TECHNOLOGY EVALUATION OF THE PILOT PERMEABLE TREATMENT WALL AND RECOMMENDED PATH FORWARD

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Executive Summary

The purpose of this technology evaluation report is to evaluate and recommend a path forward for the Pilot Permeable Treatment Wall (PTW) located on the North Plateau at the West Valley Demonstration Project (WVDP). A pilot scale PTW was installed on the 2^{nd} lobe of a Sr-90 plume under the North Plateau in the Fall of 1999. The 30 ft. pilot PTW was deployed at a location on the North Plateau where the Sr-90 plume narrows near its leading edge.

A fairly simple design configuration was used to install the pilot PTW using a sheet pile cofferdam setup and conventional construction equipment and methods. The cofferdam design would allow internal soil to be excavated and the void backfilled with the reactive media, clinoptilolite, after which the cofferdam would be removed and the PTW become operational.

Hydraulic monitoring of the pilot PTW is performed using a series of well points, monitoring wells, and piezometers, most of which were installed after PTW construction. Post-PTW monitoring indicates that unique hydraulic conditions may be preventing groundwater flow through the PTW and associated treatment of contaminated groundwater. The conclusion regarding the hydraulic conditions may be related to the complex hydrogeology and the design and construction at the pilot PTW. Possible explanation as follows:

- Sheet pile extraction during PTW construction produced commingled clinoptilolite and roundstone and a zone of fine clinoptilolite particles around the north and east edges causing a discontinuous skin of fine zeolitic material and diverted groundwater flow.

- Hydraulic conductivity of the clinoptilolite media following construction may be up to two or three orders of magnitude less than that for the clinoptilolite prior to placement, thus causing flow path diversion.

- The PTW does not appear to be fully penetrating through the upper water bearing zone causing possible underflow in its central and eastern portions.

- A highly heterogeneous and anisotropic aquifer of fine and course sediments may be causing diverted flow.

Lessons learned were evaluated and focused on the engineering detail of the PTW design and installation. These lessons learned will assist in the eventual development of engineering alternatives to enhance the performance of the pilot PTW and future deployments of PTW technology. Lessons learned are discussed in detail in Section 6.0.

Four options for modification are presented in Section 7.2; no modification, two engineering modifications (Installation of lateral barriers or installation of an extension to the PTW), and one option that comprises of a complete rebuild of the pilot PTW.

Based on the evaluation of information prepared by WVNS and Geomatrix Consultants, WVNS is recommending Option 2, Installation of Lateral Hydraulic Barriers. However, prior to implementing this option or any others, it will be very important to collect additional characterization data to integrate into the development of a three-dimensional flow model in order to optimize and ensure the effectiveness of the proposed modifications. Therefore as a first step further characterization and assessment of the local geology and hydrogeology near the pilot PTW, is needed to decrease the degree of uncertainty with the pilot PTW performance issues and to increase the potential to select and implement an effective engineering solution.

Secondly, it is recommended that 1st lobe preliminary design proceed. FY2001 preliminary design activities begin with selection of wall location then proceed with design and implementation of a comprehensive soil and groundwater characterization program. Activities will continue with evaluation of geological and hydrogeological data. Once the data is throughly analyzed, a conceptual design may commence. At completion of conceptual design, a decision will be made as to whether full-scale deployment is feasible on the 1st lobe.

By completion of the planned path forward, there will be increased confidence both that a reliable solution to the pilot's performance can be selected and further PTW design and installations can be successfully applied at the WVDP.

1.0 Introduction - Summary

The West Valley Demonstration Project (WVDP) site is a 220-acre parcel located in a rural area within the Allegheny section of the Appalachian Plateau. The site is bordered on the north, south, and east by two creeks that generally divide it into two upland subareas referred to as the north plateau and south plateau (Fig. 1). The north plateau contains a 1960's-era commercially unviable spent nuclear fuel reprocessing plant that is now governed under the control of the United States Department of Energy (DOE) pursuant to the 1980 West Valley Demonstration Project Act.

Radioactive contamination that leaked from a main plant system in the late 1960's produced a strontium-90 (Sr-90) contaminated groundwater plume that now extends north-northeasterly from beneath the former reprocessing plant across the north plateau. An extensive subsurface investigation conducted in 1994 identified the primary source of Sr-90 activity and defined the horizontal and vertical extent of contamination in the soil and groundwater. Sampling results also showed that the Sr-90 migrates via preferential pathways governed by coarse textured sedimentary layers or zones. Data from subsurface sampling programs in 1995, 1997, and 1998 as well as more recent data from the quarterly groundwater monitoring program and north plateau operational locations all indicate that the Sr-90 plume is migrating toward a drainage ditch north of the CDDL and the main CDDL area (Fig. 2).

A groundwater recovery and ion exchange treatment system was installed as an initial mitigative effort in November 1995 across a preferential pathway near the western or 1st lobe of the plume's leading edge (Fig. 4). The continuous tracking and evaluation of groundwater levels and chemical data during system operation indicates it effectively mitigates Sr-90 transport to the surface near this location. However, maintaining optimum capture is both challenging and resource intensive.

The subsurface investigations that were conducted in 1994 and 1997 further characterized the lateral and vertical distribution of radiological contamination near the leading edge of the 1st lobe and the eastern or 2nd lobe of the plume. The resulting geologic and geochemical data analyses underwent an external technical peer review (Berkey [1997]) in order to evaluate mitigation technologies for Sr-90 on the north plateau. Recommendations developed by the two review teams stated that future tasks should focus on evaluating low maintenance and low cost groundwater remediation technologies to optimize Sr-90 mitigation. Following these recommendations, extensive research into alternative technologies indicated that an in-situ permeable reactive barrier (herein referred to as a permeable treatment wall or PTW) would best suit site needs for long-term, low-cost, low-maintenance remediation of transportive subsurface contamination.

2.0 Technical Evaluation

2.1 Reactive Barrier Technology Evaluation

Pump and treat systems and permeable reactive barriers were evaluated as innovative technology because of their extensive use at groundwater cleanup projects and as a passive, low-cost way to reduce contamination. Conceptual engineering designs for a PTW and a pump and treat system on the north plateau were performed in 1994 to determine which could be carried out quickly at a reasonable cost. The PTW designs were technically feasible but the initial costs were high as compared to a pump and treat system. Consequently, a pump and treat system known as the North Plateau Groundwater Recovery System (NPGRS) was installed at the leading edge of the 1st Lobe and has been operational since November 1995 (Fig. 4).

Since the NPGRS was considered a temporary effort, a subsequent feasibility study was initiated to identify and evaluate long-term groundwater remediation programs. Studies at Brookhaven National Laboratories (BNL) (Aloysius, D. L. [1995]) used groundwater and soils from the north plateau to identify geochemical factors that influence Sr-90 sorption to various sorbent media that may be used in a subsurface barrier.

The geochemical studies and modeling indicated that clinoptilolite should be evaluated via bench scale testing and additional computer modeling. Subsequent testing results showed that clinoptilolite could reduce Sr-90 concentrations on the north plateau to 1,000 pCi/L in about ten years using a three-foot thick barrier. However the 1995-era time and cost limitations indicated that pump and treat technology still remained the preferred interim mitigative technology for the north plateau.

The 1997 Technical Peer Review (Berkey [1997]) determined that pump and treat technology was a valuable interim measure for plume control but further investigations indicated that a permeable reactive barrier would be an efficient alternative application at the WVDP.

Beginning in April 1998, several evaluations were conducted to support PTW implementation including additional geochemical analyses, potential conceptual designs, and suitable wall locations (Berkey [1999]). The geochemical analyses indicated that additional laboratory studies would significantly enhance the confidence associated with predictions of barrier performance.

2.2 Technology Application at WVDP

Batch and column tests performed by the State University of New York at Buffalo (UB) in April 1999 positively quantified Sr sorption by clinoptilolite (CH 14 x 50). Preliminary results suggested that a barrier life of about 36 years could be achieved with a barrier of clinoptilolite (Kd of 2,350 ml/g) using a cofferdam installation design. These geochemical data and additional recommendations from a second Technical Peer Review (Berkey [1999]) indicated that a pilot-scale wall in the 2nd lobe should be installed. The conceptual PTW design involved the construction of a simple cofferdam within the 2nd lobe of the plume at a location with high Sr-90 concentrations and easy access (e.g., no adjacent buildings, obstructions, underground power lines, etc.)

The chosen reactive media, clinoptilolite, is a zeolite mineral with a solid solution formula of $[(Ca, Mg, Na_2, K_2)(Al_2Si_{10}O_{24}.8H_20)]$ (Warner, 1986), which has been shown to passively and effectively reduce the concentration of Sr-90 in groundwater. This alternative remediation technology and reactive media is capable of effectively mitigating further migration of Sr-90 in groundwater over a large portion of the north plateau.

The intent of the pilot PTW installation was to assess a small-scale field version of a full-scale remedy and to define those design parameters that must be quantified to ensure successful and cost-effective implementation of an innovative full-scale remedy of a Sr-90 adsorbing zeolite in a complex hydrogeologic environment. The full-scale deployment of this technology requires a step-wise approach to determine the nature and extent of technical, regulatory, and stakeholder issues associated with deployment.

Although PTW technology has been tested at more than 40 sites in North America, it remains an innovative technology when applied to complex hydrogeologic conditions, radioactive contaminants, variable groundwater chemistry, available construction methods, and especially with clinoptilolite as the reactive medium. The PTW must be both chemically successful at remediating contaminated groundwater and function properly from a hydraulic perspective. This pilot PTW provides site-specific information imperative to developing a competent full-scale system that meets its design objectives with the greatest certainty.

The testing and monitoring program for the pilot PTW was designed to determine if the data quality objectives (DQOs) prepared to evaluate pilot PTW operation (WVDP-350) were being met. The primary criteria for assessing wall operation were to establish groundwater flow through the wall and sufficient reduction in Sr-90 activities by the clinoptilolite treatment medium. Specific assessment goals were to:

1) Determine if groundwater flows through the PTW and is not backed up or diverted around the PTW;

2) Determine if Sr-90 activity in groundwater is reduced as the groundwater passes through the PTW; and

3) Compare Sr-90 activities up gradient and downgradient of the PTW. (Sr-90 activities immediately downgradient of the pilot PTW can be expected to decrease over time.)

An initial six-month assessment used water level data from WPs 16, 25, 26 and 27 and monitoring well 8603 before and during construction, and then water levels, Sr-90, and inorganic analyte data from an additional 13 post-construction well points. These data are graphically presented in Appendix 1 and 2.

Data from the initial assessment was again technical peer reviewed to evaluate plume mitigation and confirm initial assessment results. Common opinions reached by these reviewers include the following:

C There are sufficient positive indications that the clinoptilolite is effective in removing the Sr-90;

- Placement of a PTW into a natural groundwater flow system can easily disrupt the flow system;

- Problems related to the hydraulic performance of PTW's are common but under-reported;

- The PTW monitoring system was well planned and allows for a detailed evaluation of the flow regime in the immediate vicinity of the pilot PTW;

- Other PTW installations have identified possible zones of reduced permeability, thus it is likely that a “skin effect” may be present around the pilot wall; and

- Early performance of the wall indicates that there is some hydraulic connectivity between all monitoring points, suggesting that the “skin effect” may be more permeable at some locations or not present on all sides of the treatment wall.

2.3 General Conceptual Hydrogeologic and Sr-90 Distribution Model of the North Plateau

The unconfined and semi-confined groundwater flow conditions in the PTW area prior to construction were influenced by both laterally and vertically varying hydraulic conductivity and undulations in the surface of the underlying low permeability sediments, which act as a basal hydraulic barrier to the flow system. The regional groundwater flow direction was toward the north-northeast, as determined by WVDP groundwater monitoring program data (Fig. 5).

The leading edge of the Sr-90 plume bifurcates around a topographically significant erosional remnant of lower conductivity clay and silt that is identified in the borings logs for well 0115 and B-94-13; this bifurcation is exhibited in Figure 2 where the <1,000 pCi/L zone separates the plume. The thin section of sand and gravel that overlies this remnant thickens to the west and east, thereby providing flow paths of least resistance around the clay and silt unit into thicker water-bearing zones, where the Sr-90 becomes more highly concentrated in discrete (preferred) zones of locally higher hydraulic conductivity. (See Figures 2.1 and 2.2 in Appendix 10.) Although potentiometric surfaces do not show evidence of mounding that would bifurcate the plume, the hydrograph for well 116 has a low fluctuation and thus lower local recharge, which is indicative of a hydraulically tighter media at well 116.

The pilot PTW was installed at the western edge of the eastern lobe of Sr-90 near the 10,000 pCi/L contour, which was verified by data from pre-construction well points WP-25, WP-26, and WP-27; Sr-90 varied from 500 pCi/L in the west at WP-25 to a high value of 40,000 pCi/L in the east at WP-26. These well points are all screened between 7 and 22 feet below ground surface (bgs) and traverse the hydrostratigraphic layers near the PTW. Previous Geoprobe boring data and field gamma scans of soil collected during the installation of the PTW dewatering wells suggest that the higher activity groundwater exists in the lower half (i.e., depths greater than about 15 feet bgs) of this shallow water bearing system.

3.0 Construction Methodology

3.1 PTW Design

The pilot PTW was designed as a passive groundwater treatment system that does not rely on collecting, diverting, channeling, or pumping groundwater to a media bed during operation. This guideline led to a simple "continuous" reactive barrier design configuration that could be installed safely and easily at the identified location in the 2nd lobe of the Sr-90 plume. The cofferdam construction design would allow internal soil to be excavated and the void backfilled with the reactive clinoptilolite, after which the cofferdam would be removed and the PTW become operational. Features and design specifications for the pilot PTW are as follows:

- Rectangular cofferdam: The cofferdam was designed as a rectangular 30.5 ft long (east to west), 7-ft wide (north to south), 26-ft deep cofferdam.

- Cofferdam construction materials: 42 Arbed AZ48 sheet piles; Adeka Ultraseal #50A (for application to sheet pile interlock); two W36 x 182 wales, two W18 x 86 wales; four pumps, 2-inch polyethylene pipe, 1000-gallon hold tank and sump pump for dewatering, treatment and discharge; piping for drain-line and riser-pump assembly.

- Basic installation method: Dewatering wells and monitoring well points pre-installed in excavation area; piles driven around excavation area using vibratory hammer to approximate contact with Lavery till at 28-ft below original ground surface/1356-ft above mean sea level (msl), to be driven an additional 10-feet into the till (1346 msl) or to a lesser depth if difficulty encountered, cutting off the top of sheet piles to uniform height as needed; cofferdam support provided by a single layer of external bracing formed by placing longer wales along cofferdam length with restraint brackets mounted on 7'7" centers and shorter wales along cofferdam width so that wales are installed horizontally around outer perimeter of cofferdam at 1384-ft msl, (horizontal axis at 1382-ft msl.)

- Excavation and backfill method: After structural elements are set in place, soil inside cofferdam dewatered using pumps installed in wells, placed at equal intervals directly beneath alignment of PTW; groundwater pumped from wells through 2-inch pipe to hold tank for treatment (pumps activated by pressure switch designed to engage when groundwater level in well 12-inches above pump); sump used to discharge water from hold tank to Lagoon 2; soils excavated and cofferdam subsequently backfilled with 5.5-ft of unmixed 100% clinoptilolite (CH 14 x 50) and 1.5-ft with pea gravel placed in an area separated by moveable partition at south face of excavated area; horizontal drain-line placed along bottom of gravel section and connected to vertical riser-pump assembly.

- Surface completion method: After sheet piles extracted, surface area to be created over PTW with 1.4-ft to 1.9-ft thickness of clay fill as needed to match existing grades at the edge of the excavation (1384.6 msl), mounding fill about 6-inches higher in the middle of the excavation (1385.1 msl), with 3.5-inch diameter bumper posts placed at four corners of the PTW for demarcation/protection.

See Appendix 12 for a full description of construction methods.

3.2 As-Built Construction

The following as-built analysis of the cofferdam construction, excavation, backfilling, and surface completion relied on the following as-built construction drawings:

- North Plateau Permeable Treatment Wall, Drawing no. 900D-7867, sheets 1 through 8 of 8,

- North Plateau Treatment Wall Cofferdam, Drawing no. 900D-7857, sheets 1 through 4 of 4, and

- Site, North Plateau Area, Topography and Underground Piping, Drawing no. 900D-6743, sheet 1 of 1.

The pilot PTW installed on the north plateau is an approximate 100 ft by 100 ft area and northwest of Lagoons 4 and 5, where the ground surface generally slopes downward at about 3 percent from south to north (Fig. 4).

- Construction preparation: 100-ft by 100-ft area around PTW prepared as work surface (hardstand) by laying down 7-inch thick layer of crush stone over geotextile; dewatering wells installed and piped to hold tank; two inclinometers placed about 8-ft from where sheet piles were to be driven on north and south sides of the cofferdam to monitor movement during excavation and backfilling; electrical line installed in conduit run through trench about 2-ft wide by 2-ft deep to support pumping operations; hardstand layer scraped and geotextile cutback to allow for sheet pile installation.

- Cofferdam construction: Cofferdam laid-out and sheet piles driven with vibratory hammer to approximate top of till at 1358 msl; sheet piles driven into till with impact hammer (12-ft); hardstand and native soil excavated around outside of sheet piles and geotextile cut back to allow for installation of external wale system; internal soils excavated and dewatered to about 15-ft below ground surface using well pumping system; sump pumps dropped into area to continue dewatering during excavation as dewatering pumps were removed; divider system installed within cofferdam to maintain separation between clinoptilolite and 1-inch roundstone ("pea gravel"); 6-inch diameter perforated PVC pipe placed along abase of excavation within roundstone for drainage with 10-inch PVC riser pipe attached at lower east end of drainage pipe with both pipes wrapped in geotextile; cofferdam backfilled by emptying supersacks of clinoptilolite into dry excavation from surface; 1-inch roundstone placed in 1.5-ft separated area using PVC "elephant trunk"; distance between clinoptilolite and roundstone backfill maintained at 1-ft maximum during backfilling by observation, backfilling to approximately 1382 msl before removing divider system; clinoptilolite and roundstone brought to design elevation of 1383.2 msl and wales removed; zone outside cofferdam once occupied by wales backfilled with previously excavated material.

- Sheet pile removal and surface completion: Geotextile underneath hardstand cut back until excavation sidewalls visible; starting from west end of cofferdam, sheet piles were withdrawn from ground using vibratory hammer, scraping off any material stuck to sheet piles as needed; settling of clinoptilolite recorded at about 4-ft after last sheet pile removed, with inclinometers indicating lateral movement into the excavation of about 7-inches on the south side, 3 inches on the north side; clinoptilolite added to existing material to bring it to grade at 1383.2 msl, with fill mounded in the middle to 1384.6 msl; 1-inch thick layer of granular bentonite (Volclay CG-50) placed over excavated area; hardstand stone raked over filled area around the PTW to provide working surface; four 7-ft by 3.5-ft round bumper posts placed at each corner of PTW perimeter (moving each post from position where it was originally placed during backfilling).

See Appendix 12 for a more detailed discussion of as-built construction.

4.0 Hydraulic Evaluation

This section describes observations regarding the ground water hydraulics and hydrogeologic conditions near the pilot PTW. Additional details associated with this evaluation are presented in Appendix 11, "Pilot Permeable Treatment Wall Hydraulic Evaluation Report," by Geomatrix Consultants.

Hydraulic monitoring of the pilot PTW is performed using a series of well points, monitoring wells, and piezometers, most of which were installed after PTW construction (Fig. 6). Construction details of these monitoring locations are listed in Table 2 of Appendix 11. Groundwater elevation contour maps representing conditions during various stages of the pilot test are shown in Figure 3.3 in Appendix 11. Hydrographs for select monitoring points in the PTW area are shown in Appendix 1 and are discussed in the following sections.

4.1 Hydrogeologic Conditions at the North Plateau

Post-PTW-installation monitoring indicates that a unique hydraulic condition may be preventing groundwater flow through the PTW and associated treatment of contaminated groundwater. Several PTW development (pumping) efforts and expert review of the hydrogeologic and Sr-90 data determined potential causes of this flow restriction, which are discussed presently and in Section 5.0.

The interpretation of regional hydrostratigraphy and groundwater flow conditions was derived from borehole data collected during various characterization activities performed over the last several years including geotechnical data collected during the PTW design activities, and assessment data collected after PTW installation. These data are presented on a Sr-90 distribution map, a potentiometric surface map, and in the hydrostratigraphic cross-sections shown in Figures 2 and 4, and in Appendix 11, Figures 2.1, 2.2, 2.3, 2.4, and 3.3.

Two distinct hydrostratigraphic units exist beneath the north plateau:

- The Shallow Water-Bearing Zone (SWBZ), consisting of alluvial sand and gravel (AS&G), and the Slackwater Sequence (SWS); and

- A basal confining aquitard (or low permeability zone) consisting of lacustrine clay and silt, and Lavery Till.

The SWBZ is a heterogeneous and anisotropic unit consisting of the both laterally continuous and discontinuous layers in both the AS&G and the SWS. The cross-section in Figure 2.2 of Appendix 11 shows unconfined conditions generally appear to exist in the upper portion of the SWBZ within the AS&G. The layers of clay to silty clay and silty gravel present within the SWS likely produce semi-confined conditions within the lower portion of the SWBZ.

The SWBZ near the NPGRS is composed only of the coarse-grained, unconfined deposits that extend from grade to the basal confining unit. The thickness of the AS&G unit in this area varies from 5 to 15 feet, depending on the topography of the basal layer; the water table is approximately five feet below grade.

The cross-section A-A' in Figure 2.2 of Appendix 11 traverses the PTW and NPGRS areas. It shows the lower-most basal unit to be the topographically variable Lavery Till, which is a laterally continuous, stiff, and unsorted sequence of silty clay to clayey silt and a hydraulic conductivity of less than $1x10^{-7}$cm/s.

A thin sequence of lacustrine clay with silt apparently overlies the Lavery Till in the study area; this unit is thickest in the boring log for well 0115 and boring B-94-11, where it was previously interpreted as Lavery Till. However, the alternating sequence of clayey silt and silty clay is indicative of a lacustrine depositional environment. The hydraulic conductivity of the clay with silt unit was found to be $4x10^{-8}$ cm/s by slug testing performed on well 0115, which is screened entirely in the unit. Since the Lavery Till is a silty clay that likely was derived from proglacial lacustrine deposits, such actual lacustrine layers could have been easily mistaken as Lavery Till, especially in field samples.

In the PTW area, the lacustrine clay with silt unit is overlain by the SWS, a thick sequence of water-lain deposits of alternating thin, well-sorted beds of loose silty gravel and fine sandy silt that fills a wide, channel-like depression as shown on Figure 3. Cross-section C-C' in Figure 2.4 of Appendix 11 shows how the interbeds of coarse and fine-grained sediments may cause confining groundwater conditions to exist within the water-bearing deposits of the SWS. Variations in the stratigraphy (i.e., thickness and lateral continuity of water-bearing deposits) may significantly change the local hydraulic gradient and flow directions, which cannot be accounted for on site-wide maps.

The SWS near the PTW is overlain by the AS&G unit that exhibits a finer sequence of silt and sand with less gravel and less stratification than the underlying SWS. The AS&G coarsens to the west towards the NPGRS, showing lesser fine-grained material, and is described as an alluvial gravel and fine sand on cross-section A-A' (Fig. 2.2 in Appendix 11).

The uppermost stratigraphic unit near the PTW is a silty clay fill that overlies the locally finer grained AS&G. Data on cross-sections A-A' and C-C' indicate that the fill is consistently 2 to 3 feet thick and may serve as a confining layer to the underlying water-bearing units. Such semi-confined conditions are likely to be spatially and temporally variable.

The eastern lobe of the Sr-90 plume is apparently subjected to a steeper apparent hydraulic gradient toward well 0105, causing a slightly eastward dispersion of the lobe. The steep hydraulic gradient between wells 8603 and WP-11 (illustrated in Fig. 5) may be coincident with the transition between the semi-confined and unconfined conditions. An evaluation of hydrographs and precipitation data did not reveal consistent characteristics of a confined system, so a confident surface delineation of this clay fill is not possible with current data.

In addition, stratigraphic data forming cross-section D-D' indicates that the central and eastern ends of the pilot PTW may not penetrate to the Lavery Till, but may "hang" in the SWS above the top of the Lavery Till. Additional details regarding the hydrostratigraphy near the pilot PTW can be found in Appendix 11.

4.2 Distribution of Hydraulic Conductivity

The variable hydrostratigraphy and soil texture in the SWBZ produces hydraulic conductivity values in the 10^{-3} cm/s magnitude in the western lobe of the plume near the NPGRS where the AS&G is coarser-grained. As the grain size composition of the AS&G becomes finer in an eastward direction toward the PTW, conductivity values in the AS&G decrease. Well 0116 is screened within this finer facies and yields a slug-test-based hydraulic conductivity value of 6×10^{-5} cm/s.

Near the PTW, the SWBZ consists of a finer AS&G and the alternating SWS that produces hydraulic conductivity values between $1x10^{-3}$ to $1x10^{-4}$ cm/s. The fully penetrating wells and piezometers installed near the PTW yield *average* hydraulic conductivity values. Thus, higher conductivity is possible in discrete, continuous sand and gravel layers associated with the SWS, which may mask the lower hydraulic conductivity in the locally finer AS&G. Since the AS&G near the PTW may have a lower hydraulic conductivity than the underlying SWS, the AS&G may also act as a confining bed over the SWS.

The hydraulic conductivity values obtained from the slug testing of wells in and near the PTW are shown in Figure 3.10 of Appendix 11. Slug tests commonly are less reliable for engineering-scale quantitative assessments because the results may be highly dependent on well construction (i.e. sand pack, drilling skin, and development effort). However, the resulting hydraulic conductivity data still can provide qualitative information useful for assessing approximate conditions. Slug test results outside the pilot PTW range from $6.6x10^{-5}$ cm/s to $6.0x10^{-3}$ cm/s. Hydraulic conductivity measured within the "roundstone" of the PTW ranged from $1.5x10^{-4}$ cm/s to $2x10^{-2}$ cm/s. This difference may be based on well point construction and/or variability of hydraulic conductivity in the roundstone. All slug test results obtained within the clinoptilolite portion of the PTW are in the $1x10^{-3}$ cm/s range.

4.3 Pre-Construction Groundwater Conditions

The regional groundwater flow patterns in the SWBZ prior to PTW installation were evaluated under a period of low groundwater elevation (August 1998) and high groundwater elevation (February 1997).

The groundwater flow pattern in August 1998 (Fig. 2.6 in Appendix 11) indicated that groundwater flows from southwest of the vitrification test facility generally towards the north-northeast. A uniform hydraulic gradient existed throughout much of the NPGRS/PTW area. The August 1998 groundwater distribution suggests that the groundwater flow direction before construction of the PTW was virtually perpendicular to the current long-axis of the PTW.

The groundwater flow pattern in February 1997 (Fig. 2.7 in Appendix 11) generally resembles that of the low groundwater flow pattern. The groundwater flow direction was nearly perpendicular to the current PTW during this high groundwater condition.

Pre-construction (July 1999) groundwater levels near the PTW from well points WP-25, WP-26 and WP-27 generally indicate that the groundwater was approximately 6.5 to 6.7 feet bgs and had a fairly flat gradient with a slight eastward groundwater flow component near the PTW. This departs from the regional northeastward groundwater flow direction for that area (Fig. 3.3 in Appendix 11). The absence of lithologic information from these monitoring locations limits the ability to determine if the groundwater measurements represent the same flow or a subflow zone.

4.4 Post-Construction Groundwater Conditions

The regional groundwater flow pattern after PTW construction is generally similar to historical patterns except immediately near the PTW, where the flow geometry changed and groundwater now mounds south and west of the wall and within the wall, with an apparent flattening of the horizontal hydraulic gradient to the northeast of the PTW. Unlike pre-PTW-construction flow conditions, the overall flow direction is now due eastward near the PTW, generally *parallel* to its long axis. (See Figures 2.8 and 2.9 in Appendix 11.)

Post-construction groundwater data from twenty-six well points (WP-25 through WP-40 and PZ-01 through PZ-10) allow detailed local analysis of the flow regime. A comparison of the pre- and post-construction hydraulic head data (Fig. 3.3 in Appendix 11) and hydrographs from July 1999 to February 2001 (Appendix 1) indicate the following observations:

- Water levels measured from wells located in the PTW are consistently higher than measurements from well points screened in the native sediments with the exception of water levels measured in WP-25;

- The groundwater elevations measured from well points located inside the PTW are practically identical, indicating a near zero horizontal hydraulic gradient;

Additional detailed observations are presented in Appendix 11.

4.5 PTW Development

The 6-inch lateral pipe and connected 10-inch riser in the roundstone zone were subjected to pumping in July, August, and September 2000 and January 2001. The development efforts were intended to: reduce groundwater mounding in the PTW; minimize preferential pathways around, rather than through the wall; mobilize and remove any low permeability skin that may have developed due to PTW emplacement activities; to qualitatively evaluate hydraulic response in the PTW area during longer duration pumping and; to increase the hydraulic conductivity of the zeolite, roundstone, and adjacent soil; This development has decreased the horizontal hydraulic gradient between the PTW and native soil, but has not eliminated the mounding of groundwater in and around the wall.

Prior to PTW development, several "mini-pump tests" were performed at WP-25 in January and April 2000 to generate a hydraulic pressure response that could be used to evaluate the presence of a lower hydraulic conductivity skin along the interface of the PTW with the native aquifer. Figure 7 shows the final drawdown distribution produced by pumping WP-25 in January 2000. Hydrographs in Appendix 3 show the delayed drawdown response in WP-29 located inside the PTW, which strongly indicates the importance of storage (drainage from specific yield) in the unconfined or less confined PTW. WP-27 on the opposite side of the PTW had the lowest response to pumping, indicating that PTW storage and the low hydraulic conductivity boundary at the PTW-native soil interface is minimizing hydraulic connection.

Figure 8 shows the final drawdown values for the January 2001 development effort, which is considered to be a comprehensive effect of the seven efforts, which are graphically presented in Appendices 4 through 7 and 9.

The drawdown of about 5 ft at WP-29 indicates that the PTW riser pipe and the PTW media is in good hydraulic connection (i.e., the PTW responds like a large rectangular well with uniform drawdown). The drawdown responses indicate that the PTW is best connected to the ambient soil at the western end of the PTW and apparently also moderately connected to the soil in the eastern third, but not at the eastern end, which appears to be a boundary. The drawdown data suggest that lower hydraulic conductivity zones especially exist at the PTW-soil boundary along the east end and north edge, as well as in the western third of the PTW. The possibility that a singular transmissive zone is responsible for a large amount of inflow to the PTW is also possible. The decrease in degree of confinement between the native sediments and the PTW may partially account for the mounded head observed in the PTW during periods of transient head fluctuation.

The February 2000 and 2001 hydrographs and precipitation data plot presented in Appendix 8 indicate that surface-source inflow to the PTW is subdued by the installation of the surface drain in 2001; WP-29 heads that did not exceed WP-25 heads, which had occurred in February 2000. However, the head increases within the PTW are still greater than outside the PTW, suggesting that: (1) surface water infiltration still influences water levels within the PTW and/or (2) pressure response differences between the semi-confined aquifer system and the unconfined PTW system give rise to temporal mounding. These data suggest that both the PTW development and the surface water drain have been effective in reducing, but not completely eliminating hydraulic mounding caused by surface infiltration into the PTW.

The difference in groundwater elevations between well point WP-29 inside the PTW and other external well points both before and after the PTW development and after the surface water drain was installed is shown as follows:

Groundwater Elevation Differences (feet)

	North			South		East	West
	WP-29–WP-30	WP-29–WP-34	WP-29–WP-40	WP-29–WP-28	WP-29–WP-36	WP-29–WP-27	WP-29–WP-25
Before Development	1.44	1.53	1.41	0.72	1.20	1.99	-0.57
After Development, Before Drain Installation	0.85	0.83	0.85	-0.03	0.41	1.18	-0.84
After Development and Drain Installation	0.83	0.81	0.83	-0.02	0.37	1.14	-0.81

Although the groundwater elevation inside the PTW remains consistently higher than ambient elevations (except for the west end), the decreases in head differences caused by development and the surface drain installation indicates hydraulic connection is improving. Appendix 9 provides comparative data generally indicating that the hydraulic response of the ambient system to PTW development is improving with each successive effort.

4.6 Post-Construction Distribution of Sr-90

The spatial and temporal variability in the distribution of Sr-90 following installation of the pilot PTW is presented in Figures 3.6 and 3.7 of Appendix 11 and Figure 4. The pre- and post-PTW installation distribution of Sr-90 verifies that the pilot PTW is located within the western fringe of the 2nd lobe and confirms a regional north-northeast migration pattern. Review of the Sr-90 data leads to the following observations:

- Sr-90 in groundwater sampled from within the clinoptilolite of the PTW is low to negligible indicating both removal of Sr-90 due to ion exchange processes and perhaps the influence of lower activity source water (area around WP-25);

- Elevated Sr-90 migration towards the PTW from the south-southwest is reduced in WP-28 and WP-36, located about 5 feet north and south of the west end of the PTW, due to possible treatment with the PTW. This suggests a short southerly flow vector towards WP-28 caused by the minor mounding in the wall.

- The high Sr-90 at PZ-09, which is located within the roundstone, may represent direct influx to the PTW and the absence of zeolitic fines capable of removing Sr-90 from the groundwater within this portion of the gravel section.

- Sr-90 activity in WP-27 at the east end of the PTW increased from about 10,000 pCi/L to 15,000 pCi/L after the sheet piles were installed and then to 40,000 pCi/L immediately after removal of the sheets.

