions. Bear (1979) or other groundwater hydrology books derive equation (5.1) from Darcy's equation and the continuity equation. Terms (1) through (3) in the left hand side represent the fluxes in the x, y, and z directions. The last term on the left hand side represents extraction or recharge. The term on the right hand side represents the elastic storage change in the volume of water stored in a given aquifer volume per unit change in head. Equation (5.1) can be solved for head at each location in space and time when sinks (or sources) and appropriate boundary and initial conditions are specified.

Groundwater Vistas uses an implicit (backward in time) finite difference method in which the groundwater flow system is divided into a grid of cells and time steps. Head and concentration are calculated at a single point in a cell called a node. Harbaugh and McDonald (1988) defined the groundwater flow finite difference equation as:

$$\begin{aligned}
 & R_{i,j-1/2,k}(h_{i,j-1,k}^m - h_{i,j,k}^m) + R_{i,j+1/2,k}(h_{i,j+1,k}^m - h_{i,j,k}^m) \\
 & + C_{i-1/2,j,k}(h_{i-1,j,k}^m - h_{i,j,k}^m) + C_{i+1/2,j,k}(h_{i+1,j,k}^m - h_{i,j,k}^m) \\
 & + V_{i,j,k-1/2}(h_{i,j,k-1}^m - h_{i,j,k}^m) + V_{i,j,k+1/2}(h_{i,j,k+1}^m - h_{i,j,k}^m) + M_{i,j,k} h_{i,j,k}^m + Q_{i,j,k} \\
 & = SS_{i,j,k}(WR_j \times WC_i \times TK_{i,j,k}) [(h_{i,j,k}^m - h_{i,j,k}^{m-1}) / (t^m - t^{m-1})]
 \end{aligned} \tag{5.2}$$

Where:

$h^m_{i,j,k}$  is head at cell  $i,j,k$  at time step  $m$  (L);

$V$ ,  $R$ ,  $C$  are hydraulic conductances, or branch conductances between nodes  $i,j,k$  and a neighboring node ( $L^2/T$ );

$M_{i,j,k}$  is the sum of coefficients of head from source and sink terms ( $L^2/T$ );

$Q_{i,j,k}$  is the sum of constants from source and sink terms, with  $Q_{i,j,k} < 0.0$  for extraction and  $Q_{i,j,k} > 0.0$  for recharge ( $L^3/T$ );

$SS_{i,j,k}$  is specific storage ( $L^{-1}$ );

$WR_j$  is a cell width of column  $j$  in all rows (L);

$WC_i$  is a cell width of row  $i$  in all columns (L);

$TK_{i,j,k}$  is the vertical thickness of cell  $i,j,k$  (L); and

$t^m$  is the time at time step  $m$  (T).

Subscript notation  $\frac{1}{2}$  is used to denote averaging of conductance between nodes.

Equation (5.2) solves for head at each node at each time step.

Groundwater problems are field specific and may deviate from assumptions made using the above equation. The variability and heterogeneity of hydrogeologic and field conditions affects the applicability and reliability of the model. Some presumptions made in this study may be modified to account for varying hydrogeologic and field conditions.

### 5.5.1 Governing Equation for Unconfined Aquifers

Hantush modified equation 5.1 to a transient 2-D flow in unconfined heterogeneous and anisotropic aquifer with horizontal bases. Equation 5.3 forms basis for spatial discretization to solve for head. Equation 5.3 is linear in  $h^2$  and assumes average head ( $h$ ) in a vertical section approximates water table. Hantush's modified equation for unconfined aquifers is derived by applying Darcy's law to an unconfined aquifer with saturated thickness  $h$  and utilizing the continuity equation. Hantush's modified equation is given by:

$$K_{xx} \frac{\partial^2 h^2}{\partial x^2} + K_{yy} \frac{\partial^2 h^2}{\partial y^2} + Q = \frac{\theta}{D} \frac{\partial h^2}{\partial t} \quad (5.3)$$

Where:

- $K_{xx}, K_{yy}$  = values of hydraulic conductivity along the x, and y coordinate axes, which are assumed to be parallel to the major axes of the hydraulic conductivity (L/T );
- $h$  = is average hydraulic head (L);
- $\theta$  = effective porosity;
- $D$  = average saturated thickness (L);
- $Q$  = is a volumetric flux per unit volume and represents sources or sinks (1/T ) with  $Q < 0.0$  for extraction and  $Q > 0.0$  for recharge.

Ahlfeld and Mulligan (2000) stated that solution of the full three dimensional equation for unconfined flow is complicated as the elevation of the free surface is unknown. They suggested that a quasi three-dimensional form of the flow equation should be used in which the depth of the unconfined aquifer is divided into vertical layers. The head in

each layer should be calculated from the two-dimensional flow equation and a conductance term should relate vertical flow between layers to the head difference between those layers.

### 5.5.2 Particle Tracking – MODPATH

MODPATH is a particle tracking computer software that computes three dimensional path lines based on the output from MODFLOW. Flow paths are delineated based on the assumption that each directional velocity component varies linearly within a cell. If  $v_x$ ,  $v_y$ , and  $v_z$  are the principal components of the average linear groundwater velocity vector,  $n$  is porosity, and  $Q$  is the volume rate of water created or consumed by internal sources and sinks per unit volume of aquifer, then the partial differential equation describing conservation of mass in steady state groundwater flow for an infinitesimally small volume of aquifer can be expressed as:

$$\frac{n\partial v_x}{\partial x} + \frac{n\partial v_y}{\partial y} + \frac{n\partial v_z}{\partial z} = Q \quad (5.4)$$

Pollock (1994) established a method of computing path lines that utilized the principal components of velocity vector at every node in the flow field based on the inter-cell flow rates from the flow simulation model. The method applies simple linear interpolation to compute the principal velocity components at a node within a cell. The linear

interpolation produces a continuous velocity vector field within each individual cell that satisfies equation (5.4). MODPATH has limitations relating to the embedded assumptions. These limitations include the type of discretization scheme, parameter uncertainty and boundary conditions. MODPATH is only valid for simple linear velocity interpolation schemes (Pollok, 1994).

### 5.5.3 Governing Equation for Contaminant Transport

Contaminant transport in this study is best described by a transient 3-D hydrodynamic dispersion equation with decay. The governing equation for 3-D flow of contaminants in a heterogeneous and anisotropic aquifer is given by:

$$R_d \frac{\partial C}{\partial t} = V_x \frac{\partial C}{\partial x} + \frac{\partial}{\partial x} (D_{xx} \frac{\partial C}{\partial x}) + \frac{\partial}{\partial y} (D_{yy} \frac{\partial C}{\partial y}) + \frac{\partial}{\partial z} (D_{zz} \frac{\partial C}{\partial z}) + \lambda C \quad (5.5)$$

Where:

- $D_{xx}, D_{yy}, D_{zz}$  = values of dispersion coefficients along the x, y and z coordinate axes, which are assumed to be parallel to the major axes;
- $V$  = mean flow velocity or seepage velocity;
- $C$  = contaminant concentration;
- $\lambda$  = decay constant;
- $R_d$  = retardation factor = seepage velocity/contaminant velocity

## **5.6 Development of the Optimization Model**

This study employs hydraulic gradient control to optimally remove the PCE, TCE and PCR in BPOU. The management problem includes multiple conflicting objectives. A feasible management model must mathematically express the interdependent relationships of the competing objectives such that the solution will result in optimal decision-making. The proposed methodology transforms the multi-objective problem into single-objective problem by using the multi-objective decision making theory. The annual cost of operation and maintenance for the hydraulic gradient control scheme is defined as the objective function and the remaining objectives are treated as constraints. The optimal management decision minimizes the total annual cost and includes the rest of the objectives in a constraint set that includes operational, physical, economic, institutional and legislative requirements. The goal is to minimize water transfer costs of a hydraulic gradient control scheme that removes contaminants.

Initially, groundwater flow is simulated. Then the particle-tracking program is run to simulate contaminant flow paths. RT3D generates plume contours and plume boundary. This procedure establishes base line contaminant contours to delineate the existing condition and aids in developing the hydraulic control scheme.

Preliminary strategic locations of best wells are determined in an experimental design approach. Best wells are selected by several simulation and optimization runs to determine the optimal location and flow rates for the wells. The iterative coupling of simulation with optimization procedure guarantees a successful hydraulic gradient

control scheme.

Finally, the rates for recharge and extraction are optimized using various constraints and output from the simulation model. The simulation model depicts groundwater flow and contaminant transport. The hydraulic gradient control scheme initiates inward gradient that prevents further intrusion and stabilizes the contaminant plume. The hydraulic head at the inner pumping well is significantly lower than the hydraulic heads at the surrounding recharge wells creating an inward gradient. Desirable hydraulic gradients are achieved by varying recharge and pumping rates. Cost of operation and maintenance is minimized while satisfying the set of the constraints. Plume cleanup is achieved when the PCE and TCE contaminant plume concentration is less than 5  $\mu\text{g/L}$  and the PCR contaminant plume concentration is less than 6  $\mu\text{g/L}$ .

The optimization model is developed by explicitly formulating the cost objective function. The cost function is explicitly formulated by defining the functional relationships of the state and decision variables. Sources, users and disposal sites must be clearly defined. The objective and constraint functions clearly depict the interdependent relationships affecting water allocation. Each variable is quantified in terms of cost.

### **5.6.1 Decision Variables and Associated Costs**

Decision variables are policies of the management model for which optimal solutions are sought. Decision variables define the solution to a given problem subject to given management decisions.

In this study, the decision variables are the groundwater well pumping and recharge rates, imported water, reclaimed water, precipitation, total demand allocations and the planning horizon. The water manager can define upper and lower bounds for the decision variables. A solution to the optimization problem is a set of optimal values of the decision variables that correspond to a given operational policy.

### **5.6.2 State Variables**

State variables are functions of the decision variables that reflect the dynamic state of the system. In a dynamic system, they represent the resultant change in decision variables and the effect on the specified constraints. The hydraulic head, aquifer storage and plume concentrations are such variables that depend on the decision variables.

### **5.6.3 Constraints**

The model is restrained by physical, operational, institutional, environmental and legislative restrictions on the decision variables called system constraints. Constraints define management restrictions on decision and state variables including flow rates, demand and transfers such as flux boundaries, head, hydraulic gradient and contaminant

plume concentration for a given location and management period. The multi-objective problem is transformed into a single objective problem by using the other objectives as constraints as shown below:

*water supply source upper bounds*

$$0 \leq Q_{i,t} \leq Q_{\max_{i,t}}$$

$Q_{\max_{i,t}}$  = maximum supply rate from a given source  $i$  in time period  $t$  [ $L^3/T$ ]

*well capacity constraints*

$$0 \leq Q_{i,t} \leq Q_{\max_{i,t}}$$

$Q_{\max_{i,t}}$  = maximum extraction or recharge rate for well  $i$  in time period  $t$  [ $L^3/T$ ]

*demand constraints*

$$\sum Q_{i,t} \geq D_t$$

$D_t$  = total water demand in time period  $t$  [ $L^3/T$ ]

*drawdown constraints*

$$h_{i,t} \geq h_{\min_i}$$

$h_{\min_i}$  = minimum allowable head for well  $i$  [ $L$ ]

*head constraints above which water quality and liquefaction become concerns*

$$h_{i,t} \leq h_{\max_i}$$

$h_{\max_i}$  = maximum allowable head for well  $i$  [ $L$ ]

## **5.7 Hydraulic Gradient Control Scheme**

The development of a hydraulic gradient control scheme focuses on how to alter the pre-existing groundwater flow patterns at the site in order to block, contain and clean up the contaminant plume. The primary goal of hydraulic gradient control is to minimize the concentration of the contaminants at the potable water wells.

Initially the existing contaminant flow field is studied. The design of the hydraulic gradient control scheme optimally selects the type and location of hydraulic control devices including monitoring, pumping and recharge wells and their flow rates. Complex natural forces within the aquifer system include subsurface inflow, infiltration, precipitation, leakage and aquifer geology. The pumping and recharge wells should be operated at a target hydraulic gradient in order to effectively pump out the contaminant plume and avoid excessive mounding.

Initially, the hydraulic gradient controls are defined in relation to the groundwater system. The flow model and contaminant transport models are run. The contaminant plume is delineated using the contaminant transport model RT3D. Pumping and recharge wells that ensure the capture, containment and stabilization of the plume are selected. The selected pumping and recharge wells should maintain a desirable head gradient. The hydraulic control scheme is then optimized subject to the given constraints and system requirements.

The water extracted from the plume can be treated and injected back into the hydraulic

gradient control recharge wells. For cost effective hydraulic gradient control, it is best to utilize off-line or standby wells as recharge or pumping wells.

Well locations are determined from an initial simulation, particle tracking, contaminant transport and optimization run. The groundwater simulation model and contaminant transport models are initially calibrated to match site conditions. The hydraulic control scheme is formulated in terms of the following hydraulic head expression:

$$h_i = h_i(q), \quad (5.5)$$

where :

$h_i$  = simulated head at location i (node i)

$q$  = discharge / recharge at specified locations

The aquifer hydrogeologic properties and all regional fluxes are implicitly expressed in this formulation. The contaminant plume is assumed to be continuous and forms a single plume. The constraint that bound differences in head is given by:

$$d_{il} \leq h_{i,1} - h_{i,2} \leq d_{iu} \quad (5.6)$$

$d_{iu}$  and  $d_{il}$  are specified upper and lower bounds on head differences and  $h_{i,1}$  and  $h_{i,2}$  are heads at adjoining node points respectively

## 5.8 Linear Optimization Formulation

The objective function is the key expression for optimization. It expresses variables such as the cost of the system, system operation, the value of resource allocation, or the impact of applied stresses such as overdraft and subsidence to aquifers. An objective function translates decision variables into a quantity of interest. The generic optimization formulation is given by:

$$\text{Min } P(x) \quad (5.7)$$

Subject to:

$$g_i(x) \geq 0 \quad i=1,2,\dots,m$$

$$x_j \geq 0 \quad j=1,2,\dots,n$$

where:

$P$  = mathematical objective function formulated from all costs.

$x$  = decision variable vector.

$X$  = set of feasible decision variables  $\{X/x_u \leq x \leq x_l; x \geq 0\}$

$x_u$  = upper bound on  $x$ ;

$x_l$  = lower bound on  $x$ ;

$g_i(x)$  =  $i$ th constraint.

$n$  = total number of decision variables.

$m$  = total number of constraints.

In this study, the objective function is formulated as the minimum total annual operational and maintenance cost. The total annual cost includes yearly operating and maintenance cost for pumping, recharge and imported water. Reclaimed water is used to recharge the hydraulic control wells. When new injection or pumping wells are added, the additional capital cost  $P$ , is converted to an annual cost  $A$  by including the economics formula  $P^*(A/P, A, i)$  in equation 4.7. The term  $i$ , is the effective interest rate. The objective function can be expressed as follows:

$$\text{Min (Cost)} = \text{Min} \sum (aP(I,T) + b IM(I,T) + cRC(I,T) + dR(I,T)) \quad (5.8)$$

Subject to the constraints.

Where:

$P(I,T)$  = annual volume of water extracted from the groundwater basin by user  $I$  during the period  $T$ .

$IM(I,T)$  = annual volume of imported water consumed by user  $I$  during the period  $T$ .

$RC(I,T)$  = annual volume of reclaimed water from plume extraction consumed by user  $I$  during the period  $T$ .

$R(I,T)$  = annual volume recharged into the hydraulic gradient control wells from source  $I$  during the period  $T$ .

$a(I,T)$  = cost of pumping of one gpm of water to a height of 100 feet for user  $I$  during the period  $T$ .

$b(I,T)$  = cost of importing one gpm of water from source  $I$ , during the period  $T$ .

$c(I,T)$  = cost of treating one gpm of reclaimed water extracted from the plume, at plant  $I$  during period,  $T$ .

$d(I,T)$  = cost of recharge of one gpm of water through well I, during period T.

### **5.8.1 Cost Coefficient Estimation**

Pumped local groundwater =

cost of power for lift, well operation and maintenance, cost of disinfection if water is for M&I use or for hydraulic gradient control. Power requirement depends on lift distance and volume of water pumped. Assume that under steady state operation, the lift at each well will remain constant. This allows retention of linear dependence on the pumped rate.

Imported water cost =

Total cost of imported water is the price charged by suppliers (purveyors) and depends on the amount of water imported.

Reclaimed Water cost =

the cost of transmitting pumped water to the treatment facilities and conveyance to the recharge wells for hydraulic gradient control.

## **5.9 Optimization Program - LINDO**

LINDO utilizes a mathematical linear program to determine optimal allocation of water.

A user can input several variables in LINDO to formulate a linear program, run the program, analyze the results, modify the formulation and re-iterate. Optimal well rates from LINDO are fed back into Groundwater Vistas. Groundwater Vistas then performs a simulation based on the current well rates. Well rates and locations are deemed optimal when the contaminant transport model outputs contaminant concentration contours below MCL or NL.

## **Chapter 6**

### **Case Study – Baldwin Park Operable Unit**

This study develops a methodology capable of optimally allocating water from sources to users to disposal sites under the condition of shrinking PCE and TCE plumes below MCL and PCR below NL in Baldwin Park Operable Unit (BPOU). The methodology developed identifies efficient strategies for shrinking and cleaning up the PCE, TCE and PCR plumes, while meeting total water demand, minimizing the net increase in imported water demand and providing maximum drought and emergency supplies. The finite difference groundwater flow model was calibrated to simulate responses to induced stresses. Particle tracking was used to delineate contaminant flow paths. The contaminant transport module RT3D produced the contaminant concentration contours. Recharge and extraction well locations and rates were varied to induce the inward hydraulic gradient towards the core of the plume.

The cleanup of the PCE, TCE and PCR plumes is managed by implementing the linear optimization methodology presented in the previous chapter. Usually, management is faced with multiple conflicting objectives and must seek the best feasible alternatives. The multi-objectives are transformed into a single objective by including all but one of the objectives in the constraint set. The user can choose the number and type of water sources and investigate the sensitivity of the constraints on the model results. The BPOU was selected for the purpose of developing and testing the optimized hydraulic gradient control concept for PCE, TCE and PCR removal and due to the availability of data.

## **6.1 San Gabriel Valley Basin**

The San Gabriel Valley basin is located in eastern Los Angeles County, California. The surface area is 154,000 acres or 255 square miles. The basin is bounded on the north by the Raymond fault and the Quaternary sediments and consolidated basement rocks of the San Gabriel Mountains. The basin is bounded on the south and west by exposed consolidated rocks of the Repetto, Merced and Puente hills. The San Jose fault and the Chino fault form the eastern Boundary. Precipitation in the basin varies from 15 to 31 inches and averages around 19 inches (DWR, 1966).

The San Gabriel basin comprises primarily of alluvial deposits. Most of the alluvial deposits are of Quaternary age, overlying relatively impermeable rock. These deposits range from 2,000 to 4,000 feet thick in the center of the basin and range between approximately 250 to 800 feet thick at the basin outlet in Whittier Narrows (USEPA, 1999).

The basin is underlain by two distinct sources of sediment in the basin; the coarse-grained crystalline rocks of the San Gabriel Mountains and the finer-grained sedimentary rocks of the hills to the southeast and southwest. Alluvial sediment derived from the northern San Gabriel Mountains is generally coarser than that from the hills to the south. As such, the hydraulic conductivity of the alluvium generally increases with proximity to the northern San Gabriel Mountains. Sediment distribution is directly related to the position relative to river and tributary courses. Coarse-grained sediments are predominant in the San Gabriel River proximity. The San Gabriel Basin geology is characterized by

interfingering lenses of alluvial deposits (e.g.cobbles, gravel, silt, and clay) that exhibit a high degree of variability in sediment type, both vertically and laterally. Major hydrogeologic structures include the topographic highs (i.e., San Gabriel Mountains and southern hills) and topographic lows (i.e., Whittier Narrows). Groundwater flow in the basin is potentially impacted by four major faults; the Sierra Madre Fault System, the Raymond Fault, the Lone Hill-Way Hill Fault, and the Workman Hill Fault. Data for the transient flow and contaminant transport models were obtained from the sources described in the Table 6.1 below.

Table 6.1 Sources of Model Data

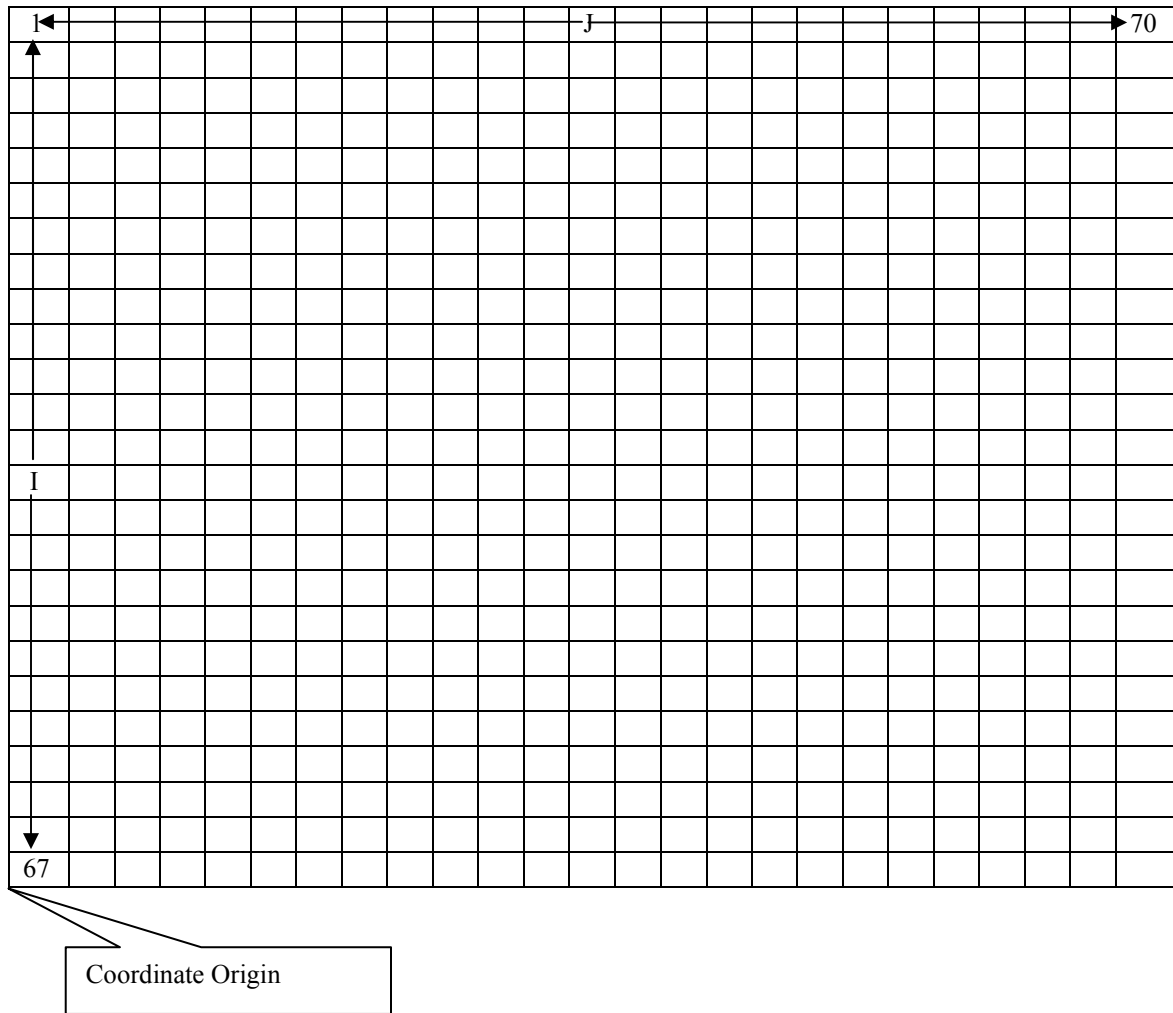
| Data               | Source                                                       |
|--------------------|--------------------------------------------------------------|
| Ground Elevations  | USEPA Database/United States Geological Survey (USGS)        |
| Aquifer geology    | USEPA Database (2008) / Department of Water Resources (1966) |
| Well Data          | USEPA database (2008)                                        |
| Precipitation      | Main San Gabriel Basin Watermaster                           |
| Water Quality Data | USEPA database (2008)                                        |

## **6.2 Simulation Model Development and Discretization**

The grid size was determined by considering the physical boundaries of the system, the temporal and spatial variability of the model parameters. A uniform grid size was selected for ease of computations in Groundwater Vistas. Three layers are assumed to exist in the vertical direction. All three layers are unconfined. The data was obtained from USEPA database (USEPA, 2008).