The Sr-90 trends at wells downgradient of the PTW indicate the following trends depending on their proximity to the PTW:

- Low Sr-90 at WP-30 (<5,000 pCi/L) is partially due to the inflow of lower activity groundwater from the west and by the local outflow of low-activity (i.e., treated) water from the PTW;

- Sr-90 at WP-34 located approximately five feet north of the PTW has steadily increased to over 30,000 pCi/L in the past 12 months, which is contrary to a three-month long decreasing trend that followed the removal of the sheet piles. The current increasing trend may be related to the possible underflow of Sr-90 below the central and eastern portions of the pilot PTW. The general Sr-90 trend at WP-34 is consistent with the Sr-90 trend at up gradient well point WP-26 suggesting that both wells may be along a similar flow path even though they are on opposite sides of the pilot PTW.

- Sr-90 at WP-35 located approximately 20 feet north of the PTW also shows a similar rate of increase as WP-34 indicating continued migration of the Sr-90 lobe to the north-northeast.

- After an initial increase, Sr-90 at WP-40 has significantly decreased since November 2000. The decrease may be caused by slowly migrating low activity (i.e., treated) groundwater emanating from the northeastern end of the PTW toward this well location.

4.7 Hydrogeologic and Hydraulic Conditions Summary

The hydrogeologic description of PTW conditions discussed in sections 4.0 through 4.6 is not without some uncertainty but it can be used to evaluate engineering solutions to either restore the intended hydraulic performance of the system, or modify the design to promote the treatment of Sr-90 contaminated groundwater.

The hydrogeologic description may be used to further develop a conceptual model that will provide input to a groundwater flow model. The overall hydraulic performance of the pilot PTW likely is controlled by the following conditions that should be accounted for in the setup of a model to ensure probable site conditions are simulated:

- a predominantly more eastward groundwater flow direction than initially anticipated (the PTW was oriented for predominantly northward flow and did not include lateral hydraulic controls to direct flow into the PTW);

- a highly heterogeneous and anisotropic aquifer sequence of fine and coarse sediments;

- a relatively narrow zone of high activity Sr-90 water that exists topographically low in the aquifer and increases in concentration from the west end of the PTW to the east end; this flow path is partially diverted around the east end of the PTW;

- a hanging central and eastern portion of the pilot PTW likely allows some underflow of high Sr-90 activity groundwater;

- a discontinuous skin of fine zeolitic material at the contact with the zeolite/roundstone backfill and native aquifer material resulting from installation activities;

- C slow discharge of dilute, low Sr-90 activity water from portions of the PTW, which is evident in some wells located close to the PTW; and,

- C continuing, although reduced, direct surface water infiltration into the PTW.

Though not a specific physical condition, the scale of the test also influences the observed performance because it is a relatively small-scale test that cannot absorb the influences from the high degree of heterogeneity and complexity (in aquifer material and the direction of the hydraulic gradient) associated with the local system. Large-scale implementation of a PTW would counteract such small-scale conditions by preventing flow adjustments and generally forcing the flow system to employ the wall in a steady-state flow net.

Although current conditions at the pilot PTW indicate that the system is not performing from a hydraulic perspective as intended, this pilot test successfully identified specific technical issues that can be addressed and designed for prior to deploying an effective full-scale system. Prior to designing such a full-scale system, the identified technical issues that likely limit the hydraulic performance of the pilot PTW should be further evaluated so that proper remedies to these issues can be appropriately engineered. To support this, a focused data collection program is proposed as described in Section 7.1.

5.0 Performance Assessment

Section 4.0 indicates that groundwater from the south and expected regional up-gradient direction likely is flowing around the PTW to the east, with groundwater from the west entering the PTW. This section discusses potential causes of limited flow through the PTW that could have resulted from the design and construction of the PTW.

5.1 Smearing of PTW Sidewalls

The driving and extraction of sheet piles likely "smeared" some fine-grained materials along the interface between the sheet piles and the native soil, possibly creating a skin of lower permeability material around the PTW. The interlayered fine-grained and coarse-grained units of the SWS are more susceptible to this smearing and resulting hydraulic conductivity reduction. This smearing would only marginally affect thicker, coarser-grained water bearing zones, but greatly affect thinner water bearing zones, which may normally act as outflow pathways from the PTW. If hydraulic heads are higher in these coarser units and smearing is prevalent in select smaller outflow zones, then higher water levels would be observed in the PTW than are observed in well-points and piezometers outside the PTW because the wells would be better developed.

The magnitude of smearing caused by driving and extracting the sheet piles, and its effect on the local hydraulic conductivity of native materials has not been extensively studied. The magnitude of this effect depends on soil strength and plasticity, the thickness of the coarse- and fine-grained units, the thickness of the sheet piles, and the length of time the sheet piles are in the ground. The installation of monitoring wells screened strictly in the interlayered SWS near the PTW will indicate whether piezometric water levels are higher in this unit than in the overlying finer-grained AS&G. If water levels in these wells are similar to water levels measured in the PTW, then smearing of smaller "outflow" layers could be the sole cause of the observed hydraulic regime.

5.2 Consolidation of Clinoptilolite

The surface of the clinoptilolite and roundstone settled about 4 feet as the sheet piles were withdrawn, which equals about 1,000 cubic feet. The inclinometers to the north and south of the PTW showed approximately 3 and 7 inches of movement into the excavation, respectively, or an estimated excavation volume loss of about 85 cubic feet to the north and 113 cubic feet to the south due to soil decompression and movement into the PTW. (See Appendix 10 for inclinometer data.) A resulting backfill volume loss estimate of 1,200 cubic feet after sheet pile removal probably resulted from four mechanisms: the volume of the extracted sheet piles (200 cf), consolidation of the clinoptilolite (150 cf), crushing of the clinoptilolite (200 cf), and movement of the clinoptilolite into the roundstone zone (530 cf). The sheet-pile volume of 200 cubic feet was estimated via a cross-sectional area of 0.192 square feet per sheet pile multiplied by 40 sheet piles inserted to an average depth of 26 feet.

The approximate 15% to 20% consolidation of clinoptilolite may affect the hydraulic performance of the PTW as shown in previous laboratory tests (Rabideau, 2000), where loose samples of clinoptilolite had a hydraulic conductivity of about 1.2×10^{-1} cm/sec and consolidated samples a hydraulic conductivity of 4.0×10^{-2} cm/s. Consequently, the consolidation of the material without crushing the grains (see below) would not sufficiently reduce hydraulic conductivity to prevent groundwater flow through the PTW.

5.3 Crushing of Clinoptilolite

The clinoptilolite used in the PTW was manufactured as a 14x50 mesh size, but can be easily crushed by mechanical disturbance, which will reduce its hydraulic conductivity. However, material testing performed at UB still produced a relatively high hydraulic conductivity of 4×10^{-3} cm/sec for compacted clinoptilolite. Mechanical disturbance of the clinoptilolite would have occurred at least three times during construction of the PTW: transportation, placing clinoptilolite in the PTW, and extraction of the sheet piles.

Manufacturer quality assurance testing of the clinoptilolite before delivery showed less than 4 percent fines in the material. The design did not prescribe any specific procedures for transportation, storage, handling or inspection of the clinoptilolite at the site prior to placement in the PTW.

The clinoptilolite was delivered by truck to WVNS from Oregon in supersacks that were susceptible to jostling and vibration, which may have caused grain breakage. Although a grain size analysis of delivered clinoptilolite was not performed as a quality assurance check, the limited amount of fines generated would not have a significant impact on the hydraulic conductivity of the material.

The clinoptilolite may have also been crushed by its placement in the PTW; the zeolite was simply dropped into the cofferdam from up to 30 feet at the beginning, which probably generated fines from crushed and abraided clinoptilolite grains. However this effect cannot be easily quantified.

In addition, the clinoptilolite nearest the cofferdam sheet piles was intensely disturbed and almost certainly crushed when the sheet piles were withdrawn with a vibratory hammer. The clinoptilolite grains greater than 2 feet from the sheet piles along the west, north, and east lengths likely suffered only minor breakage, with the roundstone along the south side of the PTW buffering the clinoptilolite grains from breakage. The clinoptilolite near the bottom of the PTW would also be subject to severe crushing because of burial stresses. The addition of water to the PTW prior to sheet pile extraction would have absorbed some of the vibratory energy and provided pore pressure to reduce burial stress and thus reduced grain crushing near the sheet piles and base. The crushed clinoptilolite grains and associated fines present within about 2 feet of the north, east, and west sides of the PTW and near the base of the PTW were likely transported around the PTW as groundwater entered from the west end and affected this crushed grain distribution.

Consequently, the total volume loss due to grain crushing may be about 200 cubic feet, or about 6 to 7 percent of the estimated clinoptilolite volume of 4,875 cubic feet, which corresponds to the volume reduction noted by Rabideau (2000) in compacting clinoptilolite, where the porosity reduced from 0.51 to 0.48, or a volume reduction of 6 percent.

5.4 Clinoptilolite Plugging Void Spaces in Roundstone

The grain size of the roundstone was 90-100% smaller than 0.5 inches, and 0-15% smaller than 0.25 inches, while the grain size of the clinoptilolite varied between 0.055 and 0.017 inches, which is about one tenth that of the roundstone. Consequently, the clinoptilolite could easily penetrate the void spaces within the roundstone as could some of the adjacent soils. Inclinometer data show the compression of the southern wall into the PTW, which promulgated clinoptilolite grains to penetrate into the roundstone and subsequently flow into PVC riser pipe at the base of the roundstone zone.

The presence of clinoptilolite within the voids of the roundstone likely reduced the hydraulic conductivity of the roundstone. If the clinoptilolite filled all of the voids of the roundstone (assuming 30 percent porosity) about 530 cubic feet of clinoptilolite would be moved into the roundstone.

5.5 Fines Movement as PTW Filled with Groundwater

The PTW was not filled with water before the cofferdam was removed, thus upon sheet-pile extraction the inflow from approximately 20 feet of hydraulic head difference (from PTW base to the exterior potentiometric surface) would have turbulently transported clinoptilolite fines to the edges of the PTW as the water filled the PTW. As the first sheet piles were removed at the west end of the PTW, groundwater under an assumed average linear flow velocity of 0.2 feet per second flowed into the cofferdam. This rate likely varied due to clinoptilolite heterogeneities from the bottom of the cofferdam (compacted) to the top (loose). Inflow would have transported fines to the edges of the PTW, eventually ceasing when the water level reached equilibrium with adjacent groundwater levels. The approximately 10,000 gallon void space of the dry PTW likely was filled on the order of tens of hours, or within one to two days.

Since the sheet piles were first extracted at the west side of the PTW near WP-25 (a high-head area), any fines along this side would have been flushed into the PTW towards the non-contributing edges (south, east, and north sides). Additional fines generated when extracting sheet piles along the south, east and north sides of the PTW would then contribute to this fine-grained zeolitic “skin.”

Since this was a transient effect that occurred when the sheet piles were withdrawn, the mechanism cannot easily be replicated, and other data that could be collected to support it may prove inconclusive. The hydraulic testing (pump tests) that were performed on the PTW indicate a less conductive skin is present.

5.6 Groundwater Flow Under PTW

As-built information suggests that the PTW was not excavated to the top of the Lavery Till, which is possibly allowing groundwater and Sr-90 to flow under the central and eastern portions of the PTW, as indicated by the following data.

C Sr-90 trends in several WPs north and south of the PTW are somewhat similar and are most notable in the Feb. 2001 data when all PTW WPs and PZs were sampled. (See Appendix 2) In addition, increasing in Sr-90 at downgradient WPs 34 and 35 may be coincident with the highest activity observed in the 1997 data, which indicate that a preferred flow zone in a possible finger-shaped lens may extend south to WP-26 and PZ-01, north to WP-16, and further north to GP-16-97. This “finger” may also be a zone of higher permeability based on soil samples from GP-20-97 (WVDP 1998).

C Head south of the PTW has been . 0.5 ft higher than north of the PTW, which would provide sufficient gradient for groundwater flow and plume transport, if the soil beneath the PTW is not influenced by higher heads within the wall.

C Since Sr-90 is apparently bypassing the wall, the high head within the PTW does not appear to have significantly raised the head in soils beneath the wall, which indicates that the low permeability skin may occur at the base of the PTW as well as the sides.

5.7 Potential Surface Influences

Permeable Cap Over PTW: The cap over the PTW was designed to have a low hydraulic conductivity to prevent infiltration of surface water into the PTW. The 1-inch thick Volclay CG-50 bentonite layer that was placed over the clinoptilolite and roundstone zone has a grain size that is 99 percent smaller than 0.008 inches and 15 percent less than 0.0003 inches, or about one tenth that of the clinoptilolite, and one hundredth that of the roundstone. Since this material could easily fall into the void spaces of the PTW materials, the 1-inch thickness is likely inconsistent due to lateral movement during soil capping and alluviation into the PTW materials, thus compromising its intent on being a low permeability layer. Although this may be an entry point for surface water to the PTW, WVNS believes it to be a minor to negligible factor on overall PTW performance.

The overlying clay capping soil is borrowed Lavery Till that has a hydraulic conductivity of about 1×10^{-7} cm/sec when compacted. However the clay was not compacted when placed over the PTW for fear of crushing the clinoptilolite. Thus potential flow paths through this uncompacted clay allow surface infiltration to enter the PTW. While the cap limits direct infiltration from above, bypass occurs because well hydrographs that show the magnitude of water-level increases within the PTW were greater than increases in the native aquifer during and after rainfall and snow events. These discrepancies became less pronounced, although not eliminated, after the installation of the surface drain to the south (up gradient).

Surface Water Inflow: Hydrographs of PTW-area wells show that water levels within the PTW rise sharply during rainfall and snow events, thus indicating a hydraulic connection between surface water and groundwater within the PTW. Surface water may enter the PTW via the hardstand surface layer that is present about the PTW, especially since the hardstand slopes slightly to the north and allows runoff to flow towards the PTW. A surface drain that was constructed in October 2000 to divert surface water around the southern side of the PTW has reduced the sharp post-precipitation water-level rises within the PTW. Although the volume of flow into the PTW has been reduced by the surface drain, water-level data still indicate that the hardstand layer is somewhat hydraulically connected to the PTW. Although this is an entry point for surface water to the PTW, it is minor to negligible, because of the outward slope of the PTW cap.

5.8 Summary of Performance Assessment

The engineering design relied on traditional construction methods that have been used successfully at other sites. Evaluation of the hydraulics in and around the PTW indicate that hydraulic heads in the PTW are higher than the surrounding aquifer, thereby limiting groundwater flow through the PTW. Consequently, the conclusions regarding the design and construction at the pilot PTW are as follows:

- Sheet pile extraction and mobilization of fines during PTW construction produced a comingled clinoptilolite and roundstone zone and a zone of crushed clinoptilolite particles around the north and east edges.

- The hydraulic conductivity of the crushed material may be up to two or three orders of magnitude less than that for uncrushed clinoptilolite.

- The hardstand is hydraulically connected to the PTW and numerous potential flow paths have been identified through the cap and other surface features.

- The PTW does not appear to be fully penetrating through the upper water bearing zone and likely "hangs" in its central and eastern portions above the top of the Lavery Till potentially allowing underflow.

6.0 Lessons Learned

The previous section has focused on the engineering detail of the PTW design and installation and will assist in the eventual development of engineering alternatives to enhancing the performance of the PTW. Based on the conclusions of the previous sections and the hydraulic evaluation section, several important "lessons learned" are identified and will help create a better future PTW design at the WVDP:

- Site characterization for PTW design purposes must focus on the location of the proposed installation and cannot rely solely on regional information

- PTW design work must include temporal and spatial data on the three-dimensional distribution of target contaminants in the proposed location

- Hydraulic head information should focus locally and include a sufficiently wide area to account for potential spatial (both lateral and vertical) and temporal changes to the direction and magnitude of the hydraulic gradient. This information should be collected before and after PTW construction so that the effect of the PTW on the local hydraulic regime can be understood.

- Stratigraphic information must be sufficiently detailed in the vicinity of the proposed location to accurately design the PTW for proper vertical coverage, and/or penetration of the affected water bearing zone. This stratigraphic information must also be considered in the engineering designing of the excavation support for the PTW.

- Generally, the use of sheet piles to support the excavation for a PTW will modify the local stratigraphy and may affect discrete flow paths. Removal of sheet piles will consolidate any loose or uncompacted material in the PTW, and will allow materials with dissimilar grain sizes to co-mingle. Sheet pile removal may also generate high dynamic stresses within the PTW materials that can break fragile particles of the reactive material within the PTW.

- The hydraulic head within a PTW excavation should be maintained at the top of the emplaced material when the excavation support system is removed to reduce the potential for rapid inflow of water that may mobilize fines or other materials within the PTW during removal of the excavation support.

- The performance of a PTW can be affected by numerous external factors, such as surface water infiltration, utility trenches, etc., that must be addressed during the detailed engineering design. Given the high cost of installation, monitoring and correcting any performance problems, the engineering design should be conservative in addressing site-specific issues that could affect PTW performance. For example, an HDPE liner placed over the PTW treatment materials and appropriately keyed into the surrounding native material would be a more effective cap than the granular bentonite and uncompacted clay cap that was installed.

- The PTW deployment approach must take greater care to avoid potential “skin” effects and pulverization of treatment material, and creation of fines.

- PTWs that hang, or do not completely penetrate an underlying low hydraulic conductivity unit, over all or part of their alignment, generally have greater potential for unintended performance than fully-penetrating PTW designs.

- Continuous wall PTW designs (similar to the WVDP pilot) typically are less complicated to design and build than “funnel and gate” designs, but must fully capture the affected groundwater, including during variations in the direction of the lateral hydraulic gradient.

- If the PTW does not perform as-designed, accurate and well-documented as-built information is critical in understanding the problem and developing suitable remedies. When constructing a pilot PTW to evaluate the technology, this information is even more important.

The conditions and issues that have reduced the intended performance of the pilot PTW also have been problematic for other sites that have deployed PTW technology over the past 10 years. Generally, the remedial effectiveness of PTW technology from a chemical standpoint (e.g., destruction of organic compounds, immobilization of inorganic compounds, and buffering of low pH conditions) has been demonstrated and is fairly well understood as per documentation from the Remediation Technology Development Forum (e.g., see http://www.rtdf.org). Examples and lessons learned from specific sites are summarized in Appendix 11. No reports of full-scale PTW failures due to chemical treatment inadequacies are apparent although laboratory bench-tests have shown limitations to various chemical treatment processes. Issues regarding plugging or fouling of a PTW from chemical processes are being studied and monitored at both research and full-scale commercial sites; these processes are anticipated and have apparently not yet diminished the effectiveness of a PTW for a given site.

As is the case for the pilot PTW, most difficulties with permeable reactive barriers generally are due to unintended hydraulic performance resulting in:

- incomplete capture of the affected groundwater (e.g., flow around or below, the PTW).

- design groundwater velocity not being achieved.

- non-uniform flow conditions within the PTW.

Consequently, this PTW assessment had led to the following list of considerations in design and construction of future permeable reactive barrier (PRB) installations at the WVDP:

- Sheet pile cofferdams are probably the most effective means of constructing the PRB, but driving and extraction of the sheet piles will affect local hydraulic conductivity. Thus a factor of safety should be included in the intended Sr-90 capture zone.

- Wing walls should be considered in design to create a remedy that will direct the flow of water through the PRB. Suitable dimensions of the wing wall sections can be developed from careful hydraulic modeling of all anticipated groundwater conditions.

- Geotechnical design should minimize sheet pile penetration of the Lavery Till to reduce the energy required to extract the sheet piles.

- A conservative cap design, using HDPE liners or other impermeable materials should be used to prevent surface water infiltration.

- All possible sources of surface water should be diverted away from the area of the PRB.

- More delicate placement techniques can be developed to place the clinoptilolite.

- The excavation should be filled with water before the sheet piles are removed.

- The divider system between the clinoptilolite and any gravel zones should be removed after the sheet piles are removed.

- Considerations for future designs also should consider using a coarser grain size distribution for the zeolite treatment media, or an aggregate that is less susceptible to grain breakage during construction of the PRB and post-construction movement of soil towards the PRB.

7.0 Proposed Data Collection and Options for Pilot PTW Modifications

The unique hydraulic conditions at the WVDP PTW can be characterized by a small-scale investigation designed to specifically fill data gaps that currently lead to some uncertainty regarding hydraulic conditions near the PTW. (See Figure 1.5 in Appendix 13.) The investigation will target the potential for vertical head distributions, groundwater flow below the PTW, and the source of high head to the west. The data collection program is discussed below:

7.1 Data Collection and Modeling Program

Additional field data will both strengthen the current conceptual model and support development of a three-dimensional numerical groundwater model that can be used to confidently assess an engineering solution to either modify the existing PTW, alter its orientation, or design a new PTW under the unique site specific conditions in the same or different portions of the north plateau. Although the current hydraulic conditions have been well explained, there remains enough uncertainty that a supplemental and cost-efficient field investigation would be beneficial to decrease the degree of uncertainty and increase the potential that an effective engineering solution can be developed and implemented.

Therefore, the following recommendations are presented by WVNS:

1. Install two new wells via hollow-stem auger method and collect continuous stratigraphic information in the vicinity of WP-25 (west-end) and WP-27 (east end) to confirm the water level conditions that appear to provide major control on the assumed groundwater flow directions in the vicinity of the PTW.

2. Drill at least three additional soil borings each adjacent to the north, south, and eastern face of the PTW that are continuously logged for stratigraphic detail to confirm whether the pilot PTW penetrates the underlying till or is “hanging” within the SWBZ. One additional soil boring for stratigraphy should be drilled toward the western end of the PTW, and one additional boring should be drilled near the south face of the PTW. The seven total borings should be converted to 2-inch monitoring wells with vertically distinct screened intervals in order to facilitate focused aquifer testing.

3. Perform a long-term (72 hours to one week) aquifer testing program consisting of a series of step-test periods, a constant-discharge period, and recovery period. The test could be performed using the vertical riser drainpipe installed within the up gradient gravel section of the pilot PTW with observation wells monitored using downhole pressure transducers. Short-term (i.e., 4 to 8 hours) step-tests also should be performed in the new wells recommended in No. 1 and 2 above.

4. Perform a series of aquifer tests on several of the proposed wells to better understand the vertical separation of hydrostratigraphy near the PTW.

The hydraulic and hydrostratigraphic data collected from the supplemental well installations and testing should be integrated into the development of a three-dimensional numerical flow model to better assess and predict the observed hydraulic conditions. The modeling would greatly improve the ability to develop an engineering solution that achieves the level of operational success required by the project stakeholders. This model would also be expanded to include other areas of the north plateau as the PTW program is implemented on a full-scale schedule.

7.2 Modification Options and Recommendation

Four options for modification for the pilot PTW program are presented in this section as follows:

Option 1 No Modifications of Pilot PTW
Option 2 Install Lateral Hydraulic Barriers
Option 3 Install Pilot PTW Extension
Option 4 Install New Pilot PTW

The basic criteria for selecting each option were as follows:

- The Basis for Selection
- Assumptions
- Cost Estimate
- Assessment of Future Performance
- Waste Generation

- Ease of Implementation
- Likelihood of Success

The four options are summarized in the following section. A more detailed description of each option can be found in Appendix 12.

7.2.1 Option 1 - No Modifications of the Pilot PTW

This option consists of completing the assessment of the pilot PTW program and moving forward with making the decision of whether or not to design the full-scale PTW for the site. This option would strictly be used as a lessons-learned that assists in development of information necessary for designing and deploying the full-scale PTW. This option would have zero costs associated with the pilot PTW program.

7.2.2 Option 2 - Install Lateral Hydraulic Barriers

This option consists of installing flow barriers at the west and, possibly, east ends of and perpendicular to the PTW, as shown in Appendix 12, Figure 2.1. The purpose of the flow barriers is to hydraulically isolate the PTW from higher water levels at WP-25, and to redirect the flow of groundwater through the PTW. The length of the barrier walls necessary to cerate the required groundwater flow conditions would be determined from analytical modeling and detailed design. For cost purposes it was assumed a 60 foot barrier would be needed and would cost approximately $80,000 for one end and $130,000 for both ends.

7.2.3 Option 3 - Install Extension to PTW

This option consists of installing an extension on the east side of the existing PTW, as shown in Appendix 12, Figure 2.2. The purpose of the PTW extension is to capture the flow of groundwater that appears to be flowing around the eastern end of the existing PTW. The conceptual design of this alternative assumes an extension of approximately equal length (approximately 30 feet) and width of the existing pilot PTW. The conceptual design of this alternative assumes an extension of approximately equal length (approximately 30 feet) and width of the existing pilot PTW. Again it will be important to use a groundwater flow model to design the final geometry and alignment of this option. This option would be installed using lessons-learned from the original pilot installation and is estimated to cost approximately $400,000.

7.2.4 Option 4 - Install a New Pilot PTW

This option consists of installing a new pilot PTW at a suitable location, either up or down gradient of the existing PTW. Additional soil and groundwater investigation would be performed to locate and design the new pilot PTW, and the new PTW would be constructed in a manner that incorporates the lessons learned from the design, construction and monitoring of the existing PTW. The new pilot would have similar dimensions and construction as the existing pilot PTW. The general cost estimate for this option is $720,000.

The main point of these examples is that designing for hydraulic performance is critical to any PTW application. Comprehensive site characterization is key, and will more likely result in a reliable PTW design that becomes a cost-effective remedy for a given site.

8.0 Proposed Path Forward

8.1 Pilot PTW Path Forward

Based on the evaluation of information prepared by WVNS and Geomatrix Consultants, WVNS is recommending Option 2, Installation of Lateral Hydraulic Barriers. By installing the lateral hydraulic barriers on the western end, it is expected to hydraulically isolate the PTW from higher water level conditions in WP-25 and to redirect the flow of groundwater through the PTW. The eastern sheet pile barrier wall will also isolate the PTW from any anomalous conditions that may be present at the eastern end of the PTW.

The flow barriers will consist of sheet piles driven approximately 1 foot into the Lavery Till. The sheet piles will be driven in interlock to provide a continuous barrier. The length of the barrier walls necessary to create the required groundwater flow conditions will be determined in detailed design after the development of a three-dimensional flow model.

Prior to implementation of this modification option or any other, it will be very important to collect additional characterization data to integrate into the development of the three-dimensional flow model. The following is the proposed recommended path forward for both the pilot PTW and future deployment of PTW technology at the WVDP.

As a first step, further characterization and assessment of the local geology and hydrogeology near the pilot PTW, to decrease the degree of uncertainty with pilot PTW performance issues and increase the potential to select and implement an effective engineering solution is recommended. This effort will include the installation of at least seven continuously sampled boreholes that will be completed with a 2-inch diameter groundwater monitoring well. Four of the boring wells will be located along the western, southern, and eastern sides of the pilot PTW and completed with a 15-foot well screen. The additional three wells will be completed with 5-foot well screens, installed at various depths (Fig. 3.1 in Appendix 12). Water levels and groundwater sampling for these new wells would be integrated into the operational monitoring program to provide vertical gradient information. A hydraulic testing program will also be conducted to confirm the influence and distribution of skin at locations around the pilot PTW and to better assess the distribution of hydraulic conductivity in and around the pilot PTW. Cost estimates for this data collection program are listed in Appendix 13.

8.2 Full-Scale PTW Path Forward

Secondly, it is recommended that the1st lobe preliminary design proceed. FY2001 preliminary design activities begin with selection of wall location then proceed with design and implementation of a comprehensive soil and groundwater characterization program. This will be carried out on a local scale and include the collection of hydrogeologic data through the installation of boreholes and conventional 2-inch wells, and conducting seasonal pumping tests to thoroughly understand the local hydrogeology.

Activities will continue with evaluation of geological and hydrogeological data. Once the data is thoroughly analyzed, a conceptual design may commence. At completion of conceptual design a decision will be made as to whether full-scale deployment is feasible on the 1st lobe. A project schedule can be found in Appendix 14.

By the completion of the aforementioned steps, there will be increased confidence that both a reliable solution to the pilot's performance can be selected and further PTW design and installations can be successfully applied at the WVDP.

9.0 References

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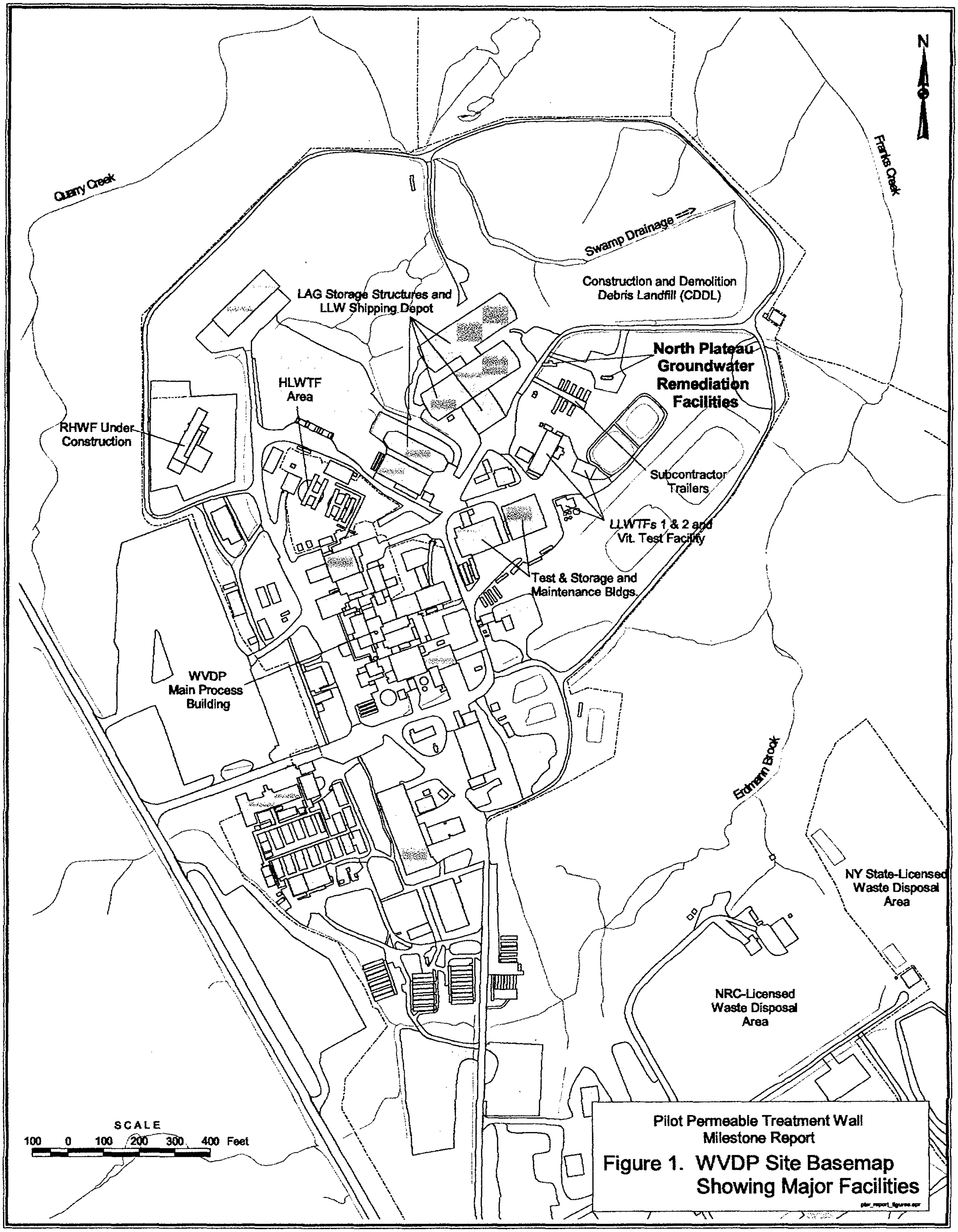
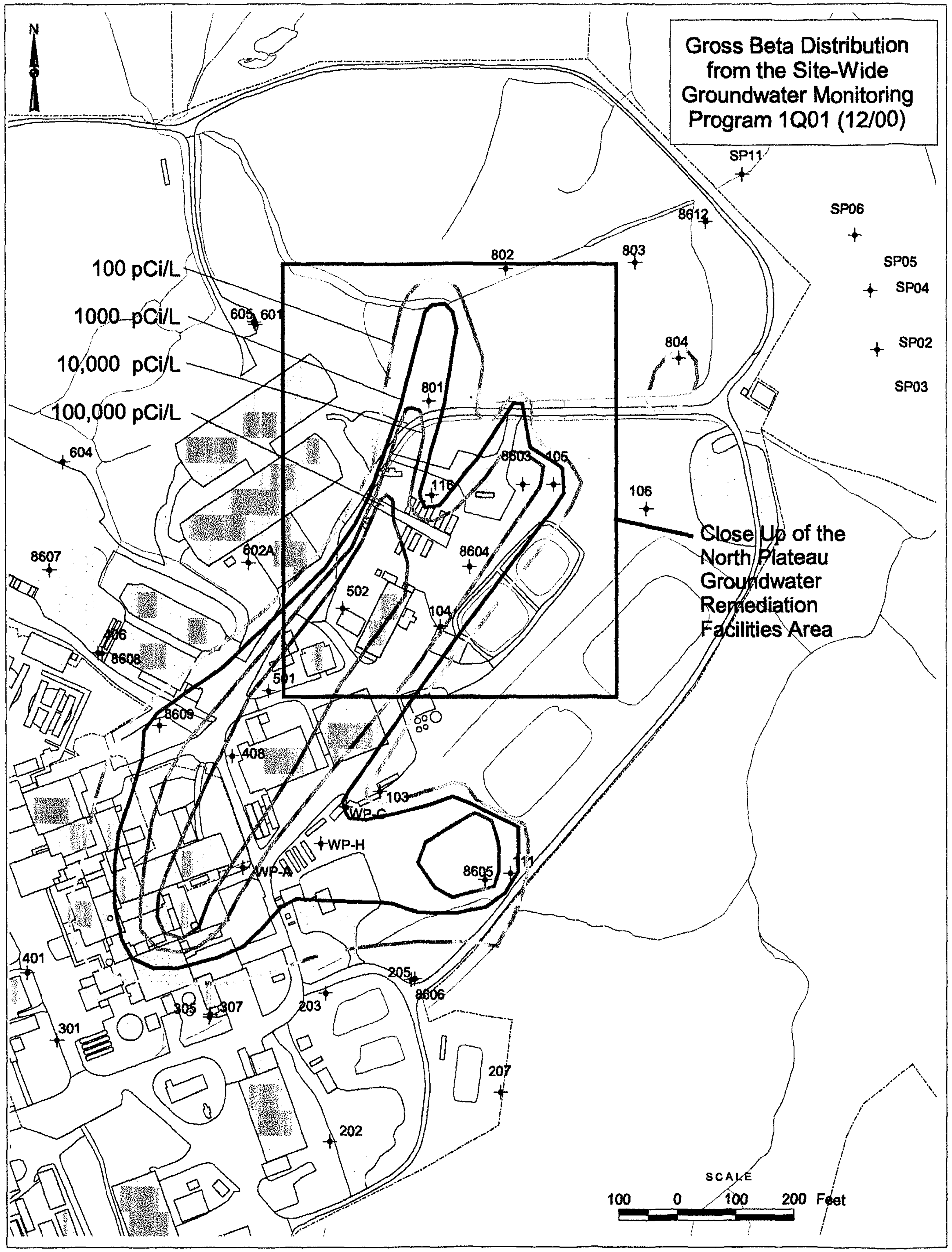
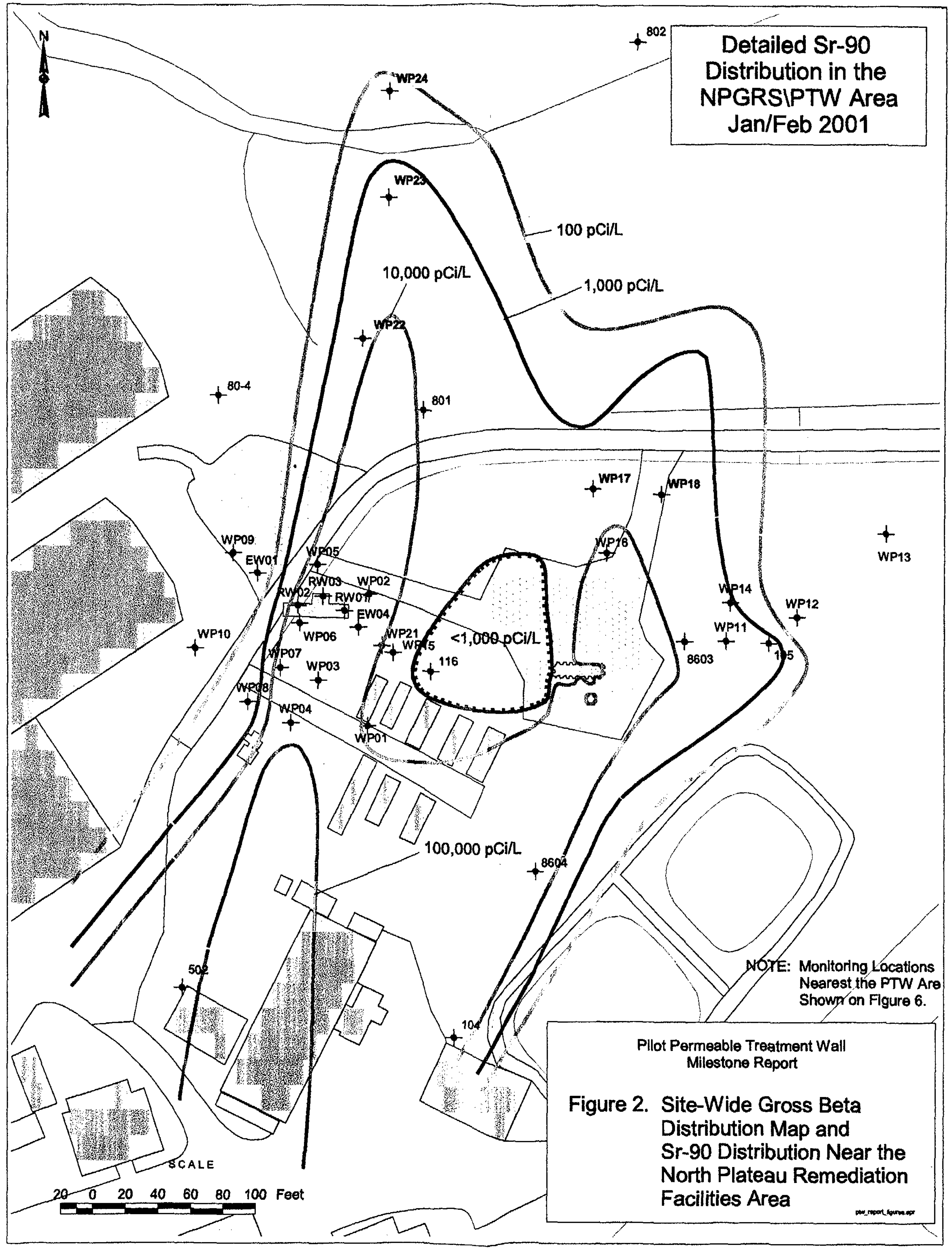
FIGURES