The Baldwin Park model area was discretized into 67 rows and 70 columns. Each grid block is 150 m by 150 m. Small constant grid was used to clearly define contaminant plume. All values are averaged over the cell dimensions for a given cell. Thus, models with finer grids output larger head or drawdown responses than those with coarser grids for the same flow rates.

Figure 6.1 Finite Difference Grid



### 6.2.1 Boundary and Initial Conditions

The northern boundary of Foothill Boulevard, western boundary of Route 605 freeway, eastern boundary of Azusa Avenue and southern boundary of Route 10 freeway are assumed to be general head boundaries based on historical groundwater levels.

### **6.3 Study Area – Baldwin Park Operable Unit**

The Baldwin Park Operable Unit lies in the central portion of the San Gabriel Valley Basin (Figure 1.2), approximately 25 miles from the Pacific Ocean, in eastern Los Angeles County. The Baldwin Park Operable Unit lies south of the San Gabriel mountains, east of the 605 freeway, north of the 10 freeway and west of Azusa Avenue. The Baldwin Park area is fully developed and has a mixture of residential, commercial and industrial facilities. The region has large parcels of open land with active and inactive gravel pits and the Santa Fe Flood control basin.

Alluvial deposits are predominant in the region and are unconsolidated to partially unconsolidated non-marine sediments of Recent and Pleistocene age. These sediments were deposited by fluvial and geomorphic processes resulting from San Gabriel River and its tributaries. Massive gravel pits exist in the northern and central portions of Baldwin Park. Lithologic evaluation of well logs reveal that gravel depths exceed 500 ft in the north mixed with 10 to 30 feet of clay in the south. Alluvial sediments range from a few hundred feet in the north to approximately 2000 ft in the south.

#### **6.3.1 Aquifer Parameters**

Table 6.1 summarizes the calibrated values of aquifer parameters used in the BPOU flow model. Initial values of hydraulic conductivity and specific yield were obtained from the USEPA database 2008 were divided into 7 zones of hydraulic conductivity and specific yield. Final calibrated values were used to generate model output.

### 6.3.2 Hydraulic Conductivity

Hydraulic Conductivity values in the BPOU are higher than other regions in the San Gabriel Valley. Aquifer tests at 25 wells (from well lithology in USEPA database) yielded hydraulic conductivity values from 20 to 350 ft/day. High values of hydraulic conductivity exist in the north and central regions. BPOU hydraulic conductivity values are the highest in the San Gabriel Valley. Values of hydraulic conductivities from 25 wells were plotted to yield 7 zones for hydraulic conductivity (Figure 6.2).

### 6.3.3 Specific Yield

Specific yield values range from 0.1 to 0.2, reflecting the presence of coarse grained material. Values of specific yield from the 25 wells were plotted to yield 7 zones.

Aquifer parameters for BPOU are summarized in Table 6.4 below.

Table 6.2 Aquifer Parameters

| Parameter                  | Range      | Unit   | Input method | Source         |
|----------------------------|------------|--------|--------------|----------------|
| Hydraulic Conductivity (K) | 20-350     | ft/day | Zone         | USEPA Database |
| Specific Yield             | 0.01-0.20  |        | Zone         | USEPA Database |
| Porosity                   | 0.2 – 0.25 |        | Zone         | USEPA Database |

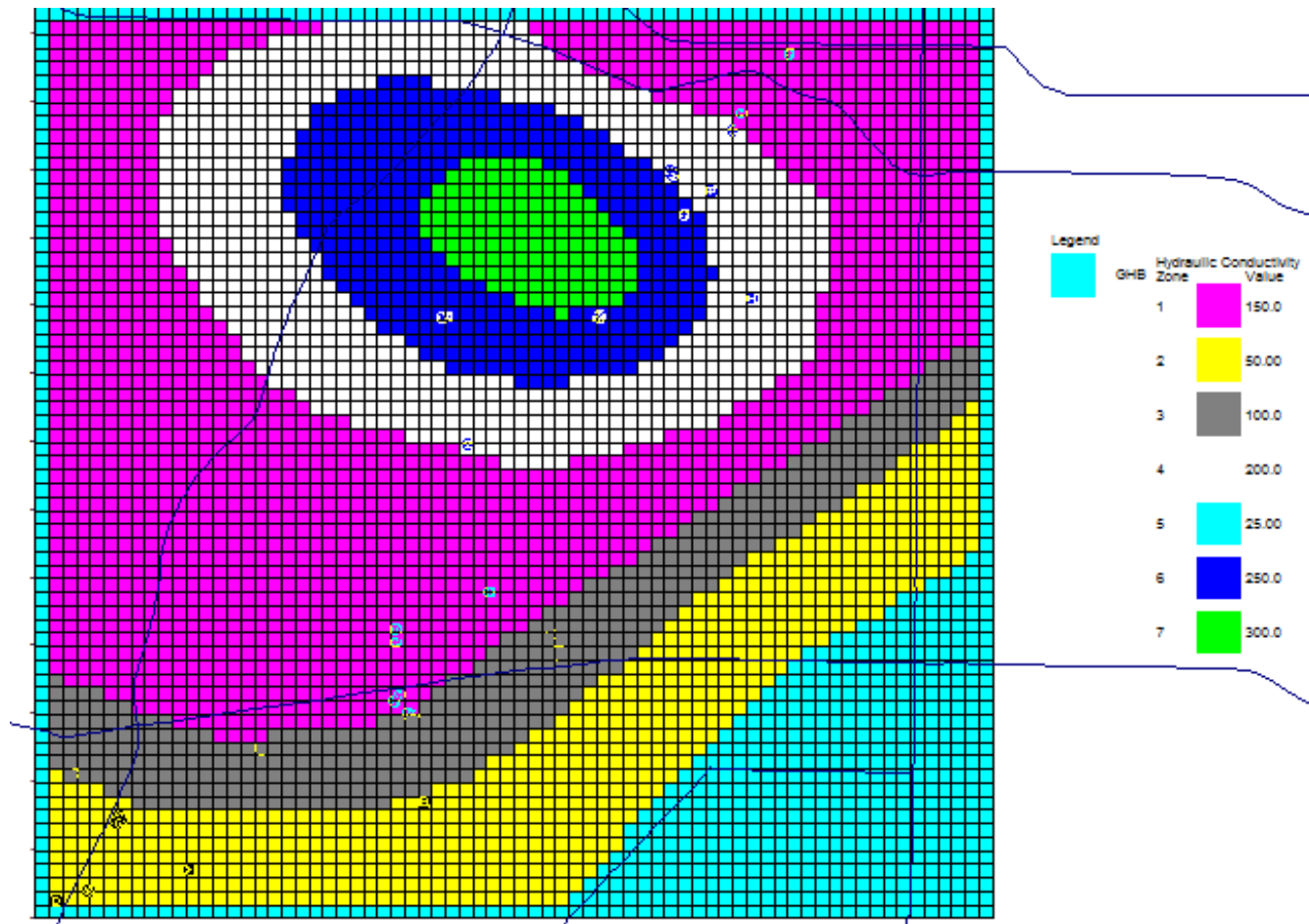


Figure 6.2 Hydraulic Conductivity Zones

## 6.4 Groundwater Data

Data from the tables below were used to develop the groundwater flow model, particle tracking and contaminant transport models. Table 6.3 contains the active pumping wells and their respective depths in layers 2 and 3. No wells penetrate layer 3. Table 6.4 depicts monitoring well data. Transient pumping data (ft<sup>3</sup>/day) were obtained from the USEPA database (2008).

Table 6.3 Active Pumping Wells

| Well Name | X-Coordinate | Y-Coordinate | Ground Elevation | Well Depth | Layer |
|-----------|--------------|--------------|------------------|------------|-------|
| 01903067  | 410859.15    | 3768990.5    | 334              | 558        | 1     |
| 01903072  | 406826.68    | 3767902.25   | 292              | 120        | 1     |
| 61900718  | 407483.34    | 3768755      | 286              | 510        | 1     |
| 71903093  | 410537.15    | 3770102.5    | 340              | 526        | 1     |
| 01900027  | 411362.18    | 3774289.5    | 424              | 600        | 1     |
| 01900032  | 411118.18    | 3774341.5    | 410              | 600        | 1     |
| 01900034  | 412794.21    | 3774347.5    | 456              | 540        | 1     |
| 01902356  | 411058.18    | 3774326.5    | 423              | 525        | 1     |
| 08000060  | 412777.21    | 3774321.5    | 452              | 600        | 1     |
| 61900719  | 407532.09    | 3768773.5    | 284              | 516        | 1     |
| 01901460  | 410687.15    | 3769953.5    | 337              | 947        | 2     |
| 01902859  | 410768.15    | 3769894.5    | 335              | 800        | 2     |
| 08000097  | 407179.09    | 3768017.5    | 285              | 980        | 2     |
| 08000174  | 407051.81    | 3769306.31   | 300              | 890        | 2     |
| 78000098  | 410593.15    | 3770172.5    | 340              | 1078       | 2     |
| 51902858  | 409047.12    | 3769572.5    | 339              | 1178       | 2     |
| 51902947  | 409072.12    | 3769564.5    | 315              | 1154       | 2     |
| 08000112  | 407511.28    | 3768822.25   | 285              | 1276       | 2     |

Table 6.4 Monitoring Wells

| Well Name | X-Coordinate | X-Unit | Y-Coordinate | X-Unit | Ground Elevation |
|-----------|--------------|--------|--------------|--------|------------------|
| V10VCMW1  | 415058.13    | M      | 3777961.50   | M      | 627              |
| W10OSMW2  | 413736.91    | M      | 3775950.00   | M      | 526              |
| W10OSMW1  | 413629.91    | M      | 3776013.25   | M      | 529              |
| W10NCMW1  | 413711.50    | M      | 3775468.75   | M      | 503              |
| W10JHMW1  | 411667.03    | M      | 3774304.00   | M      | 430              |
| W10BDMW1  | 414866.00    | M      | 3777240.50   | M      | 593              |
| V10VCMW4  | 415103.75    | M      | 3777955.00   | M      | 629              |
| V10VCMW2  | 415022.56    | M      | 3777975.50   | M      | 627              |
| W10OSMW5  | 413602.59    | M      | 3775884.50   | M      | 520              |
| V10QGDW1  | 415605.34    | M      | 3777394.25   | M      | 609              |
| V10PIEW1  | 414955.06    | M      | 3777445.75   | M      | 605              |
| V10CAMW1  | 415569.16    | M      | 3777572.50   | M      | 616              |
| V10VCMW3  | 415080.69    | M      | 3778001.50   | M      | 629              |
| W11AZW01  | 414093.03    | M      | 3775256.00   | M      | 504              |
| W11AZW2R  | 414648.84    | M      | 3775234.75   | M      | 511              |
| W11AZW1R  | 414093.03    | M      | 3775240.00   | M      | 504              |
| W11AZW10  | 415004.19    | M      | 3775810.50   | M      | 536              |
| W11AZW09  | 414516.69    | M      | 3776394.50   | M      | 554              |
| W11AZW08  | 414423.38    | M      | 3775234.50   | M      | 506              |
| W11AZW06  | 414004.03    | M      | 3775720.50   | M      | 522              |
| W10OSMW3  | 413563.44    | M      | 3775957.50   | M      | 519              |
| W11AZW02  | 414640.66    | M      | 3775244.50   | M      | 510              |
| W10OSMW4  | 413568.38    | M      | 3775920.75   | M      | 520              |
| W11AJMW5  | 414184.59    | M      | 3777092.75   | M      | 537              |
| W11AJMW4  | 414344.56    | M      | 3776582.00   | M      | 555              |
| W11AJMW3  | 414793.47    | M      | 3776800.00   | M      | 571              |
| W11AJMW2  | 414244.22    | M      | 3776392.25   | M      | 552              |
| W11AJMW1  | 414665.63    | M      | 3776350.75   | M      | 557              |
| W10WOMW1  | 414844.38    | M      | 3776920.75   | M      | 581              |
| W10RUMW1  | 415155.53    | M      | 3777222.75   | M      | 592              |
| W10PPMW1  | 414549.66    | M      | 3774973.75   | M      | 495              |
| V10AMMW2  | 414505.28    | M      | 3778436.25   | M      | 642              |
| W11AZW03  | 414144.00    | M      | 3776418.25   | M      | 551              |
| V10AMMW4  | 414177.25    | M      | 3778250.00   | M      | 618              |

### **6.4.1 Stress Periods**

Table 5.5 depicts the quarterly stress periods used in this study. One hundred and fifty (150) quarterly stress periods were used from October 1974 to the end of March 2012.

The model was run from July 1, 2006 to June 30, 2011 to predict future clean up scenarios. The pumping data in the USEPA database are monitored and recorded quarterly. Hence, quarterly stress periods were used in the simulation. Quarterly stress periods depict model results better than annual or semi-annual stress periods. Seasonal or periodic system responses are output more accurately using quarterly stress periods.

Incorporating quarterly (shorter) stress periods into the model allows better depiction of these seasonal changes. In the simulations of future scenarios and in the optimization analyses described later, quarterly (3-month) stress-periods are used to more accurately capture local and global optima and minima.

Table 6.5 Stress periods

| Stress Period | Date    | Stress Period | Date    | Stress Period | Date    | Stress Period | Date    | Stress Period | Date    |
|---------------|---------|---------------|---------|---------------|---------|---------------|---------|---------------|---------|
| 1             | 10/1/74 | 31            | 4/1/82  | 61            | 10/1/89 | 91            | 4/1/97  | 121           | 10/1/04 |
| 2             | 1/1/75  | 32            | 7/1/82  | 62            | 1/1/90  | 92            | 7/1/97  | 122           | 1/1/05  |
| 3             | 4/1/75  | 33            | 10/1/82 | 63            | 4/1/90  | 93            | 10/1/97 | 123           | 4/1/05  |
| 4             | 7/1/75  | 34            | 1/1/83  | 64            | 7/1/90  | 94            | 1/1/98  | 124           | 7/1/05  |
| 5             | 10/1/75 | 35            | 4/1/83  | 65            | 10/1/90 | 95            | 4/1/98  | 125           | 10/1/05 |
| 6             | 1/1/76  | 36            | 7/1/83  | 66            | 1/1/91  | 96            | 7/1/98  | 126           | 1/1/06  |
| 7             | 4/1/76  | 37            | 10/1/83 | 67            | 4/1/91  | 97            | 10/1/98 | 127           | 4/1/06  |
| 8             | 7/1/76  | 38            | 1/1/84  | 68            | 7/1/91  | 98            | 1/1/99  | 128           | 7/1/06  |
| 9             | 10/1/76 | 39            | 4/1/84  | 69            | 10/1/91 | 99            | 4/1/99  | 129           | 10/1/06 |
| 10            | 1/1/77  | 40            | 7/1/84  | 70            | 1/1/92  | 100           | 7/1/99  | 130           | 1/1/07  |
| 11            | 4/1/77  | 41            | 10/1/84 | 71            | 4/1/92  | 101           | 10/1/99 | 131           | 4/1/07  |
| 12            | 7/1/77  | 42            | 1/1/85  | 72            | 7/1/92  | 102           | 1/1/00  | 132           | 7/1/07  |
| 13            | 10/1/77 | 43            | 4/1/85  | 73            | 10/1/92 | 103           | 4/1/00  | 133           | 10/1/07 |
| 14            | 1/1/78  | 44            | 7/1/85  | 74            | 1/1/93  | 104           | 7/1/00  | 134           | 1/1/08  |
| 15            | 4/1/78  | 45            | 10/1/85 | 75            | 4/1/93  | 105           | 10/1/00 | 135           | 4/1/08  |
| 16            | 7/1/78  | 46            | 1/1/86  | 76            | 7/1/93  | 106           | 1/1/01  | 136           | 7/1/08  |
| 17            | 10/1/78 | 47            | 4/1/86  | 77            | 10/1/93 | 107           | 4/1/01  | 137           | 10/1/08 |
| 18            | 1/1/79  | 48            | 7/1/86  | 78            | 1/1/94  | 108           | 7/1/01  | 138           | 1/1/09  |
| 19            | 4/1/79  | 49            | 10/1/86 | 79            | 4/1/94  | 109           | 10/1/01 | 139           | 4/1/09  |
| 20            | 7/1/79  | 50            | 1/1/87  | 80            | 7/1/94  | 110           | 1/1/02  | 140           | 7/1/09  |
| 21            | 10/1/79 | 51            | 4/1/87  | 81            | 10/1/94 | 111           | 4/1/02  | 141           | 10/1/09 |
| 22            | 1/1/80  | 52            | 7/1/87  | 82            | 1/1/95  | 112           | 7/1/02  | 142           | 1/1/10  |
| 23            | 4/1/80  | 53            | 10/1/87 | 83            | 4/1/95  | 113           | 10/1/02 | 143           | 4/1/10  |
| 24            | 7/1/80  | 54            | 1/1/88  | 84            | 7/1/95  | 114           | 1/1/03  | 144           | 7/1/10  |
| 25            | 10/1/80 | 55            | 4/1/88  | 85            | 10/1/95 | 115           | 4/1/03  | 145           | 10/1/10 |
| 26            | 1/1/81  | 56            | 7/1/88  | 86            | 1/1/96  | 116           | 7/1/03  | 146           | 1/1/11  |
| 27            | 4/1/81  | 57            | 10/1/88 | 87            | 4/1/96  | 117           | 10/1/03 | 147           | 4/1/11  |
| 28            | 7/1/81  | 58            | 1/1/89  | 88            | 7/1/96  | 118           | 1/1/04  | 148           | 7/1/11  |
| 29            | 10/1/81 | 59            | 4/1/89  | 89            | 10/1/96 | 119           | 4/1/04  | 149           | 10/1/11 |
| 30            | 1/1/82  | 60            | 7/1/89  | 90            | 1/1/97  | 120           | 7/1/04  | 150           | 1/1/12  |

## 6.5 Hydraulic Gradient Control System Components

The hydraulic gradient control system comprised 18 extraction wells with variable pumping rates and 6 injection wells. Injection rates were alternated between 519 gpm and 779 gpm to create transient effects. Pumping rates were alternated between 1558 gpm

and 2338 gpm to create transient effects. An efficient hydraulic gradient scheme is developed capable of cleaning up the PCE and TCE contaminant plume below the MCL of 5µg/L and the PCR below the notification limit of 6 µg/L.

## **6.6 Contaminant Plume**

The contaminants are commingled plumes comprising VOC's (PCE and TCE) and PCR. The target goal was to clean the PCR plume below advisory notification level of 6 µg/L and to clean the PCE and TCE plume below maximum contaminant level (MCL) of 5µg/L. Analysis of the water quality data focused primarily on the PCE and TCE concentrations because both PCE and TCE are primary indicators of the level of VOC contamination in the groundwater plume. High VOC contamination exceeding 1000 times drinking water standard were discovered in the USEPA database (2008).

## **6.7 Water Budget**

The basin is made up of fully developed residential, commercial and industrial regions. Agricultural (AG) flow includes water for farm irrigation and landscape irrigation. The annual return flow amount of 4500 acre-ft was reported by DWR (1966). M & I and AG demands for the study area were assumed to be a fifth (1/5) of the total annual water demand. Five principal water supply purveyors have been identified for the study area.

### **6.7.1 Imported Water Sources**

Imported water is purchased from Metropolitan Water District of Southern California

(MWD) and water is delivered by the City of Los Angeles (City of LA) from its Owens Valley Aqueduct. Groundwater extraction amounts are limited by the terms of the Baldwin Park Operable Unit Judgment and the adoption of programs to encourage in-lieu groundwater replenishment. Groundwater is used to satisfy Municipal and Industrial (M&I), agricultural (AG) and aquifer recharge demands. The model area has an average precipitation of 19 in. The extracted amount used is less than the average, 10,550 acre-ft/year. Growing demand has been met by supplemental sources, effectively by imports.

Aquifer water balance varies yearly and is directly related to variations in recharge, import and extraction. Groundwater inflow and outflow are assumed to be balanced by changes in storage initially and eventually by underflow.

### **6.7.2 Recharge Package**

Recharge of the basin is mainly from direct percolation of precipitation and percolation of stream flow. A small portion of San Gabriel River passes through the basin. The small portion of San Gabriel River was not included in the model because the river is dry 90% of the year. The water in the river percolates into the groundwater basin.

The long term average rainfall in the San Gabriel Valley Basin is 18.52 inches. In 2004-05 fiscal year, an average of 46 inches of rainfall fell on the San Gabriel Valley Basin. In 2005-06, the San Gabriel Valley Basin received an average of 16 inches of rain. Figure 2.3 shows average annual rainfall for the San Gabriel Basin from 1974 to 2006. The

precipitation data is used to develop the transient groundwater flow and contaminant transport model. The precipitation data is used to develop the hydrologic base period and hydrologic model. Annual rainfall values were used in the recharge package. It is assumed that the annual volume of rainfall infiltrating the basin is 8% of the total rainfall volume per year.