Pilot Permeable Treatment Wall
Milestone Report

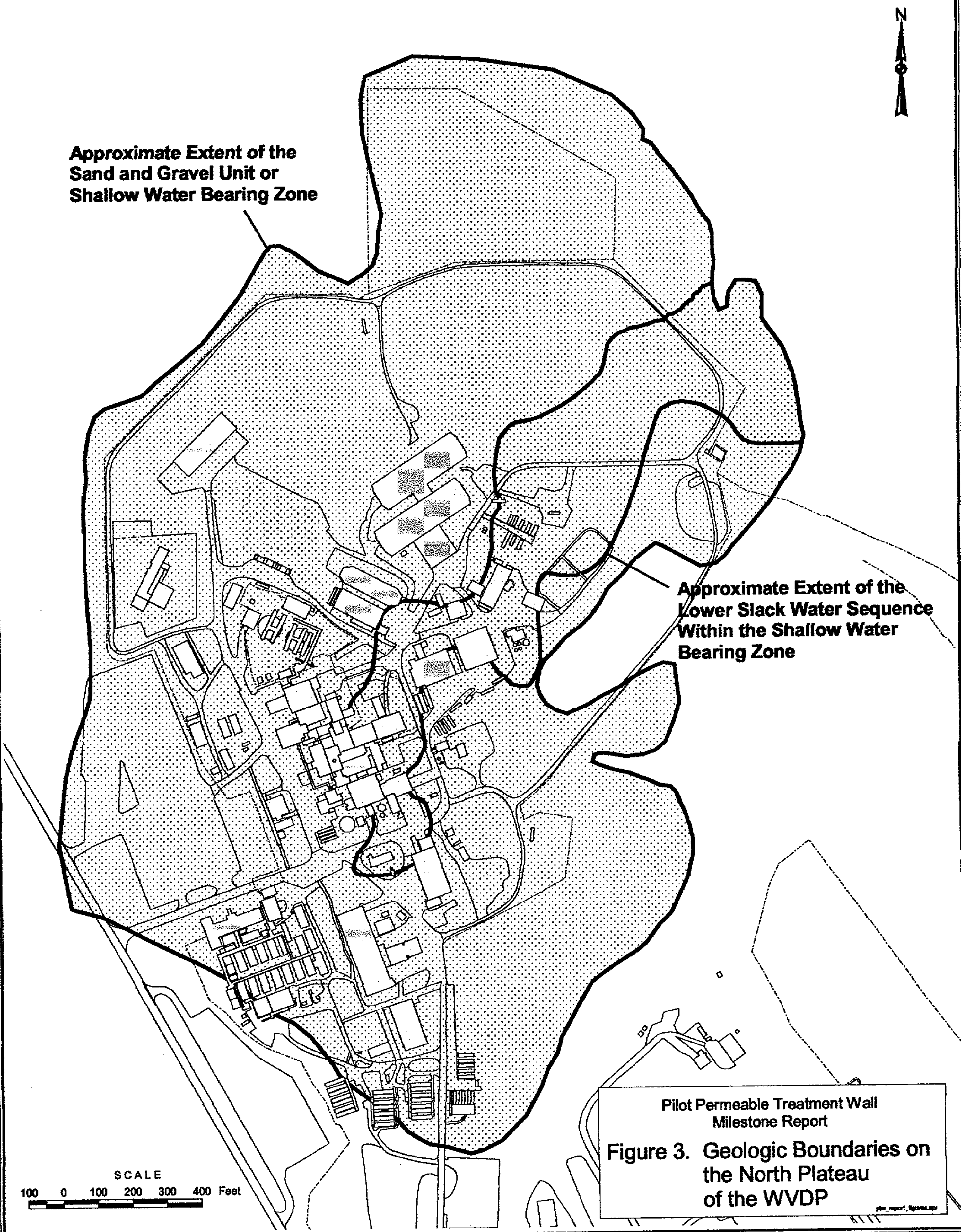
Figure 1. WVDP Site Basemap Showing Major Facilities





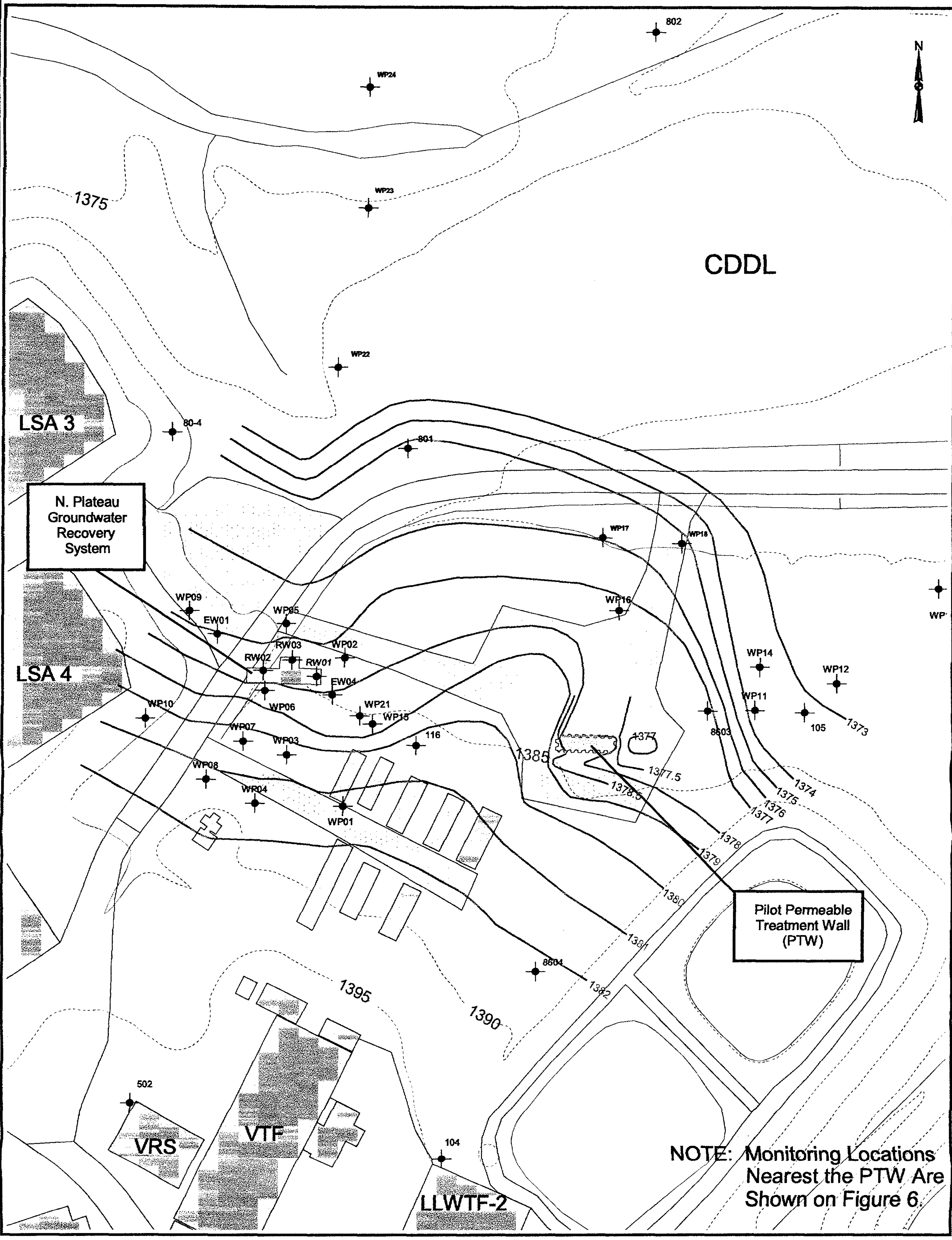
Pilot Permeable Treatment Wall Milestone Report

Figure 2. Site-Wide Gross Beta Distribution Map and Sr-90 Distribution Near the North Plateau Remediation Facilities Area



Pilot Permeable Treatment Wall
Milestone Report

Figure 3. Geologic Boundaries on the North Plateau of the WVDP



SCALE

20 0 20 40 60 80 100 120 Feet

Legend

- NPGRS/PTW-Operational Wellpoints, Piezometers, and Monitoring Wells
- Piezometric Contours (C.I. = 10. ft.) January 24, 2001 Pre-Development
- Topographic Contours (C.I. = 5.0 ft.)

Pilot Permeable Treatment Wall
Milestone Report

Figure 4. North Plateau Groundwater Remediation Facilities: Monitoring Locations, Topography, and Groundwater Elevations

SCALE

100 0 100 200 300 400 Feet

Legend

801

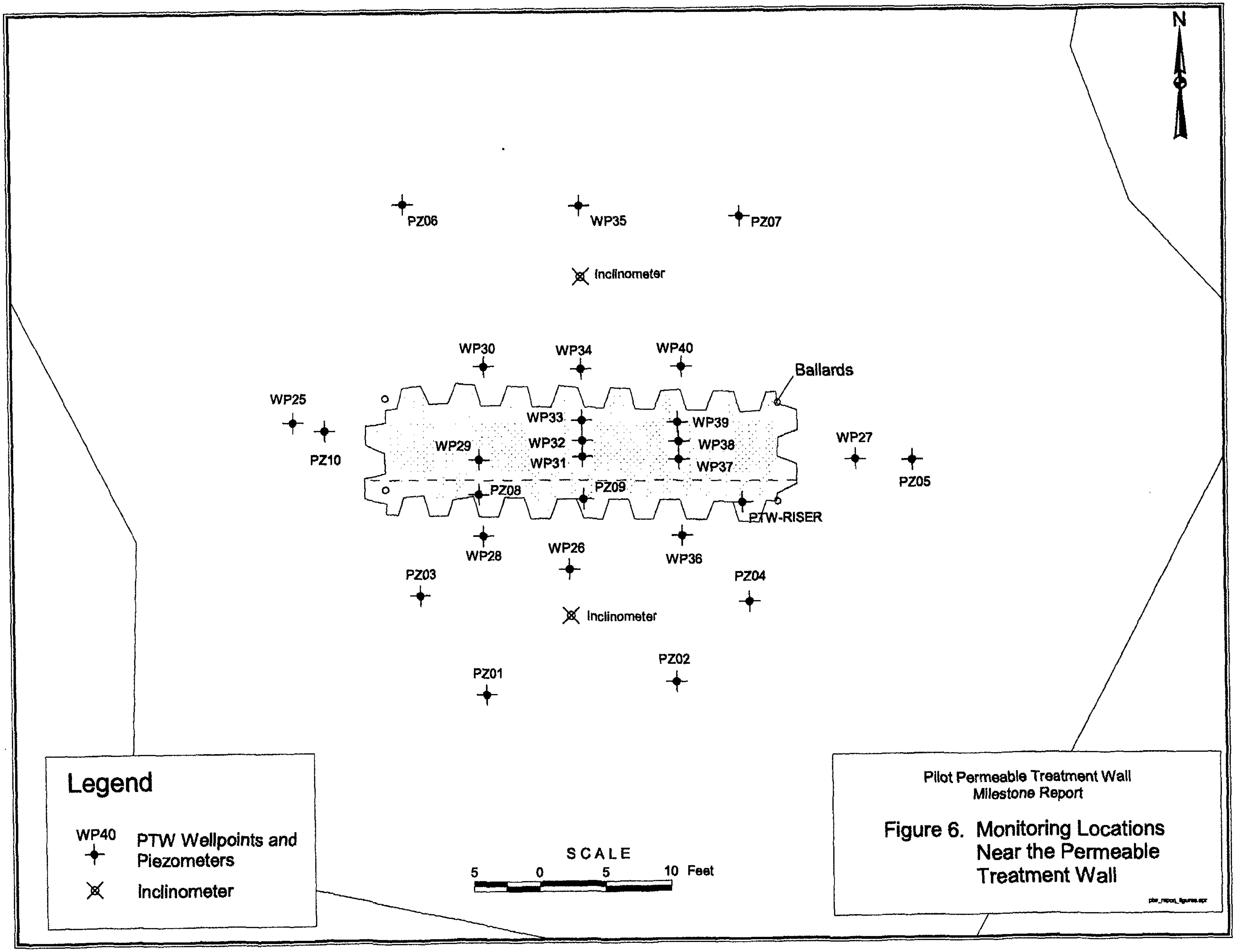
Groundwater Monitoring Program Location

Piezometric Contours (C.I. = 2 ft.)
11/30/00 or 1Q01 Data

Pilot Permeable Treatment Wall
Milestone Report

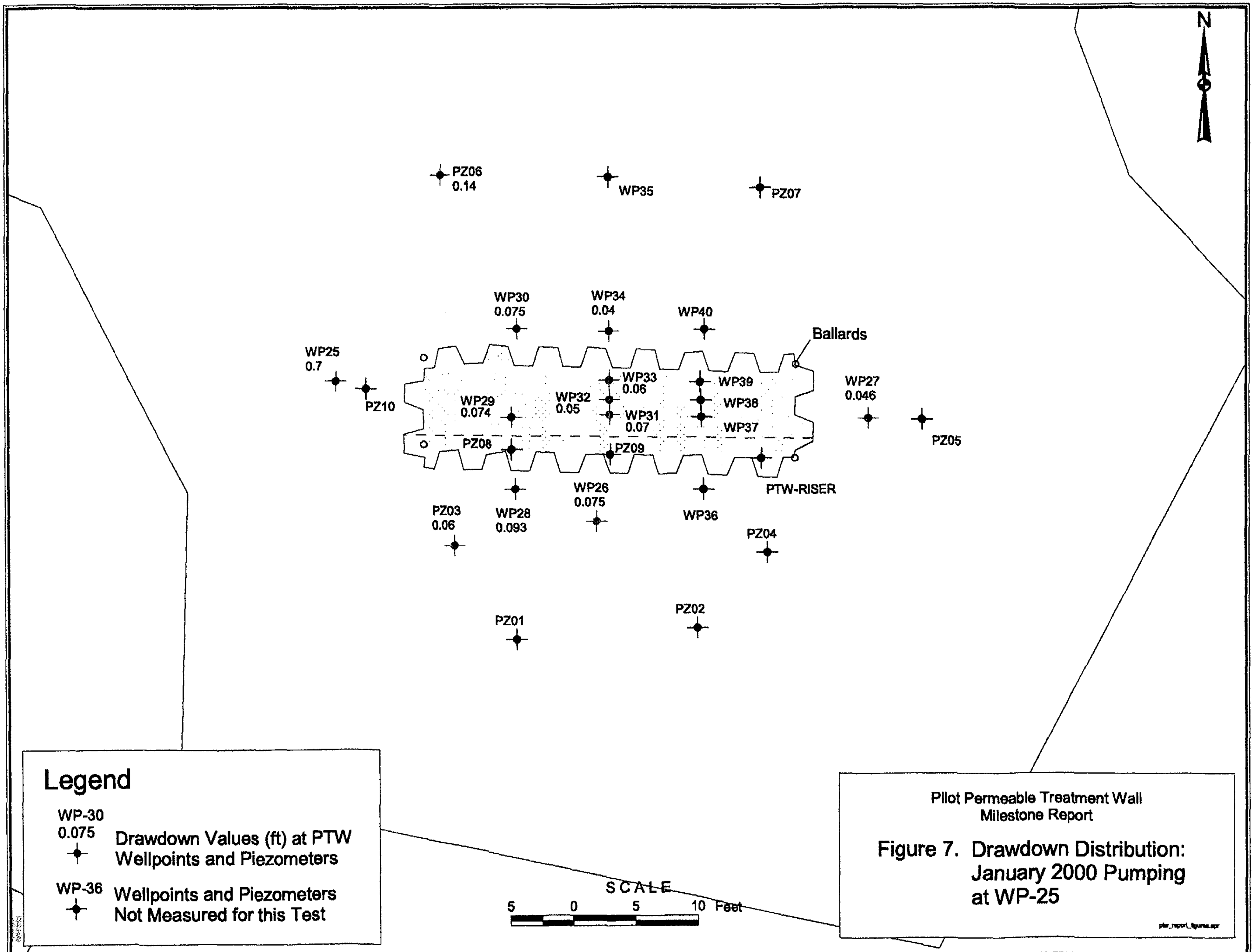
Figure 5. Regional Groundwater Elevations

pile_report_figures.apr



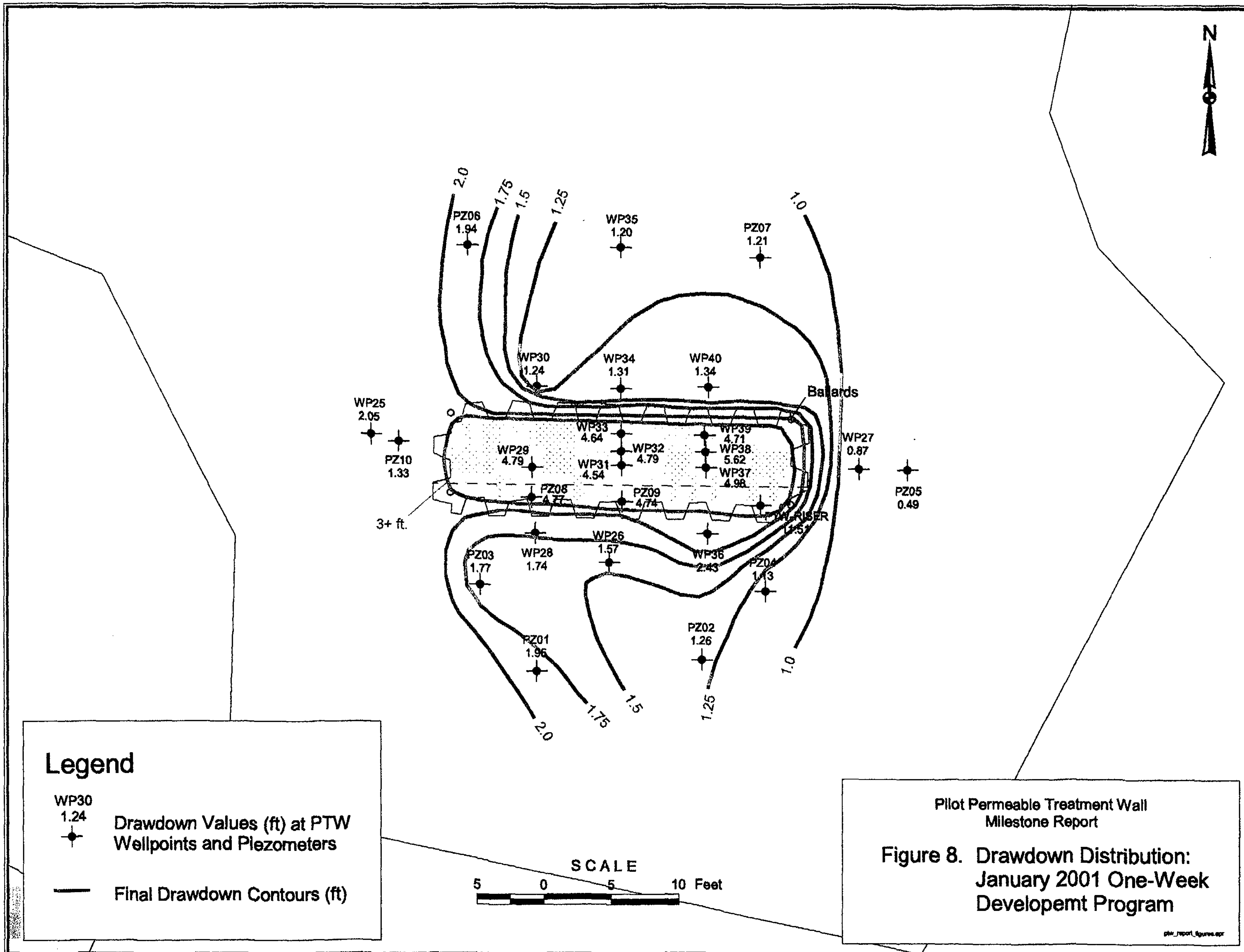
Pilot Permeable Treatment Wall
Milestone Report

Figure 6. Monitoring Locations Near the Permeable Treatment Wall



Pilot Permeable Treatment Wall
Milestone Report

Figure 7. Drawdown Distribution: January 2000 Pumping at WP-25



Legend

WP30
1.24
Drawdown Values (ft) at PTW Wellpoints and Piezometers

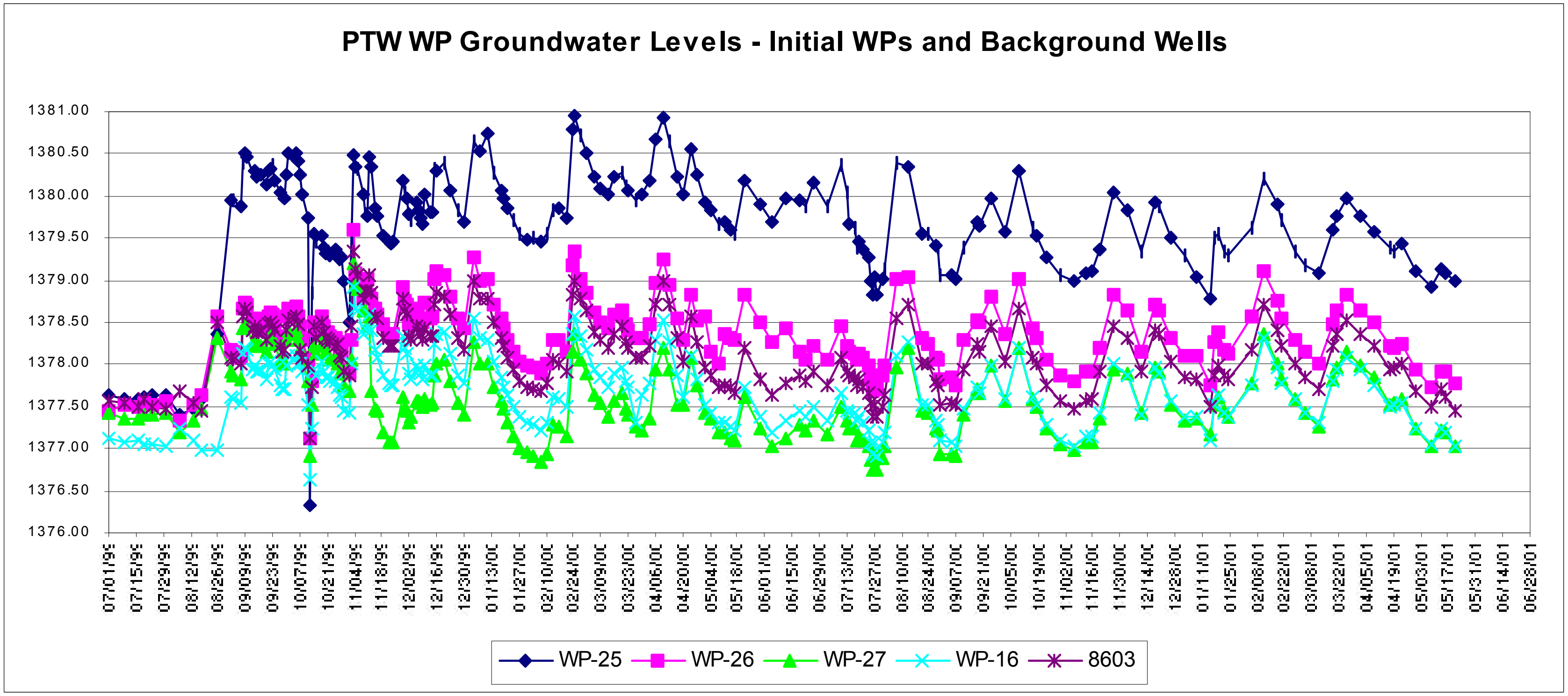
Final Drawdown Contours (ft)

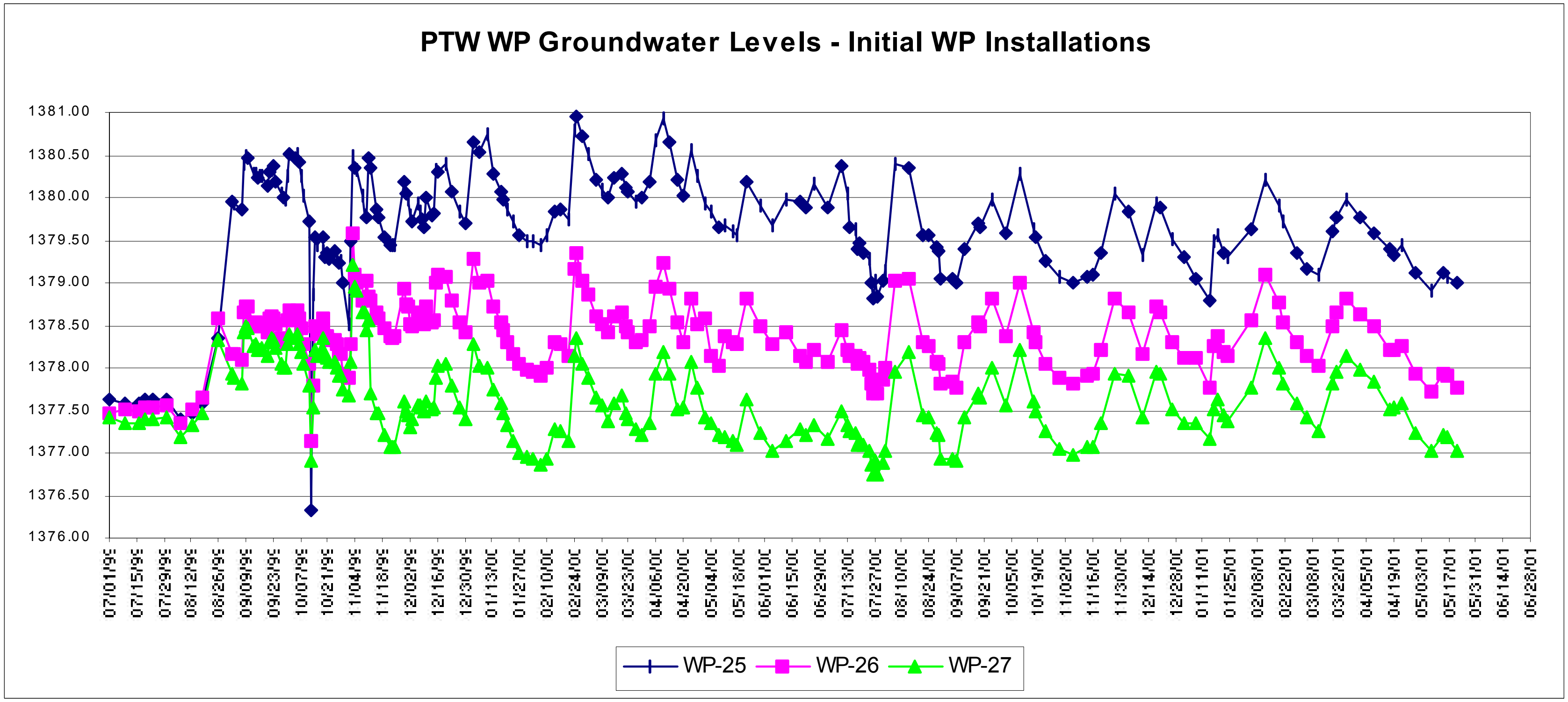
Pilot Permeable Treatment Wall
Milestone Report

Figure 8. Drawdown Distribution: January 2001 One-Week Developemt Program

ptw_report_figures.apr

APPENDIX 1
GROUNDWATER ELEVATION DATA


PTW WP Groundwater Levels - Initial WPs and Background Wells
1381.00
1380.50
1380.00
1379.50
1379.00
1378.50
1378.00
1377.50
1377.00
1376.50
1376.00
WP-25
WP-26
WP-27
WP-16
8603