### 6.7.3 Hydrologic Base Period

The cumulative departure from the mean annual rainfall was plotted with time for a 30 year period. Figure 5.3 depicts the hydrologic base period for BPOU. The hydrologic base period is approximately from 1988 to 2003 (15 years).

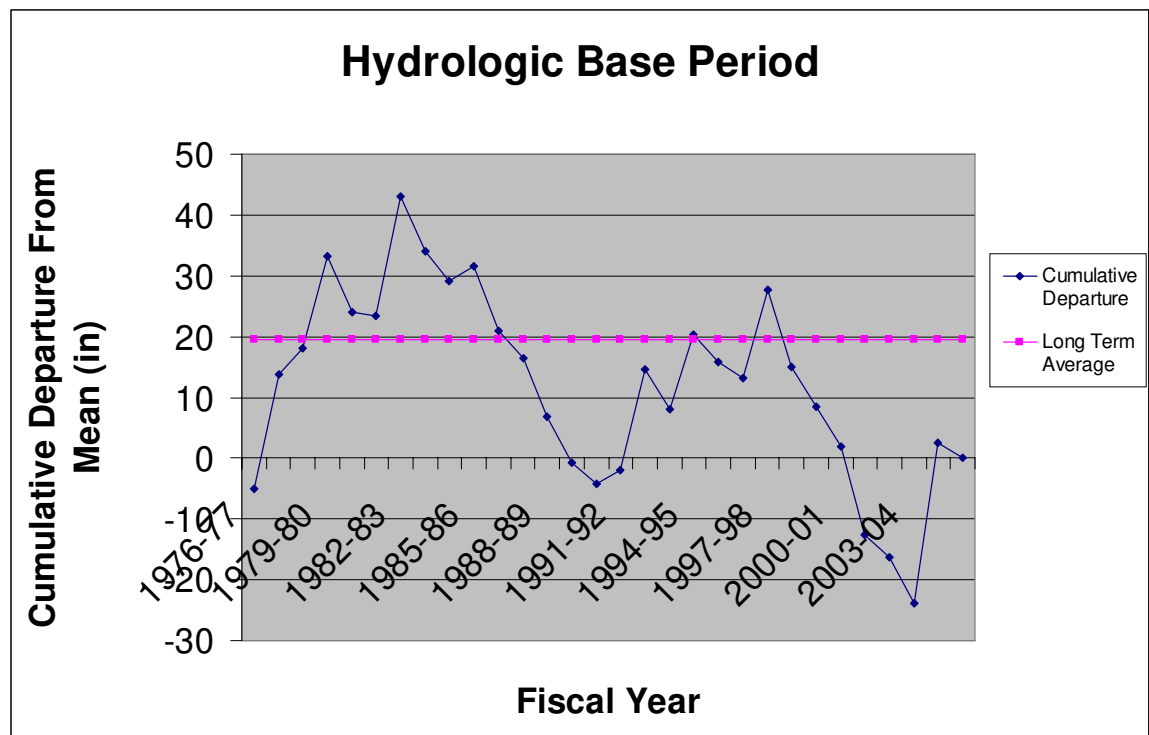


Figure 6.3 Hydrologic Base Period

Some facts about a hydrologic base period include:

- Representative of long-term hydrologic conditions;
- Includes wet, dry and average years of precipitation;
- Spans a 15 to 30 year period;
- Starts and ends at years preceded by comparatively similar rainfall quantities;
- Preferably starts and ends in a dry period. This minimizes any water draining (in transit) through the vadose zone, and includes recent cultural conditions.

#### **6.7.4 General Head Boundary Package**

A transient general head boundary was computed for the north, south, east and west boundaries. Four monitoring wells were used as reference targets for computing water levels at general head boundary cells. The reference targets were used to develop relations between boundary grid cells and reference target water levels. The relationship for the base stress period Oct 1974 was replicated for the remaining 149 stress periods. One hundred and fifty (150) quarterly stress periods were used. Conductance was calculated by multiplying the effective aquifer depth by the average hydraulic conductivity. Figure 6.4 shows the transient general head boundary cells (green) used in this study.

#### **6.7.5 Well Package**

Data for 18 active pumping wells and 27 monitoring wells were obtained from USEPA database and input into the model. Figure 5.1 shows the pumping wells (red) and monitoring wells (blue). Pumping wells pumped at different rates in quarterly stress periods. Monitoring wells contained water level and water quality data.

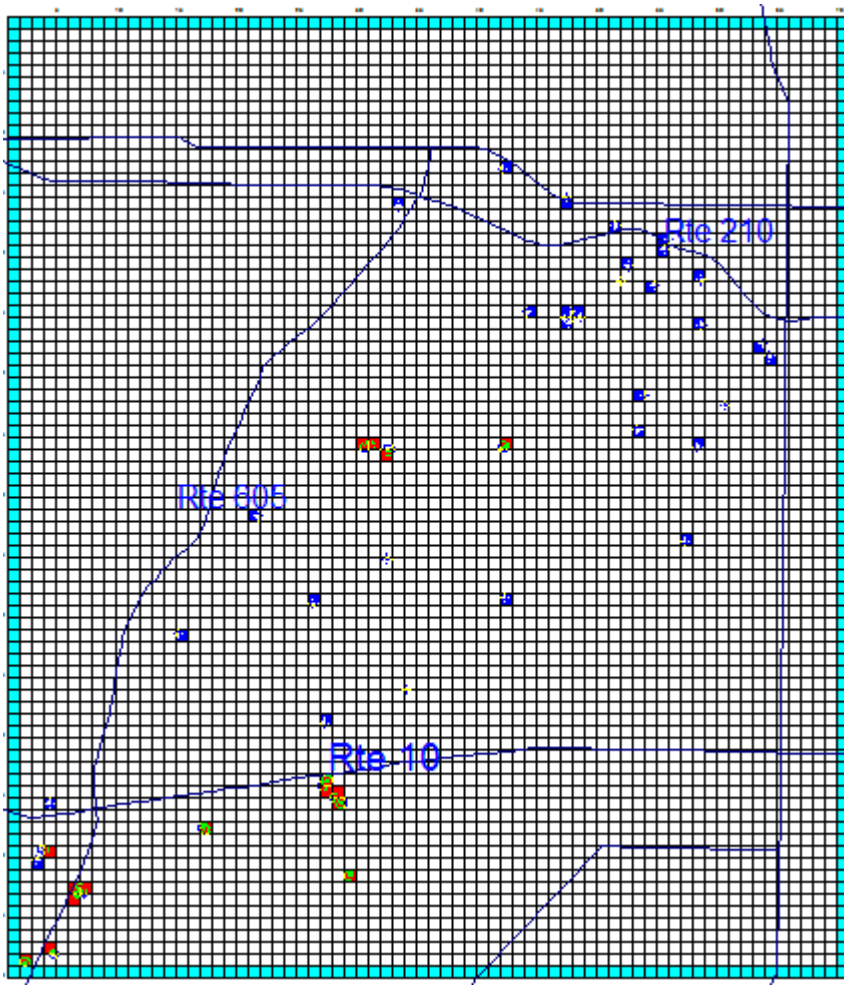


Figure 6.4 Transient General Head Boundary, Pumping and Monitoring Wells

### 6.7.6 Evapotranspiration Package

Extinction depth for evapotranspiration is 15 feet or less below ground surface.

Groundwater levels are 50 to 150 feet below ground surface far exceeding the extinction depth. Therefore evapotranspiration was not considered.

## 6.8 Model Calibration

The groundwater simulation model was calibrated by adjusting the hydraulic properties and boundary conditions to match head values at specified locations. The parameters in the simulation model are adjusted so that the mathematical model correctly solves the governing equations. Physical parameters that govern contaminant transport include: horizontal hydraulic conductivity ( $K_h$ ), vertical hydraulic conductivity ( $K_v$ ), specific yield ( $S_y$ ) and vertical distribution of injection and extraction. Initial values for each zone were obtained from the USEPA database (2008). Values were adjusted to obtain reasonable agreement between simulated and observed values. Vertical hydraulic conductivities were determined to be 10% of the horizontal hydraulic conductivities from calibration of the flow model. The continuous coarse-grained deposits dictate horizontal conductance. The fine-grained materials dictate the vertical conductance.

A goal of this research was to develop and test a hydraulic gradient control scheme for contaminant removal. The model was accepted as calibrated when adequate capability of replicating target data in the plume was achieved. Water levels from October 1, 1974 to September 30, 2000 were used to calibrate the flow model. Head target values for the study area were obtained from the USEPA database (2008).

Transient groundwater calibration was performed. PEST, a sub-module of GWV was used to calibrate the model. Calibration of a groundwater flow model is the process of adjusting hydraulic parameters including hydraulic conductivity, specific yield, initial conditions and boundary conditions within reasonable ranges to obtain a match between

observed and field measured values or calibration targets. Calibration targets are observed, measured or estimated hydraulic heads, groundwater flow rates or other aquifer parameter that the model must reproduce or replicate closely, for the model to be considered calibrated. A model must be calibrated to a desired accuracy (usually within 10% of the absolute residual standard deviation error) to be credible.

Model calibration is frequently accomplished through trial-and-error adjustment of the model's input data to match field observations. Automatic inverse techniques are another type of calibration procedure. The calibration process continues until the degree of correspondence between the simulation output and the observed data from the physical hydrogeologic system is consistent with the objectives of the project.

Types of Calibration:

- Steady State
- Transient

Water level data from monitoring wells from October 1, 1974 to September 30, 2000 were used for transient calibration. October 1974 was used as baseline for transient model.

Steady-state simulations produce average or long-term results and require a true physical equilibrium. Transient analyses are typically performed when boundary conditions change with time or when transient analysis of model output is required.

Potentiometric Head Residuals calculate the difference between the computed or

simulated heads and the measured heads:

$$r_i = h_i - H_i$$

where:

$r_i$  is the residual,

$H_i$  is the measured head at point  $i$ ,

$h_i$  is the computed or simulated head at the approximate location where

$H_i$  was measured.

If the residual is positive, then the computed head is too high; if negative, the computed head is too low. Residuals cannot be calculated from censored data.

Model data determines the range over which model parameters and boundary conditions may be varied. When parameters are well characterized by field measurements, the range over which that parameter is varied in the model should be consistent with the range observed in the field. Statistical techniques quantify the degree of fit between model simulations and field measurements.

When vertical gradients are negligible, two-dimensional modeling may be appropriate. If vertical gradients are significant or if there are several aquifers in the flow system, a quasi three-dimensional model may be appropriate. A quasi three-dimensional model may be used for aquitards that are not explicitly discretized but are approximated using a leakage term.

Residual Statistics:

Residual statistics measure maximum and minimum residuals, residual mean and standard deviation of the residuals.

Maximum and Minimum Residuals – The maximum residual is the highest residual or the largest difference (residual) between the simulated or calculated head and the measured head. The minimum residual is the lowest difference between the simulated or calculated head and the measured head.

Residual Mean is the arithmetic mean of the residuals computed in a given simulation:

$$R = \frac{\sum_{i=1}^n r_i}{n} \quad \text{where:}$$

R = residual mean

n = number of residuals.

A residual mean close to zero is desired assuming no correlation among residuals.

The individual residuals can also be weighted to account for differing degrees of confidence in the measured head resulting in a weighted residual mean given by:

$$R = \frac{\sum_{i=1}^n w_i r_i}{n \sum_{i=1}^n w_i}$$

A higher weighting factor should be used for measurement with a higher degree of confidence and vice versa.

Standard Deviation of the Residual - The standard deviation of the residuals measures the spread of the residuals about the residual mean. The standard deviation of residuals is

given by:

$$S = \left[ \frac{\sum_{i=1}^n (r_i - R)^2}{(n-1)} \right]^{\frac{1}{2}}$$

Smaller values of the standard deviation indicate better degree of correspondence than larger values of standard deviation. An objective of calibration is to minimize the standard deviation of the residual. The standard deviation can be weighted to calculate the weighted standard deviation given by:

$$S = \left[ \frac{\sum_{i=1}^n w_i (r_i - R)^2}{(n-1) \sum_{i=1}^n w_i} \right]^{\frac{1}{2}}$$

Spatial or temporal correlation among residuals indicates a systematic trend or bias in the model. The correlation among residuals is inversely proportional to the degree of correspondence.

### 6.8.1 Calibration Sensitivity Analysis

Sensitivity analysis was performed after calibrating the groundwater flow model. The sensitivity of calibration residuals provided a means for assessing the adequacy of the model. One or more of the input variables (hydraulic properties or boundary conditions) were varied to determine the value of the hydraulic heads. Sensitivity analysis test parameter uncertainty and is a quantitative analysis of variability or uncertainty in

aquifer model parameters on the degree of calibration of the model.

Sensitivity analysis plays a key role in the calibration process by identifying those parameters that influence model reliability. Sensitivity analysis is used extensively in inverse techniques to adjust model parameter values.

Model inputs (aquifer properties such as hydraulic conductivity and specific yield) that have been estimated from calibration were varied in a given range. Calibration statistics (maximum residual, minimum residual, standard deviation of residual) were computed subject to the constraints of maximum and minimum water table elevations.

Groundwater Vistas can automatically calibrate model runs from MODFLOW but a more powerful sub-routine, PEST (Parameter Estimation), is better suited for advanced calibration and sensitivity analysis. PEST is the world's most advanced software for model calibration, parameter estimation and predictive uncertainty analysis.

### **6.8.2 Calibration Process**

PEST is a nonlinear parameter estimation package that differs from most other packages. The difference is that PEST can be used to estimate parameters for several existing computer models, whether or not a user has access to the model's source code. PEST is able to "take control" of a model, running it multiple times while adjusting its parameters until the discrepancies between selected model outputs and the field measurements are

reduced to a minimum using the weighted least squares method. PEST communicates with a model through the model's own input and output files. PEST implements a robust variant of the Gauss-Marquardt-Levenberg method of nonlinear parameter estimation.

PEST calibration was performed by allowing values of hydraulic conductivity (K) in Zones 1 to 7 and specific yield ( $S_y$ ) in Zones 1 to 7 to change.

### 6.8.3 Calibration Results

Table 6.6 shows the results of the PEST calibration.

Table 6.6 PEST Calibration Results

| Calibrated Parameter       | Unit   | Zone 1 | Zone 2 | Zone 3 | Zone 4 | Zone 5 | Zone 6 | Zone 7 |
|----------------------------|--------|--------|--------|--------|--------|--------|--------|--------|
| Hydraulic Conductivity (K) | ft/day | 150    | 50     | 100    | 200    | 25     | 250    | 300    |
| Specific Yield ( $S_y$ )   |        | 0.100  | 0.150  | 0.200  | 0.200  | 0.200  | 0.100  | 0.100  |
| Initial Parameter          |        |        |        |        |        |        |        |        |
| Hydraulic Conductivity (K) | ft/day | 135    | 50     | 125    | 200    | 20     | 270    | 300    |
| Specific Yield ( $S_y$ )   |        | 0.150  | 0.150  | 0.200  | 0.200  | 0.200  | 0.200  | 0.100  |

Table 6.7 below shows the calibration and verification periods in days and years. The calibration period is from October 1, 1974 to September 30, 2000. The verification period is from October 1, 2000 to June 30, 2006.

Table 6.7 Calibration and Verification Periods

| Time<br>(days) | Date         | Time<br>(days) | Date         | Stress<br>Period | Date         |
|----------------|--------------|----------------|--------------|------------------|--------------|
| 0              | Oct. 1,1974  | 9490           | July 1,2000  | 11041.25         | Oct. 1,2004  |
| 91.25          | Jan. 1,1975  | 9581.25        | Oct. 1,2000  | 11132.5          | Jan. 1,2005  |
| 182.5          | April 1,1975 | 9672.5         | Jan. 1, 2001 | 11223.75         | April 1,2005 |
| 273.75         | July 1,1975  | 9763.75        | April 1,2001 | 11315            | July 1,2005  |
| 365            | Oct. 1,1975  | 9855           | July 1,2001  | 11406.25         | Oct. 1,2005  |
| 456.25         | Jan. 1,1976  | 9946.25        | Oct. 1,2001  | 11497.5          | Jan. 1,2006  |
| 547.5          | April 1,1976 | 10037.5        | Jan. 1,2002  | 11588.75         | April 1,2006 |
| 638.75         | July 1,1976  | 10128.75       | April 1,2002 | 11680            | July 1, 2006 |
| 730            | Oct. 1,1976  | 10220          | July 1,2002  | 11771.25         | Oct.1, 2006  |
| 821.25         | Jan. 1,1977  | 10311.25       | Oct. 1,2002  | 11862.5          | Jan. 1, 2007 |

Figures 6.5, 6.6 and 6.7 below show transient calibration results at monitoring wells Z1000006, W10OSMW1 and Z1000008 respectively.

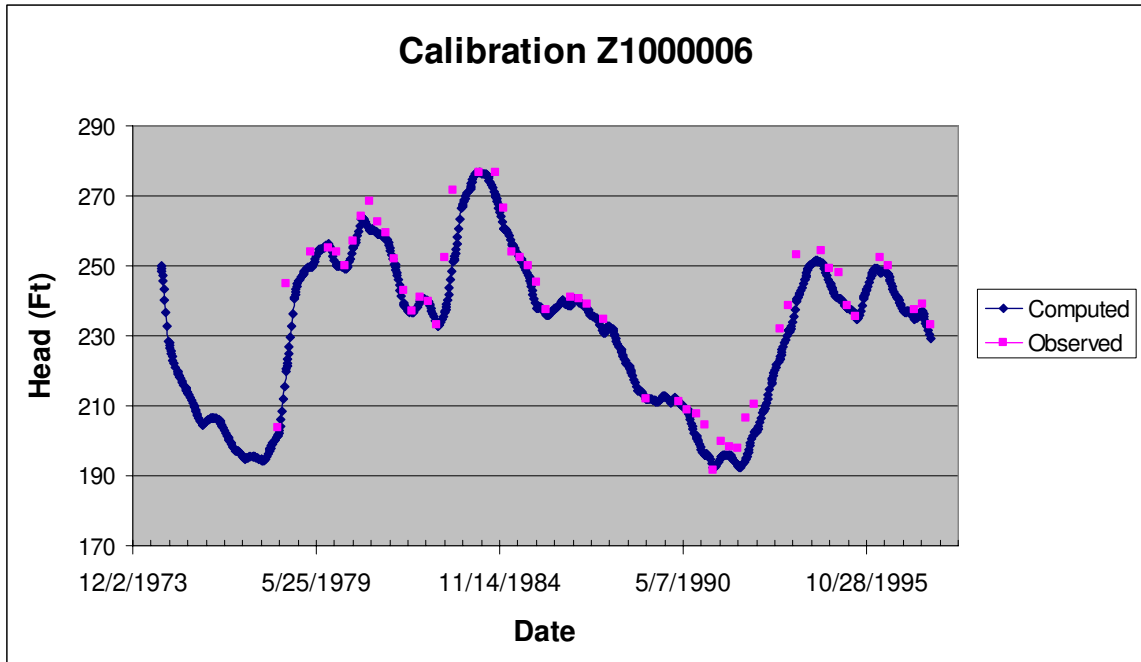


Figure 6.5 Transient Calibration at Monitoring Well Z1000006

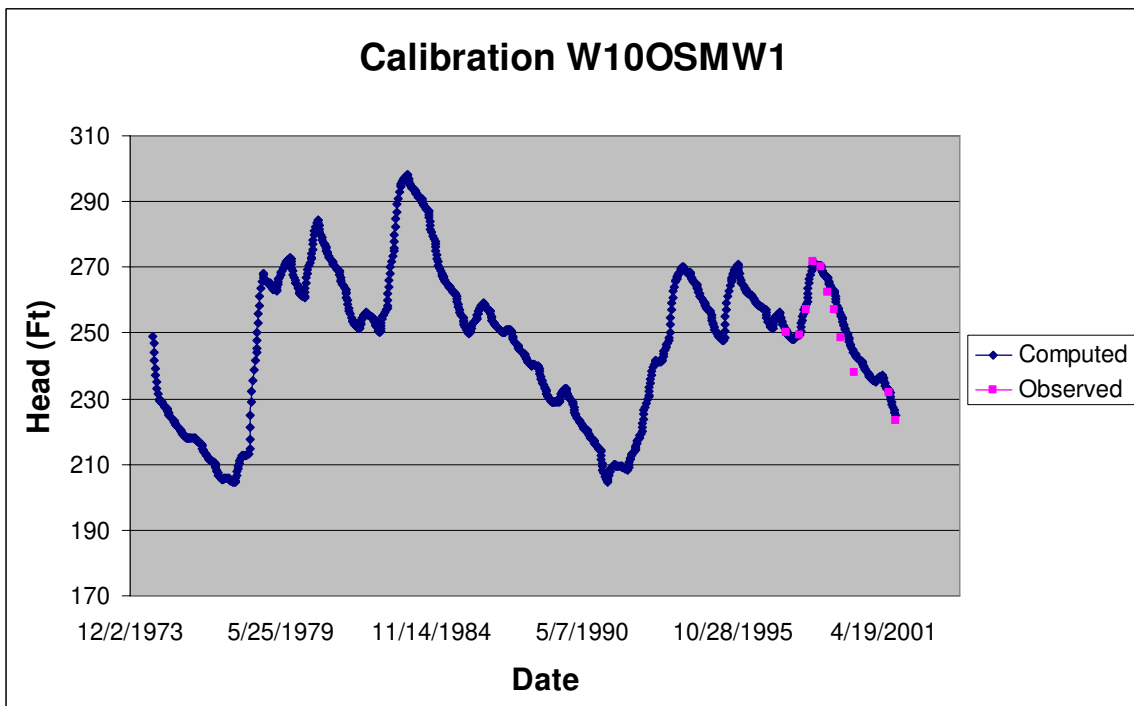


Figure 6.6 Transient Calibration at Monitoring Well W10OSMW3

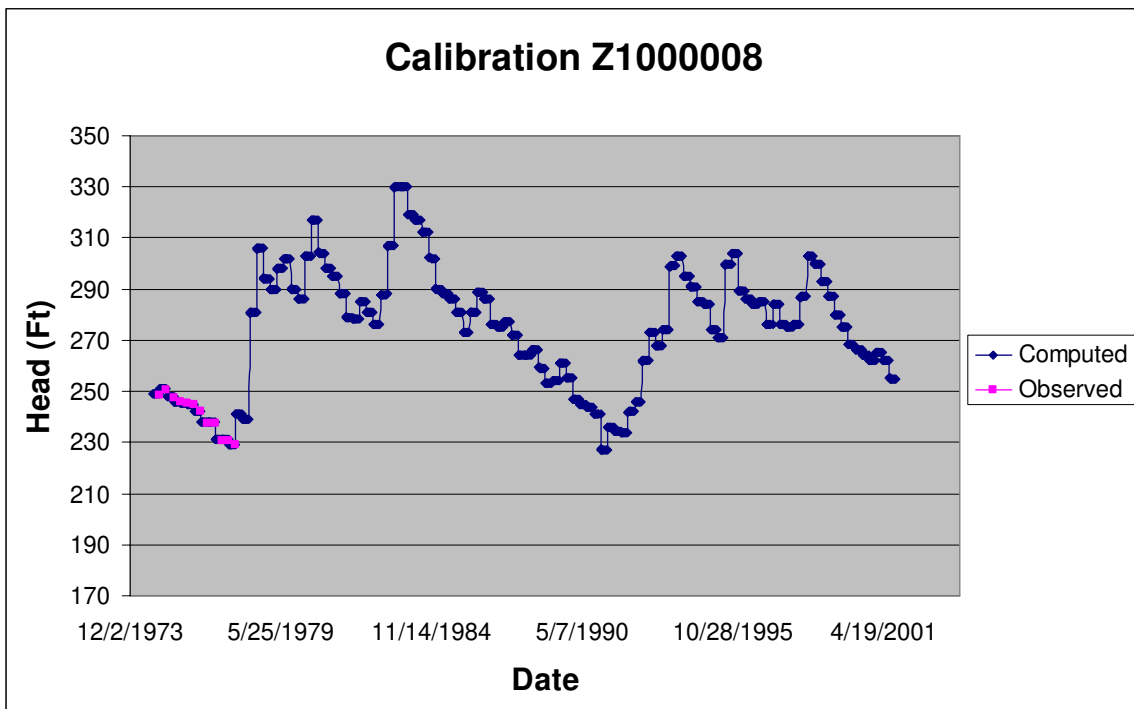


Figure 6.7 Transient Calibration at Monitoring Well Z1000008

## 6.9 Model Verification

The calibrated groundwater simulation model was run from October 1, 2000 to June 30, 2006 and target water levels verified. Model verification provided a testing phase for the calibrated model. The calibrated model closely replicated the verification target water levels as shown in the figures below.