PTW WP Groundwater Levels - Initial WP Installations
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1380.50
1380.00
1379.50
1379.00
1378.50
1378.00
1377.50
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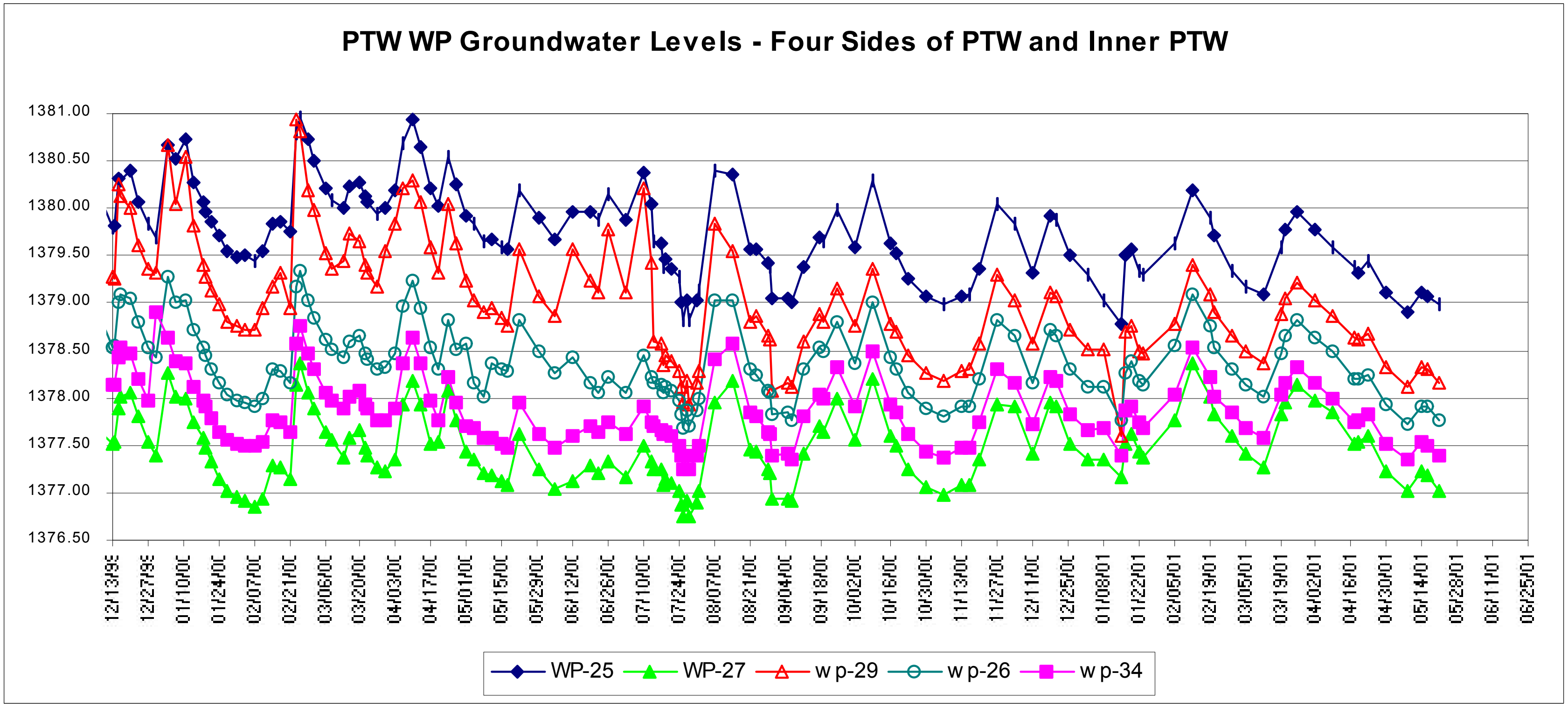
1376.50
1376.00
WP-25
WP-26
WP-27


PTW WP Groundwater Levels - Four Sides of PTW and Inner PTW
1381.00
1380.50
1380.00
1379.50
1379.00
1378.50
1378.00
1377.50
1377.00
1376.50
12/13/99
12/27/99
01/10/00
01/24/00
02/07/00
02/21/00
03/06/00
03/20/00
04/03/00
04/17/00
05/01/00
05/15/00
05/29/00
06/12/00
06/26/00
07/10/00
07/24/00
08/07/00
08/21/00
09/04/00
09/18/00
10/02/00
10/16/00
10/30/00
11/13/00
11/27/00
12/11/00
12/25/00
01/08/01
01/22/01
02/05/01
02/19/01
03/05/01
03/19/01
04/02/01
04/16/01
04/30/01
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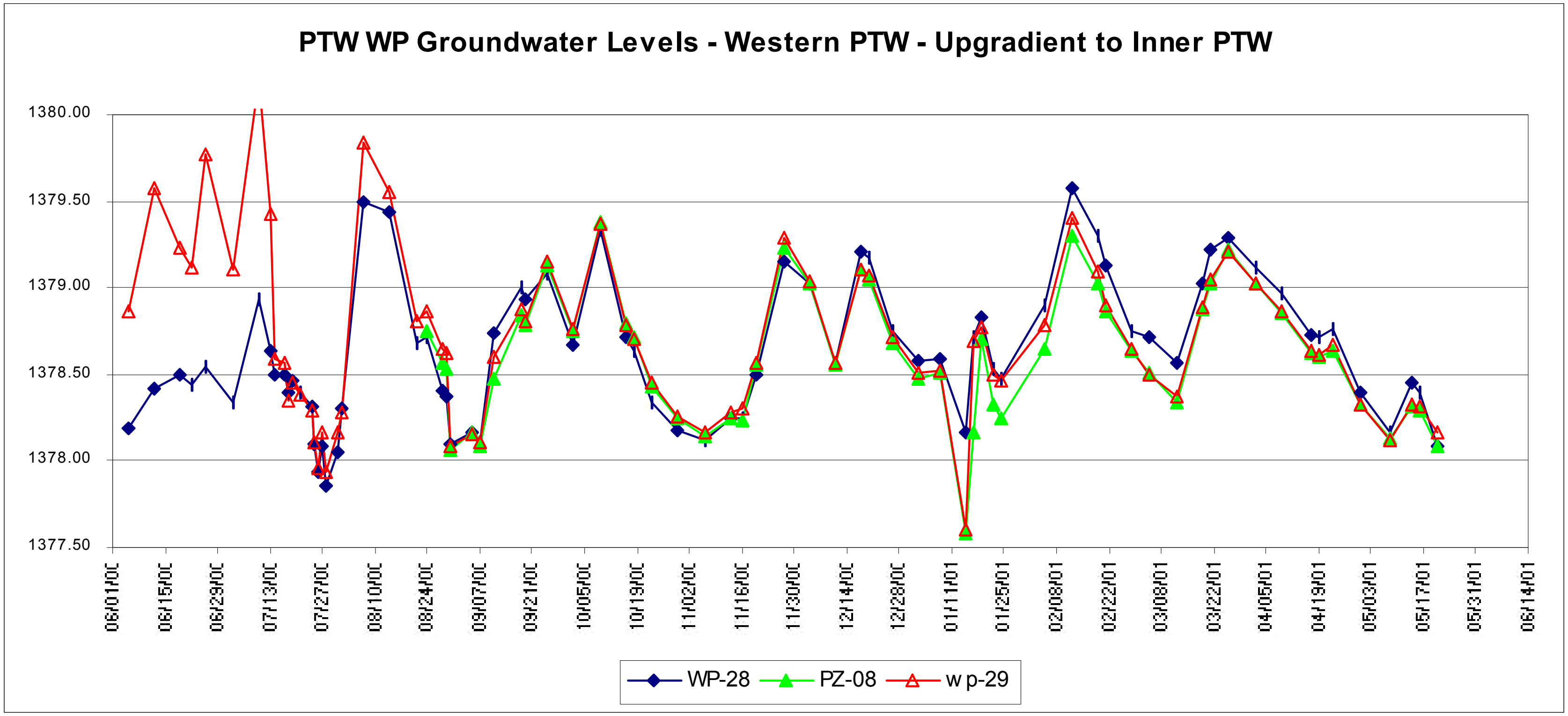
06/11/01
06/25/01
WP-25
WP-27
wp-29
wp-26
wp-34



PTW WP Groundwater Levels - Western PTW - Upgradient to Inner PTW
1380.00
1379.50
1379.00
1378.50
1378.00
1377.50
06/01/00
06/15/00
06/29/00
07/13/00
07/27/00
08/10/00
08/24/00
09/07/00
09/21/00
10/05/00
10/19/00
11/02/00
11/16/00
11/30/00
12/14/00
12/28/00
01/11/01
01/25/01
02/08/01
02/22/01
03/08/01
03/22/01
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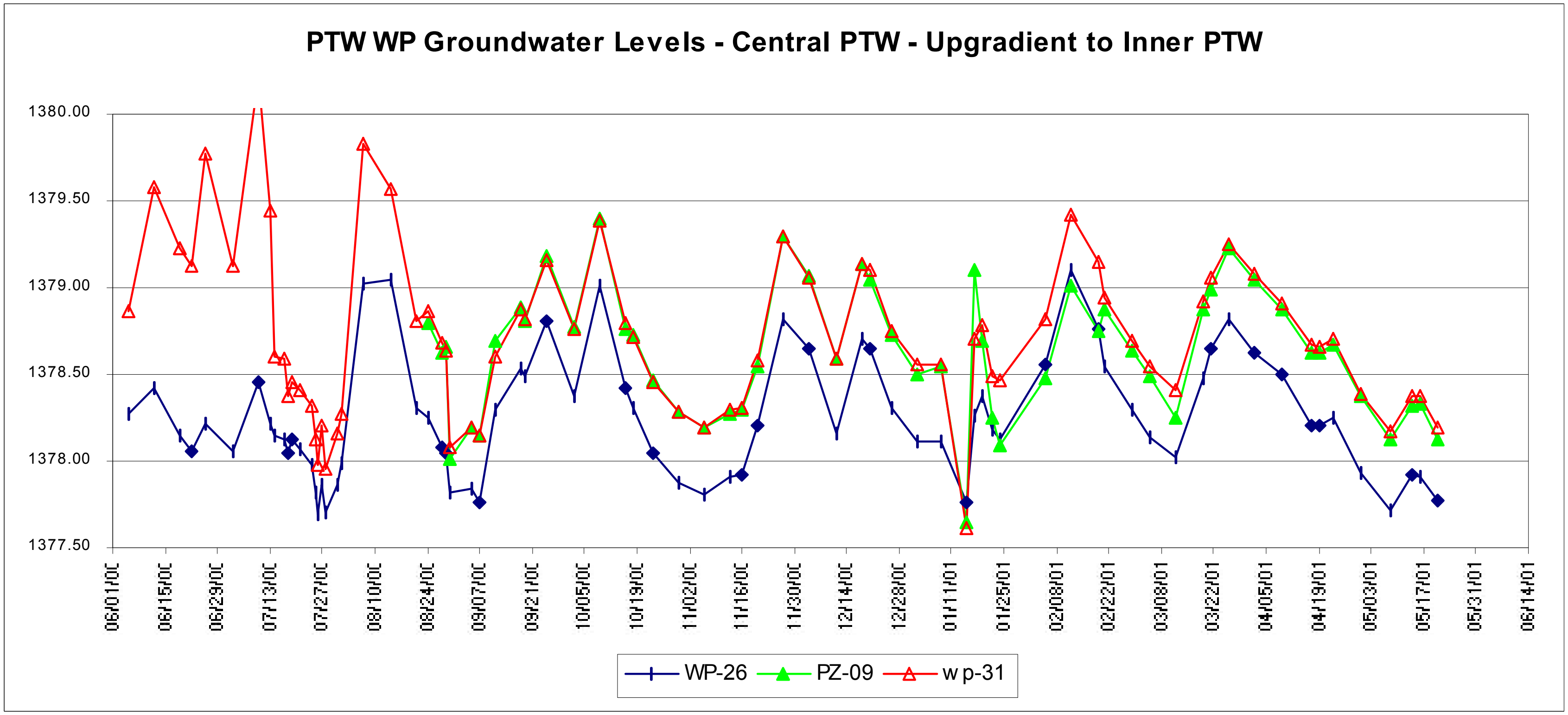
WP-28
PZ-08
wp-29



PTW WP Groundwater Levels - Central PTW - Upgradient to Inner PTW
1380.00
1379.50
1379.00
1378.50
1378.00
1377.50
06/01/00
06/15/00
06/29/00
07/13/00
07/27/00
08/10/00
08/24/00
09/07/00
09/21/00
10/05/00
10/19/00
11/02/00
11/16/00
11/30/00
12/14/00
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01/11/01
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03/08/01
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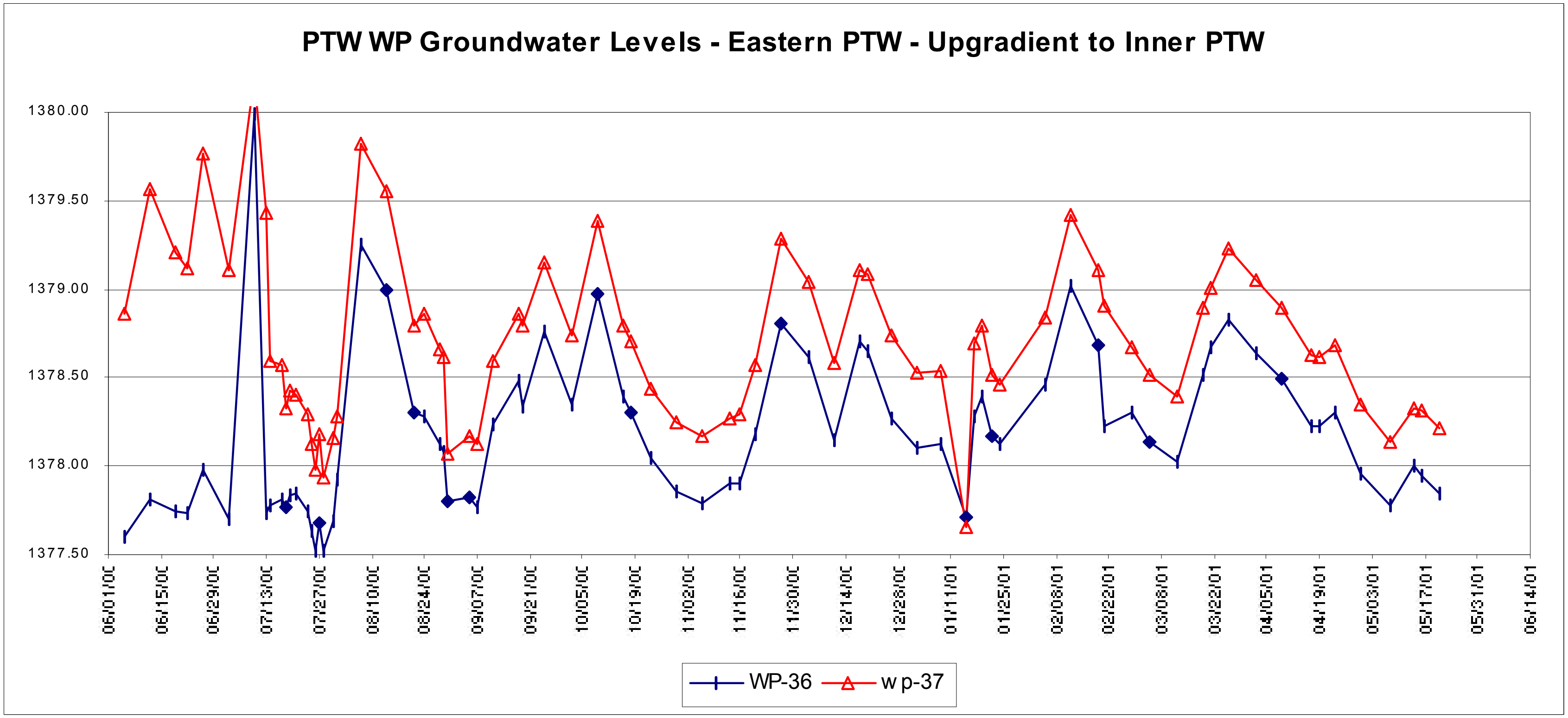
WP-26
PZ-09
wp-31



PTW WP Groundwater Levels - Eastern PTW - Upgradient to Inner PTW
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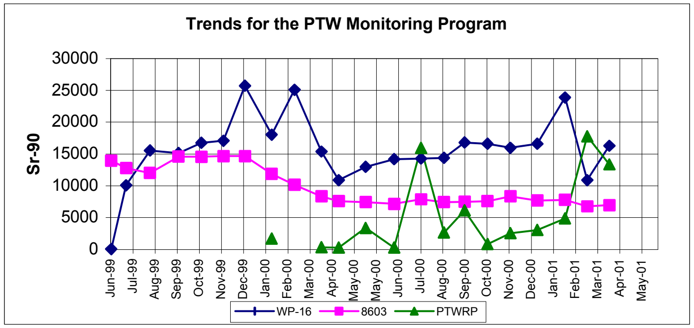
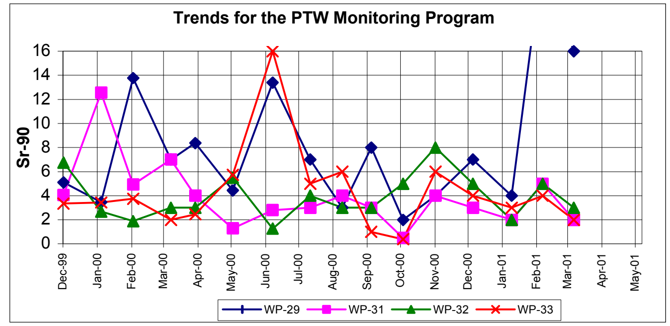
1379.50
1379.00
1378.50
1378.00
1377.50
06/01/00
06/15/00
06/29/00
07/13/00
07/27/00
08/10/00
08/24/00
09/07/00
09/21/00
10/05/00
10/19/00
11/02/00
11/16/00
11/30/00
12/14/00
12/28/00
01/11/01
01/25/01
02/08/01
02/22/01
03/08/01
03/22/01
04/05/01
04/19/01
05/03/01
05/17/01
05/31/01
06/14/01
WP-36
wp-37

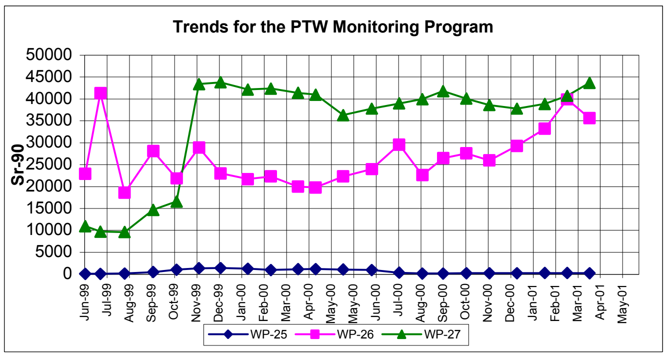
APPENDIX 2
GROUNDWATER SAMPLING RESULTS



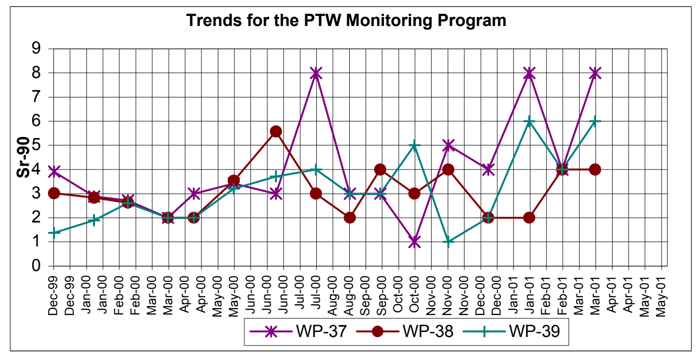
Local Background Wells and PTW Riser



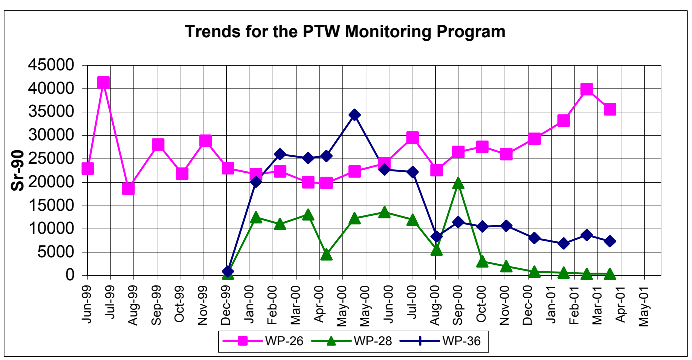
Wells Inside PTW



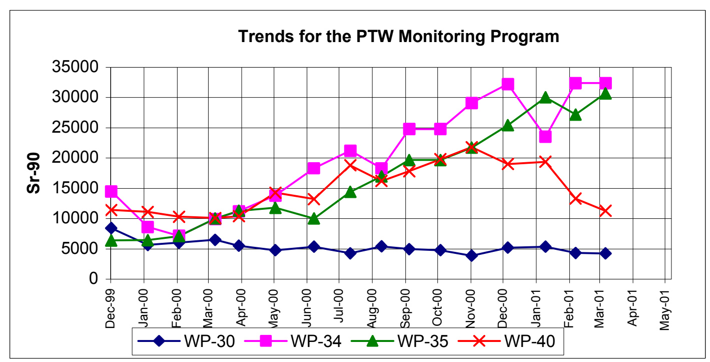
Pre-Installation Wells



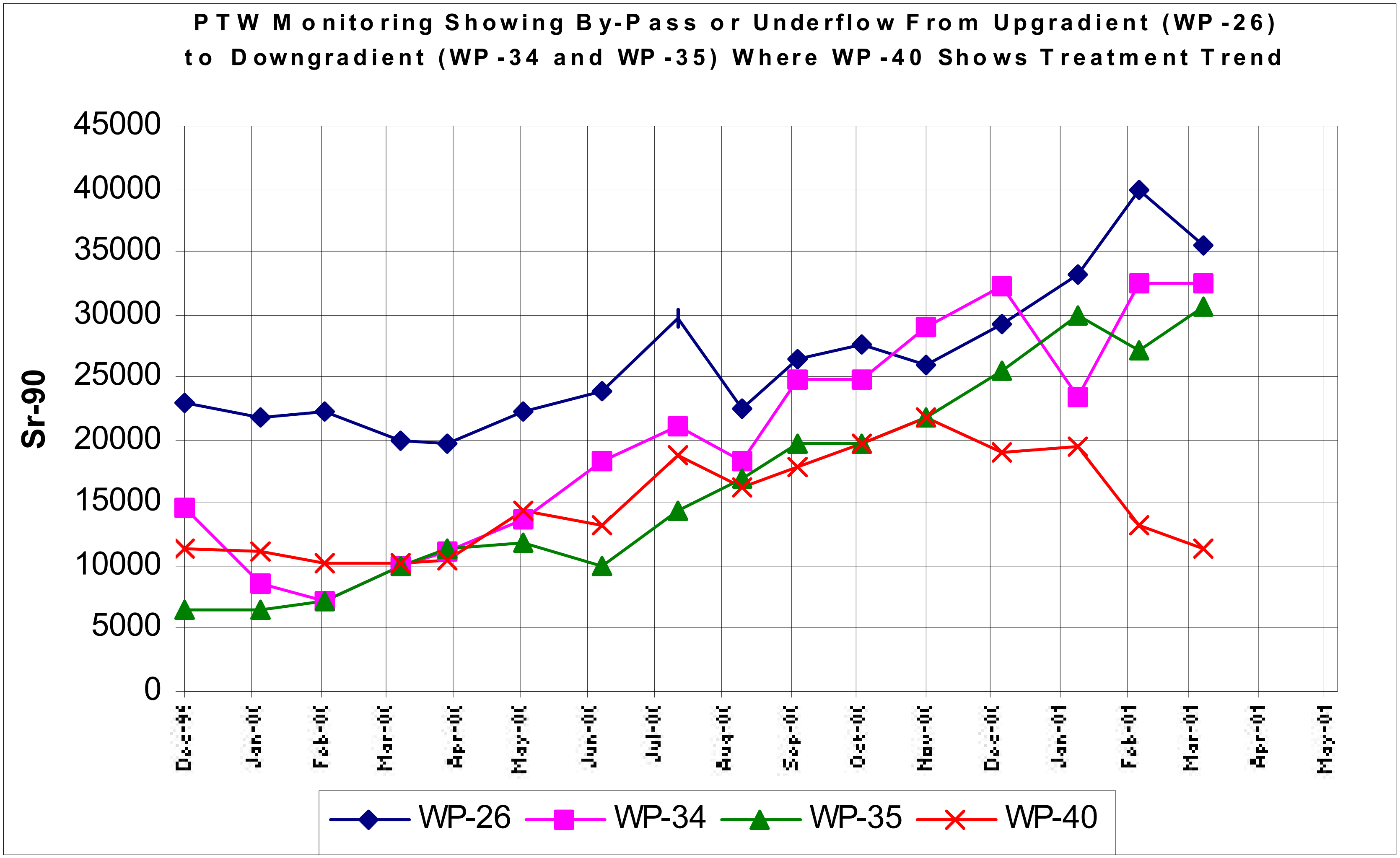
Wells Inside PTW



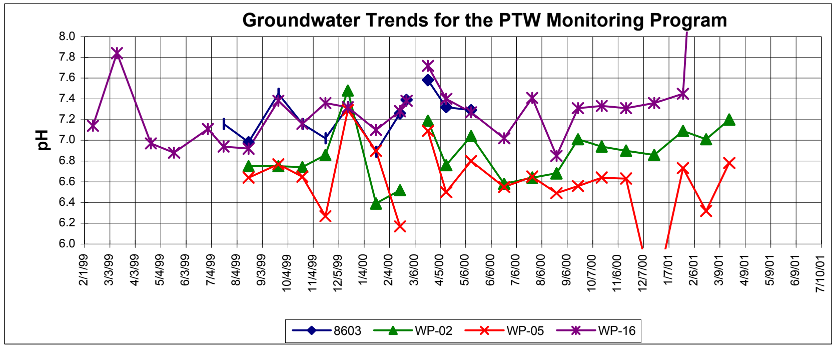
Upgradient Wells



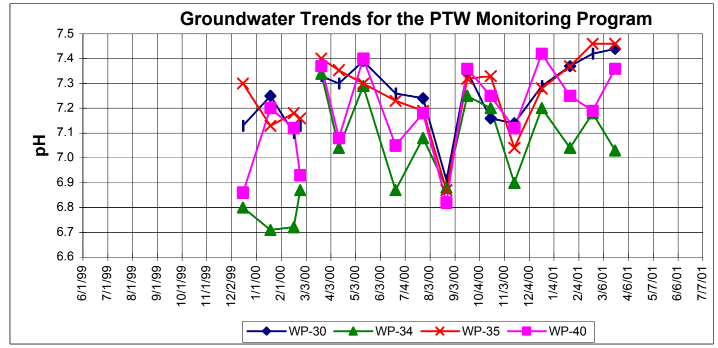
Downgradient Wells



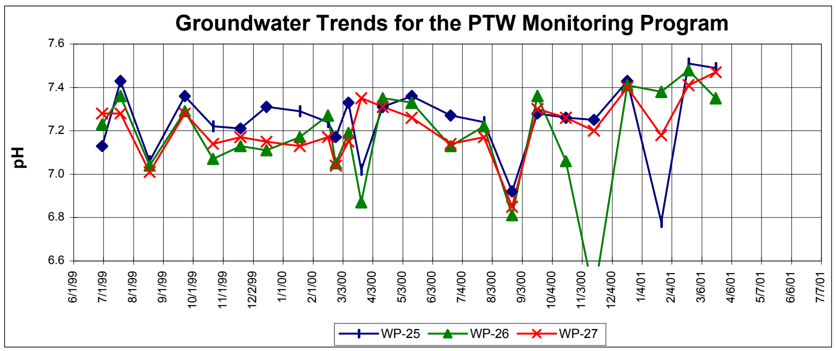
PTW Monitoring Showing By-Pass or Underflow From Upgradient (WP-26) to Downgradient (WP-34 and WP-35) Where WP-40 Shows Treatment Trend
Sr-90
45000
40000
35000
30000
25000
20000
15000
10000
5000
0
WP-26
WP-34
WP-35
WP-40



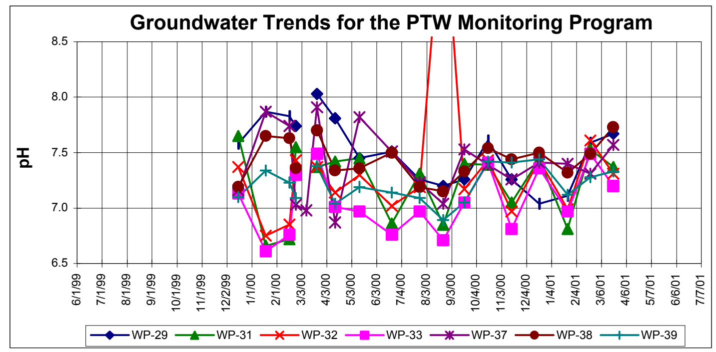
Background Wells



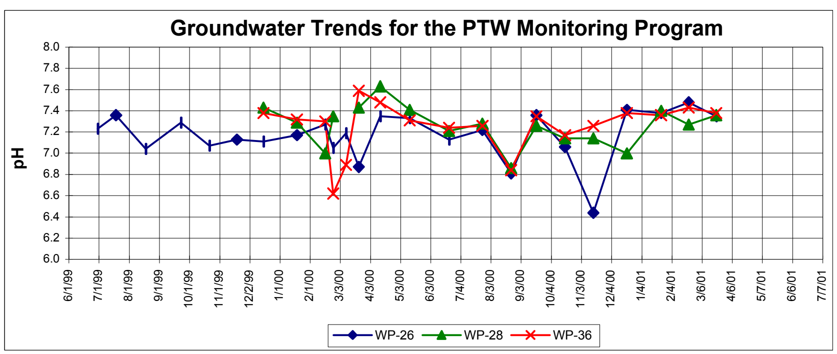
Downgradient Wells



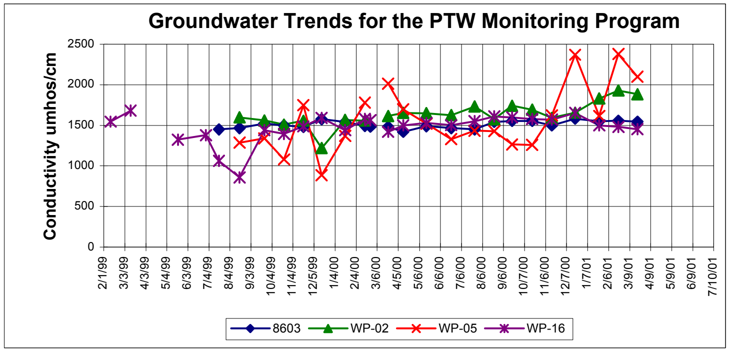
Pre-Installation Wells



Wells Inside PTW



Upgradient Wells



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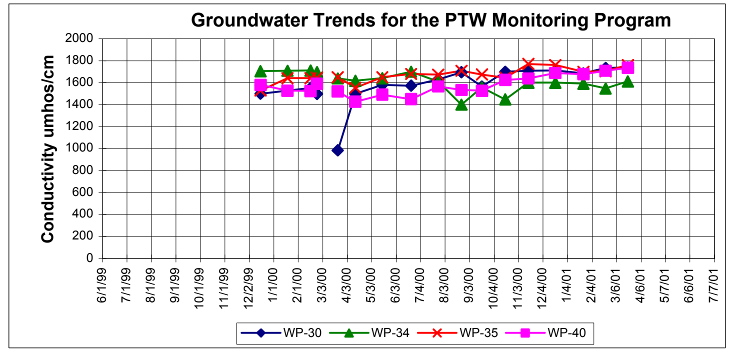
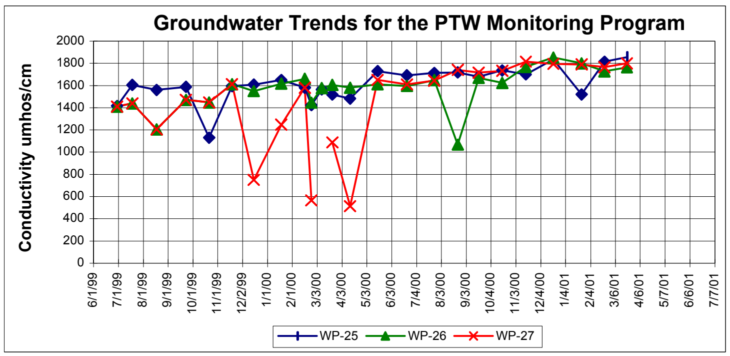


Downgradient Wells



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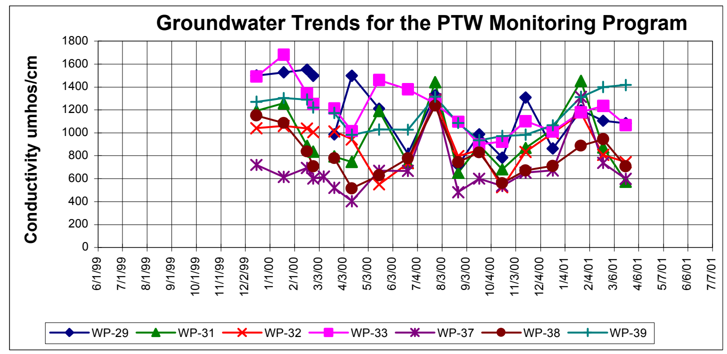

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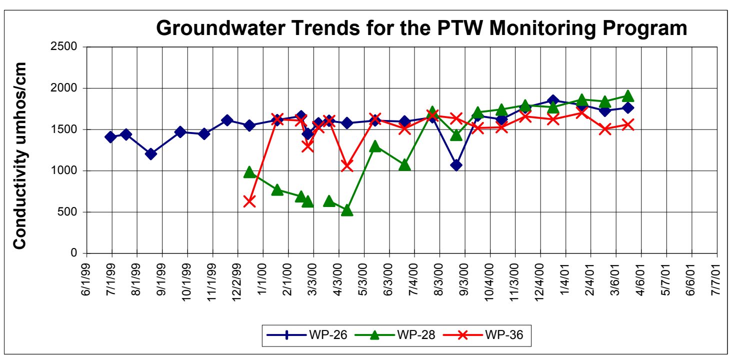

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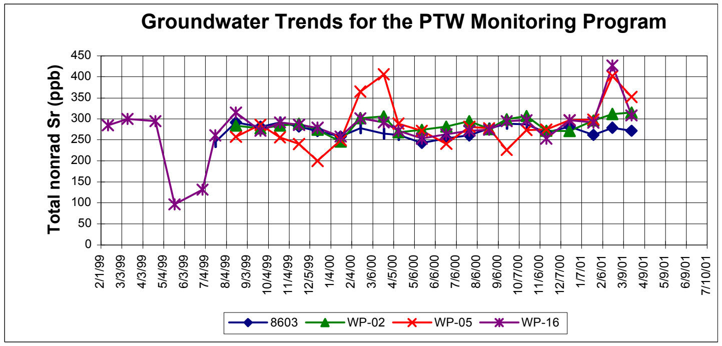

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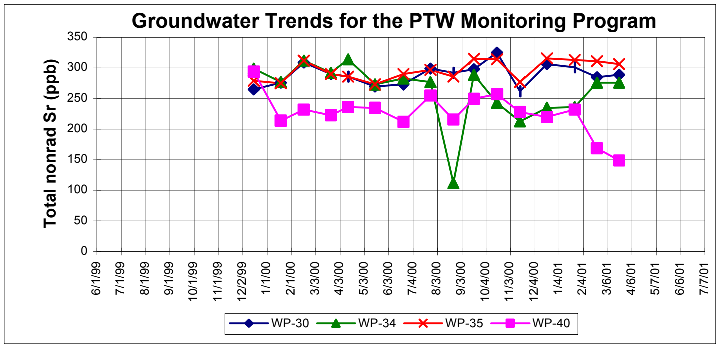
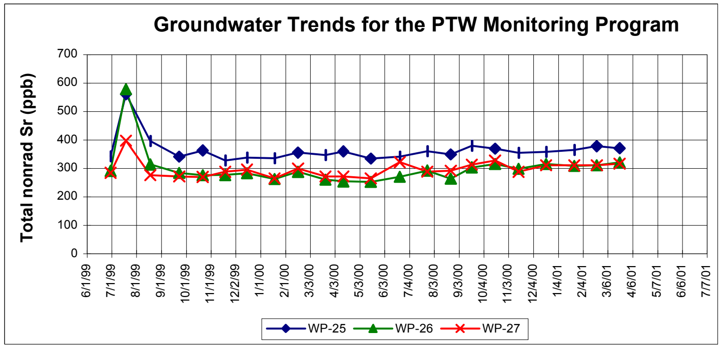


Downgradient Wells



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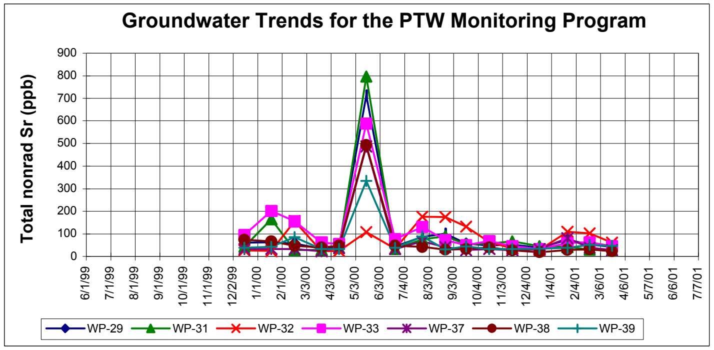
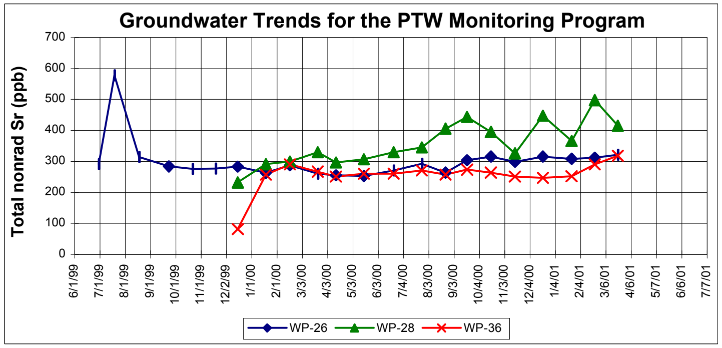


Wells Inside PTW



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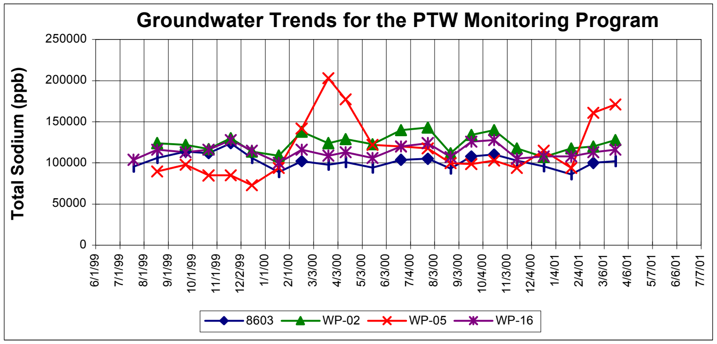

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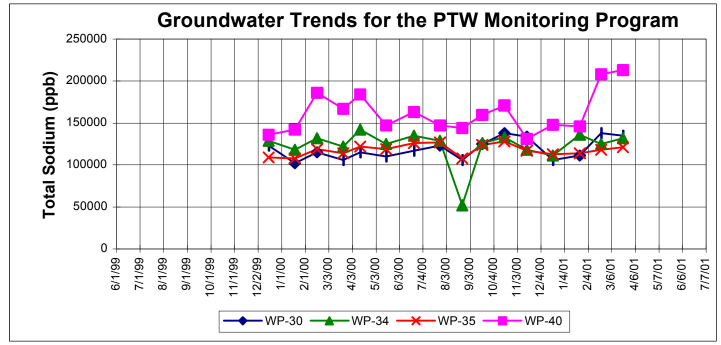
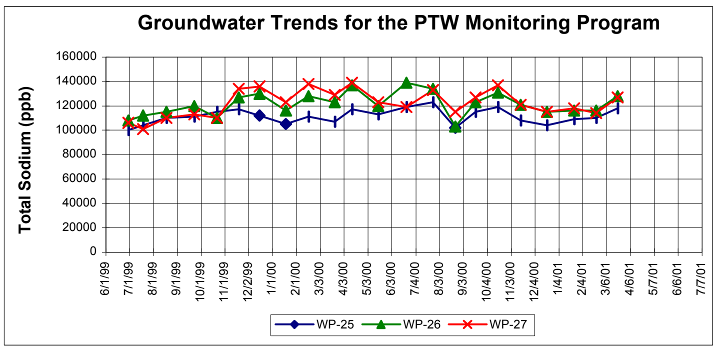


Downgradient Wells



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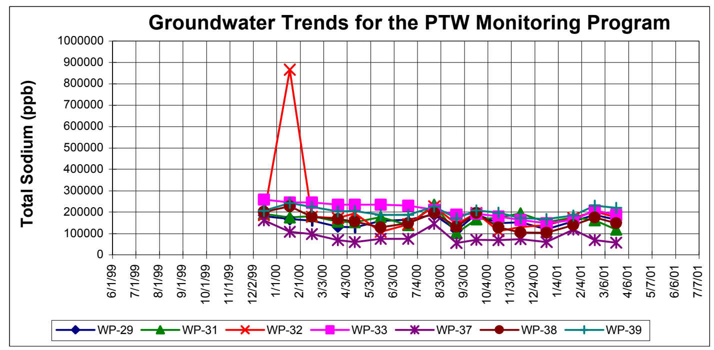
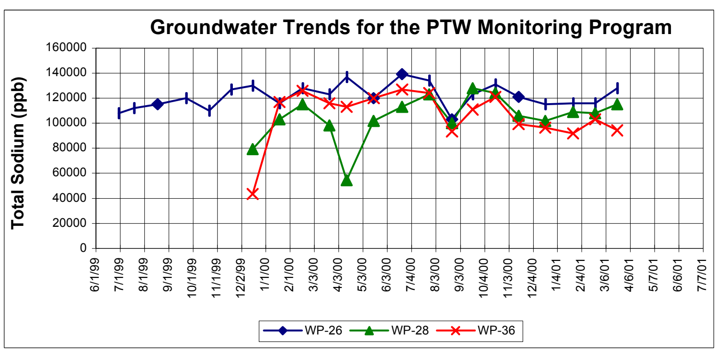
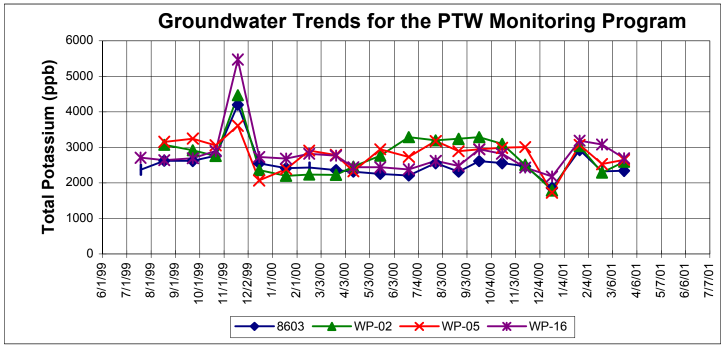


Wells Inside PTW



Upgradient Wells



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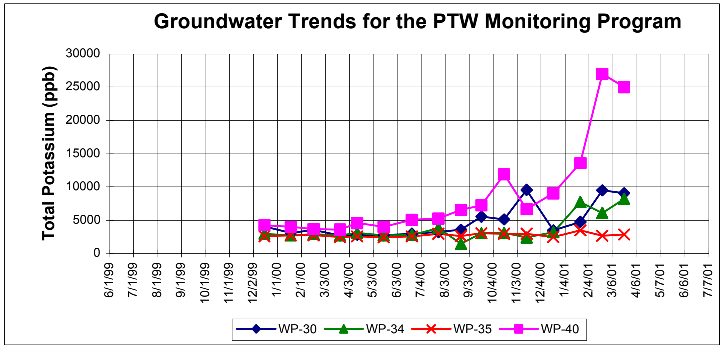
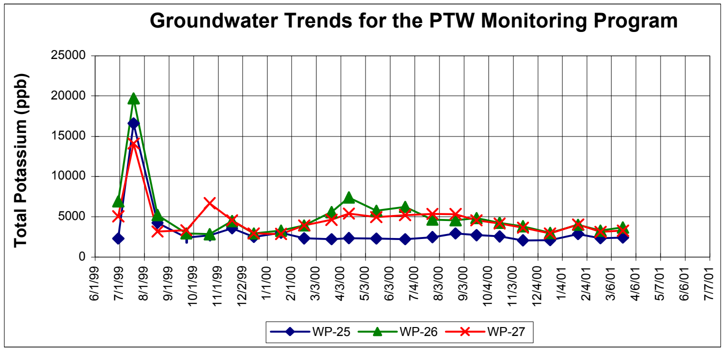
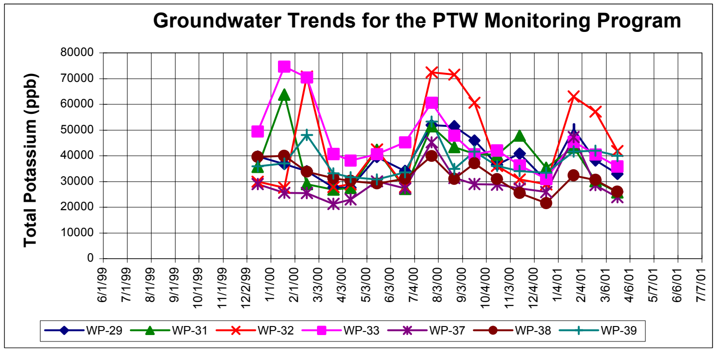


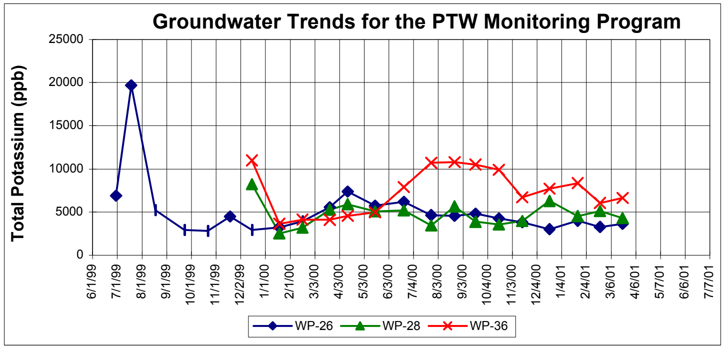
Downgradient Wells



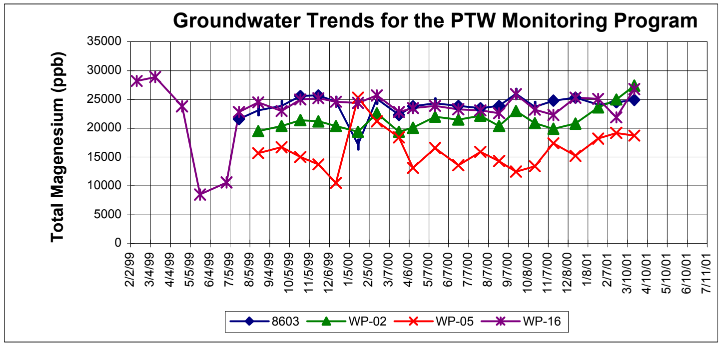
Pre-Installation Wells



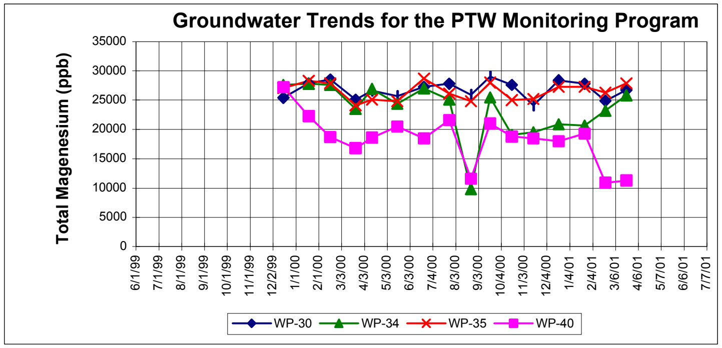
Wells Inside PTW



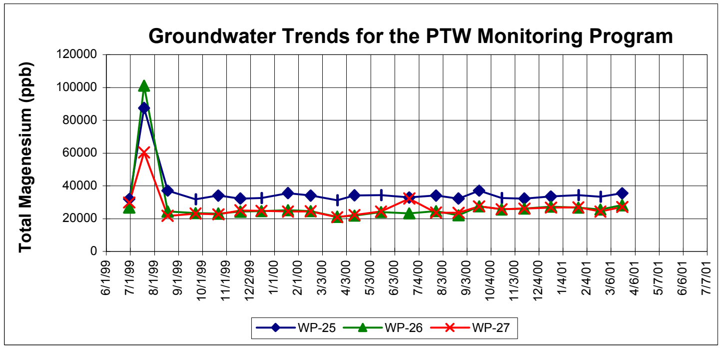
Upgradient Wells



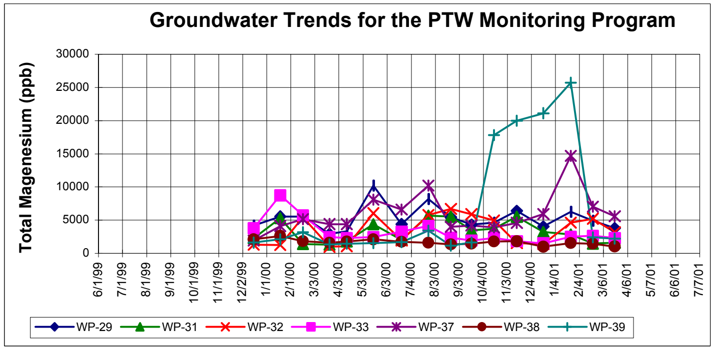
Background Wells



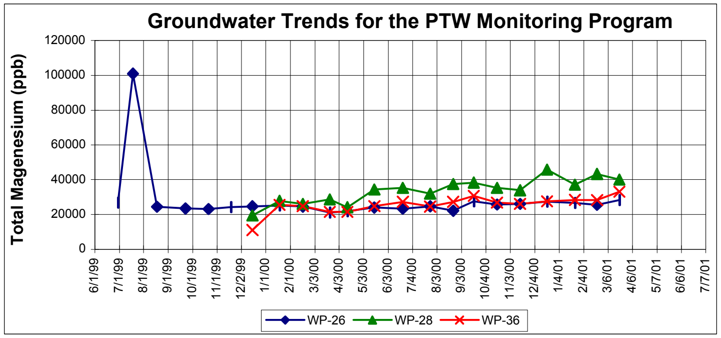
Downgradient Wells



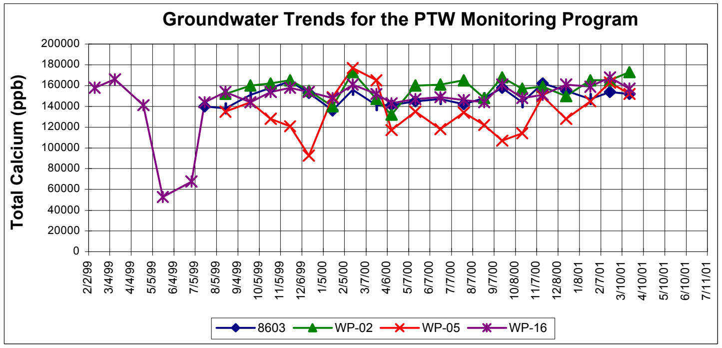
Pre-Installation Wells



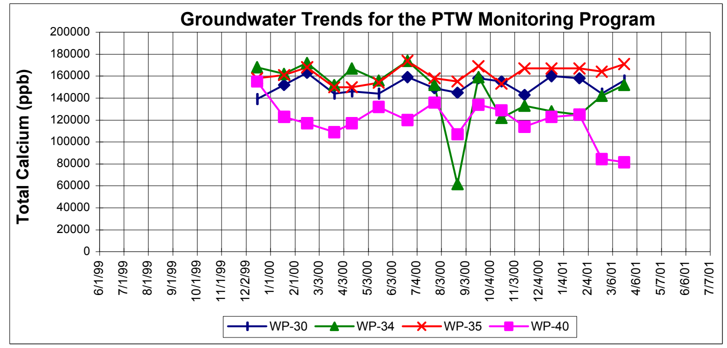
Wells Inside PTW



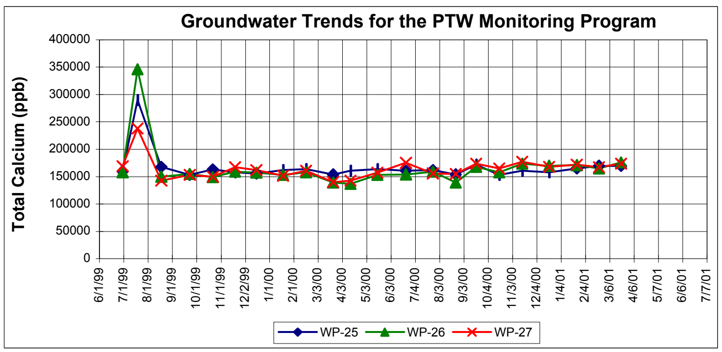
Upgradient Wells



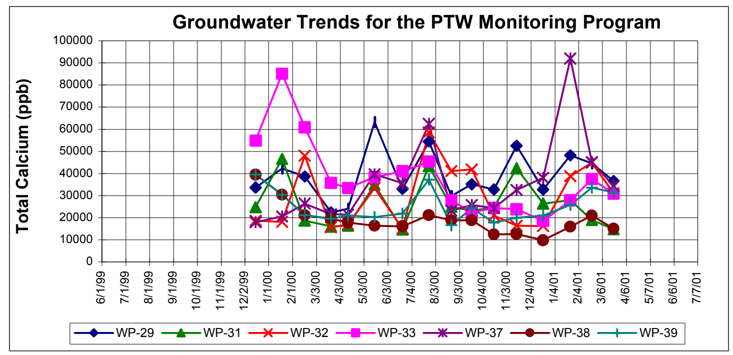
Background Wells



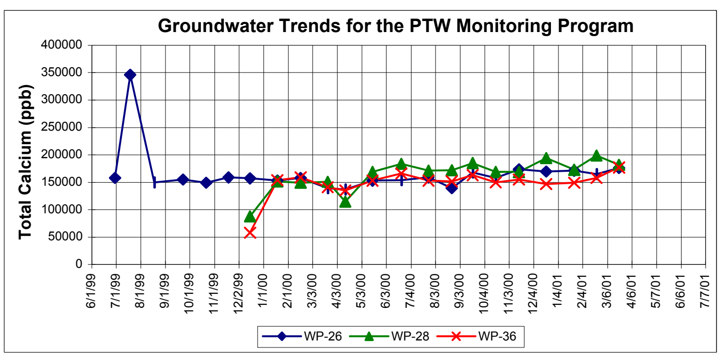
Downgradient Wells



Pre-Installation Wells

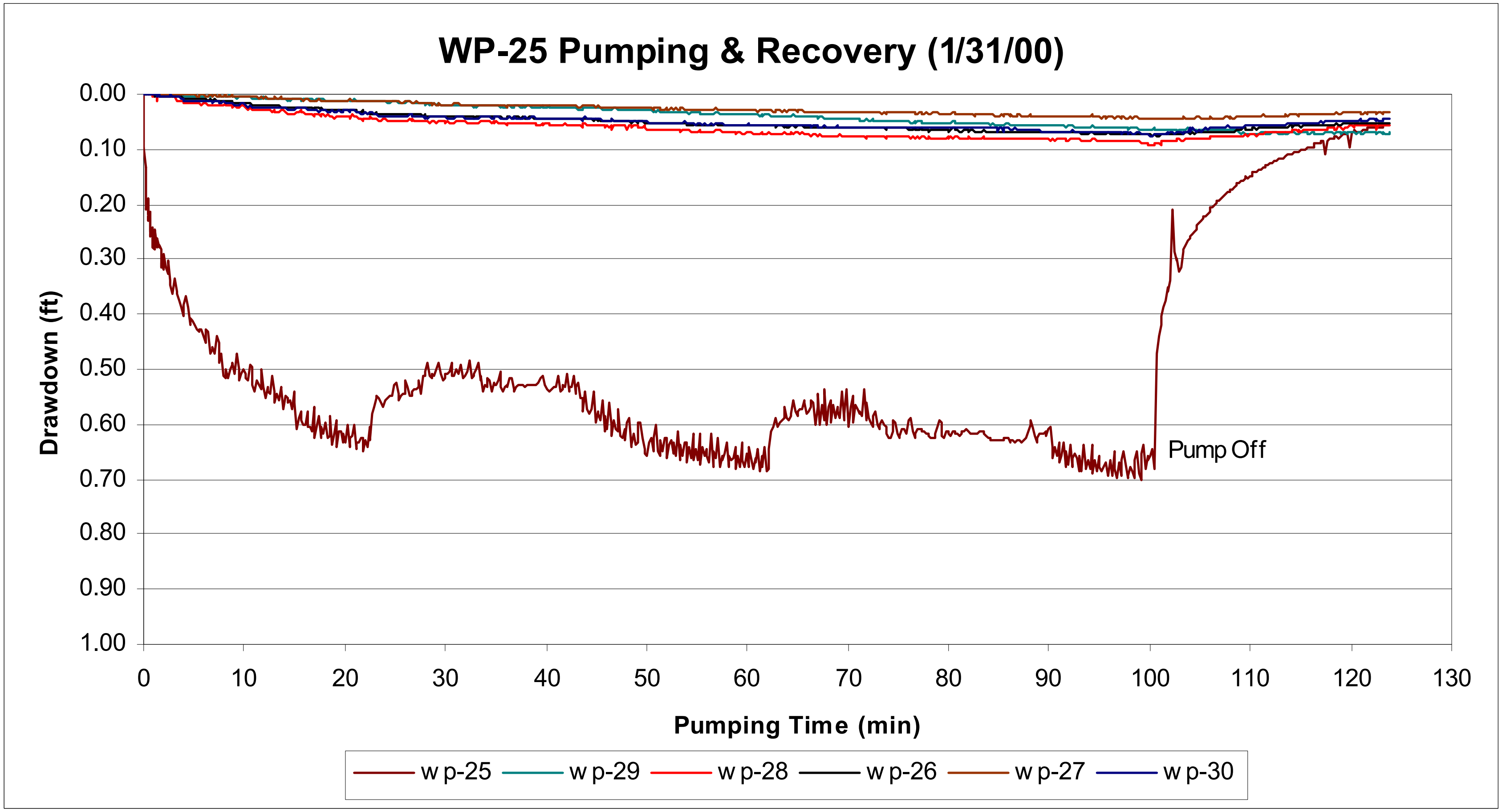


Wells Inside PTW

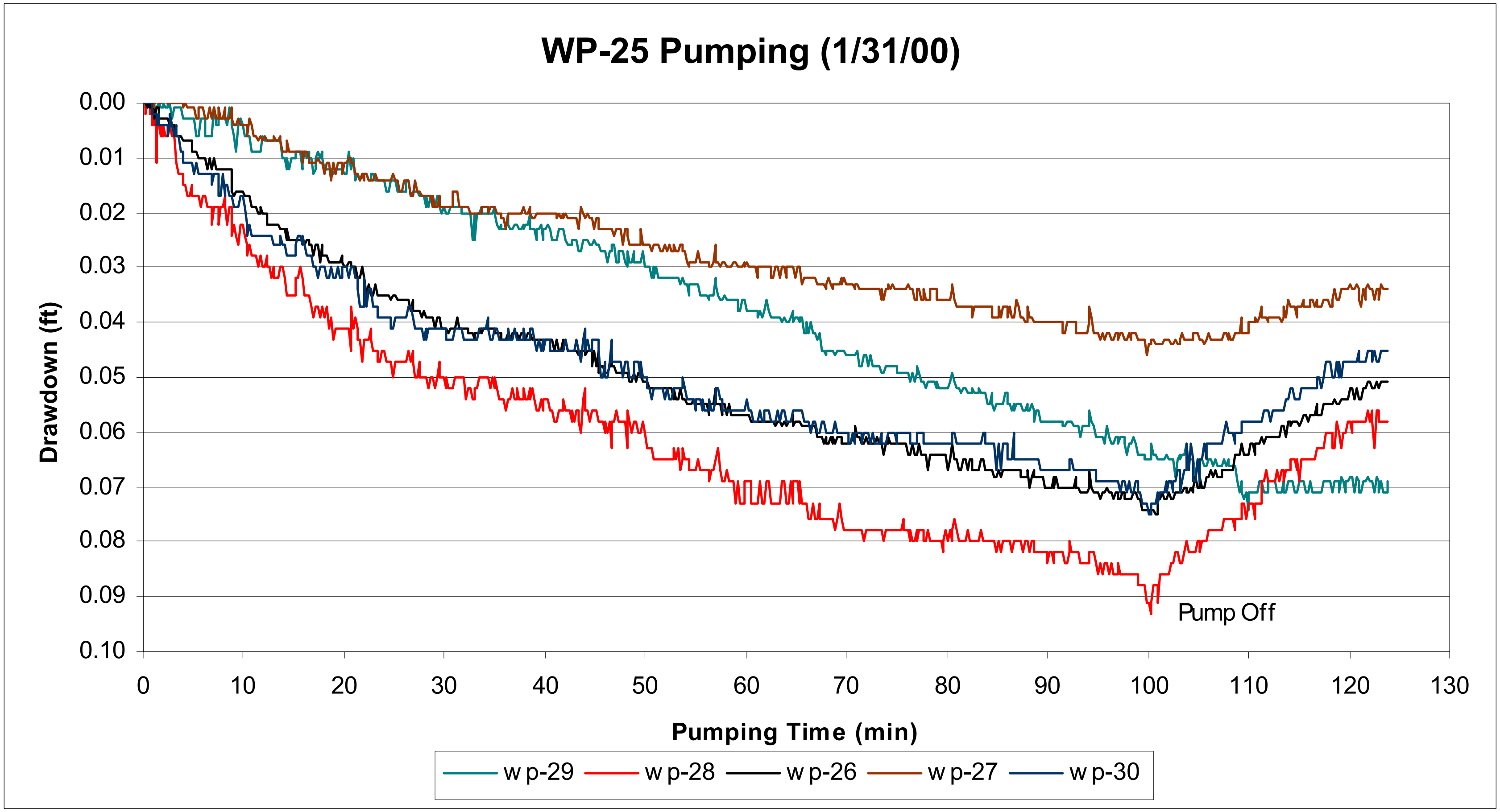


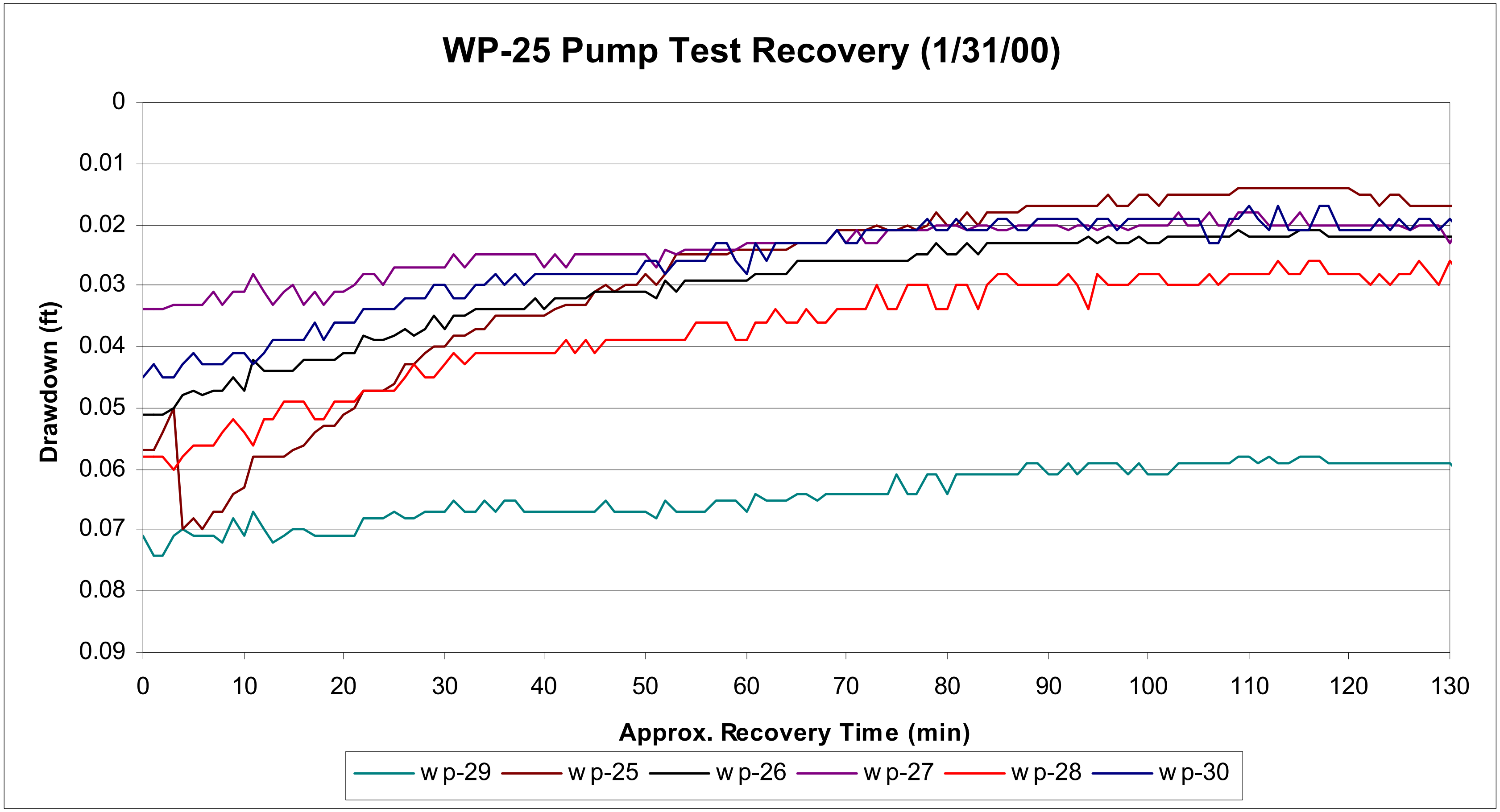
Upgradient Wells

APPENDIX 3
WELL WP-25 PUMPING TEST DATA



WP-25 Pumping & Recovery (1/31/00)
Drawdown (ft)
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.00
0
10
20
30
40
50
60
70
80
90
100
110
120
130
Pumping Time (min)
Pump Off
w p-25
w p-29
w p-28
w p-26
w p-27
w p-30


WP-25 Pumping (1/31/00)
Drawdown (ft)
0.00
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08
0.09
0.10
0
10
20
30
40
50
60
70
80
90
100
110
120
130
Pumping Time (min)
Pump Off
w p-29
w p-28
w p-26
w p-27
w p-30



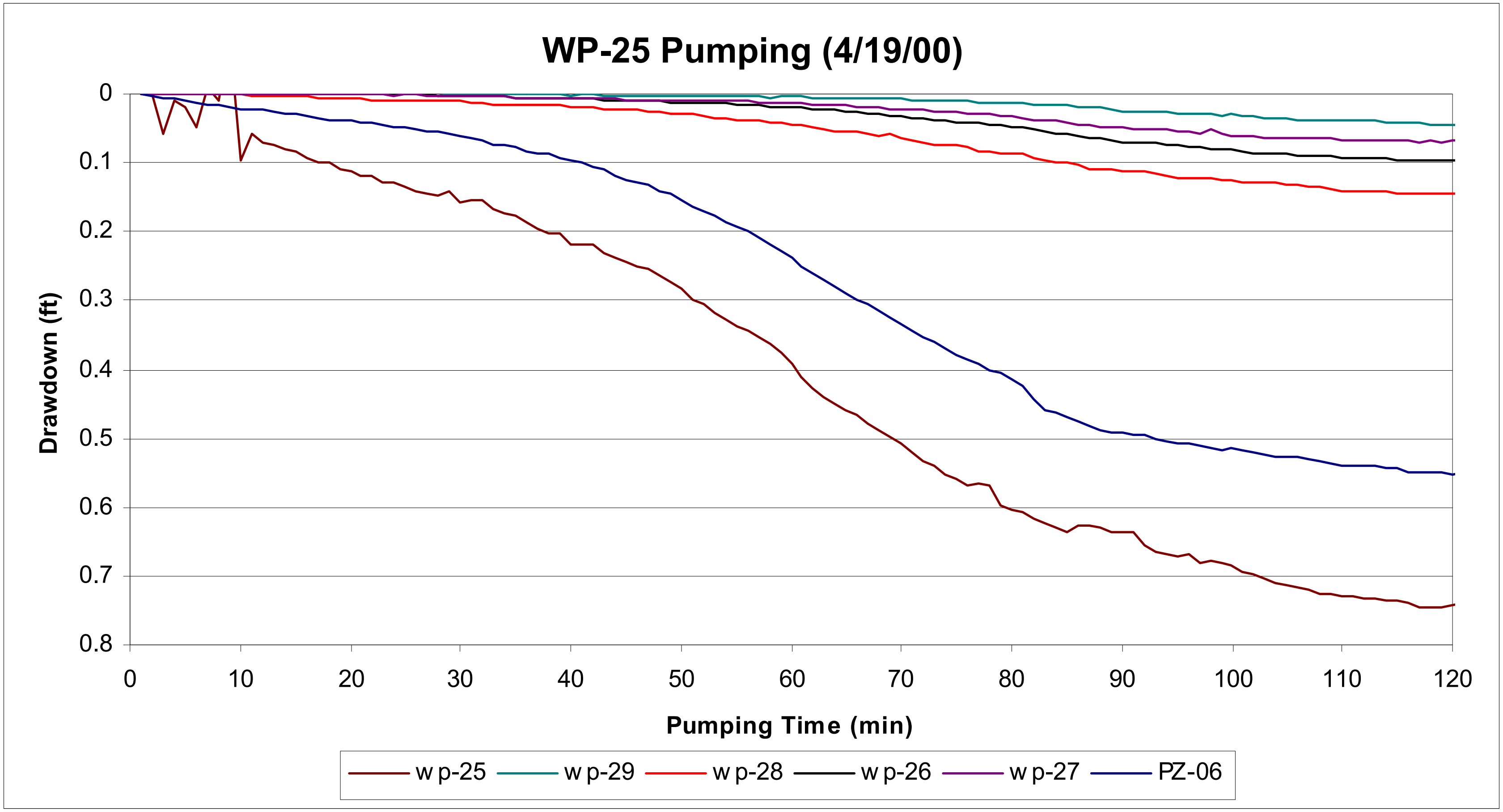
WP-25 Pump Test Recovery (1/31/00)
Drawdown (ft)
0
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08
0.09
0
10
20
30
40
50
60
70
80
90
100
110
120
130
Approx. Recovery Time (min)
w p-29
w p-25
w p-26
w p-27
w p-28
w p-30