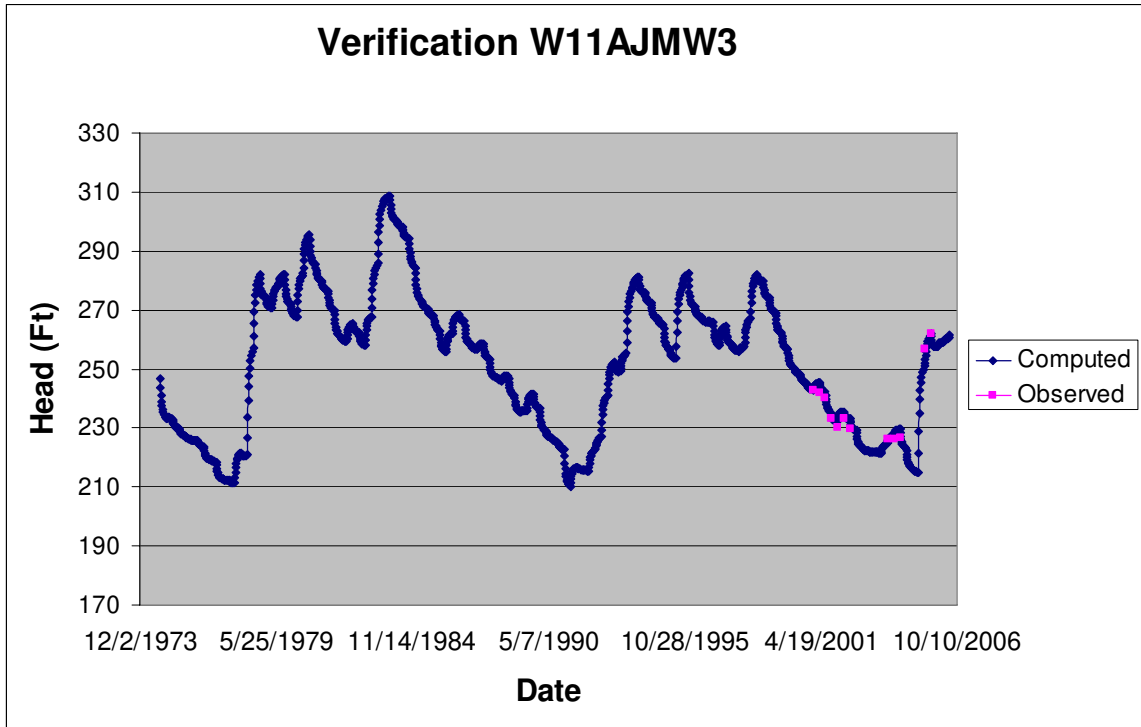


Figure 6.8 Transient Verification at Monitoring Well W11AJMW3

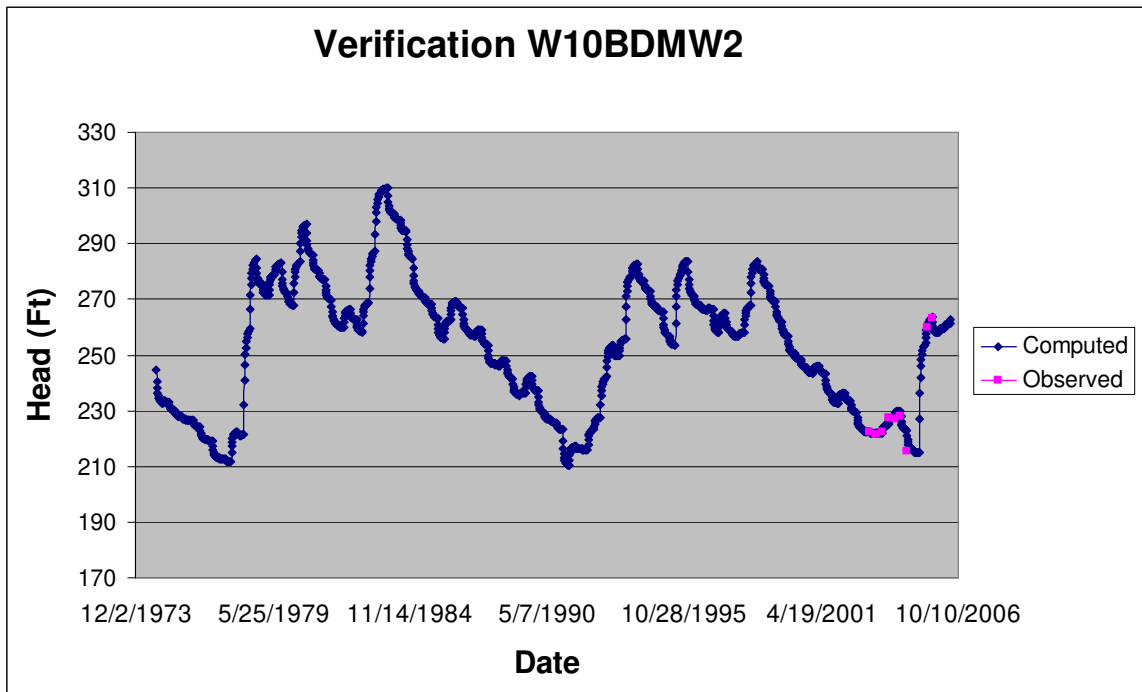


Figure 6.9 Transient Verification at Monitoring Well W10BDMW2

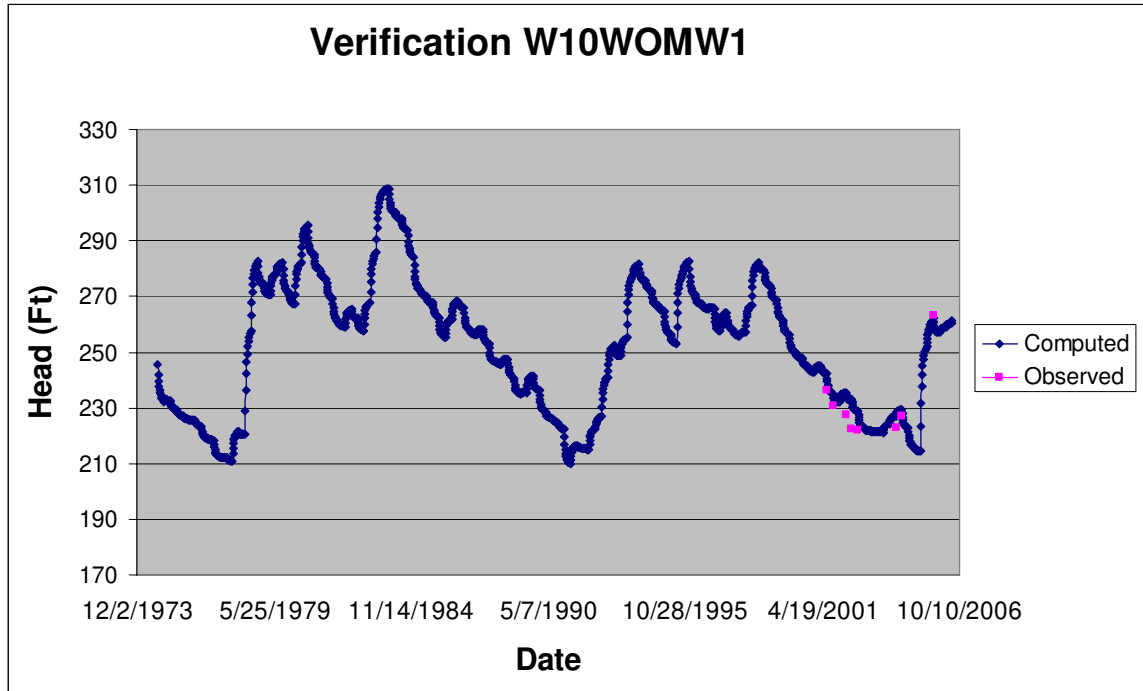


Figure 6.10 Transient Verification at Monitoring Well W10WOMW1

## 6.9 Management Model

A hydraulic gradient control scheme is developed to remove the PCE, TCE and PCR contaminants in the groundwater. A multi-stage procedure described in Chapter 1 was used to successfully select best wells and their optimal pumping and recharge rates to contain and extract the plume below the MCL.

### 6.9.1 State Variables

Assessing the dynamic state of the system is important and an essential step in contaminant plume cleanup. State variables represent the changing dynamics of the system as a result of the change in decision variables and the effect on the specified constraints. The head,  $h$ , PCE, TCE and PCR concentrations are such variables that

depend on decisions and specified constraints.

### 6.9.2 Management Decision Variables

The contaminant plume in the Baldwin Park Operable Unit is moving south west.

Several water saving alternatives were examined to satisfy the water demand in the basin.

The following decision variables were considered; pumped water, imported water, recharged water and reclaimed water rates. The management model addressed the quantity of groundwater extracted, recharged, imported and reclaimed and type of users.

Tables 6.8 list the decision variables employed by the optimization model.

| <b>Table 6.8 Decision Variable Summary</b> |              |                                     |
|--------------------------------------------|--------------|-------------------------------------|
| Variable                                   | Variable No. | Variable Representation             |
| Pumping Wells<br>$P(l,t)$                  | (1- 18)      | $P(l,t) \ l=1, 2,3 \dots 18; t=1$   |
| Imported Water<br>$IMP(l,t)$               | (19)         | $IMP(l,t) \ l=1; t=1$               |
| Reclaimed water<br>$QT(l,t)$               | (20-21)      | $RC1(l,t) \ l=1,2; t=1$             |
| Recharge Water<br>$R1(l,t)$                | (22- 27)     | $R1(l,t) \ l=1, 2, 3, \dots 6; t=1$ |

The variables are described below:

1. *Pumped water  $P(I,T)$*

The variable  $P(I,T)$  is contaminated water extracted from the groundwater basin and treated to satisfy water supply and M&I demand. Pumping rates are varied.

Decision variable is generally  $P(I,T)$ ,  $I = 1,2,...,18$  and  $T=1,2,3...n$ . In this study, only one management period ( $T=1$ ) is considered during the optimization.

## *2. Imported water $IM(I,T)$*

The management model uses one source for imported water (MWD). Decision variable is  $IM(I,T)$ ,  $I=1$  and  $T=1$  denotes the annual volume of imported water used to satisfy potable water and agricultural demand.

## *3. Reclaimed Water $QT(I,T)$*

Decision variable for reclaimed water is  $QT(I,T)$  with  $I=1,2$  and  $T=1$ . Reclaimed water is groundwater that is extracted and treated by treatment facilities. Some of the treated reclaimed water is used for aquifer recharge in the hydraulic control wells. The remainder is used for water supply and M&I offsite.

## *4. Recharged water $R(I,T)$*

Recharge water is reclaimed water used to create the hydraulic gradient control required to stabilize and cleanup the PCE, TCE and PCR plumes. Decision variable  $R(I,T)$  denotes the rate of water recharged to the hydraulic gradient control wells.  $I = 1,2,3$  and  $T=1$ .

### **6.9.3 Management Constraints**

Several mitigating factors influence management decisions. The management model is subject to several constraints including system, operational, institutional, environmental,

economic and statutory constraints. These limitations impose operational, economic, and functional restrictions on the proposed model. Operational constraints for this study considered maximum and minimum values of the decision variables and combinations of the decision variables. Flow rates are limited by safe yield, well capacity, water supply and water quality constraints. Groundwater extraction is limited by the terms of the Baldwin Park Operable Unit judgment. The judgment restricts the amount of groundwater that can be pumped each year. The restricted amount is less than the safe yield. Water availability and distribution system capacity constraints also limit imported water amounts. Reclaimed water amounts are also constrained by extraction rates and treatment facility constraints. Water supply and demand constraints are also considered. The water quality constraint is assumed satisfied when the PCE and TCE concentration contours are less than the maximum contaminant level (MCL=5 µg/l) and when the PCR concentration contours are less than the advisory notification level of 6 µg/L established by the California Department of Health Services (CDHS).

#### **6.9.4 Objective Function**

The objective function minimizes the total annual energy, operational and maintenance cost. The total cost includes cost for pumping, recharge and imported water used to maintain the existing system and the hydraulic gradient system. Costs for the hydraulic gradient control system are reduced by utilizing existing pumping, off-line or standby wells. When new injection or pumping wells are added, the additional capital cost  $P$ , is converted to an annual cost  $A$  by including the economics formula  $A^*(P/A, A, i)$  in equation 5.1. The term  $i$ , is the effective interest rate. The objective function, which

analyzes operation and maintenance cost is defined as:

$$\text{Min (Cost)} = \text{Min} \sum (a * P(I, T) + b * IM(I, T) + c * RC(I, T) + d * R(I, T)) \quad (6.1)$$

Subject to: all constraints.

Where:

$P(I, T)$  = annual volume of water extracted from the groundwater basin from well I during period T.

$IM(I, T)$  = annual volume of imported water consumed by user I during the period T.

$QT(I, T)$  = annual volume of reclaimed water from plume extraction consumed by user I during the period T.

$R(I, T)$  = annual volume of recharged into the hydraulic gradient control location I from source J during the period T.

$a(I, T)$  = cost of pumping of one gpm of water 300 feet for use by source I during the year T.

$b(I, T)$  = cost of importing one gpm of water from source I, during the period T.

$c(I, T)$  = cost of treating one gpm of reclaimed water extracted from the plume, at plant I during period, T.

$d(I, T)$  = operation and maintenance cost to recharge of one gpm of water through well I, during period T.

### **6.9.5 Optimization – LINDO**

The optimization process has been described in Chapter 5. The MODFLOW module from Groundwater Vistas runs the simulation model. The output from the simulation model is fed into LINDO. LINDO tests new policies of decision variables for feasibility and optimality during several iterations. LINDO outputs the solution to the objective function. Constraints are assessed in the post-optimization phase to determine whether they have been satisfied. LINDO outputs optimal decisions by minimizing cost of operation and maintenance. The plume boundary is delineated using RT3D and the water quality constraint (MCL) is analyzed. Optimal solution is achieved when the decision variables that attained minimum cost also satisfy the water quality constraint (MCL or NL).

## **Chapter 7**

### **Case Study Results**

This chapter discusses the results of the case study presented in the chapter 6. The focus of this study was to determine the origin of the contaminants TCE, PCE and PCR, predict the fate and transport of the contaminants PCE, TCE and PCR, develop the groundwater flow model using MODFLOW, the particle tracking model using MODPATH and the solute transport model using RT3D. Then “layer” on the optimization code to simulate several cleanup scenarios. Use the groundwater flow model, particle tracking, solute transport models and optimization models to determine the optimum cleanup schemes for PCE, TCE and PCR in BPOU.

An optimized hydraulic gradient control scheme was devised to clean up the PCE, TCE and PCR in the BPOU in the San Gabriel Valley Basin. The output of the simulation and optimization models is discussed. The management model is analyzed to determine whether the results reflect optimal policies for the given objective and constraints.

#### **7.1 Results of Simulation Model**

The Baldwin Park Operable Unit was selected as suitable case study primarily due to the availability of data from the USEPA database and to develop and test the research objectives. Assumptions and approximations made in this study were consistent with the principles of groundwater flow and contaminant transport. The simulation models were calibrated by replicating head targets within the contaminant plume. Generally, the

system output obeyed mass balance equations but the potential for minor errors exist due to unreported extractions and subsurface inflow and outflow.

The transient simulation model was run for 150 quarterly stress periods. Each simulation period was divided into 10 time steps. The maximum number of iterations per time step is 50. The flow model was calibrated to match existing historical observations. Figure 7.1 shows MODFLOW head contours at the end of stress period 127 (June 30, 2006). The head contours from the MODFLOW simulations reveal that flow is southwest.

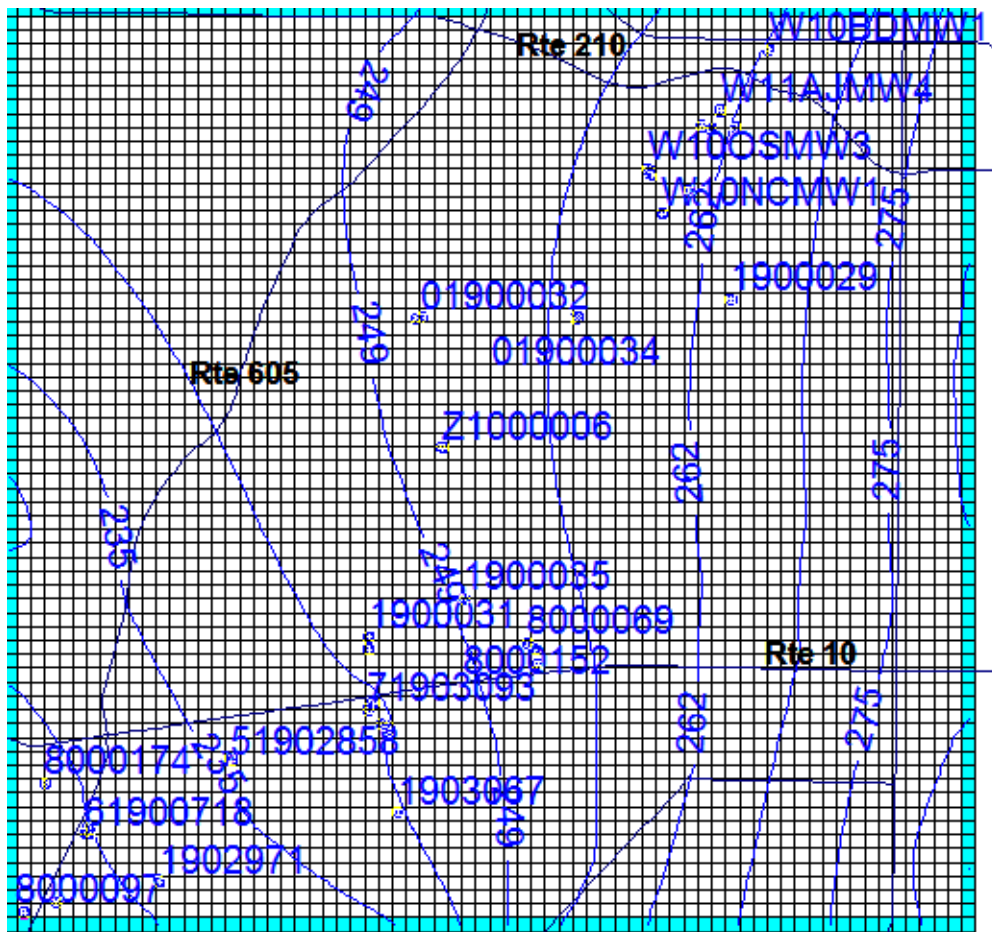


Figure 7.1 Head Contours at the end of June 30, 2006- Layer 1

## **7.2 Particle Tracking Model - MODPATH Results**

MODPATH was run to delineate contaminant flow paths. RT3D was run to simulate contaminant transport and generate contaminant concentration contours.

Figure 7.2 shows forward tracking MODPATH results for layer 1. Figure 7.3 shows forward tracking MODPATH results for layer 2. MODPATH computes paths for contaminant particles and keeps track of the travel time of the particles. In the forward tracking mode, MODPATH delineates the path of a contaminant particle from its source to a sink (pumping well) or a boundary. A line of 20 particles was added to the east side from north to south. The particles move south-west delineating flow paths.

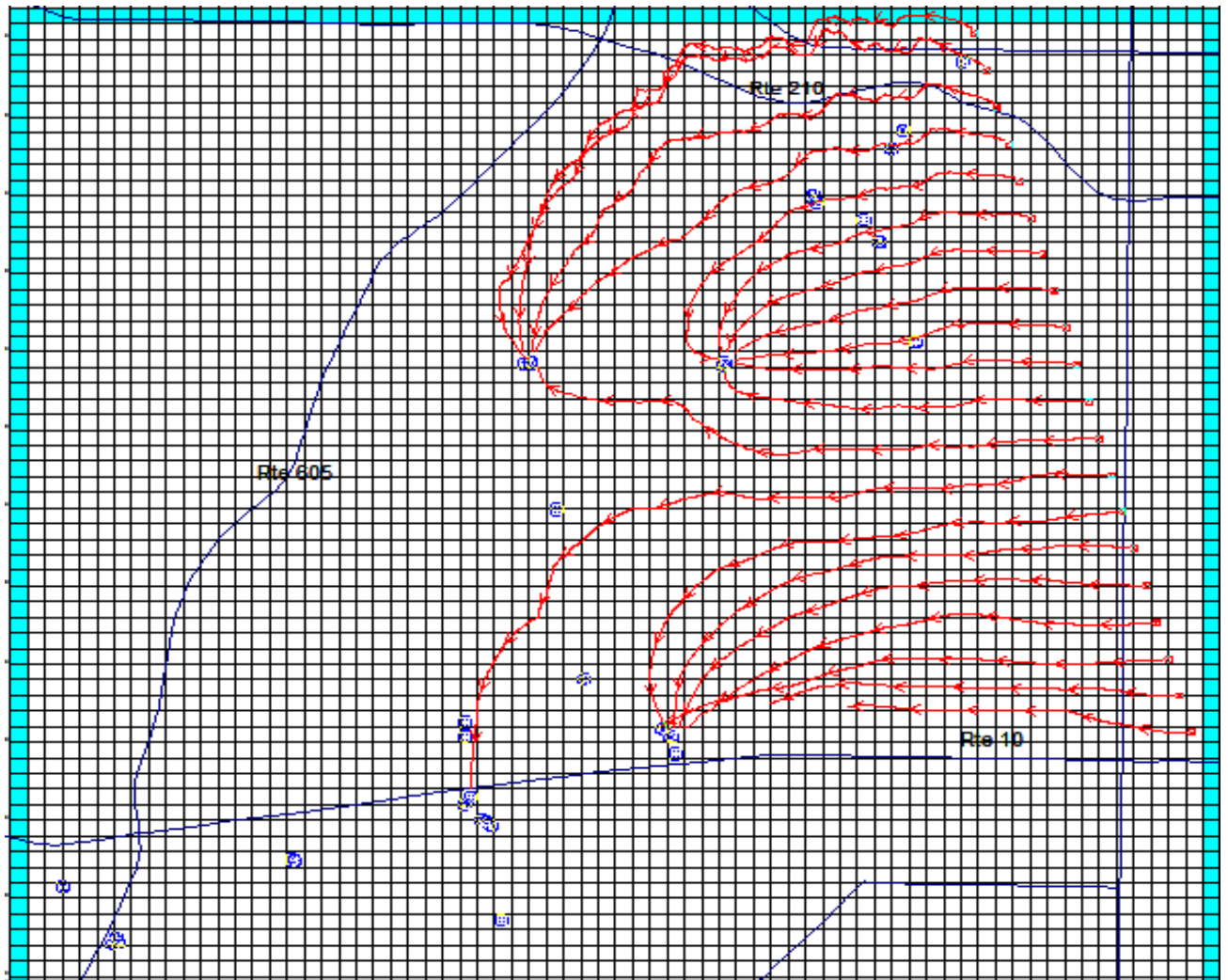


Figure 7.2 MODPATH Pathlines at the end of June 30, 2006 - Layer 1

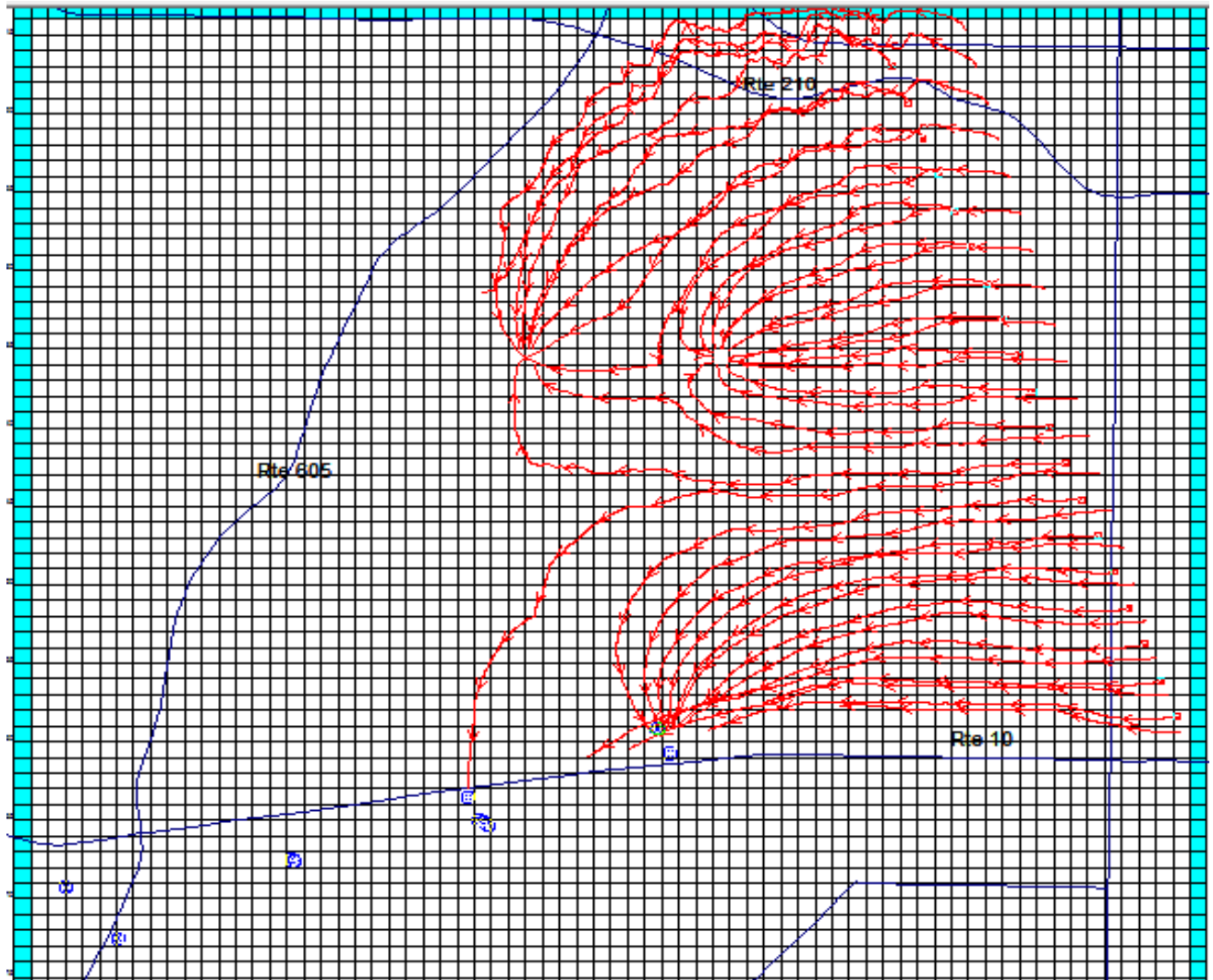


Figure 7.3 MODPATH Pathlines at the end of June 30, 2006 - Layer 2

### 7.3 Developing the Hydraulic Gradient Scheme

The hydraulic gradient scheme is developed by studying the contaminant flow and determining optimum locations of the pumping and recharge wells. Utilizing existing pumping wells inside the plume is cost effective. Standby or offline wells can also be utilized. If an existing well is not available, a new well must be constructed to achieve optimal cleanup. Recharge wells are located in an arc around the outer edge of the

plume. The rate of pumping at pumping well P1 was increased to 300,000 ft<sup>3</sup>/day (and 450,000 ft<sup>3</sup>/day alternately) to counteract the effects of injection wells I1, I2 and I3 injecting at 100,000 ft<sup>3</sup>/day (and 150,000 ft<sup>3</sup>/day alternately). Thus the total rate of injection at the three injection wells is equal to the total rate of pumping at the centralized production well forming a recycle mode.

#### **7.4 Contaminant Transport Model - RT3D Results**

Initial concentrations of PCE, TCE and PCR were input in Groundwater Vistas. Most of the PCE, TCE and PCR have migrated south west and are prevalent in Layers 1 and 2. Layer 3 does not contain any contaminants. The contaminant transport model was calibrated from October 1, 1974 to June 30, 2002. The contaminant transport model was verified from July 1, 2002 to June 30, 2006. Figures 7.4, 7.5 and 7.6 show results for contaminant calibration and verification for PCE, TCE and PCR respectively.

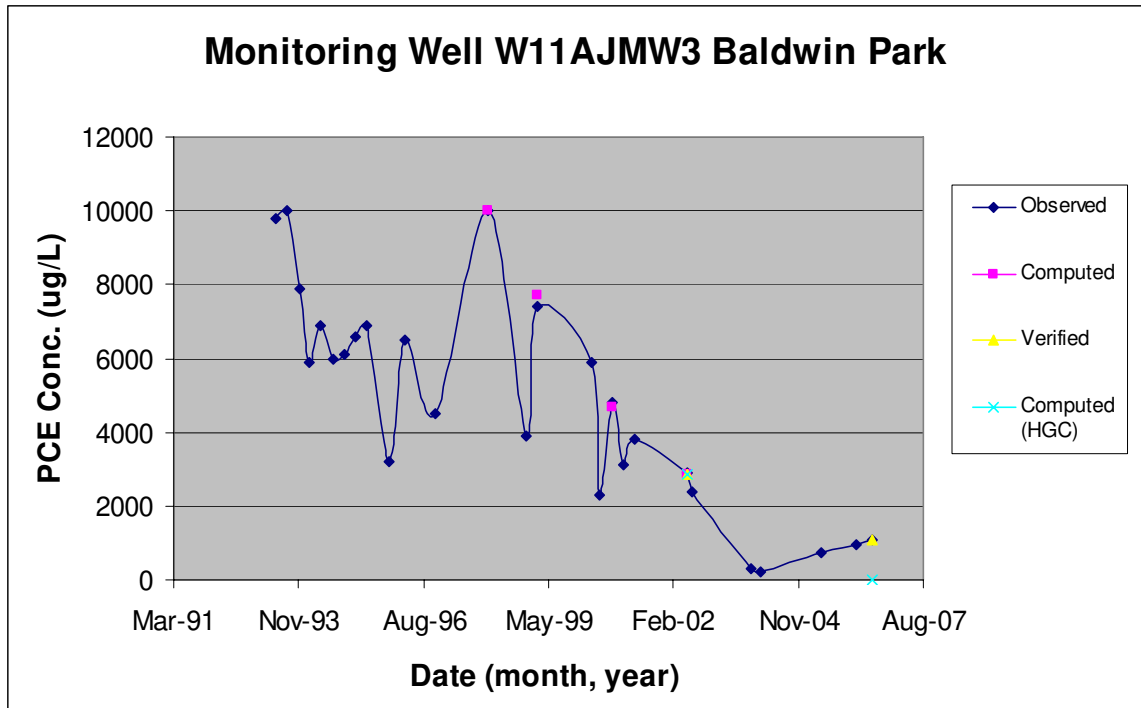


Figure 7.4 Contaminant Transport Calibration and Verification - PCE

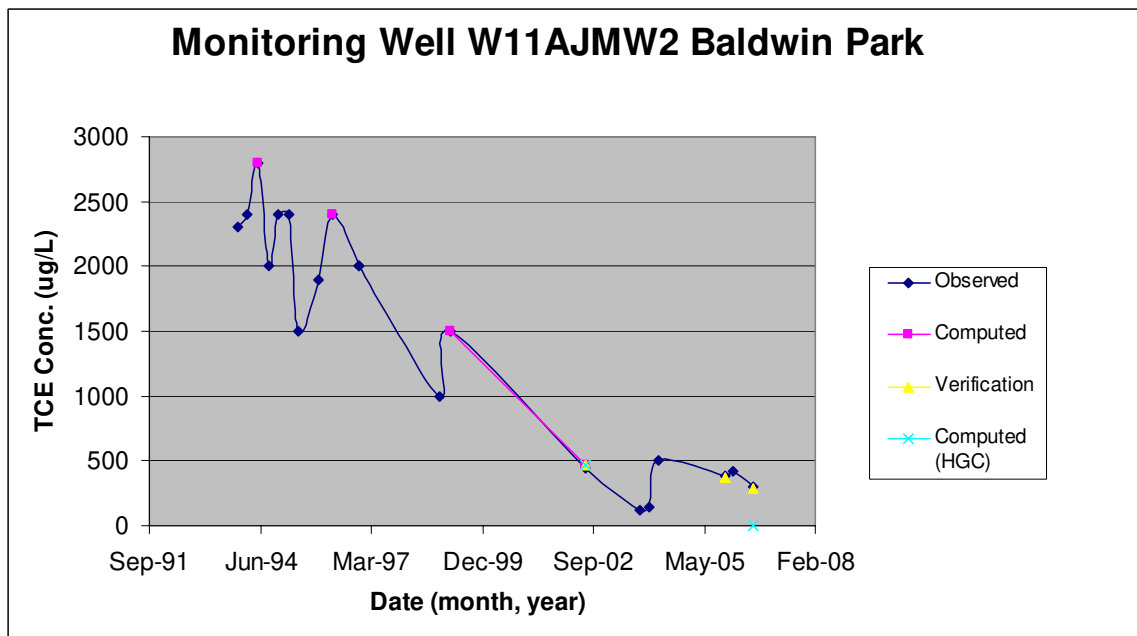


Figure 7.5 Contaminant Transport Calibration and Verification – TCE

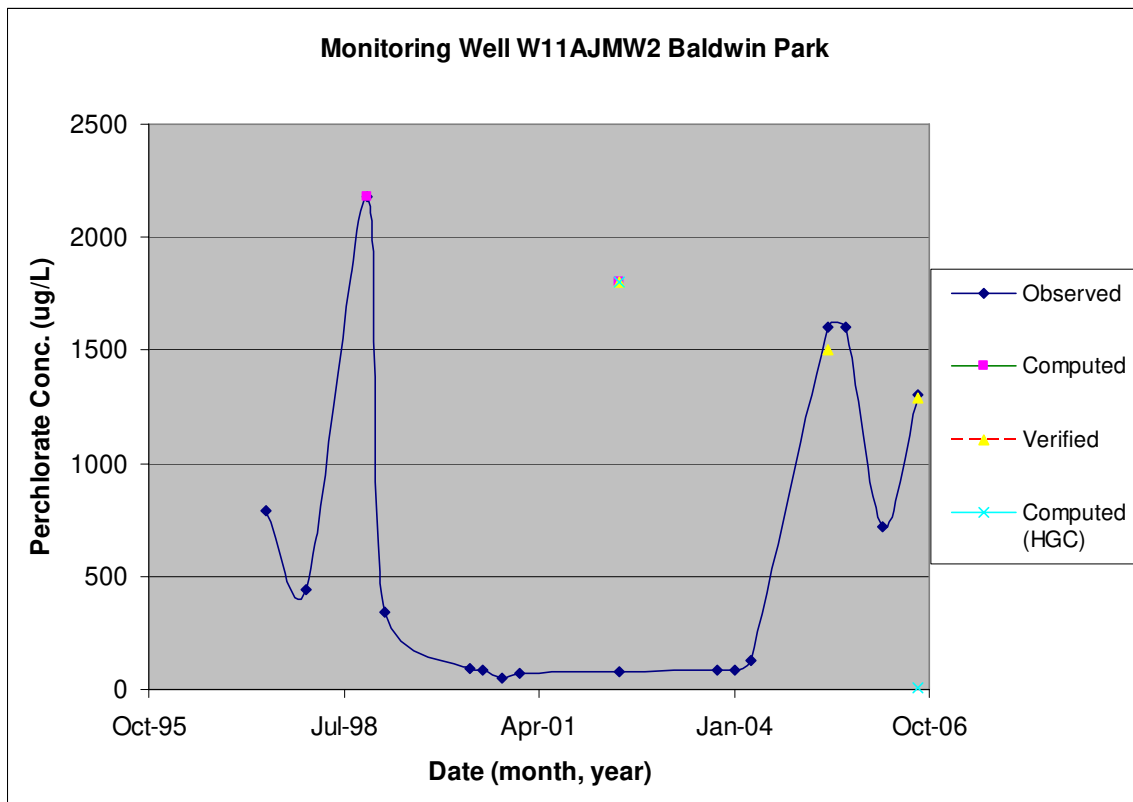


Figure 7.6 Contaminant Transport Calibration and Verification – PCR

The contaminant concentrations for the stress period ending in June 30, 2006 served as base line concentrations for the application of the hydraulic gradient scheme for future scenarios. Future scenarios were run from July 1, 2006 to June 30, 2011 to compare with Watermaster’s projected results. Figures 7.7 and 7.8 show the PCE concentration at the end of June 2006 in layers 1 and 2 respectively. Figures 7.9 and 7.10 show the TCE concentration at the end of June 2006 in layers 1 and 2 respectively. Figures 7.11 and 7.12 show the PCR concentration at the end of June 2006 in layers 1 and 2 respectively. Figures 7.13, 7.14 and 7.15 show the head contours at the end of June 2010 for layers 1, 2 and 3 respectively. Figures 7.16 and 7.17 show the PCE concentration at the end of June 2010 in layers 1 and 2 respectively. Figures 7.18 and 7.19 show the TCE concentration at the end of June 2010 in layers 1 and 2 respectively. Figures 7.20 and 7.21 show the

PCR concentration at the end of June 2010 in layers 1 and 2 respectively. The hydraulic gradient scheme developed in this scheme reduces the PCE and TCE concentrations below the MCL of 5 µg/L and the PCR concentration below the NL of 6 µg/L in 4 years. Given the Watermaster's projection of a 60% contaminant concentration reduction by 2011, it will take Watermaster an additional 25 years to bring current contaminant levels below MCL or NL.

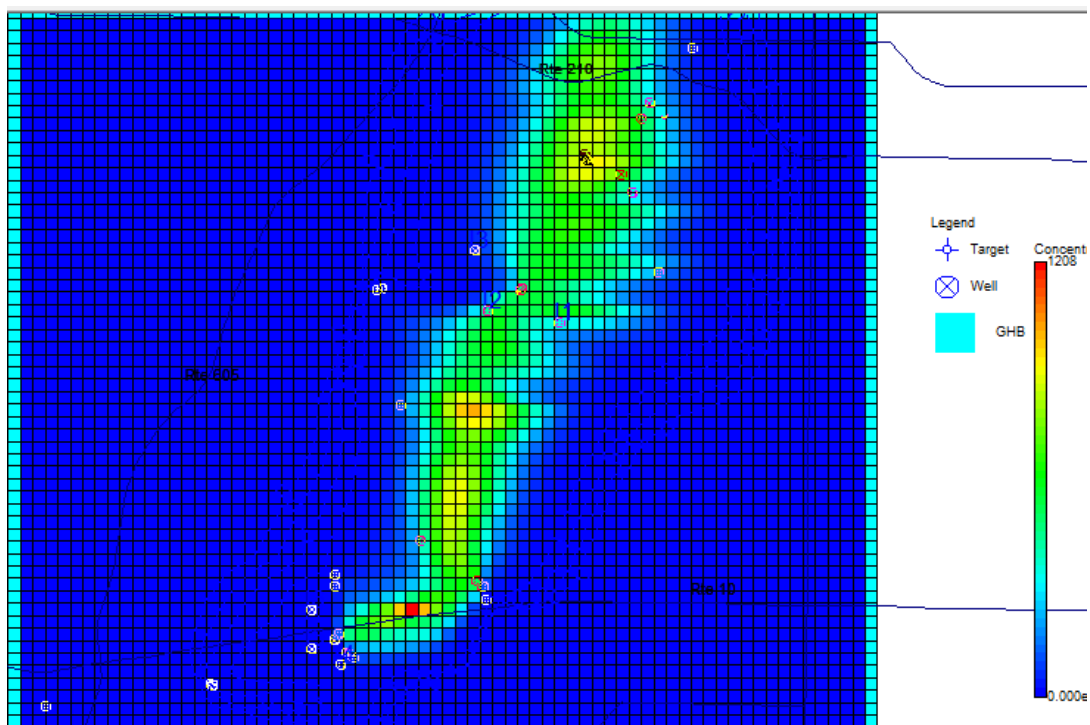


Figure 7.7 PCE Concentration Contours (µg/L) at the end of June 2006 – Layer 1

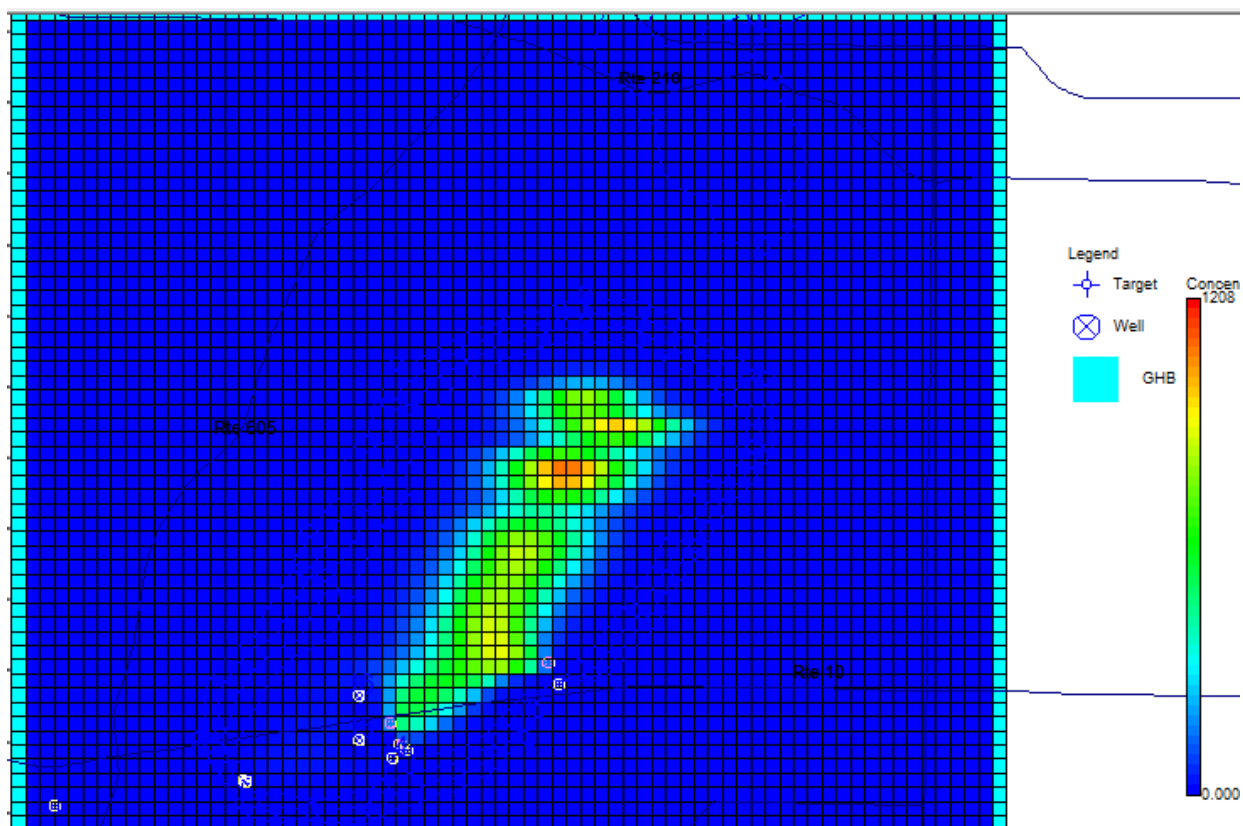


Figure 7.8 PCE Concentration Contours ( $\mu\text{g/L}$ ) at the end of June 2006 – Layer 2