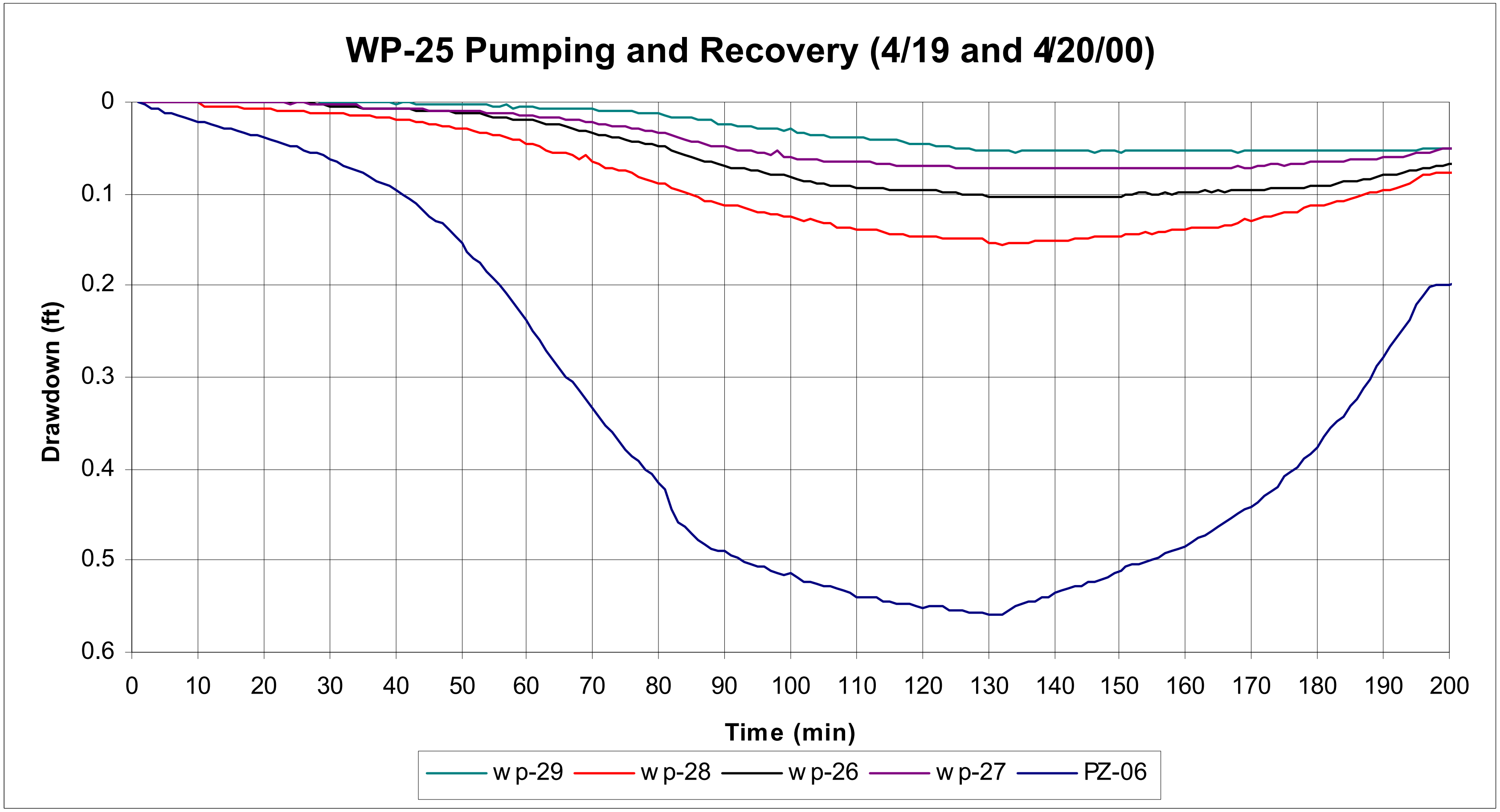
Hand Measured Water Level Data from WP-25 Mini-Pump Test 1/31/2000

Time Since Start of Pumping (min)	WP-31 Water Level (ft)	WP-31 Drawdown (ft)	WP-32 Water Level (ft)	WP-32 Drawdown (ft)	WP-33 Water Level (ft)	WP-33 Drawdown (ft)	WP-34 Water Level (ft)	WP-34 Drawdown (ft)	PZ-03 Water Level (ft)	PZ-03 Drawdown (ft)	PZ-06 Water Level (ft)	PZ-06 Drawdown (ft)
0	9.87	0.00	9.98	0.00	9.95	0.00	11.28	0.00	9.78	0.00	9.39	0.00
5									9.79	0.01	9.47	0.08
35					9.96	0.01			9.83	0.05	9.57	0.18
60	9.92	0.05	9.99	0.01	9.97	0.02	11.32	0.04	9.85	0.07	9.60	0.21
120	9.94	0.07	10.03	0.05	10.01	0.06	11.32	0.04	9.84	0.06	9.53	0.14
Distance from wp-25 (ft)	22.20		22.20		22.00		22.30		16.10		18.40	

	WP-25	WP-26	WP-27	WP-28	WP-29	WP-30
Distance from wp-25 (ft)	0.00	23.80	43.00	16.70	14.50	15.20

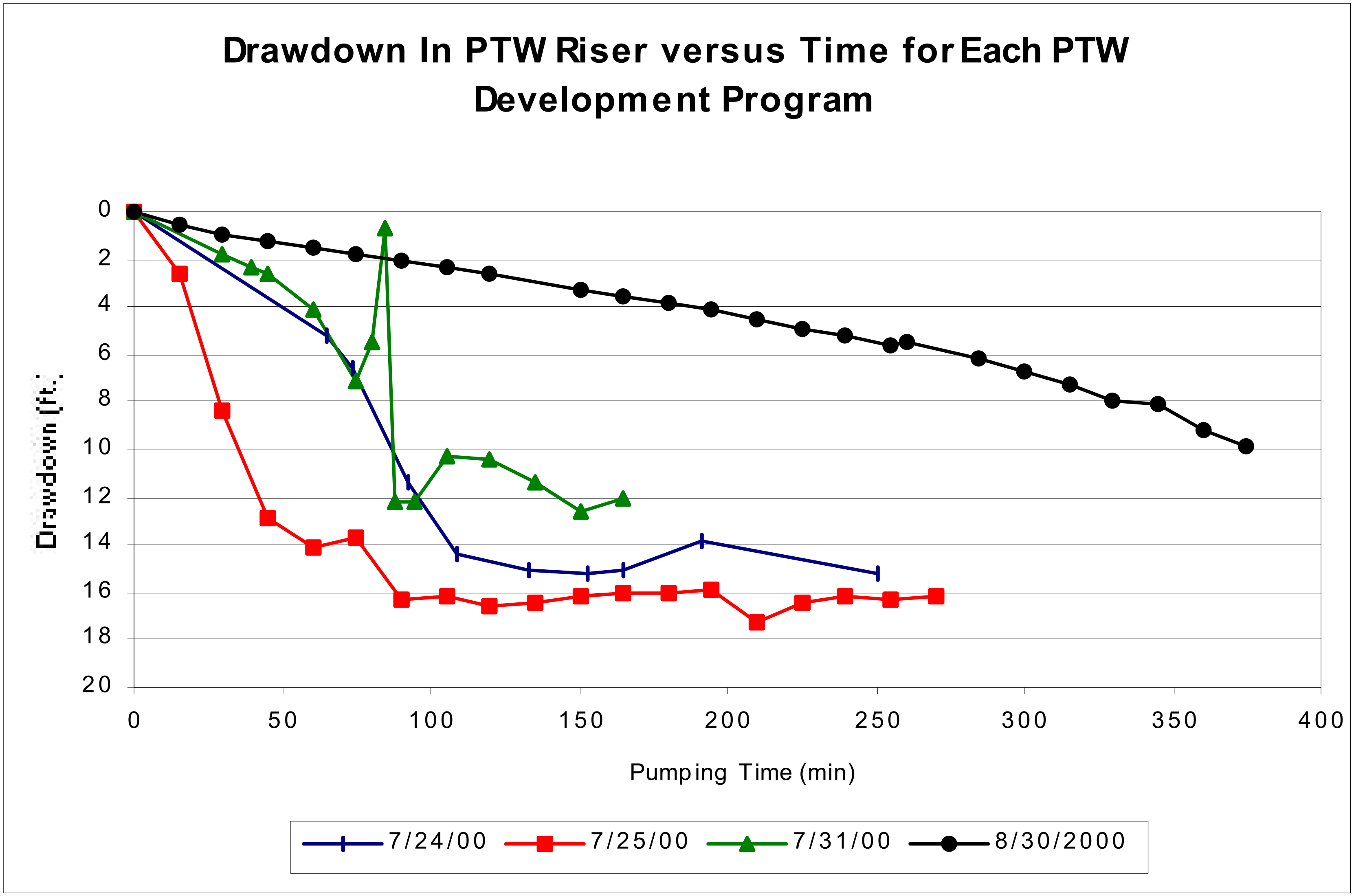
Shut off pump at 100 mins


WP-25 Pumping (4/19/00)
Drawdown (ft)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0
10
20
30
40
50
60
70
80
90
100
110
120
Pumping Time (min)
w p-25
w p-29
w p-28
w p-26
w p-27
PZ-06

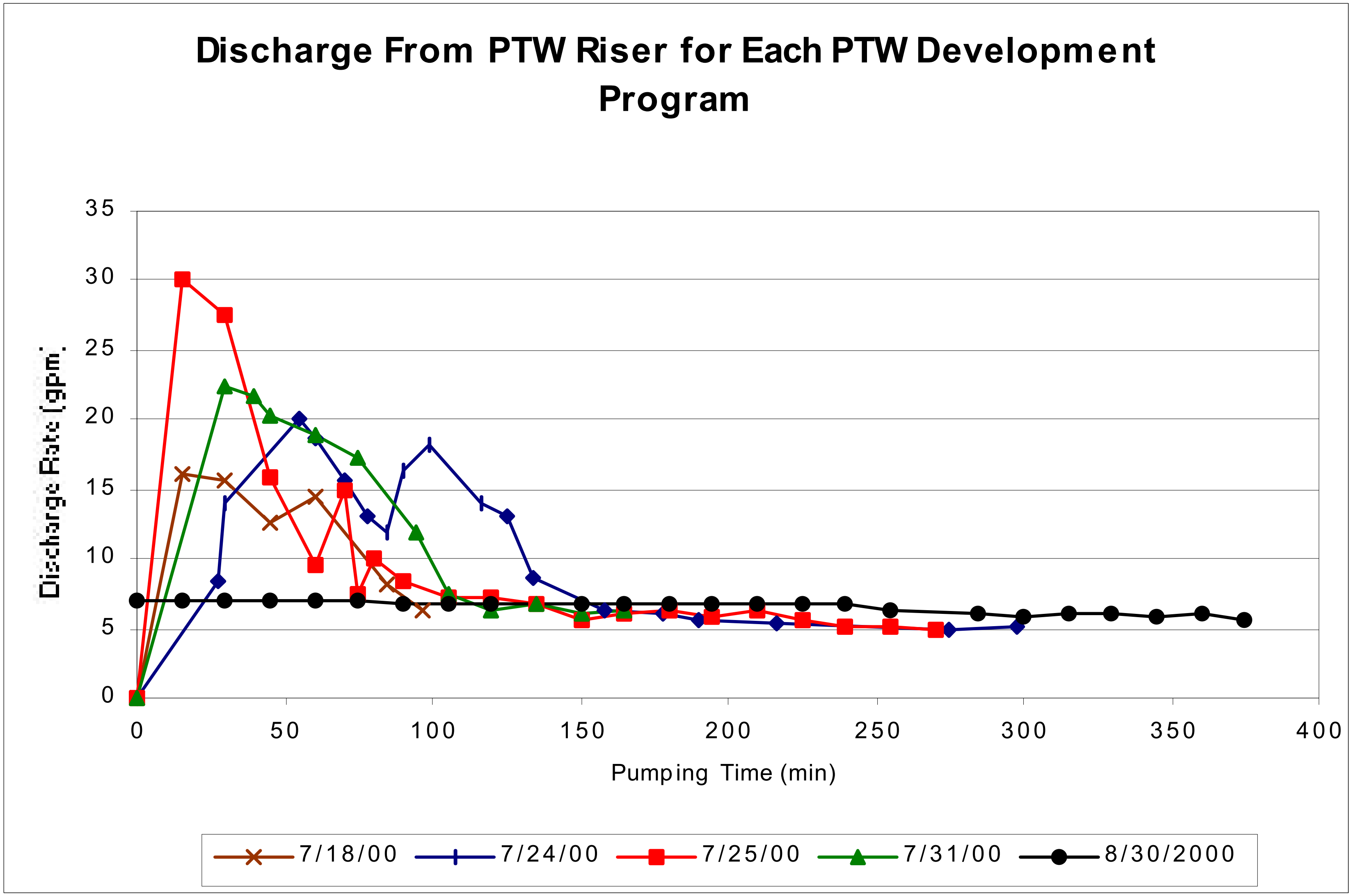


WP-25 Pumping and Recovery (4/19 and 4/20/00)
Drawdown (ft)
0
0.1
0.2
0.3
0.4
0.5
0.6
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
Time (min)
w p-29
w p-28
w p-26
w p-27
PZ-06

APPENDIX 4
PTW DEVELOPMENT DATA,
DATA & GRAPHS FROM PTW DEVELOPMENT ACTIVITIES

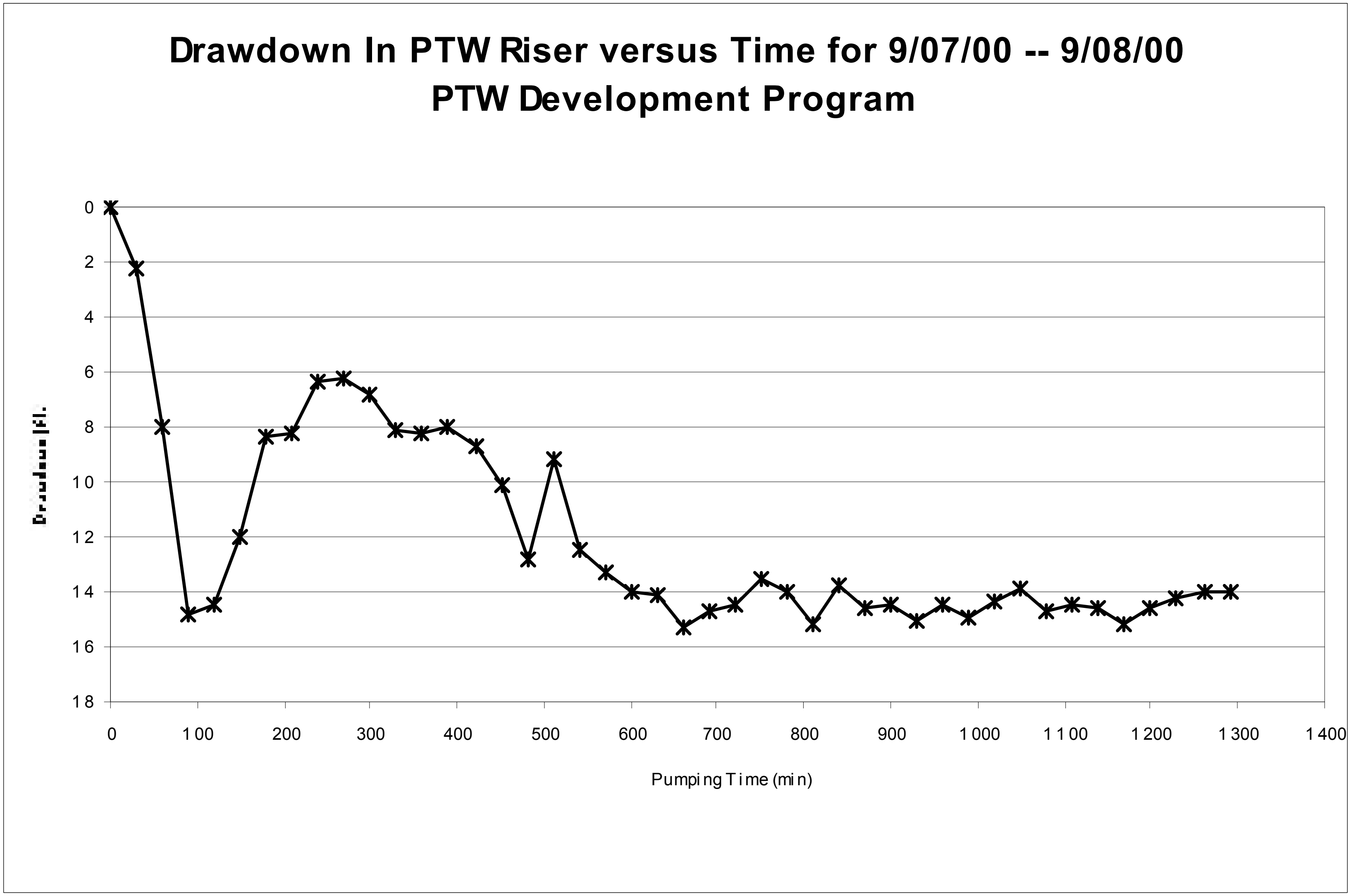


Drawdown In PTW Riser versus Time for Each PTW Development Program
Drawdown (ft.)
0
2
4
6
8
10
12
14
16
18
20
0
50
100
150
200
250
300
350
400
Pumping Time (min)
7/24/00
7/25/00
7/31/00
8/30/2000

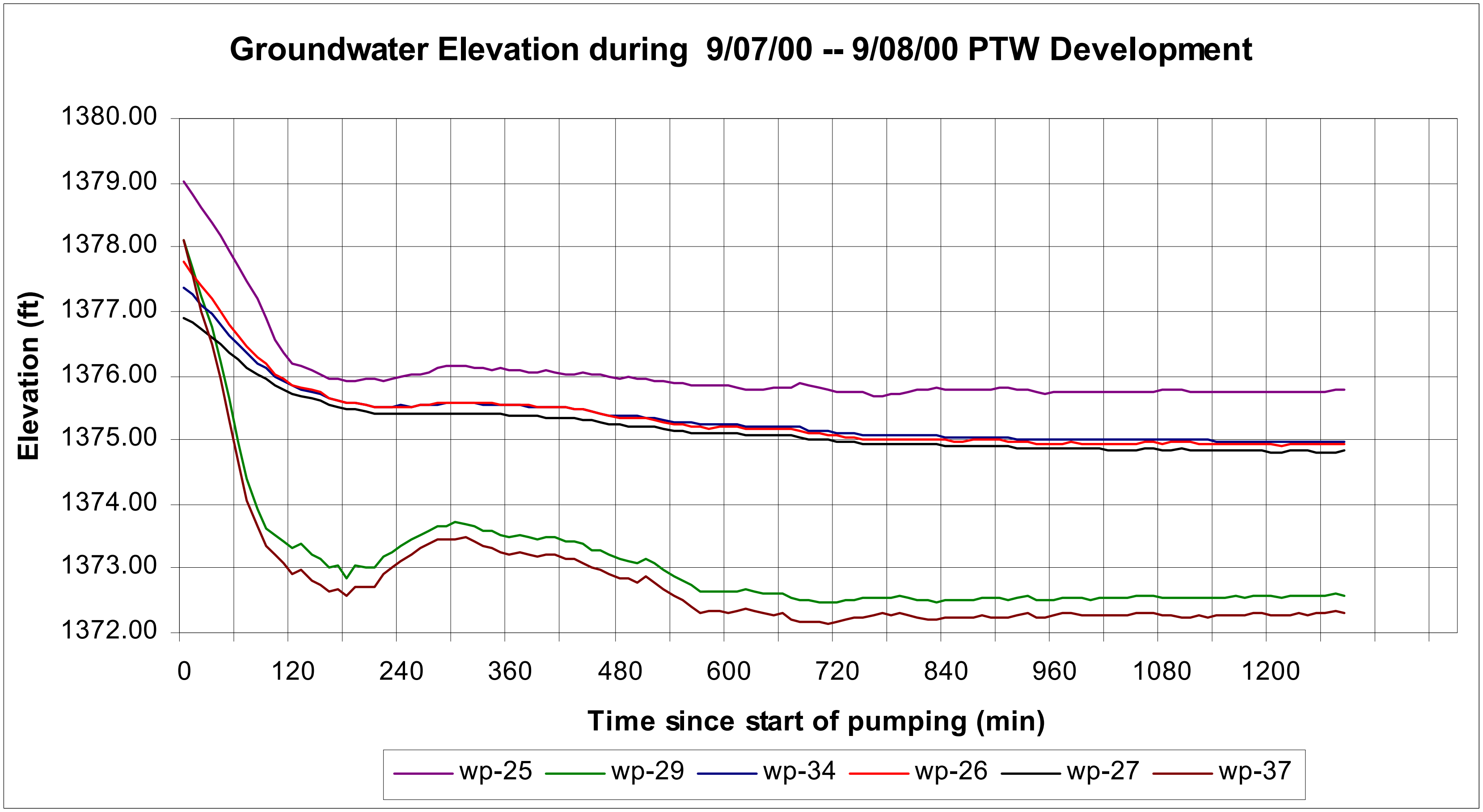


Discharge From PTW Riser for Each PTW Development Program
Discharge Rate (gpm)
Pumping Time (min)
0
5
10
15
20
25
30
35
0
50
100
150
200
250
300
350
400
7/18/00
7/24/00
7/25/00
7/31/00
8/30/2000

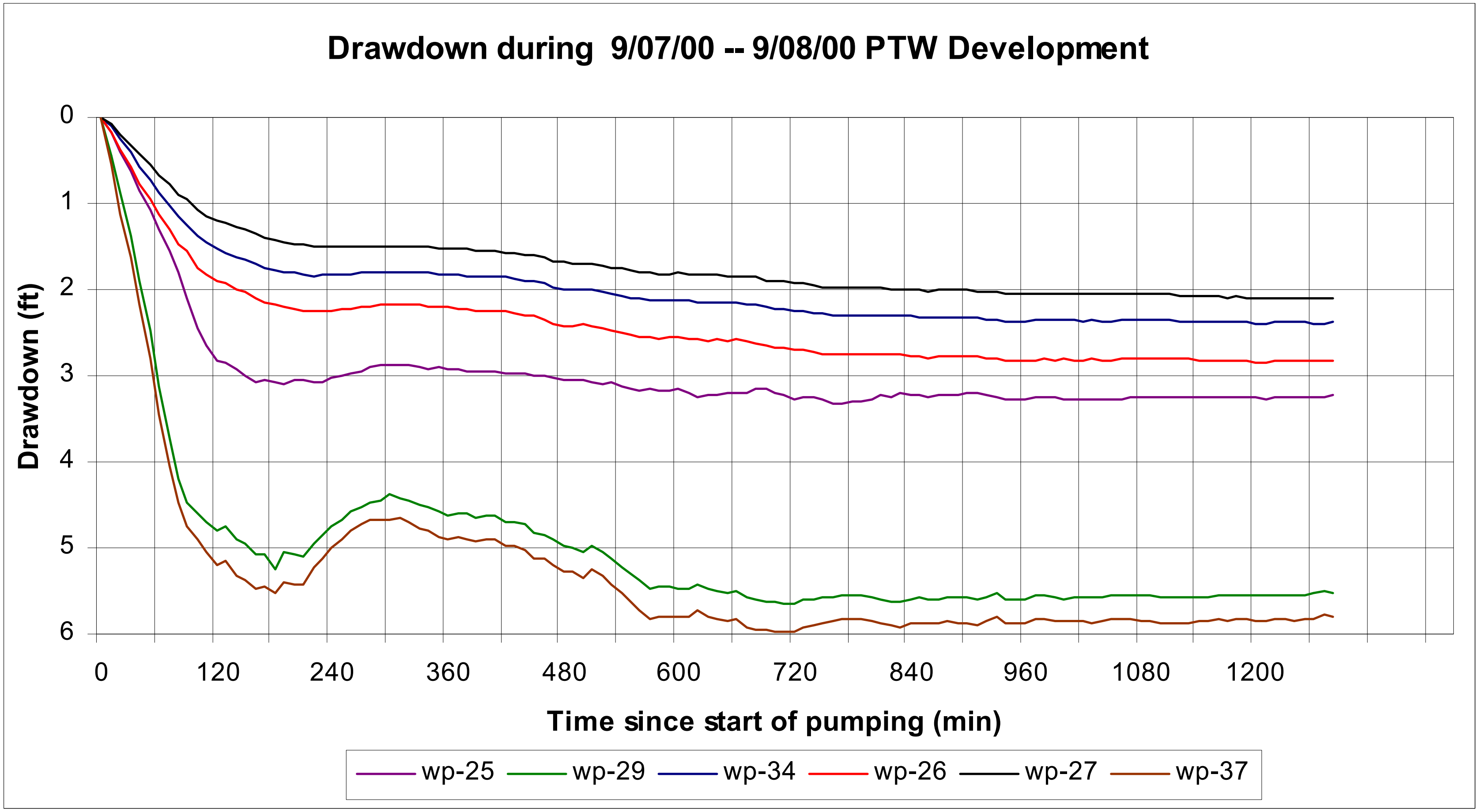
APPENDIX 5
22-HOUR PTW PUMPING TEST DATA



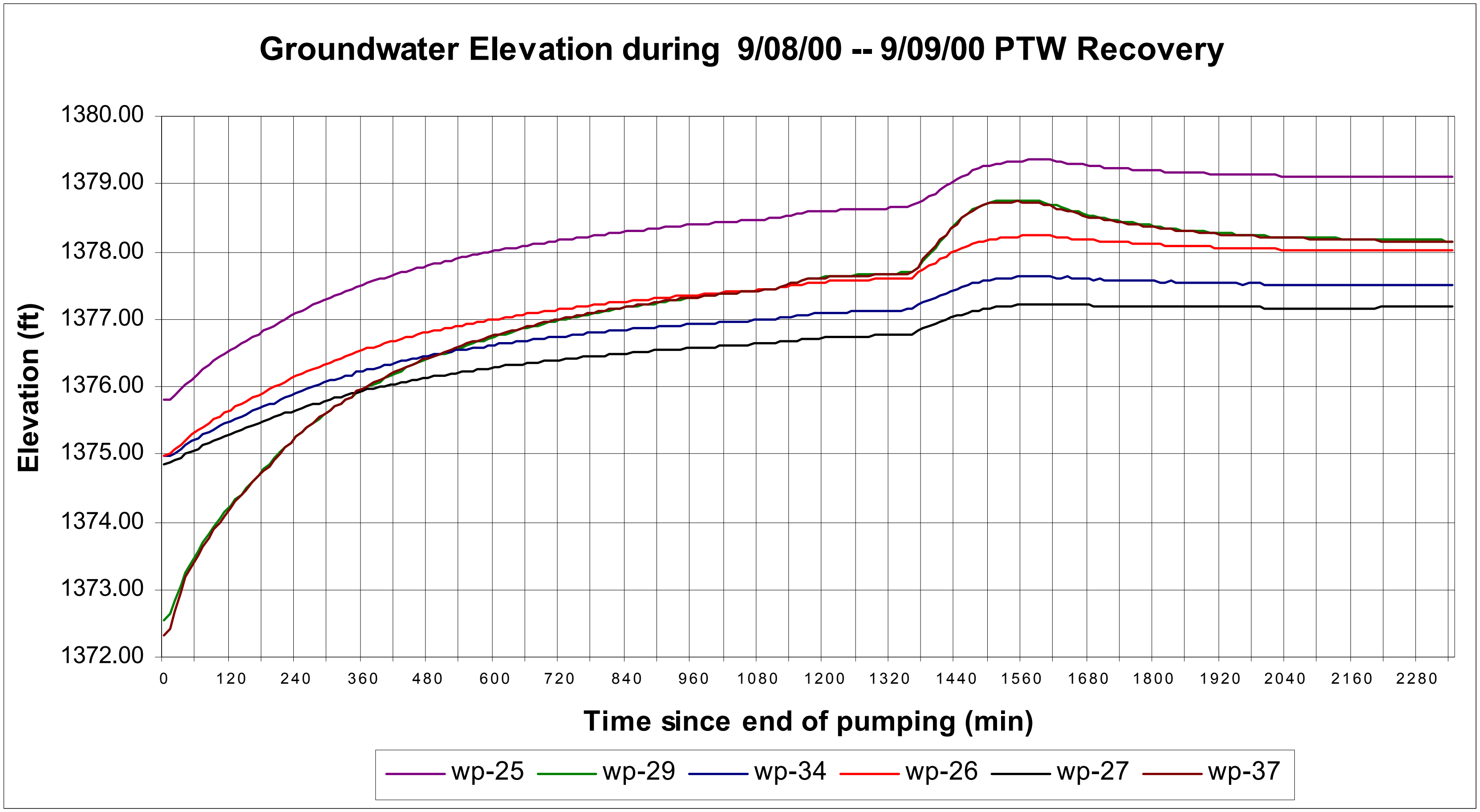
Drawdown In PTW Riser versus Time for 9/07/00 -- 9/08/00
PTW Development Program
0
2
4
6
8
10
12
14
16
18
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
Pumping Time (min)



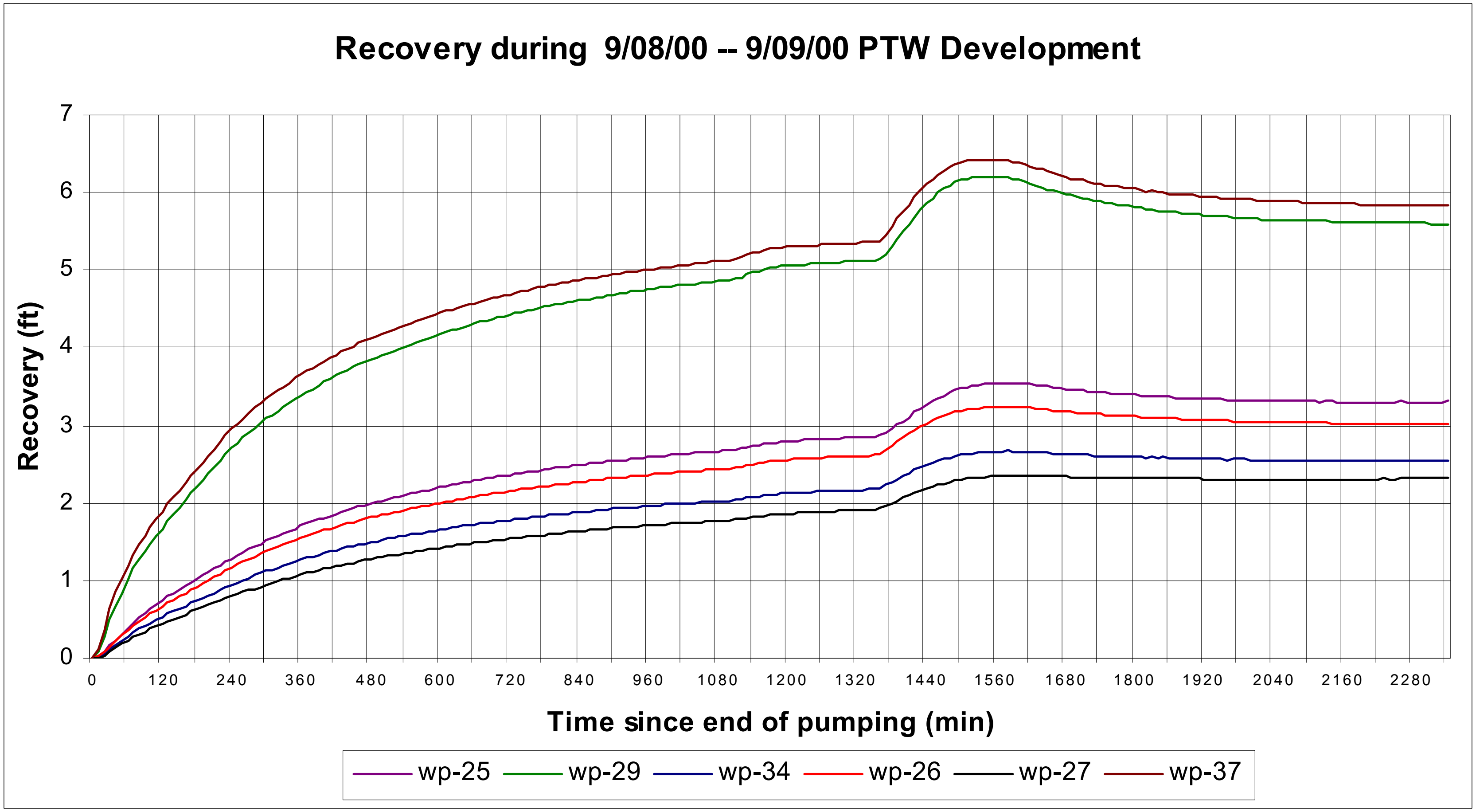
Groundwater Elevation during 9/07/00 -- 9/08/00 PTW Development
1380.00
1379.00
1378.00
1377.00
1376.00
1375.00
1374.00
1373.00
1372.00
Elevation (ft)
0
120
240
360
480
600
720
840
960
1080
1200
Time since start of pumping (min)
wp-25
wp-29
wp-34
wp-26
wp-27
wp-37