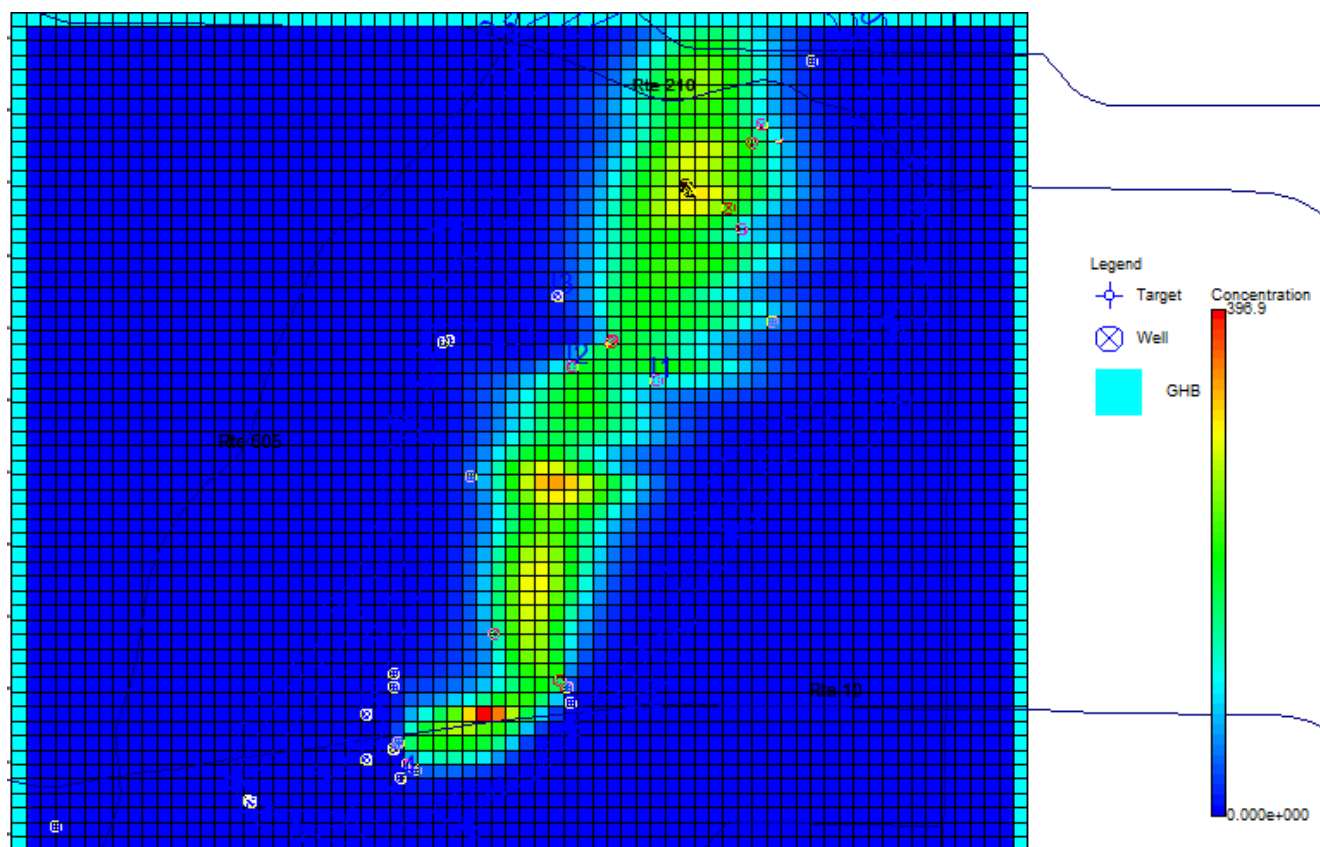


Figure 7.9 TCE Concentration Contours ( $\mu\text{g/L}$ ) at the end of June 2006 – Layer 1

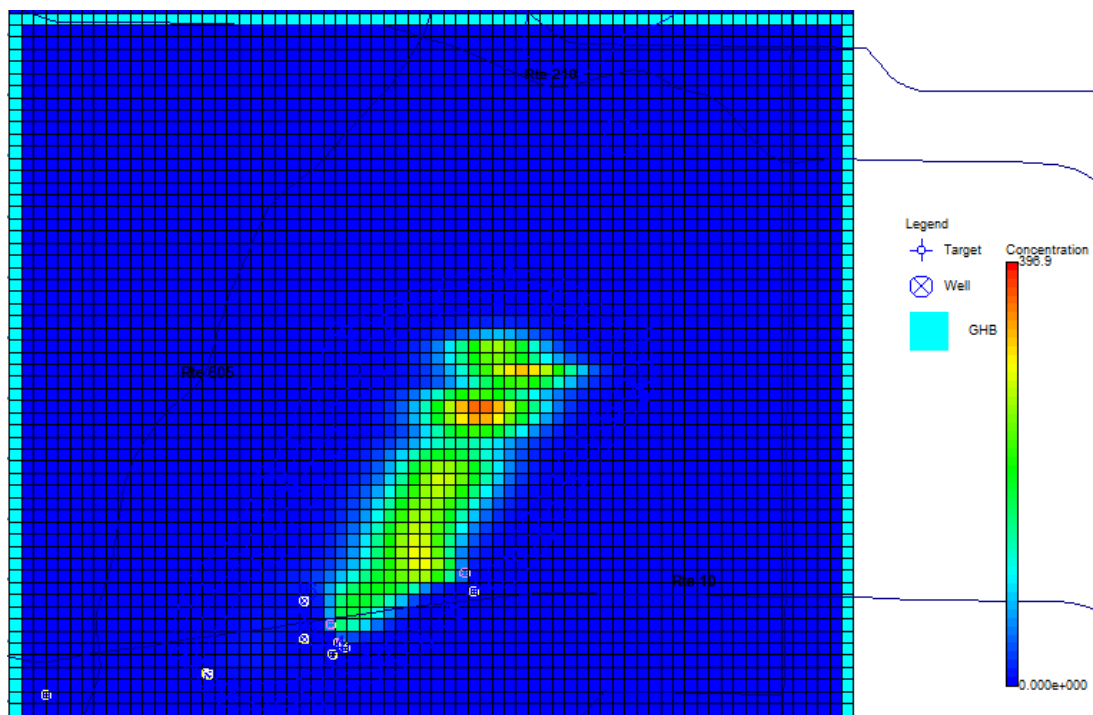


Figure 7.10 TCE Concentration Contours (µg/L) at the end of June 2006 – Layer 2

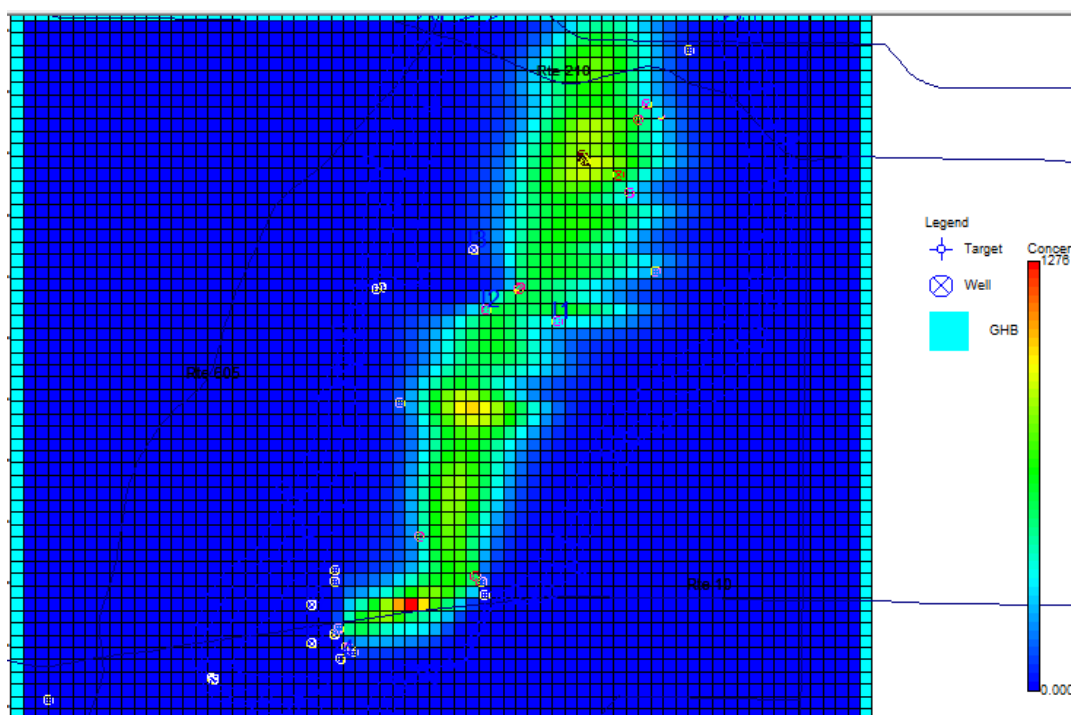


Figure 7.11 PCR Concentration Contours (µg/L) at the end of June 2006 – Layer 1

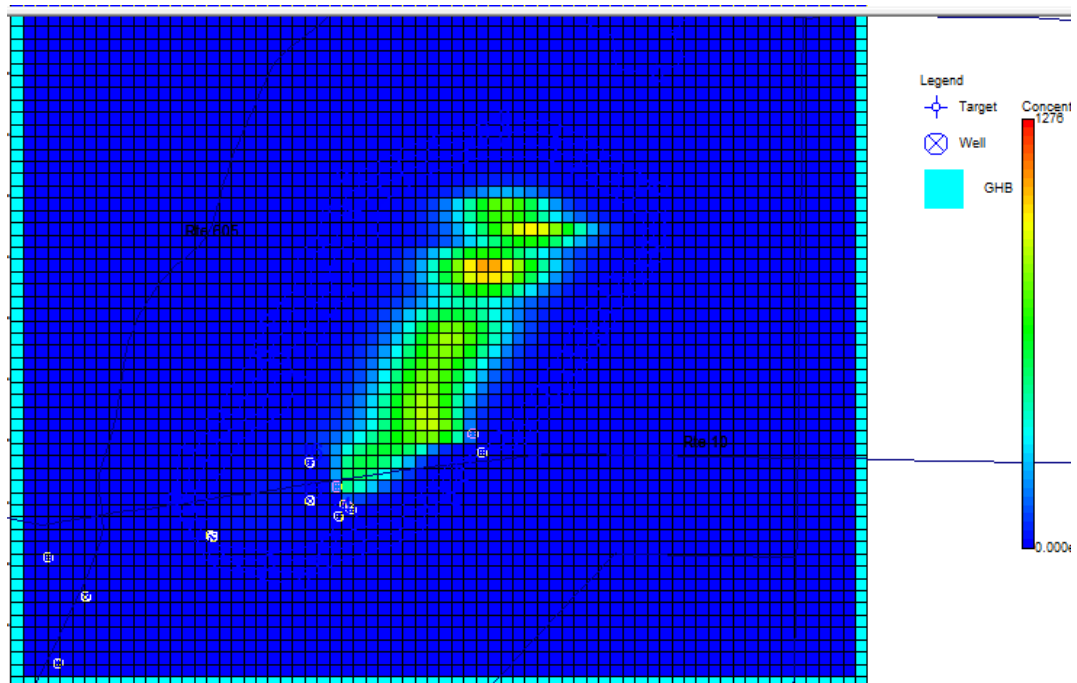


Figure 7.12 PCR Concentration Contours ( $\mu\text{g/L}$ ) at the end of June 2006 – Layer 2

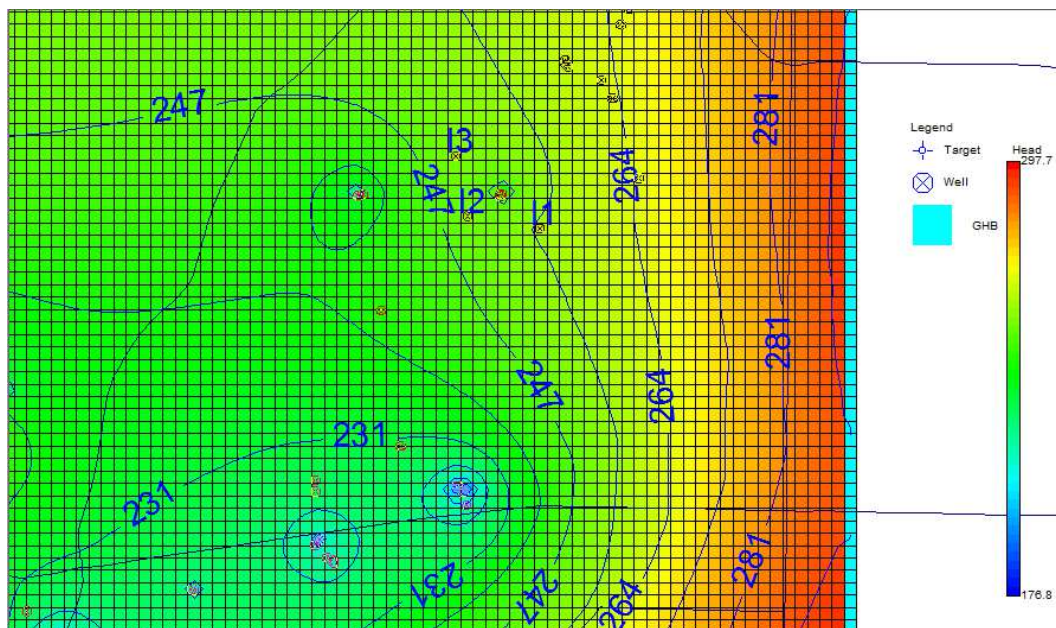


Figure 7.13 Head Contours (ft) at the end of June 2010 Layer 1

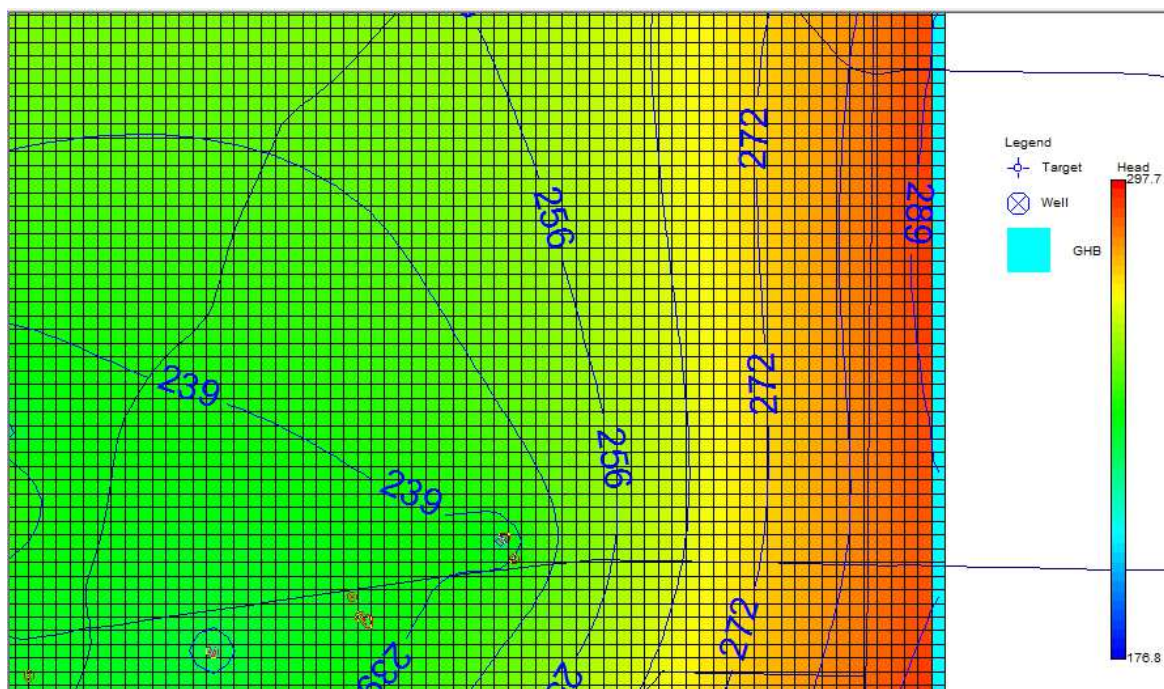


Figure 7.14 Head Contours (ft) at the end of June 2010 - Layer 2

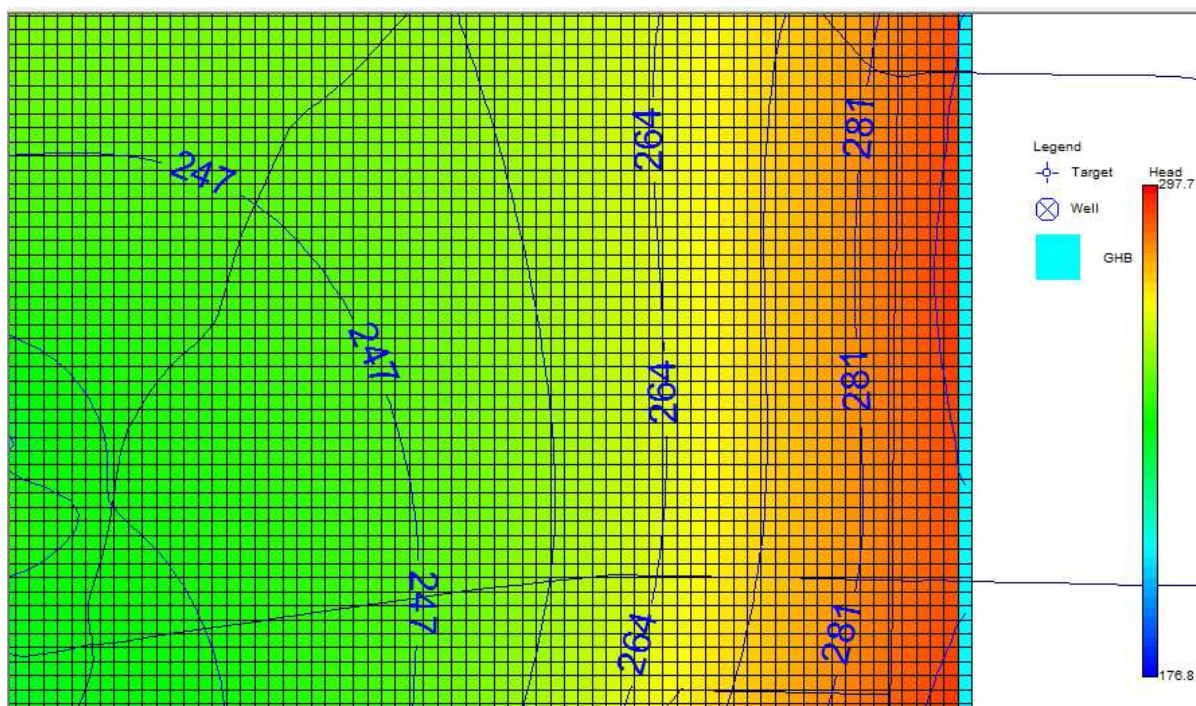


Figure 7.15 Head Contours (ft) at the end of June 2010 - Layer 3

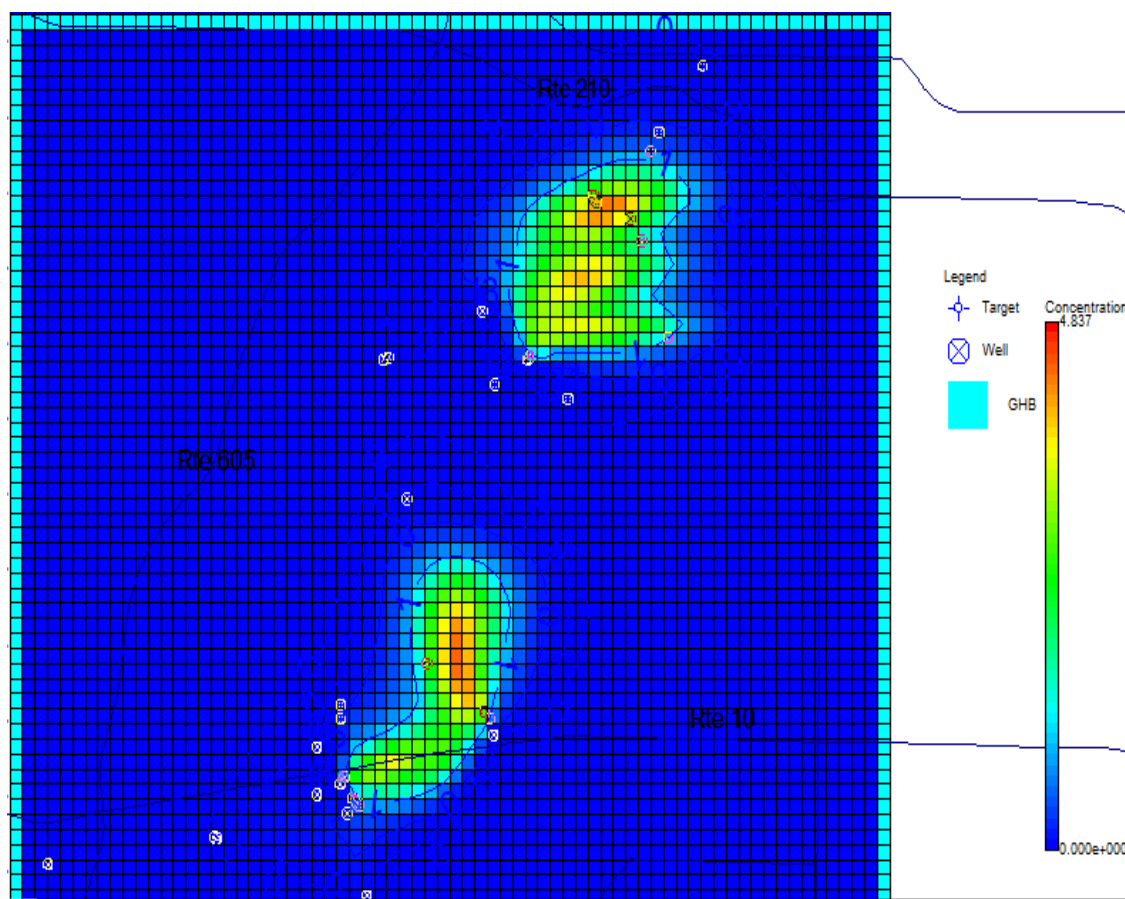


Figure 7.16 PCE Concentration Contours ( $\mu\text{g/L}$ ) at the end of June 2010 - Layer 1.

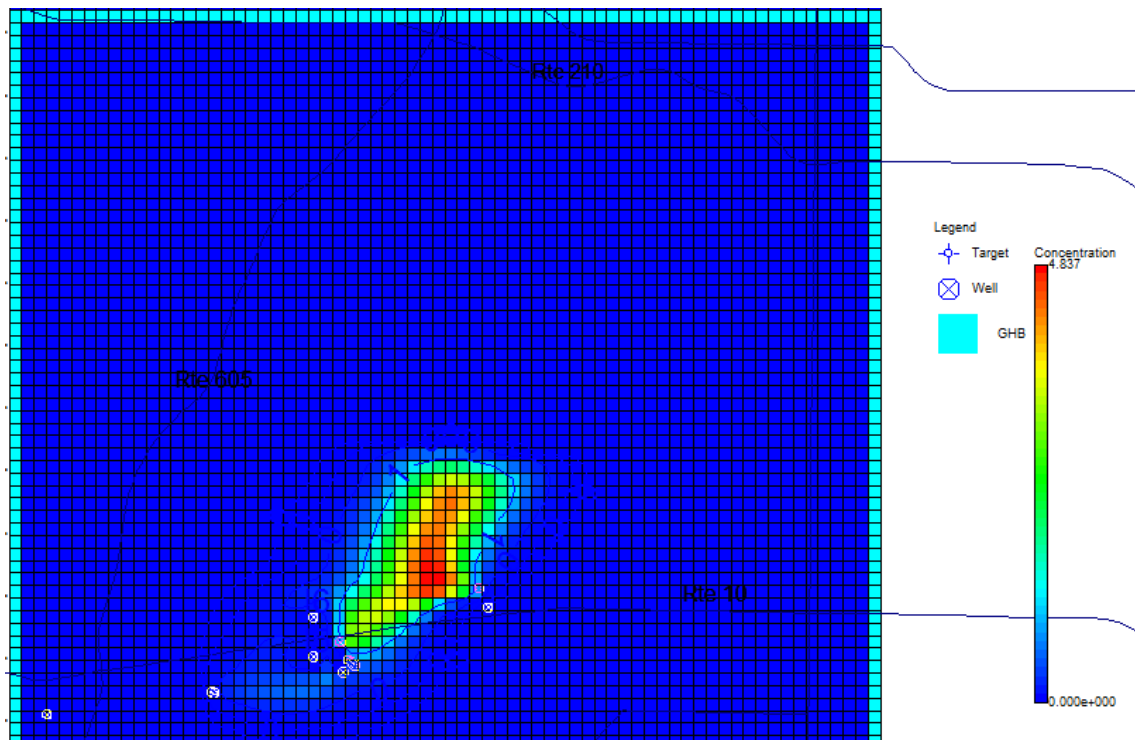


Figure 7.17 PCE Concentration Contours ( $\mu\text{g/L}$ ) at the end of June 2010 - Layer 2

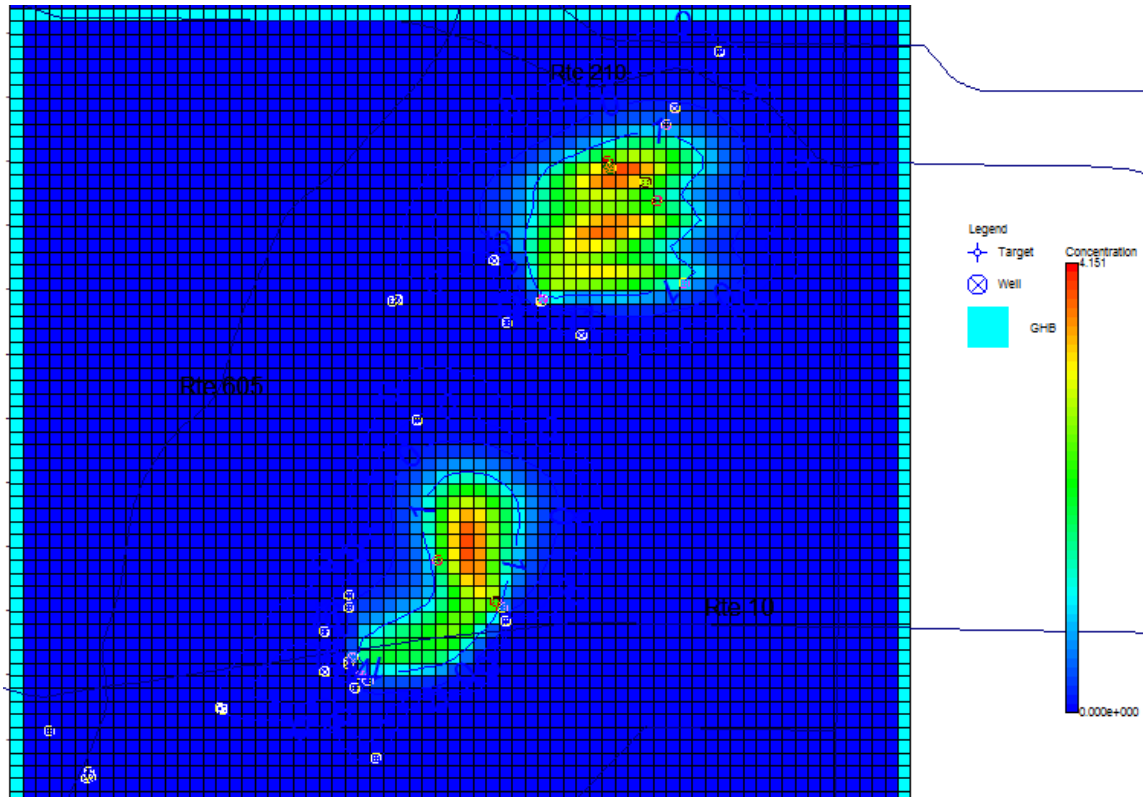


Figure 7.18 TCE Concentration Contours ( $\mu\text{g/L}$ ) at the end of June 2010 – Layer 1

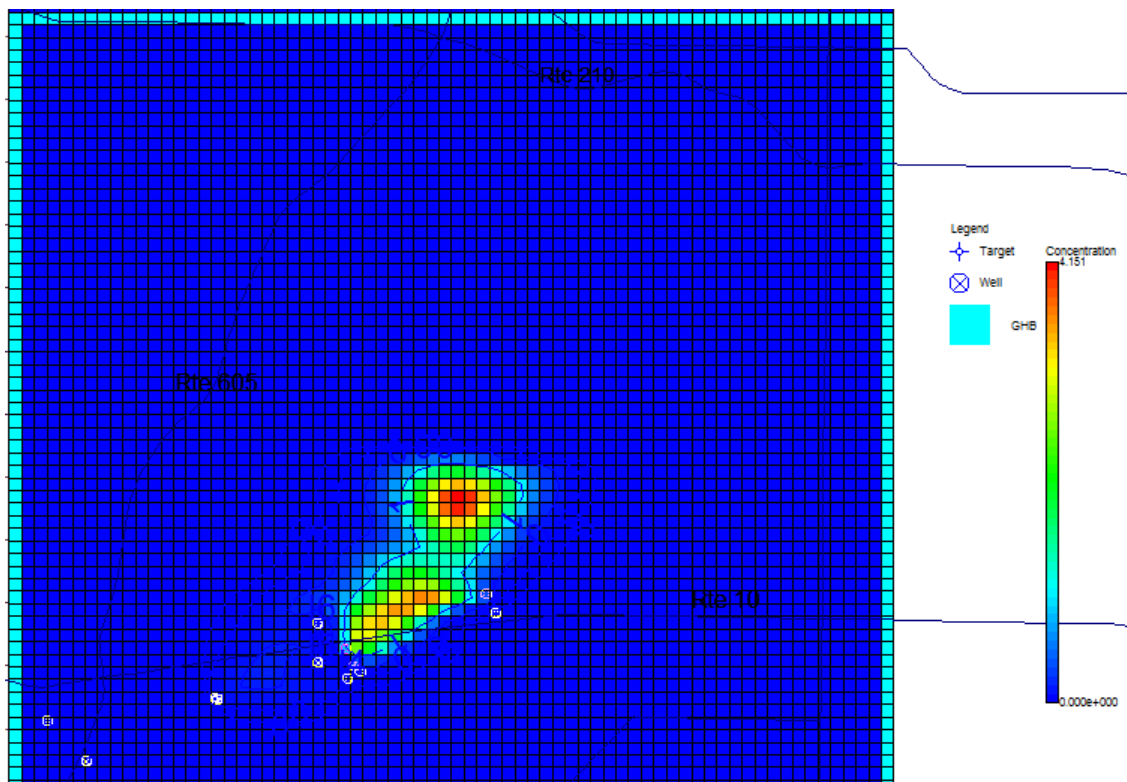


Figure 7.19 TCE Concentration Contours ( $\mu\text{g/L}$ ) at the end of June 2010 – Layer 2

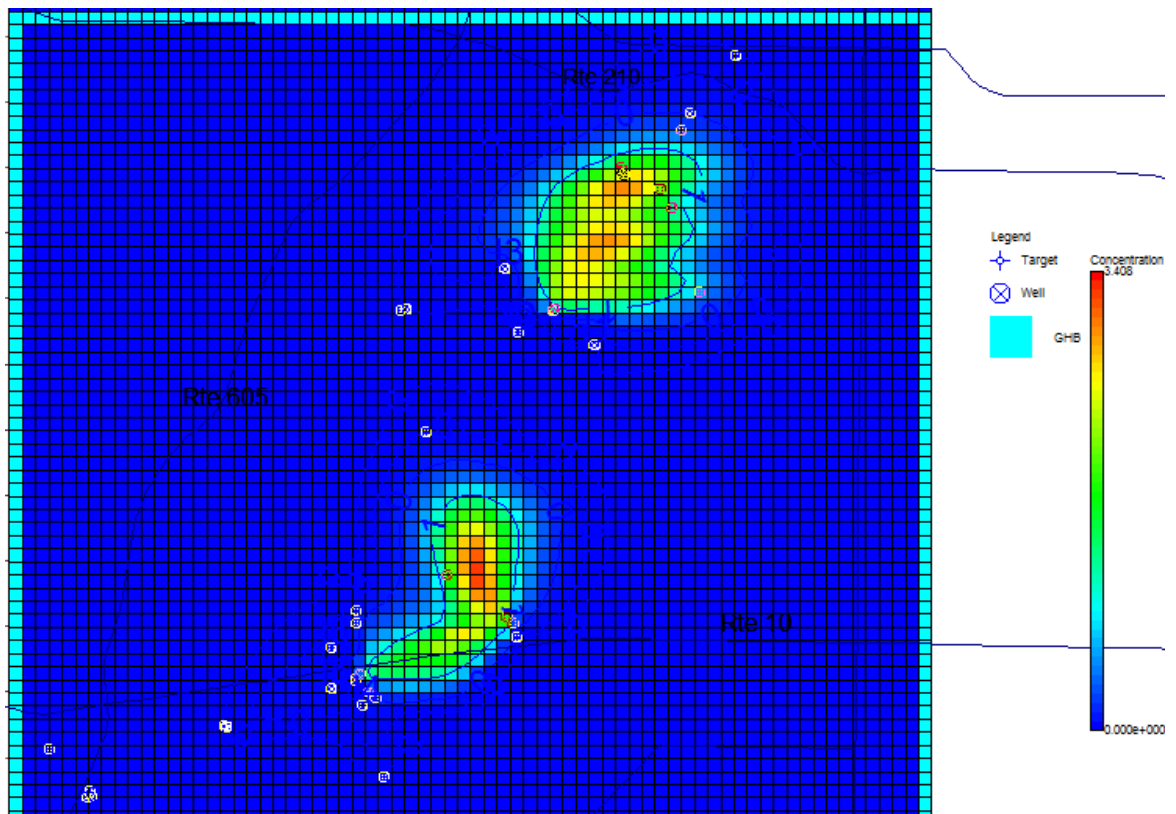


Figure 7.20 PCR Concentration Contours ( $\mu\text{g/L}$ ) at the end of June 2010 – Layer 1

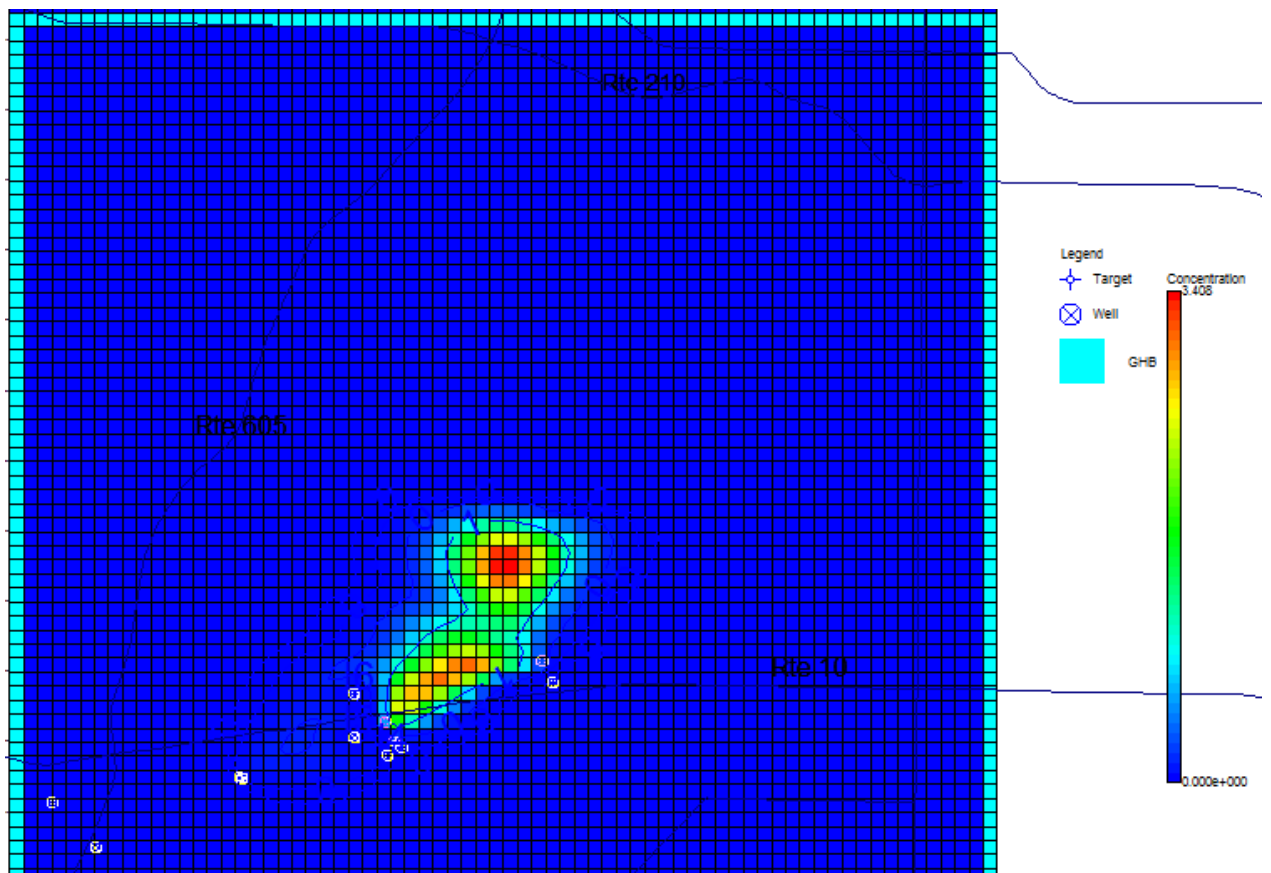


Figure 7.21 PCR Concentration Contours (µg/L) at the end of June 2010 – Layer 2

## 7.5 Summary of RT3D Results

Table 7.5 shows the contaminant concentrations at different stress periods. Initial contaminant concentrations were input from the USEPA database. The contaminant transport model was calibrated from Oct 1974 to June 2000. The calibrated model was verified from July 2000 to June 2006. The hydraulic gradient scheme was applied from stress period 127 to stress period 143 using presumed wet and dry cycles from the Hydrologic Base period (Figure 6.3). The results of RT3D output are shown in the table below. Watermaster projected concentrations for 2011 are depicted in the last row.

Table 7.1 Summary of Contaminant Concentration

| End Stress Period | Hydrologic Cycle | Date      | PCE Concentration | TCE Concentration | PCR Concentration |                        |
|-------------------|------------------|-----------|-------------------|-------------------|-------------------|------------------------|
| 127               |                  | 30-Jun-06 | 1208              | 397               | 1276              | Research Output        |
| 143               | Wet              | 30-Jun-10 | 4.8               | 4.1               | 3.4               | Research Output        |
| 143               | Dry              | 30-Jun-10 | 4.6               | 3.8               | 3.3               | Research Output        |
|                   |                  | 30-Jun-11 | 725               | 252               | 766               | Watermaster Projection |

## 7.6 System Economic Analysis

The incremental cost to operate the hydraulic gradient control scheme for 4 years versus the incremental cost for USEPA to continue the current clean up strategy (pump and treat) for 25 additional years was analyzed. Annual costs include energy, operation and maintenance costs. Capital costs include the cost of installing 6 injection wells. A typical injection well costs \$1,000,000. The cost of the injection wells is a small

fraction of the operation and maintenance cost. Offline and stand-by wells can be utilized for the hydraulic gradient control scheme.

Table 7.2 shows the unit costs of injected, recycled and pumped water. Costs are in dollars per gallon in year 2008 from various Southern California water purveyors.

Table 7.2 Unit Cost of Water

|                                           |             |               |
|-------------------------------------------|-------------|---------------|
| Injection                                 | \$528/ac-ft | \$0.00162/gal |
| Treated Imported                          | \$645/ac-ft | \$0.00198/gal |
| Treated Recycled                          | \$430/ac-ft | \$0.00132/gal |
| Treated Pumped Water (Primary Treatment)  | \$541/ac-ft | \$0.00166/gal |
| Treated Pumped Water (Tertiary Treatment) | \$410/ac-ft | \$0.00126/gal |

Assuming that all 18 production wells are operated daily for 24 hours. The minimum annual cost for USEPA to continue their current clean up strategy is \$16.08 million from the optimization results in Section 6.7.  $A_{\text{system}} = \$16.08 \text{ million/year}$ .

Water to recharge the basin by injection costs \$645/ac.ft. In this study, since the water for recharge is recycled from treated pumped water from the hydraulic gradient scheme, the cost to the existing system is a saving for imported water that would have been

bought to replenish the groundwater.

Thus Cost of HGC + Existing System using primary treatment = Cost of Existing System

– Cost of imported water to recharge 6 wells + Power cost to operate 6 Recharge wells.

$A_{HGC}$  = Annual cost of imported water to recharge 6 recharge wells daily injecting at a rate of 779 gpm =  $\$.00198 * 779 \text{ gpm} * 60 \text{ min/hr} * 24 \text{ hrs/day} * 365 \text{ days per year} = 4.86 \text{ million/yr.}$

$A'_{HGC}$  = Annual power cost to recharge 6 recharge wells daily injecting at a rate of 779 gpm =  $\$.00061 * 6 * 779 \text{ gpm} * 60 \text{ min/hr} * 24 \text{ hrs/day} * 365 \text{ days per year} = 1.50 \text{ million/yr}$

Cost of HGC + Existing System using primary treatment =  $16.08 - 4.86 + 1.5 = \$12.72$  million/yr.

In this study six recharge wells will be constructed at a cost of \$1,000,000 each. Some of the offline or standby pumping wells could have been converted to recharge wells but to show that the methodology developed in this research is applicable to any contaminant site, the recharge wells were constructed. Thus initial capital cost for the wells is \$6 million.

Note that the hydraulic gradient control system also reduces cost of litigation regarding the contamination and potential cancer risk for VOCs. Also after 4 years, the recharge wells can be converted to production wells to enhance water supply. This conversion offsets the initial capital cost of the recharge wells.

Future Cost for HGC + Existing System using primary treatment at the end of 4 years =  
 $P_{HGC}(F/P, 7\%, 4) + A_{System}(F/A, 7\%, 4) = 6 * 1.3108 + 12.72 * 4.4399 = \$64.34$  million.

Future Costs for the Existing System using primary treatment to treat the PCE, TCE and PCR at the end of 25 years =  $A_{System}(F/A, 7\%, 25) = 16.08 * 63.249 = \$1017.04$  million = \$1.02 billion.

At the end of 4 years, the HGC System is no longer needed. Since the PCE, TCE and PCR have been cleaned up below MCL and NL, only tertiary treatment will be needed for water pumped from the existing system.

Future Costs at the end of 25 years for the HGC + Existing System using primary treatment (from Year 1 to Year 4) and for HGC+Existing System using tertiary treatment (from year 5 to year 25) to treat the PCE, TCE and PCR =  $P_{System}(F/P, 7\%, 21) + A_{System}(F/A, 7\%, 21) = \$64.34 * 4.1406 + \$11.52 * (44.8652) = \$783.25$  million.

Present worth costs depict the cost in the base year of the analysis. Costs are discounted in a present worth analysis. Future costs represent the cost of the system at the end of the term for the analysis, 4 years for the HGC and 25 years for the existing system. Future costs factor in the compounded interest rate. The compounded interest rate of 7% includes adjustments for inflation. Note that the future cost of operating the existing facility is exorbitant (\$1.02 billion).

## 7.7 Optimization Model Results

This research develops a linear optimization model to manage the cleanup of the PCE, TCE and PCR in Baldwin Park Operable Unit. An efficient hydraulic gradient control scheme was developed capable of optimally selecting well locations and their corresponding pumping and injection rates and schedules corresponding to the minimum cost.

The Baldwin Park Operable Unit was selected to test the methodology developed. This research developed a methodology capable of optimally removing entrapped PCE, TCE and PCR contaminant plumes migrating with the groundwater. The cost coefficients determined the relative strengths of constraints. Cost is implicitly formulated in all the constraints.

Best wells are those that can most effectively decontaminate the plume. Best wells were selected by trial and error process by studying the flow and contaminant transport regime. Initially, Groundwater Vistas (MODFLOW module) was run to establish the head contours and to ensure that head constraints were satisfied. Then the contaminant transport module RT3D was run to delineate contaminant concentration contours. The contaminant concentration contours were then analyzed. Best wells were then selected by adjusting pumping and recharge rates and determining their locations. The hydraulic gradient control scheme developed allowed for periodic updating of the plume boundary as decontamination progresses. The pre-optimization stage yielded plume geometry and potential recharge and decontamination well locations.

Results of the initial optimization output become input for the simulation model. Results from the corrected simulation model (that satisfy the water quality constraint) become input for the optimization model. The optimization model output was verified to be optimal with respect to the simulation model used when the net change in the simulation-optimization iteration process is negligible.