Drawdown during 9/07/00 -- 9/08/00 PTW Development
Drawdown (ft)
0
1
2
3
4
5
6
0
120
240
360
480
600
720
840
960
1080
1200
Time since start of pumping (min)
wp-25
wp-29
wp-34
wp-26
wp-27
wp-37



Groundwater Elevation during 9/08/00 -- 9/09/00 PTW Recovery
Elevation (ft)
1380.00
1379.00
1378.00
1377.00
1376.00
1375.00
1374.00
1373.00
1372.00
0
120
240
360
480
600
720
840
960
1080
1200
1320
1440
1560
1680
1800
1920
2040
2160
2280
Time since end of pumping (min)
wp-25
wp-29
wp-34
wp-26
wp-27
wp-37

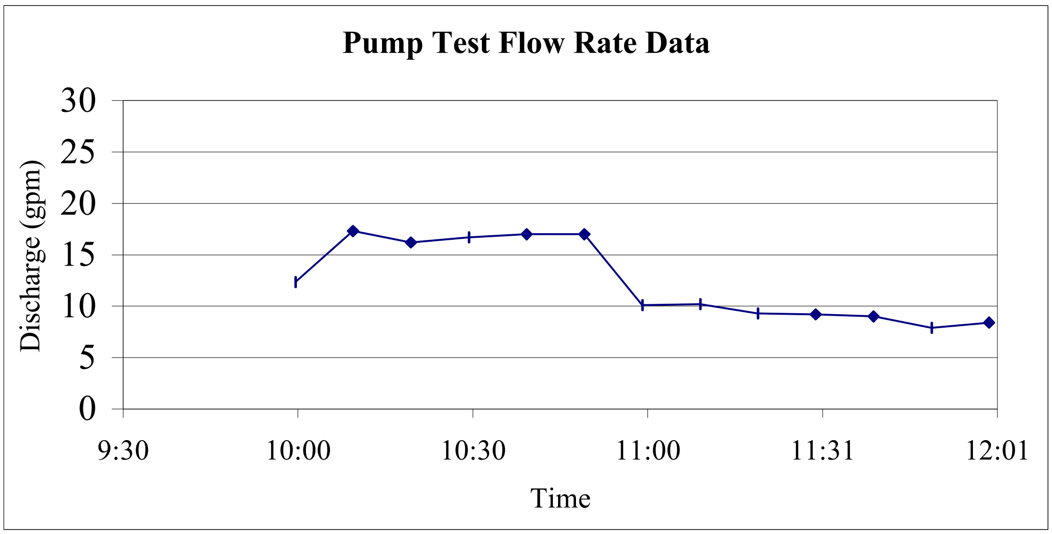


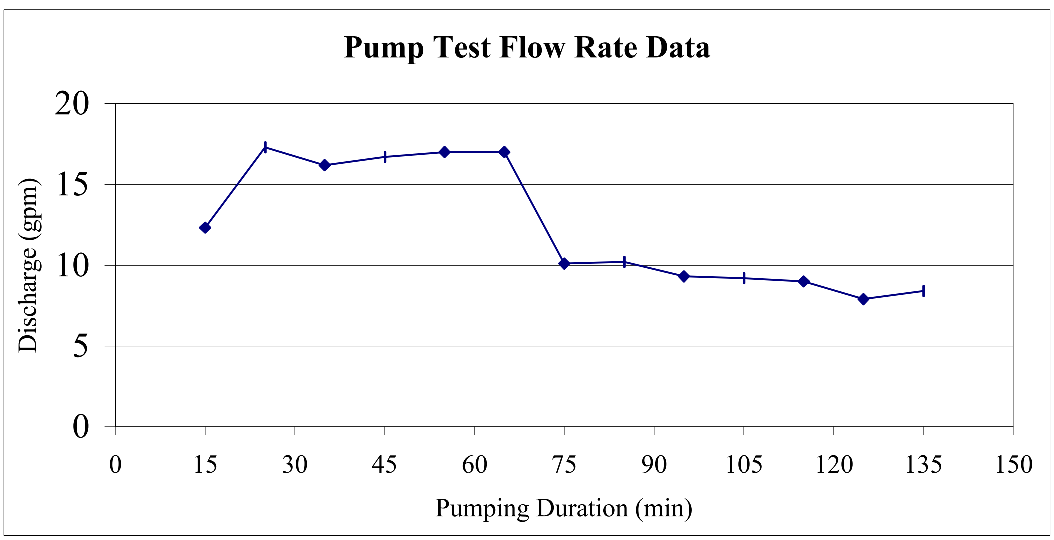
Recovery during 9/08/00 -- 9/09/00 PTW Development
Recovery (ft)
0
1
2
3
4
5
6
7
0
120
240
360
480
600
720
840
960
1080
1200
1320
1440
1560
1680
1800
1920
2040
2160
2280
Time since end of pumping (min)
wp-25
wp-29
wp-34
wp-26
wp-27
wp-37

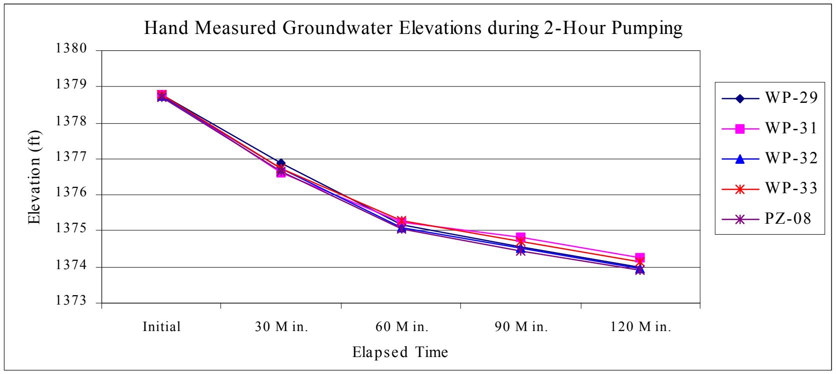
APPENDIX 6
2-HOUR PTW PUMPING TEST DATA

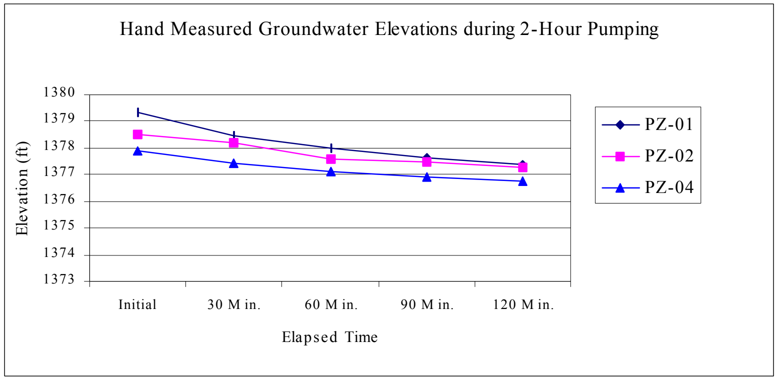
Pumping Rate During the 1/19/01 Two Hour Pump Test

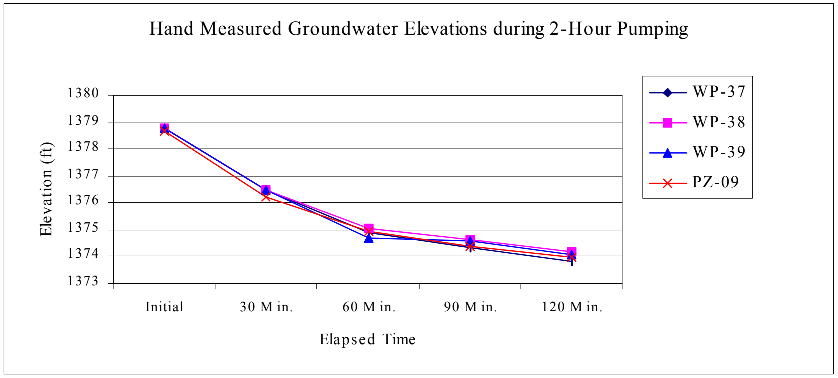
Clock Time	Pumping Duration (minutes)	Elapsed Time (minutes)	Pump Reading (gallons)	Gallons Pumped	Flow Rate (gal/min)
9:45 AM	0		870		
10:00 AM	15	15	1055	185	12.3
10:10 AM	25	10	1228	173	17.3
10:20 AM	35	10	1390	162	16.2
10:30 AM	45	10	1557	167	16.7
10:40 AM	55	10	1727	170	17.0
10:50 AM	65	10	1897	170	17.0
11:00 AM	75	10	1998	101	10.1
11:10 AM	85	10	2100	102	10.2
11:20 AM	95	10	2193	93	9.3
11:30 AM	105	10	2285	92	9.2
11:40 AM	115	10	2375	90	9.0
11:50 AM	125	10	2454	79	7.9
12:00 PM	135	10	2538	84	8.4

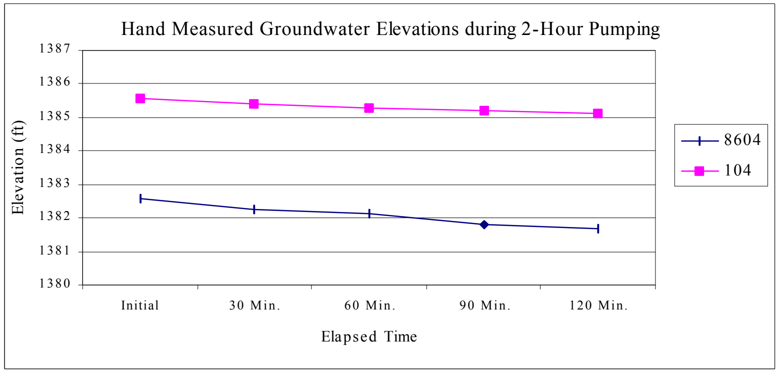


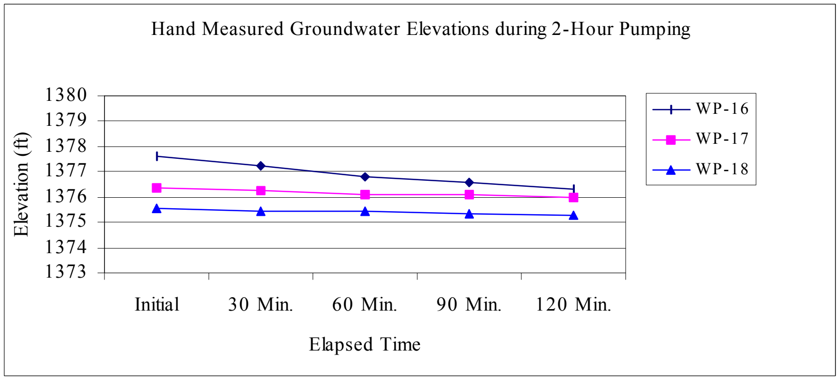


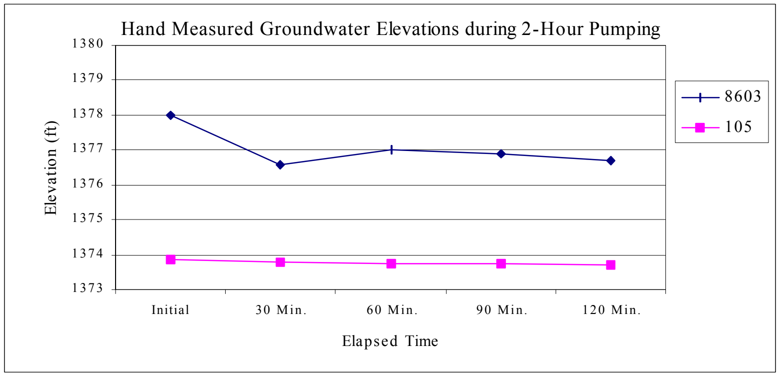

Hand Measured Groundwater Elevations during 2-Hour Pumping
1380
1379
1378
1377
1376
1375
1374
1373
Elevation (ft)
Initial
30 Min.
60 Min.
90 Min.
120 Min.
Elapsed Time
WP-29
WP-31
WP-32
WP-33
PZ-08

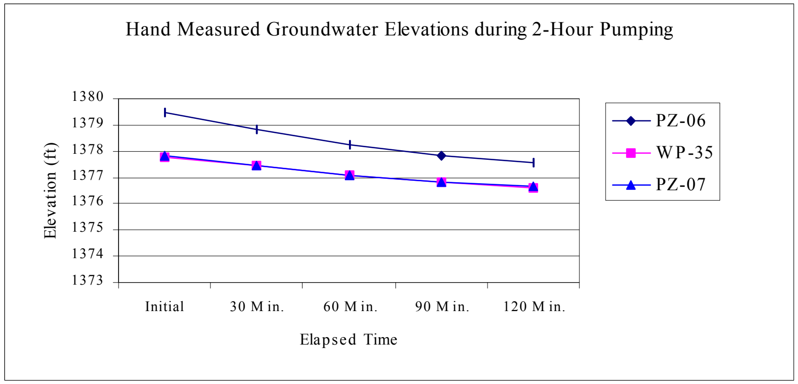

Hand Measured Groundwater Elevations during 2-Hour Pumping
1380
1379
1378
1377
1376
1375
1374
1373
Elevation (ft)
Initial
30 Min.
60 Min.
90 Min.
120 Min.
Elapsed Time
PZ-01
PZ-02
PZ-04

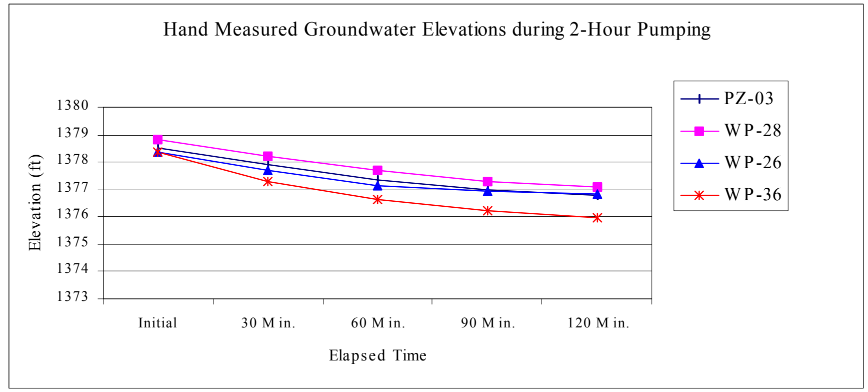

Hand Measured Groundwater Elevations during 2-Hour Pumping
1380
1379
1378
1377
1376
1375
1374
1373
Elevation (ft)
Initial
30 Min.
60 Min.
90 Min.
120 Min.
Elapsed Time
WP-37
WP-38
WP-39
PZ-09

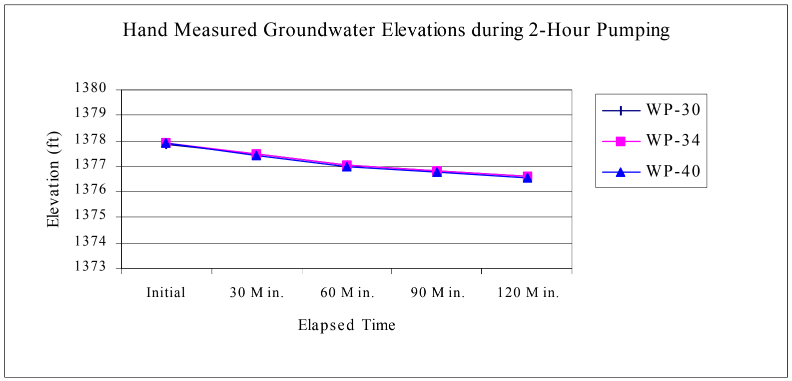

Hand Measured Groundwater Elevations during 2-Hour Pumping
1387
1386
1385
1384
1383
1382
1381
1380
Elevation (ft)
Initial
30 Min.
60 Min.
90 Min.
120 Min.
Elapsed Time
8604
104

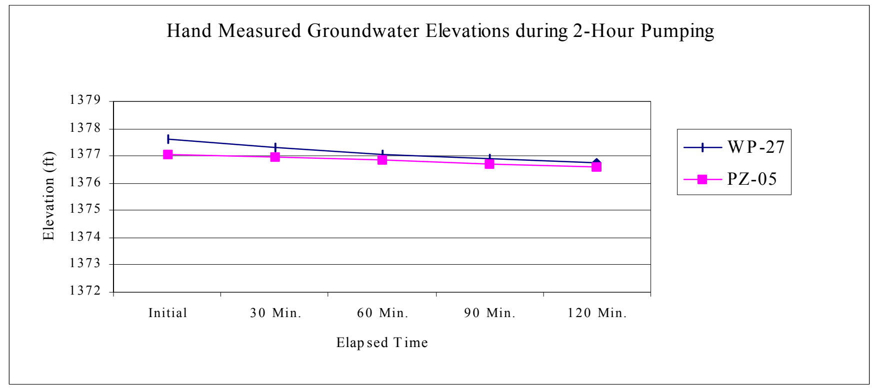

Hand Measured Groundwater Elevations during 2-Hour Pumping
1380
1379
1378
1377
1376
1375
1374
1373
Elevation (ft)
Initial
30 Min.
60 Min.
90 Min.
120 Min.
Elapsed Time
WP-16
WP-17
WP-18

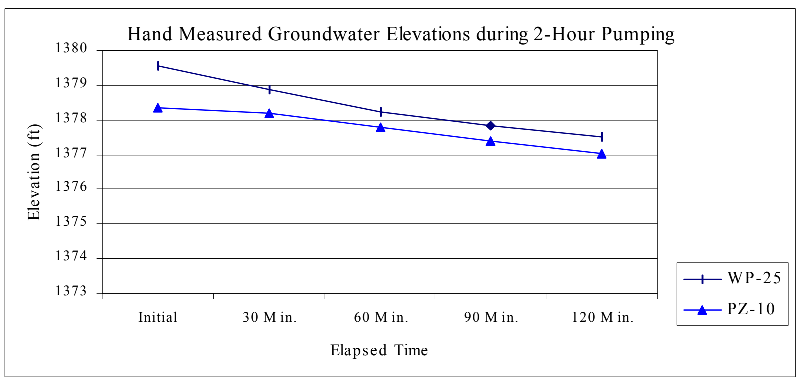

Hand Measured Groundwater Elevations during 2-Hour Pumping
1380
1379
1378
1377
1376
1375
1374
1373
Elevation (ft)
Initial
30 Min.
60 Min.
90 Min.
120 Min.
Elapsed Time
8603
105


Hand Measured Groundwater Elevations during 2-Hour Pumping
1380
1379
1378
1377
1376
1375
1374
1373
Elevation (ft)
Initial
30 M in.
60 M in.
90 M in.
120 M in.
Elapsed Time
PZ-06
WP-35
PZ-07


Hand Measured Groundwater Elevations during 2-Hour Pumping
1380
1379
1378
1377
1376
1375
1374
1373
Elevation (ft)
Initial
30 M in.
60 M in.
90 M in.
120 M in.
Elapsed Time
PZ-03
WP-28
WP-26
WP-36


Hand Measured Groundwater Elevations during 2-Hour Pumping
1380
1379
1378
1377
1376
1375
1374
1373
Elevation (ft)
Initial
30 M in.
60 M in.
90 M in.
120 M in.
Elapsed Time
WP-30
WP-34
WP-40


Hand Measured Groundwater Elevations during 2-Hour Pumping
1379
1378
1377
1376
1375
1374
1373
1372
Elevation (ft)
Initial
30 Min.
60 Min.
90 Min.
120 Min.
Elapsed Time
WP-27
PZ-05

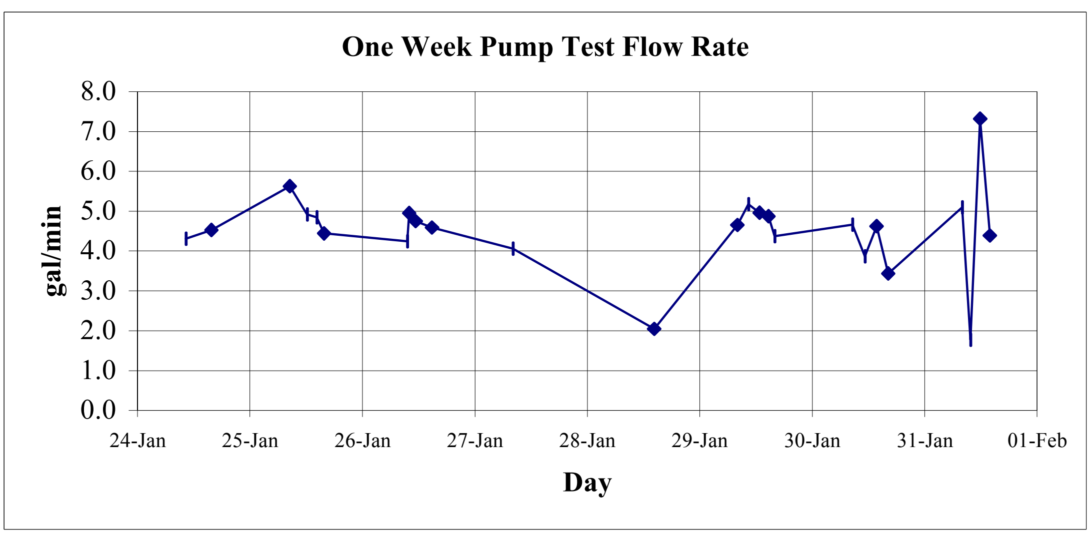

Hand Measured Groundwater Elevations during 2-Hour Pumping
1380
1379
1378
1377
1376
1375
1374
1373
Elevation (ft)
Initial
30 M in.
60 M in.
90 M in.
120 M in.
Elapsed Time
WP-25
PZ-10

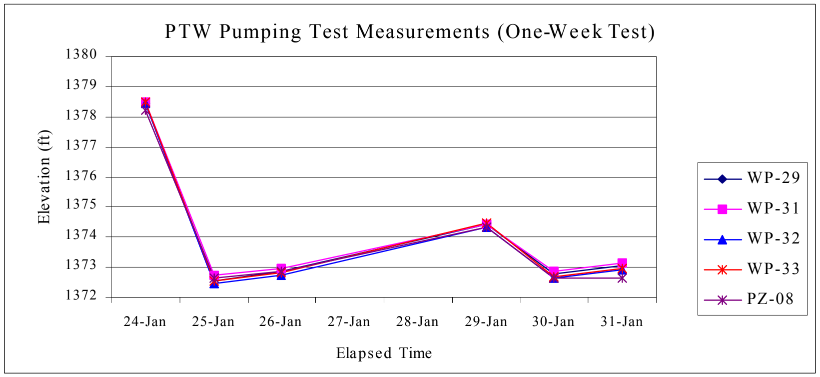
APPENDIX 7
1-WEEK PTW PUMPING TEST DATA

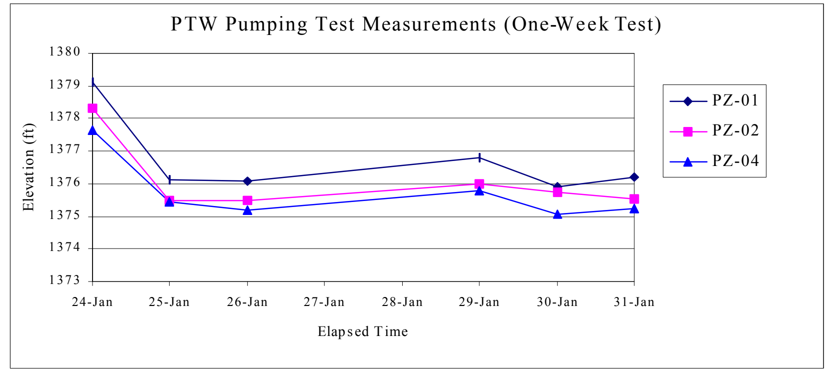
ONE WEEK PUMP TEST DATA

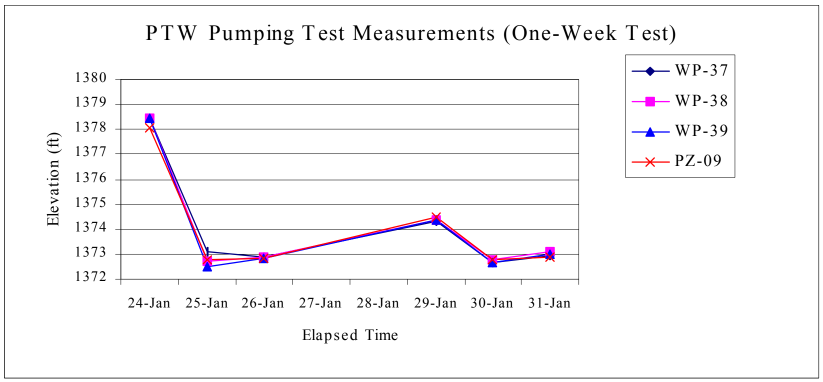
DATE & TIME	ELAPSED TIME (min)	PUMP READING	GALLONS PUMPED	PUMP RATE (gal/min)
1/24/01 10:15		2538		
1/24/01 15:45	330	3960	1422	4.31
1/25/01 8:30	1005	8507	4547	4.52
1/25/01 12:11	221	9750	1243	5.62
1/25/01 14:15	124	10360	610	4.92
1/25/01 15:50	95	10820	460	4.84
1/26/01 9:35	1065	15548	4728	4.44
1/26/01 10:00	25	15654	106	4.24
1/26/01 11:22	82	16060	406	4.95
1/26/01 14:50	208	17048	988	4.75
1/27/01 8:05	1035	21793	4745	4.58
1/28/01 14:15	1810	29145	7352	4.06
1/29/01 8:00	1065	31320	2175	2.04
1/29/01 10:23	143	31985	665	4.65
1/29/01 12:46	143	32725	740	5.17
1/29/01 14:42	116	33300	575	4.96
1/29/01 16:00	78	33680	380	4.87
1/30/01 8:35	995	38035	4355	4.38
1/30/01 11:15	160	38780	745	4.66
1/30/01 13:45	150	39360	580	3.87
1/30/01 16:12	147	40040	680	4.63
1/31/01 8:00	948	43300	3260	3.44
1/31/01 9:48	108	43850	550	5.09
1/31/01 11:52	124	44070	220	1.77
1/31/01 13:55	123	44970	900	7.32
1/31/01 15:02	67	45264.1	294.1	4.39

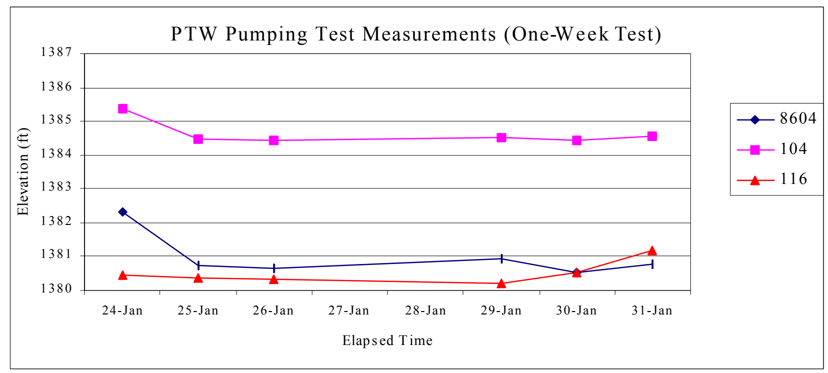
Average discharge = 4.12 gpm

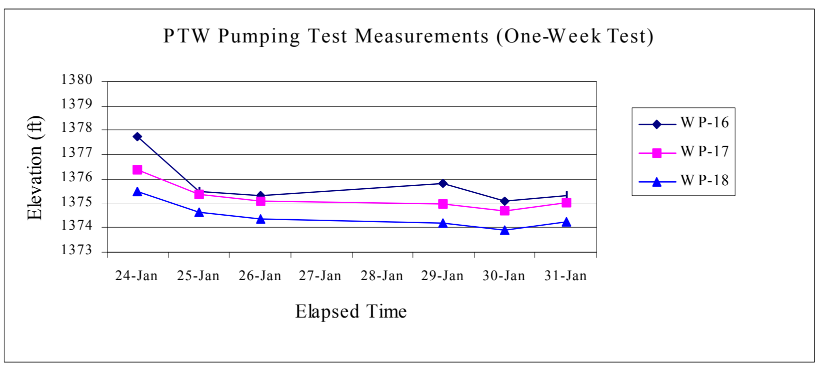


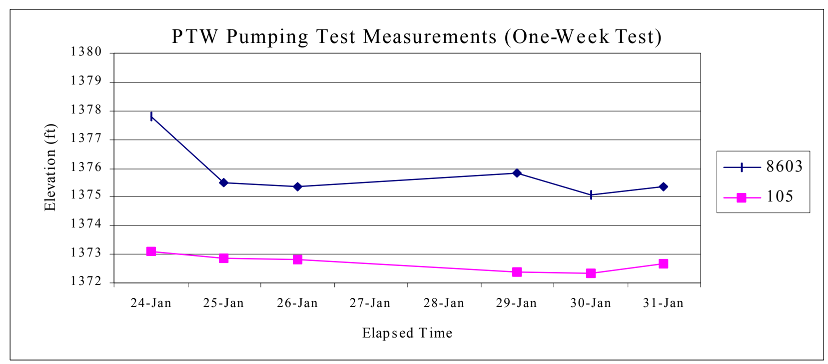

PTW Pumping Test Measurements (One-Week Test)
Elevation (ft)
1380
1379
1378
1377
1376
1375
1374
1373
1372
24-Jan 25-Jan 26-Jan 27-Jan 28-Jan 29-Jan 30-Jan 31-Jan
Elapsed Time
WP-29
WP-31
WP-32
WP-33
PZ-08

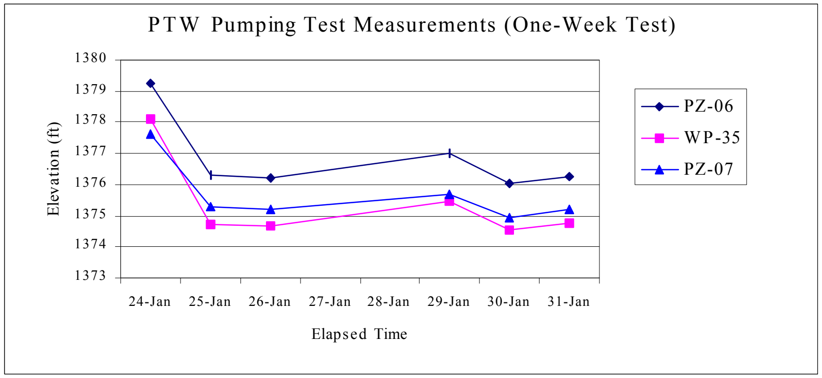

PTW Pumping Test Measurements (One-Week Test)
Elevation (ft)
1380
1379
1378
1377
1376
1375
1374
1373
24-Jan 25-Jan 26-Jan 27-Jan 28-Jan 29-Jan 30-Jan 31-Jan
Elapsed Time
PZ-01
PZ-02
PZ-04

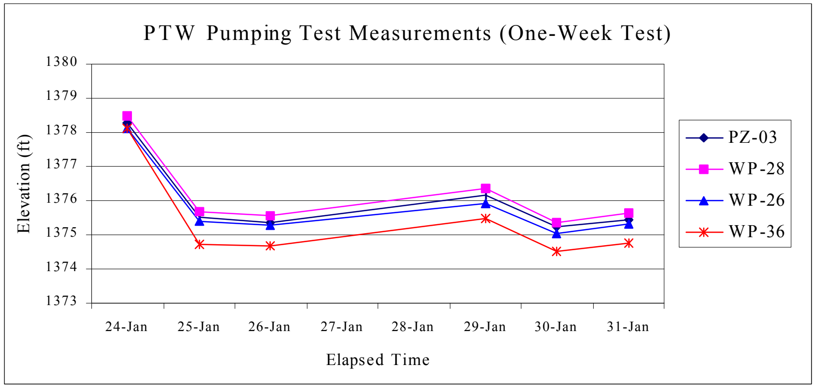

PTW Pumping Test Measurements (One-Week Test)
Elevation (ft)
1380
1379
1378
1377
1376
1375
1374
1373
1372
24-Jan 25-Jan 26-Jan 27-Jan 28-Jan 29-Jan 30-Jan 31-Jan
Elapsed Time
WP-37
WP-38
WP-39
PZ-09