Decontamination is a dynamic process as candidate pumping and recharge wells are selected as the clean up scheme progresses. Low optimal costs can be obtained by utilizing standby or out of service wells. Standby or out of service wells can be converted to pumping or recharge wells.

The objective function in equation 6.1 minimized the water transfer costs. The hydraulic gradient control constraints previously discussed in Chapter 5 ensured that inward gradients were directed toward the center of the plume.

Cost coefficients depend on the market value of energy for pumping, imported water, reclaimed water and include operation and maintenance cost. Table 7.3 lists initial and optimal policies for management problem. The minimum total annual cost of operating the existing system with primary treatment is \$16.08 million. The minimum total annual cost of operating the hydraulic gradient scheme and the existing system with primary treatment is \$12.72 million for BPOU. The minimum total annual cost of operating the existing system with tertiary treatment is \$11.52 million. Unit costs are time and location sensitive and should be verified. Table 7.4 depicts the cost summary.

| Table 7.3 Optimal Policy for Optimization Model |                       |           |                                |                    |                    |
|-------------------------------------------------|-----------------------|-----------|--------------------------------|--------------------|--------------------|
| Decision Variable                               | Decision Variable No. | Source    | User                           | Initial Rate (gpm) | Optimal Rate (gpm) |
| P(1,1)                                          | 1                     | GW        | Water Supply                   | 1450               | 1400               |
| P(2,1)                                          | 2                     | GW        | Water Supply                   | 150                | 150                |
| P(3,1)                                          | 3                     | GW        | Water Supply                   | 1000               | 525                |
| P(4,1)                                          | 4                     | GW        | Water Supply                   | 1352               | 912                |
| P(5,1)                                          | 5                     | GW        | Hydraulic Gradient Control     | 2450               | 2340               |
| P(6,1)                                          | 6                     | GW        | Water Supply                   | 1450               | 1354               |
| P(7,1)                                          | 7                     | GW        | Hydraulic Gradient Control     | 2450               | 2340               |
| P(8,1)                                          | 8                     | GW        | Water Supply                   | 1038               | 935                |
| P(9,1)                                          | 9                     | GW        | Water Supply                   | 1250               | 525                |
| P(10,1)                                         | 10                    | GW        | Water Supply                   | 935                | 735                |
| P(11,1)                                         | 11                    | GW        | Water Supply                   | 935                | 735                |
| P(12,1)                                         | 12                    | GW        | Water Supply                   | 1300               | 525                |
| P(13,1)                                         | 13                    | GW        | Water Supply                   | 935                | 735                |
| P(14,1)                                         | 14                    | GW        | Water Supply                   | 935                | 825                |
| P(15,1)                                         | 15                    | GW        | Water Supply                   | 935                | 935                |
| P(16,1)                                         | 16                    | GW        | Water Supply                   | 935                | 535                |
| P(17,1)                                         | 17                    | GW        | Water Supply                   | 935                | 735                |
| P(18,1)                                         | 18                    | GW        | Water Supply                   | 935                | 525                |
| IMP(1,1)                                        | 28                    | MWD       | Water Supply + Recharge        | 1945               | 1235               |
| R(1,1)                                          | 29                    | Reclaimed | Hydraulic Gradient Control     | 945                | 779                |
| R(2,1)                                          | 30                    | Reclaimed | Hydraulic Gradient Control     | 945                | 779                |
| R(3,1)                                          | 31                    | Reclaimed | Hydraulic Gradient Control     | 945                | 779                |
| R(4,1)                                          | 32                    | Reclaimed | Hydraulic Gradient Control     | 945                | 779                |
| R(5,1)                                          | 33                    | Reclaimed | Hydraulic Gradient Control     | 945                | 779                |
| R(6,1)                                          | 34                    | Reclaimed | Hydraulic Gradient Control     | 945                | 779                |
| QT(1,1)                                         | 40                    | GW        | Water Supply + Recharge to HGC | 12000              | 12000              |
| QT(2,1)                                         | 41                    | GW        | Water Supply + Recharge to HGC | 10000              | 10000              |

Table 7.4 Cost Summary

|                                                  | Minimum Annual Cost (\$Millions) | Timeline for Cleanup (Years) | Total Future Cost (\$ Millions) |
|--------------------------------------------------|----------------------------------|------------------------------|---------------------------------|
| HGC + Existing System with Primary Treatment (1) | 12.72                            | Years 1-4                    |                                 |
| Existing System with Tertiary Treatment (2)      | 11.52                            | Year 5-25                    |                                 |
| HGC + Existing System (1) + (2)                  |                                  | Years 1-25                   | 783                             |
| Existing System with Primary Treatment           | 16.08                            | Years 1-25                   | 1017                            |

## 6.8 Sensitivity Analysis

The model was run for 150 stress periods beginning in October 1, 1974. Pumping data was available in quarterly stress periods. Model was calibrated from October 1, 1974 to the end of the October 1, 2000. Model results closely replicated historic data. The model was also run from July 1, 2006 to March 31, 2012 to predict future scenarios using wet and dry hydrologic cycles.

The groundwater model reflected changes in pumping and recharge scenarios. High pumping periods with low recharge periods resulted in lower water levels. Similarly high recharge stress periods with low pumping periods resulted in higher water levels. The water levels are sensitive to recharge and pumping rates as expected.

Sensitivity analysis was performed using 7 zones of hydraulic conductivity as shown in

Figure 6.2. Hydraulic conductivity values were perturbed +10% and -10% and model output using the hydraulic gradient scheme tested. The results of the hydraulic gradient scheme were not very sensitive to small perturbations in hydraulic conductivity within 10%. However, the contaminants plume moved faster when hydraulic conductivities were increased as expected and the hydraulic gradient scheme was more effective in rapidly removing the contaminants. The hydraulic gradient scheme utilized in contaminant removal is more effective in unconfined aquifers than in confined aquifers.

The hydraulic gradient scheme was run to test future scenarios using assumed wet and dry cycles from the hydrologic base period (Table 7.1). In wet cycles, the PCE and TCE in the form DNAPL dissolve more but the hydraulic gradient scheme performed well in both cycles.

## **Chapter 8**

### **Conclusion**

#### **8.1 Summary**

A goal of this research was to develop a methodology capable of optimally removing the entrapped PCE, TCE and PCR in the Baldwin Park Operable Unit (BPOU). The Baldwin Park Operable Unit (BPOU) in the San Gabriel Valley is one of the largest Superfund cleanups in the United States. High concentration of VOCs and Perchlorate exceeding 100 times the maximum contaminants levels still exist. Peak concentrations of 38000 µg/L for PCE and 4000 µg/L for TCE have been detected at this site.

The current methodology employed by regulatory agencies performing the cleanup is not effective. High levels of PCE, TCE, other VOC's and Perchlorate still exist in BPOU. Currently, a pump and treat methodology is employed to spot clean critical areas of contamination in BPOU. The current focus of the regulatory agencies overseeing the cleanup is to treat and supply groundwater that meets the MCL or NL of contaminants. Their primary goal lacks an effective strategy to clean up the entire aquifer. This research provides a viable basin-wide methodology to rapidly and effectively remove contaminant pollution in BPOU.

The migration of the contaminant plume is facilitated by a combined gradient produced by injection and production wells. The management problem addresses the optimal removal of the contaminant plume and ultimately maximizes the use of valuable potable water. The methodology developed identified efficient strategies for the cleanup of the

contaminant plume below MCL or NL while meeting total water demand, minimizing the net increase in imported water demand and providing maximum drought and emergency supply capabilities.

An efficient hydraulic gradient control scheme was designed to contain and extract the contaminant plume. Numerical groundwater flow, particle tracking and contaminant transport models were developed and used to simulate responses to induced stresses and approximately delineate plume migration. The Baldwin Park Operable Unit in the San Gabriel Valley Basin in Southern California was used to test the methodology developed. Data from USEPA (2008) and DWR (1966) were used to develop the groundwater simulation model. The data was calibrated by adjusting the parameters as required. Situation specific assumptions were made to enhance the accuracy of the models.

A linear management model was developed to manage the cleanup of the contaminant plume at a minimum cost. The multi-objective management model selected the best available alternative. The multi-objective problem was reduced to a single objective by including all but one of the objectives into the constraint set. The key management objective was to minimize the cost of operation and maintenance of the system. The objective function and associated constraints were formulated so that the optimization problem was feasible. The linear management problem utilized a hydraulic gradient control scheme in which optimal well locations and their corresponding pumping and injection rates were selected. The user can employ experimental design to modify several options such as the number and rate of recharge and pumping wells.

The minimum total annual cost of operating the existing system with primary treatment is \$16.08 million. The minimum total annual cost of operating the hydraulic gradient scheme and the existing system with primary treatment is \$12.72 million for BPOU. The minimum total annual cost of operating the existing system with tertiary treatment is \$11.52 million. The hydraulic gradient scheme developed in this study reduces the PCE, TCE and PCR concentrations below MCL and NL in 4 years. At the current rate of contaminant removal using “pump and treat” methodology adopted by the USEPA, the PCE, TCE and PCR concentrations will be reduced below MCL and NL in 25 years.

## **8.2 Conclusion**

The goals of this research have been met. Groundwater flow, particle tracking, contaminant transport and optimization models were successfully developed and tested in the Baldwin Park Operable Unit (BPOU) in San Gabriel Valley. Also, an efficient hydraulic gradient control scheme was developed that contained, captured and cleaned up the PCE, TCE and PCR contaminant plumes in BPOU. The goal of developing a multi-objective optimization model that effectively cleans up the contaminants whilst meeting surface and groundwater supply demand was achieved. The plume clean-up reclaimed a number of potable water wells initially lost to contamination. The reclamation of several groundwater production facilities significantly enhanced drought and emergency water supply capability. Maximizing reclaimed water use and maximizing groundwater production reduced the demand and reliance on imported water. A well-developed hydraulic gradient control scheme facilitated the containment and cleanup of the contaminant plume. The output of the management model shows that the contaminant

plume was optimally extracted. The hydraulic gradient control scheme is operated in the recycle mode. Groundwater pumped is treated and recycled as water for injection. The hydraulic gradient control system has the added benefit of reduced cost of water to replenish groundwater system. The hydraulic gradient control system also reduces cost of litigation regarding potential cancer and other health risks for VOCs.

The case studies exemplify the use of an efficient hydraulic gradient scheme for the clean up of a contaminant plume. The groundwater simulation model tested the hydraulic gradient control concept. The numerical model solved average conditions within each cell, depicted the generic attributes and accurately reproduced the natural system. The results of the management model are subject to the limitations and assumptions of the case studies. The validity of the numerical model is limited by the assumptions made in developing and calibrating the model.

BPOU's population continues to grow. This growth places an increased demand and reliance on groundwater. Also, the semi-arid climate of Southern California prompts the development of groundwater resources to supplement our surface water. Meeting the increasing demand will require significant innovations in the optimal management of the existing water supplies. Water demand was assumed proportional to population growth.

Iterative simulation coupled with optimization is a valuable tool for managing aquifer contaminant reclamation. Certain constraints including availability of imported and reclaimed water for projected demand, future groundwater demand, water conveyance,

well capacity and storage, etc., have assumptions embedded. This study considered capital costs but the capital costs were negligible when compared to the high annual costs of the operating the existing facilities. Existing facilities including storage, conveyance etc., were assumed adequate. Off-line or standby wells can be activated as needed in the hydraulic gradient control. All pumping wells were assumed connected to the treatment plants by a network of pipelines. The multi-objective optimization is subject to operational, institutional, legislative, environmental and budgetary constraints.

Sensitivity analysis was performed using 7 zones of hydraulic conductivity. Hydraulic conductivity values were perturbed +10% and -10% and model output using the hydraulic gradient scheme tested. The results of the hydraulic gradient scheme were not very sensitive to perturbations in hydraulic conductivity within 10%. The contaminants plume moved faster when hydraulic conductivities were increased as expected. The hydraulic gradient scheme was more effective in rapidly removing the contaminants for higher hydraulic conductivities. The hydraulic gradient scheme utilized in contaminant removal is more effective in unconfined aquifers than in confined aquifers.

The hydraulic gradient scheme was run to test future scenarios using assumed wet and dry cycles from the hydrologic base period (Table 7.1). In wet cycles, the PCE and TCE in the form DNAPL dissolve more but the hydraulic gradient scheme performed well in both cycles.

### **8.3 Recommendations for Implementation**

Historically, the contaminants from the Baldwin Park Operable Unit have been flowing south to the Puente Valley Operable Unit. In order to circumvent this problem, this study recommends that the proposed set of 3 injection wells south of the Baldwin Park Operable Unit be installed first so that the southerly hydraulic gradient control scheme will block, contain and rapidly remove the contaminant plume that is migrating to Puente Valley Operable Unit. The northerly hydraulic gradient control scheme could be installed as phase 2 of this study after the implementation and testing of the southerly hydraulic gradient scheme (Phase 1) of this study.

## REFERENCES

- Anderson, M. and W. Woessner (1992). Applied Groundwater Modeling Simulation of Flow and Advective Transport, Academic Press, San Diego, California.
- Ahfeld, D. P. and M. Heidari, (1994). "Applications of Optimal hydraulic Control to Groundwater Systems," ASCE, J. Water Resources Planning and Management. 120(3), 350-365.
- Ahfeld, D. P. and Mulvey, J. M., Pinder, G. F. and E. F. Wood, (1988a). "Contaminated Groundwater Remediation Design using Simulation, Optimization and Sensitivity Theory," Model Development: Water Resources Research. 24 (3), p 431-441.
- Ahfeld, D. P. and A. E. Mulligan (2000). Optimal Management of Flow in Groundwater Systems, Academic Press.
- Ahfeld, D. P., Mulvey J.M. and G.F. Pinder (1986). "Designing Optimal Strategies for Contaminated Groundwater Remediation," Advances in Water Resources Research, 9(2), 77-84.
- Atwood, D. F. and S. M. Gorelick, (1985). "Hydraulic Gradient Control for Groundwater Contaminant Removal," J. Hydrology, Vol. 76, 85-108.
- Ayers, J. F. and H. L. Vacher (1983). "A Numerical Model Describing Unsteady Flow in a Fresh Water Lens," Water Resources Bulletin, 19(5), 785-792.
- Bear, J. (1979). Hydraulics of Groundwater, McGraw-Hill, New York.
- Bear, J. (1972). Dynamics of Fluids in Porous Media, Elsevier, New York.
- Bedient, P.B., Rifai H.S., and C. J. Newell (1994). Groundwater Contamination Transport and Remediation, Prentice-Hall, Englewood Cliffs, New Jersey.
- Bowles, M. W., L. R. Bentley, B. Hoyne and D. A. Thomas, (1999). "In Situ Groundwater Remediation Using the Trench and Gate System," Groundwater, 38(2), 172-181.
- Bumb, A. C., J. T. Mitchell, and S. K. Gifford, (1996). "Design of a Groundwater Extraction/Reinjection System at a Superfund Site using MODFLOW," Groundwater, 35(3), 400-408.
- California Department of Water Resources (DWR). 1966. "Planned utilization of Groundwater basins, San Gabriel Valley; Appendix A: Geohydrology." Bulletin 104-2. 203.

- Chang L.C., Shoemaker, C.A. and Liu, P. L-F, (1992). "Optimal Time-Varying Pumping Rates for Groundwater Remediation: Application of a Constrained Optimal Control Algorithm," *Water Resources Research* 28(12), 3157-3173.
- Colarullo, S, Heidari, Manoutcher and Maddock III (1984). "Identification of an Optimal Groundwater Management Strategy in a Contaminated Aquifer, *Water Resources Bulletin*, 20(5), 747-760.
- Culver, T.B. and C.A. Shoemaker, (1992). "Dynamic Optimal Control for Groundwater Remediation with Flexible Management Periods," *Water Resources Research*, 28(3) 629-641.
- Finney, B. A., Samsuhadi, and R. Willis, (1992). Quasi Three Dimensional Optimization Model of Jakarta basin, *J. Water Resources Planning and Management*, ASCE, 118(1), 18-31.
- Freeze, R.A. and J.A. Cherry, (1979). *Groundwater*, Prentice-Hall, Englewood Cliffs, New Jersey, 604 pp.
- Farrar, C. D., Metzger L.F., Nishikawa T, Koczot K. M., and E. G. Reichard (2006). "Geohydrologic Characterization, Water-Chemistry, and Ground-Water Flow Simulation Model of the Sonoma Valley Area, Sonoma County, California," *Scientific Investigations Report* 2006–5092.
- Fowler, K. R., Kelly C. T., Kees C. E., and Cass. T. Miller, (2004). "A Hydraulic Capture Application for Optimal remedial Design, " *Proceedings of the XV International Conference on Computational Methods in Water Resources*.
- Georgakakos, A.P. and D.A. Valsta, (1991). "Stochastic Control of Groundwater Systems," *Water Resources Research*, 27(87) 2077-2090.
- Gorelick, S.M., (1983). "A Review of Distributed Parameter Groundwater Modeling Methods," *Water Resources Research*, 19(2), 305-319.
- Gorelick, S.M. C. I. Voss, P. E. Gill, W. Murray, M. A. Saunders, and M. H. Wright, (1984). "Aquifer Reclamation Design: The Use of Contaminant Transport Confined with Nonlinear Programming," *Water Resources Research*, v 20, No.4, 415-427.
- Gorelick, S. M. and B.J. Wagner, (1986). "Evaluating Strategies for Groundwater Contaminant Plume Stabilization," *U.S. Geological Survey Water-Supply paper* 2290.
- Goicoechea, A., D. R. Hansen, and L. Duckstein, (1982). *Multiobjective Decision Analysis with Engineering and Business Applications*, John Wiley and Sons, Inc., New York.

- Greenwald R.M., (1993). MODMAN: An Optimization Module for MODFLOW, GeoTrans Inc., Sterling, Virginia.
- Harbaugh, A.W., E. R. Banta, M.C. Hill and M.G. McDonald, (2000). "User's Guide to Modularization Concepts and the Groundwater Flow Process," U.S. Geological Survey Open-file Report 00-92.
- Harbaugh, A.W., E. R. Banta, M.C. Hill and M.G. McDonald, (1996). "User's Documentation for MODFLOW-96 an update to the U. S. Geological Survey Modular Finite Difference Groundwater Model," U.S. Geological Survey Open-file Report 96-485, Reston Virginia.
- Harbaugh, A.W. and M.G. McDonald, (1988). Techniques of Water Resources Investigations of the U. S. Geological Survey: A Modular Three Dimensional Finite Difference Groundwater Flow Model, U.S. Government Printing Office, Washington, District of Columbia.
- Heidari, M., J. Sadeghipour, and O. Druici (1987). "Velocity Control as a Tool for Optimal Plume Containment in the Equus Beds Aquifer, Kansas", Water Resources Bulletin, 23(2) 325-335.
- Kelley, C. T. (1999). Iterative Methods for Optimization, Society for Industrial and Applied Mathematics, Philadelphia, Pennsylvania.
- Lefkoff, L. J. and S. M. Gorelick, (1985). "Groundwater Contamination and Reclamation," American Water Resources Association, Bethesda Maryland, 125-131.
- Lehr, J. H. and D. M. Nielsen, 1982. "Aquifer Restoration and Groundwater Rehabilitation: A light at the End of the Tunnel," Groundwater, 20(6), 650-656.
- Main San Gabriel Basin Watermaster (MSGBW) 2006. "Annual Report 2005-2006." Azusa, California.
- P&D Technologies, (1988). "Dominguez Gap Barrier Project Deficiency Study", a report prepared for the Los Angeles County Department of Public Works.
- Pollock, D.W., 1994. "User's Guide for MODPATH/MODPATH-Plot Version 4.1: a Particle Tracking Post-Processing Package for MODFLOW" U.S. Geological Survey Open-file Report 94-464.
- Polo, J. F. and R. F. Ramis, "Simulation of Saltwater-Freshwater Interface Motion," Water Resources Research, 19(1), 61-68, 1983.

- Reichard E. G. et al, "Geohydrologic Characterization, Water-Chemistry, and Ground-Water Flow Simulation – Optimization of the Central and West Coast Basins, Los Angeles County, California." Water-Resources Investigations Report 03-4065.
- Schrage, L. E. (1997). Optimization Modeling with LINDO, Duxbury press, Brooks/Cole Publishing Co., Pacific Groove, California.
- Sewell G. W., Matthews J. E. and Russell H. H. (1992). "PCE and TCE Removal from Contaminated Soil and Groundwater," EPA Groundwater Issue, EPA/540/S-92/002.
- Shamir, U., and G. Dagan (1971). "Motion of the Seawater Interface in Coastal Aquifer: a Numerical Solution," Water Resources Research, 7(3), 644-657.
- Strikweda, J. C. (1989). Finite Difference Schemes and Partial Differential Equations, Wadsworth, Belmont California.
- Sun Ne-Zheng, (1989). "Application of Numerical Models to Simulate the movement of Contaminants in Groundwater," J. Environmental Health Perspectives, Vol 83, 97-115.
- Sun Ne-Zheng, (1995). Mathematical Modeling of Groundwater Pollution, Springer-Verlag, New York Inc. and Geological Publishing House, New York.
- Tiedeman, C. and S.M. Gorelick (1993). Analysis of uncertainty in optimal groundwater contaminant capture design, Water Resources, Res., 29(7), 2139-2153.
- USEPA (1999). United States Environmental Protection Agency, Superfund Record of Decision, San Gabriel Valley, Area 2, Baldwin Park, CA
- USEPA (2008). United States Environmental Protection Agency, Superfund Database, San Gabriel Valley, CA
- Varlien, M.D. and J.M. Shafer, (1993). "Coupled Simulation-Optimization Modeling for Municipal Groundwater Supply Protection," Groundwater 31(3), 401-409.
- Volker and Rushton (1982). "An Assessment of the Importance of Some Parameters for Seawater Intrusion in Aquifer and a Comparison Dispersive and Sharp-Interface Modeling Approach," J. Hydrology, Vol. 56, 239-250.
- Wagner, B. J. (1997). "Recent Advances in Simulation-Optimization Groundwater Management Modeling, U. S. National Report to the International Union of Geodesy and Geophysics," Reviews of Geophysics, Supplement
- Willis, R. And W. W-G. Yeh, (1987). Groundwater Systems Planning and Management,

Prentice-Hall, Englewood Cliffs, New Jersey.

Yeh, W. W-G, (1992). "Systems Analysis in Groundwater Planning and Management,"  
J. Water Resource Planning and Management, 118(3), 224-237

Yeh, W. W-G et al, (1995). "Development of An Optimal Data Collection Strategy for  
the Hemet Basin Management Program," Prepared for MWD.