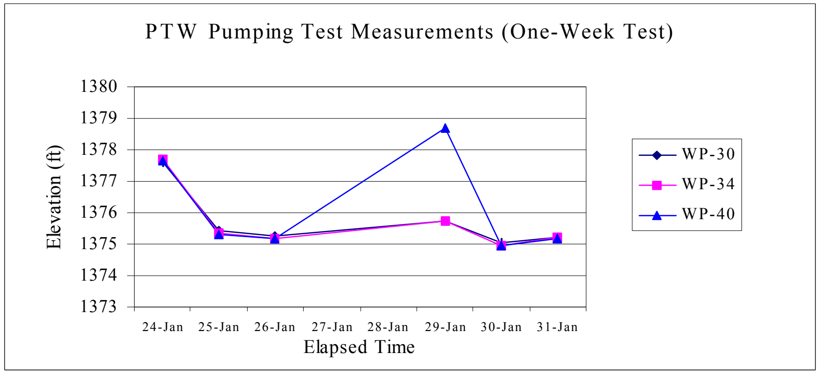

PTW Pumping Test Measurements (One-Week Test)
Elevation (ft)
1387
1386
1385
1384
1383
1382
1381
1380
24-Jan 25-Jan 26-Jan 27-Jan 28-Jan 29-Jan 30-Jan 31-Jan
Elapsed Time
8604
104
116

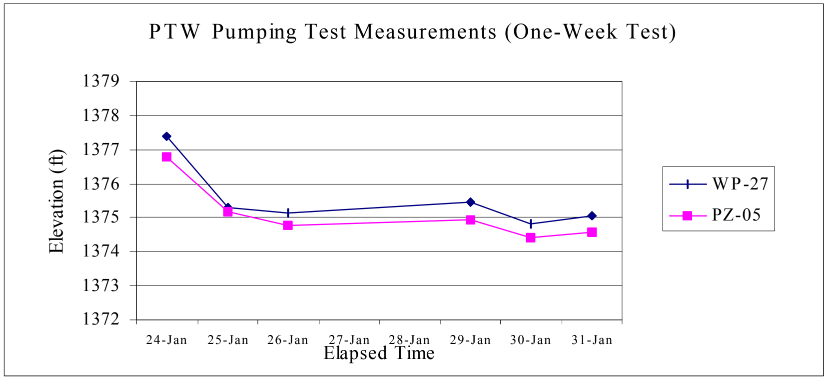

PTW Pumping Test Measurements (One-Week Test)
Elevation (ft)
1380
1379
1378
1377
1376
1375
1374
1373
24-Jan 25-Jan 26-Jan 27-Jan 28-Jan 29-Jan 30-Jan 31-Jan
Elapsed Time
WP-16
WP-17
WP-18

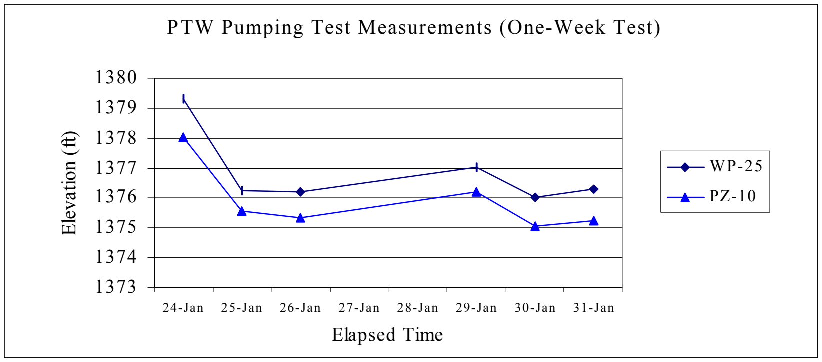

PTW Pumping Test Measurements (One-Week Test)
Elevation (ft)
1380
1379
1378
1377
1376
1375
1374
1373
1372
24-Jan 25-Jan 26-Jan 27-Jan 28-Jan 29-Jan 30-Jan 31-Jan
Elapsed Time
8603
105


PTW Pumping Test Measurements (One-Week Test)
Elevation (ft)
1380
1379
1378
1377
1376
1375
1374
1373
24-Jan
25-Jan
26-Jan
27-Jan
28-Jan
29-Jan
30-Jan
31-Jan
Elapsed Time
PZ-06
WP-35
PZ-07


PTW Pumping Test Measurements (One-Week Test)
Elevation (ft)
1380
1379
1378
1377
1376
1375
1374
1373
24-Jan
25-Jan
26-Jan
27-Jan
28-Jan
29-Jan
30-Jan
31-Jan
Elapsed Time
PZ-03
WP-28
WP-26
WP-36

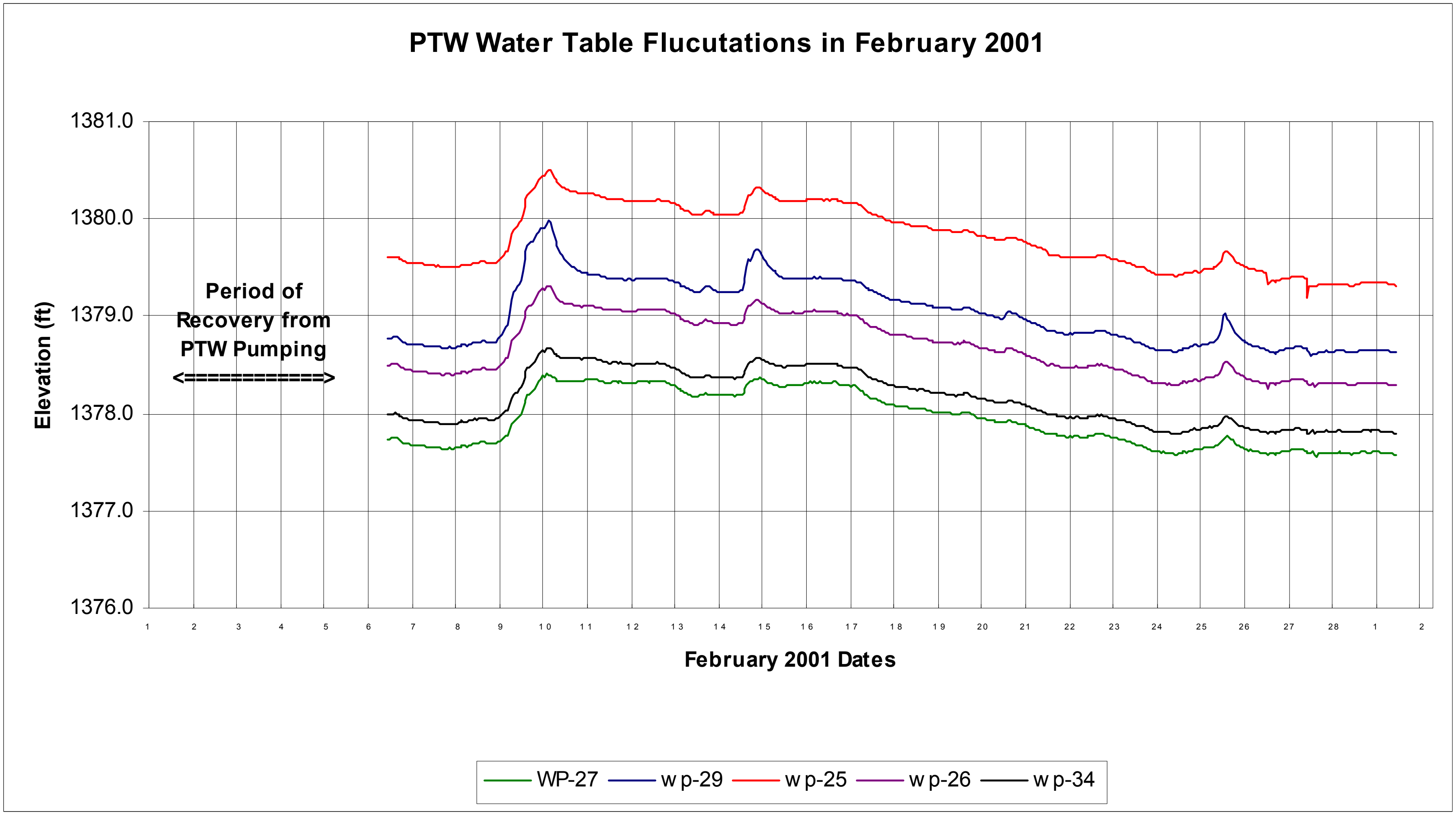

PTW Pumping Test Measurements (One-Week Test)
Elevation (ft)
1380
1379
1378
1377
1376
1375
1374
1373
24-Jan
25-Jan
26-Jan
27-Jan
28-Jan
29-Jan
30-Jan
31-Jan
Elapsed Time
WP-30
WP-34
WP-40

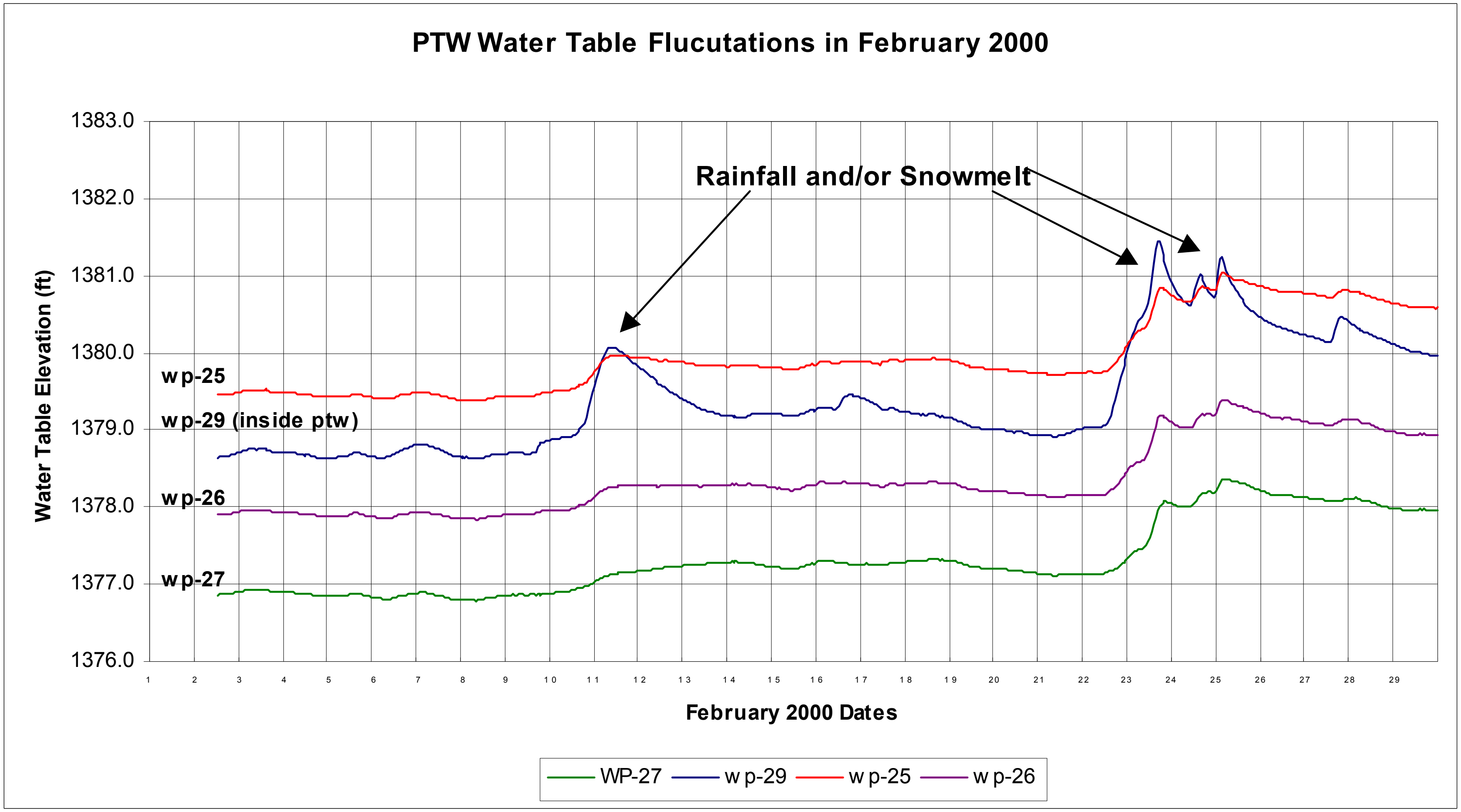

PTW Pumping Test Measurements (One-Week Test)
Elevation (ft)
1379
1378
1377
1376
1375
1374
1373
1372
24-Jan
25-Jan
26-Jan
27-Jan
28-Jan
29-Jan
30-Jan
31-Jan
Elapsed Time
WP-27
PZ-05


PTW Pumping Test Measurements (One-Week Test)
Elevation (ft)
1380
1379
1378
1377
1376
1375
1374
1373
24-Jan
25-Jan
26-Jan
27-Jan
28-Jan
29-Jan
30-Jan
31-Jan
Elapsed Time
WP-25
PZ-10

APPENDIX 8

PTW WATER LEVEL HYDROGRAPHS SHOWING CAP INFILTRATION CHARACTERISTICS


PTW Water Table Flucutations in February 2001
Elevation (ft)
1381.0
1380.0
1379.0
1378.0
1377.0
1376.0
Period of
Recovery from
PTW Pumping
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 1 2
February 2001 Dates
WP-27
w p-29
w p-25
w p-26
w p-34



PTW Water Table Flucutations in February 2000
Water Table Elevation (ft)
1383.0
1382.0
1381.0
1380.0
1379.0
1378.0
1377.0
1376.0
Rainfall and/or Snowmelt
wp-25
wp-29 (inside ptw)
wp-26
wp-27
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29
February 2000 Dates
WP-27
wp-29
wp-25
wp-26

APPENDIX 9
PTW DEVELOPMENT RESULTS SUMMARY DATA

Maximum Hand-Measured Drawdown During Seven PTW Development Efforts

Drawdown in Feet at Measurement Location:

Date/Time	Standpipe	WP-16	WP-25	WP-26	WP-27	WP-28	WP-29	WP-30	WP-31	WP-32	WP-33	WP-34	WP-35	WP-36	WP-37	WP-38
07/17/00	11.69	1.09	1.87	1.27	0.77	1.5		1.05	3.93	3.16	3.7	0.99	1	0.92	3.9	3.76
07/24/00	15.12	2.03	2.9	2.48	1.35	2.59	4.84	2.08	5.15	5.17	5.02	1.99	1.99	2.29	5.17	5
07/25/00	17.2	2.2	3.34	2.67	1.69	2.82	5.16	2.25	5.5	5.56	5.43	2.21	2.17	2.55	5.56	5.39
07/31/00	22.65	2.08	3.05	2.51	1.56	2.67	5.15	2.10	5.42	5.54	5.29	2.07	1.99	2.48	5.51	5.27
08/30/00	9.15	1.7	3.22	2.09	1.34	2.12	4.72	1.72	5.04	5.11	4.99	1.7	1.58	2.9	5.21	5.02
09/08/00	15.33	2.37	3.31	2.8	2.65	2.86	5.63	2.43	5.83	6.02	5.84	2.42	2.32	3.43	6.03	5.79
01/19/01	11.51	1.32	2.05	1.57	0.87	1.74	4.79	1.24	4.54	4.79	4.64	1.31	1.2	2.43	4.98	5.62

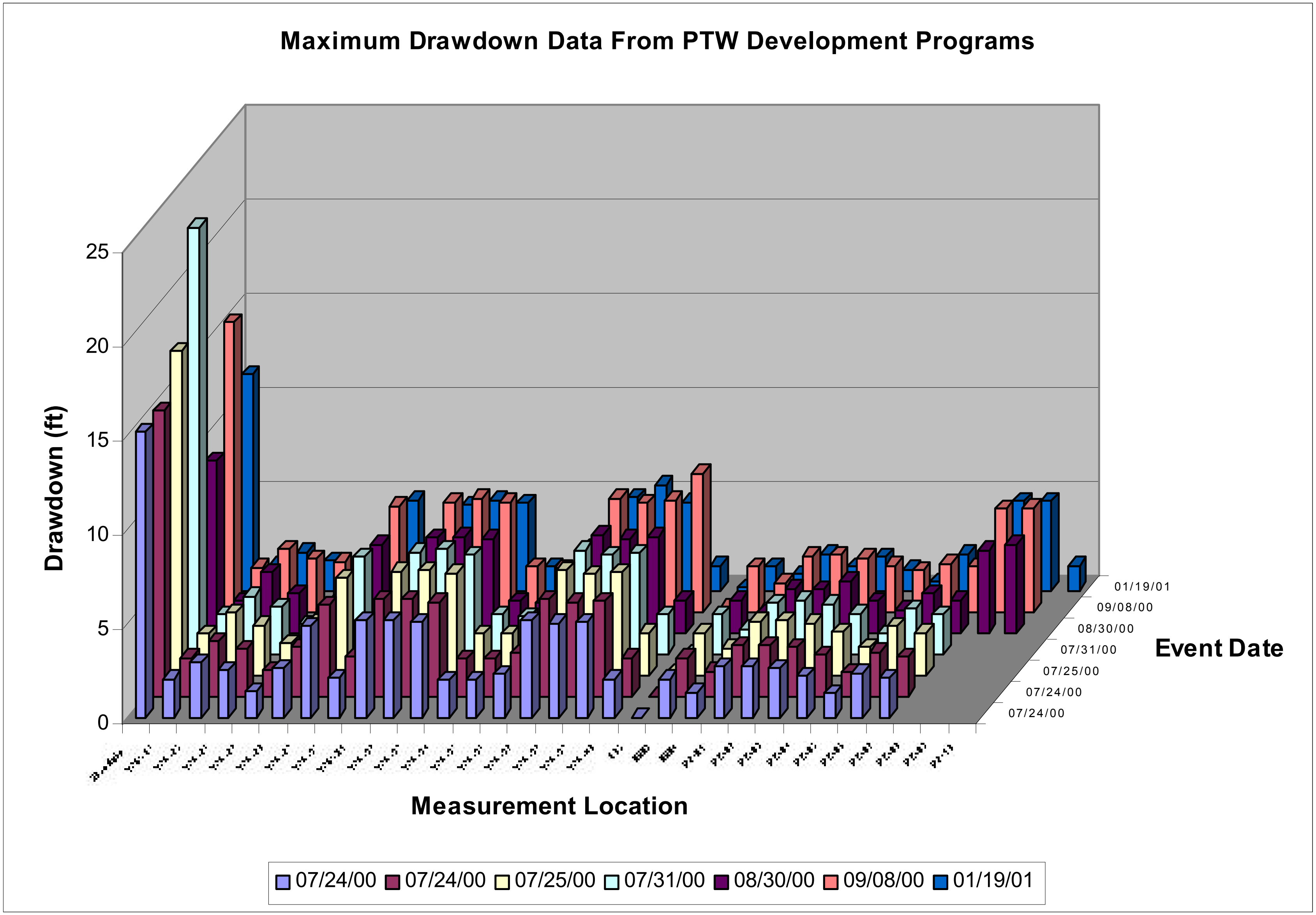
Date/Time	WP-39	WP-40	105	8603	8604	PZ-01	PZ-02	PZ-03	PZ-04	PZ-05	PZ-06	PZ-07	PZ-08	PZ-09	PZ-10
07/17/00	3.3	1.05	0.04	1.14	0.74	1.54	1.51	1.44	1.15	0.44	1.35	1.04			
07/24/00	5.06	2.06	0.02	2.05	1.34	2.69	2.73	2.6	2.21	1.26	2.36	2.07			
07/25/00	5.46	2.24	0.13	2.23	1.42	2.87	2.89	2.75	2.32	1.48	2.62	2.25			
07/31/00	5.35	2.11	0.04	2.07	1.34	2.72	2.87	2.64	2.14	1.10	2.39	2.10			
08/30/00	5.09	1.73	0.01	1.73	1.11	2.35	2.29	2.75	1.71	1.23	2.07	1.73	4.39	4.69	
09/08/00	5.93	7.35	0.22	2.4	1.55	2.96	3.02	2.88	2.46	2.2	2.58	2.41	5.48	5.5	
01/19/01	4.71	1.34	0.16	1.28	0.89	1.95	1.26	1.77	1.13	0.49	1.94	1.21	4.77	4.74	1.33

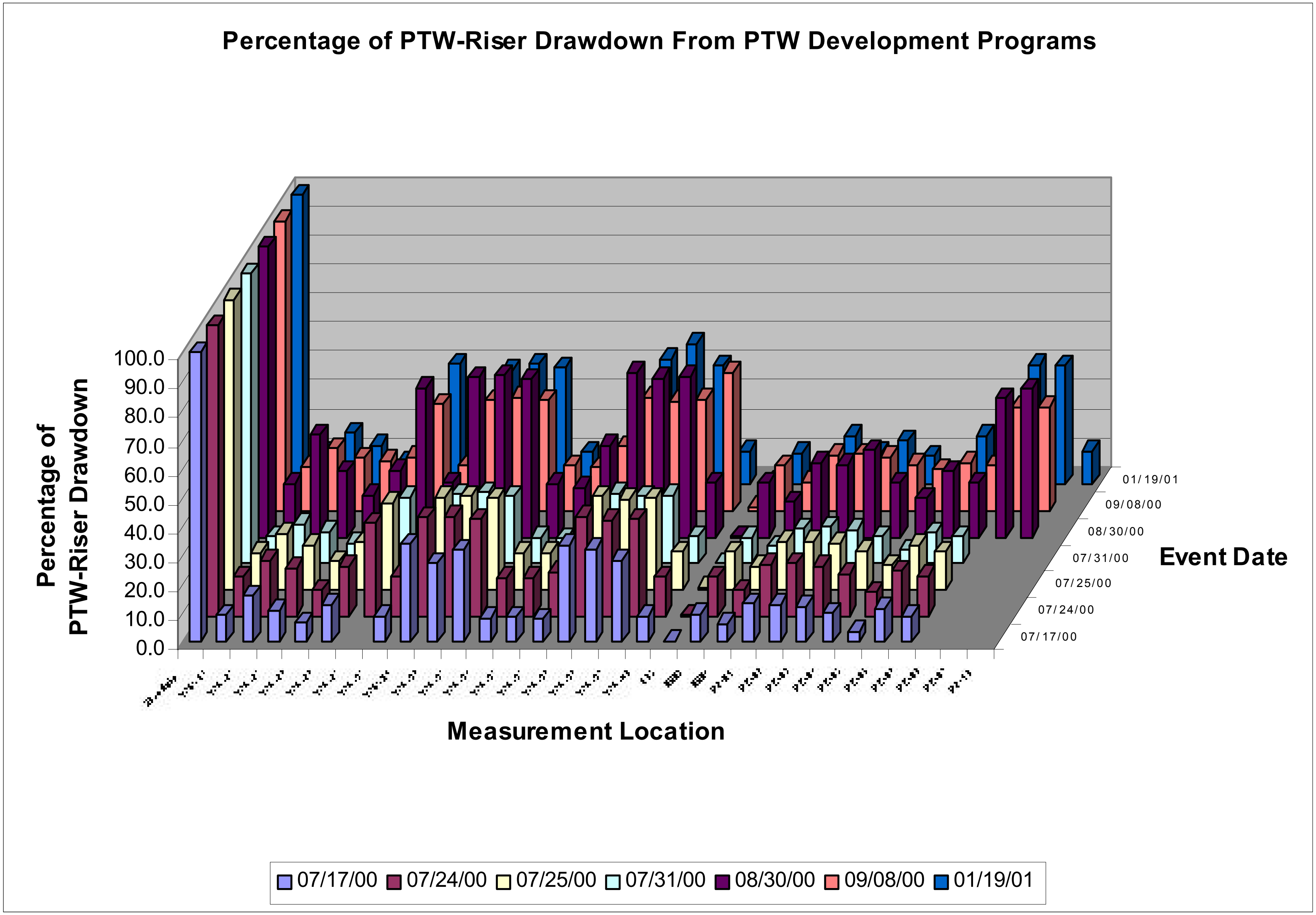
Maximum Hand-Measured Drawdown During Seven PTW Development Efforts

Percentage of PTW-Riser Drawdown in Feet at Measurement Location:

Date/Time	Standpipe	WP-16	WP-25	WP-26	WP-27	WP-28	WP-29	WP-30	WP-31	WP-32	WP-33	WP-34	WP-35	WP-36	WP-37	WP-38
07/17/00	100.0	9.3	16.0	10.9	6.6	12.8		9.0	33.6	27.0	31.7	8.5	8.6	7.9	33.4	32.2
07/24/00	100.0	13.4	19.2	16.4	8.9	17.1	32.0	13.8	34.1	34.2	33.2	13.2	13.2	15.1	34.2	33.1
07/25/00	100.0	12.8	19.4	15.5	9.8	16.4	30.0	13.1	32.0	32.3	31.6	12.8	12.6	14.8	32.3	31.3
07/31/00	100.0	9.2	13.5	11.1	6.9	11.8	22.7	9.3	23.9	24.5	23.4	9.1	8.8	10.9	24.3	23.3
08/30/00	100.0	18.6	35.2	22.8	14.6	23.2	51.6	18.8	55.1	55.8	54.5	18.6	17.3	31.7	56.9	54.9
09/08/00	100.0	15.5	21.6	18.3	17.3	18.7	36.7	15.9	38.0	39.3	38.1	15.8	15.1	22.4	39.3	37.8
01/19/01	100.0	11.5	17.8	13.6	7.6	15.1	41.6	10.8	39.4	41.6	40.3	11.4	10.4	21.1	43.3	48.8

Date/Time	WP-39	WP-40	105	8603	8604	PZ-01	PZ-02	PZ-03	PZ-04	PZ-05	PZ-06	PZ-07	PZ-08	PZ-09	PZ-10
07/17/00	28.2	9.0	0.3	9.8	6.3	13.2	12.9	12.3	9.8	3.8	11.5	8.9			
07/24/00	33.5	13.6	0.1	13.6	8.9	17.8	18.1	17.2	14.6	8.3	15.6	13.7			
07/25/00	31.7	13.0	0.8	13.0	8.3	16.7	16.8	16.0	13.5	8.6	15.2	13.1			
07/31/00	23.6	9.3	0.2	9.1	5.9	12.0	12.7	11.7	9.4	4.9	10.6	9.3			
08/30/00	55.6	18.9	0.1	18.9	12.1	25.7	25.0	30.1	18.7	13.4	22.6	18.9	48.0	51.3	
09/08/00	38.7	47.9	1.4	15.7	10.1	19.3	19.7	18.8	16.0	14.4	16.8	15.7	35.7	35.9	
01/19/01	40.9	11.6	1.4	11.1	7.7	16.9	10.9	15.4	9.8	4.3	16.9	10.5	41.4	41.2	11.6


Maximum Drawdown Data From PTW Development Programs
Drawdown (ft)
0
5
10
15
20
25
Measurement Location
Event Date
07/24/00
07/24/00
07/25/00
07/31/00
08/30/00
09/08/00
01/19/01
07/24/00 07/24/00 07/25/00 07/31/00 08/30/00 09/08/00 01/19/01



Percentage of PTW-Riser Drawdown From PTW Development Programs
Percentage of PTW-Riser Drawdown
100.0
90.0
80.0
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
Measurement Location
Event Date
07/17/00
07/24/00
07/25/00
07/31/00
08/30/00
09/08/00
01/19/01

APPENDIX 10
GEOMATRIX REPORT: PILOT PERMEABLE TREATMENT WALL HYDRAULIC EVALUATION REPORT

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TABLES

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(Continued)

FIGURES

PILOT PERMEABLE TREATMENT WALL HYDRAULIC EVALUATION REPORT
West Valley Nuclear Services, LLC
West Valley, New York

1.0 INTRODUCTION AND SCOPE OF REPORT

This *Pilot Permeable Treatment Wall Hydraulic Evaluation Report* was prepared by Geomatrix Consultants, Inc. at the request of West Valley Nuclear Services, LLC (WVNS). The report was commissioned by WVNS (Project 19-098745-C-JK) to assist in assessing the hydraulic performance of a pilot permeable treatment wall (PTW) designed to remediate groundwater affected by Strontium-90 (Sr-90) beneath a portion of the West Valley Demonstration Project (WVDP) located in western New York State (Figure 1.1). A second evaluation report prepared by Geomatrix for WVNS, the Pilot Permeable Treatment Wall Engineering Evaluation Report, is prepared under separate cover. Both this hydraulic evaluation report and the companion engineering evaluation report support preparation of the Pilot Permeable Treatment Wall Modification Report, to be submitted to WVNS by April 25, 2001.

A pilot PTW, composed of the mineral clinoptilolite, a zeolite whose general solid solution formula is $[(Ca, Mg, Na_2, K_2)(Al_2Si_{10}O_{24}.8H_20)]$ (Warner, 1986), was deployed by WVNS at WVDP in the Fall of 1999 to assess the ability of PTW technology to passively and effectively reduce the concentration of Sr-90-affected groundwater. The pilot PTW was installed to treat a portion of what is referred to as the "2nd lobe" of the Sr-90 plume beneath the North Plateau of the site. Initial mitigation of the "1st lobe" of the Sr-90 plume located beneath the western portion of the North Plateau currently is being addressed by a groundwater recovery and aboveground ion exchange treatment system ("pump-and-treat") which was installed in 1995. While the pump-and-treat remedy has been reported by WVNS to reduce local migration of the Sr-90 plume, it is considered by WVNS not to be capable of completely capturing and remediating the affected groundwater beneath the North Plateau. Thus a review of alternative remediation technologies was conducted by WVNS, and PTW technology was identified as a method potentially capable of effectively mitigating further migration of Sr-90-affected groundwater over a large portion of the North Plateau.

The general purpose of this report is to assess the hydraulic performance of the pilot PTW through its first approximately 15 months of operation. This hydraulic performance evaluation was commissioned because monitoring information from the initial assessment of the test as performed by WVNS indicates that Sr-90-affected groundwater from south, or the presumed hydraulic downgradient side, of the pilot PTW may not be flowing through the pilot PTW as intended.

WVNS has identified several objectives, which are to be addressed for the pilot PTW assessment to move forward. Meeting these objectives are important if the potential effectiveness of this remedial approach as a final remedy to this and other portions of the North Plateau is to be accurately evaluated. The objectives include:

- Evaluate the groundwater hydraulics in the pilot PTW area.
- Evaluate performance of the pilot PTW.
- Collect applicable lessons-learned.
- Develop and compare PTW modification options.
- Provide a recommendation for modifying the pilot PTW.

The basic premise for conducting a pilot test of an innovative approach to groundwater remediation is to assess a small-scale field version of a potential full-scale remedy so as to refine those design parameters that must be met to assure successful, and cost-effective implementation of the full-scale remedy. The approach taken by WVNS to first assess a smaller-scale version of the PTW is valid due to the innovative nature of the remedy (PTW composed of a zeolite) in a complex hydrogeologic environment. Although PTW technology (often referred to as permeable reactive barrier technology) has been tested at more than 40 sites in North America, it remains an innovative technology with a limited database for use in characterizing its potential use over the wide variety of hydrogeologic conditions that exist at affected groundwater sites. Very few of those sites use the clinoptilolite as reactive medium. We also know that perhaps more limiting to the success of a PTW than its ability to chemically remediate a contaminant in groundwater, is its ability to function properly from a hydraulic perspective. Pilot testing can provide additional detail and information imperative to developing a full-scale system that meets its design objectives. We believe that the pilot PTW test has met those objectives and that information from this pilot provides greater certainty that a full-scale system, or deployment of the PTW technology in other areas of the North Plateau, can be effectively designed and implemented.

1.1 APPROACH OF ASSESSMENT AND ORGANIZATION OF FINDINGS

The goals of this report are to assess the hydraulic performance of the pilot PTW. Specifically, the objectives of the report are to provide the following:

- A description of the current performance of the pilot PTW from a hydraulic perspective.
- A discussion of conditions that may be contributing to the current performance including local and regional hydrostratigraphic and topographical features, zones of lower or higher permeability, and preferential flow paths
- A discussion of key factors that must be overcome or modified to establish appropriate groundwater flow through the PTW
- Identification of data gaps or other needs essential to understanding and restoring the hydraulics around the pilot PTW.

The activities undertaken to meet these objectives included: (1) initially meeting with WVNS staff and stakeholders to discuss the test program; (2) reviewing data and reports made available by WVNS; (3) requesting additional data, or clarification of information, from WVNS staff; (4) reviewing and discussing the results of hydraulic modeling efforts with researchers at the State University of New York at Buffalo (UB) who have been commissioned by WVNS to develop a hydraulic model of the pilot PTW; and (5) preparing this report.

We understand that the design of the pilot PTW was based on a basic review of regional hydrogeologic conditions, supplemented with additional characterization of the location selected for the pilot PTW test. Because the hydrogeologic conditions at the pilot PTW, and its performance, are intimately related to the regional hydrogeologic characteristics of the North Plateau, our general approach to this study has focused on integrating our interpretation of regional conditions to the local behavior in and around the PTW. Also, although a detailed engineering evaluation of the pilot PTW is contained in a separate report, we have integrated certain aspects of the installation methods and as-built conditions that may have affected the hydraulic performance. We also have been in regular communication with UB researchers as they continue to develop a groundwater model of the pilot PTW area. We have integrated aspects of the UB work in this report; however, we do not present specific results from that work. The results of our assessment, however, may be considered important to continuing model development by UB or others, and we include conceptual recommendations for such work.

Aside from our recommendation to WVNS to sample four piezometers south of the pilot PTW (which was performed by WVNS in late February 2001) for which Sr-90 activity data was not available, we have not performed field work, nor have requested additional field work be performed for this assessment. However, based on our interpretation of the current information, we provide recommendations for additional fieldwork that we consider important for developing an approach to modify the pilot PTW and assuring success of future deployments.

Following this introductory section, which includes a general description of the pilot PTW and a summary of previously published technical evaluations of the pilot PTW performance, this report consists of the following Sections:

- Section 2.0 – Hydrogeologic Conditions at the North Plateau (including discussions of site hydrostratigraphy, regional groundwater conditions, distribution of hydraulic conductivity, distribution of Sr-90, the conceptual hydrogeologic model of the North Plateau).
- Section 3.0 – Hydrogeologic Conditions in Proximity to the Pilot PTW (including discussions of local hydrostratigraphy, potentiometric conditions pre- and post-PTW construction; distribution of Sr-90 pre- and post-PTW construction; assessment of hydraulic testing in and near the PTW; and the conceptual model of the pilot PTW performance including an assessment of numerical modeling exercises conducted by UB).
- Section 4.0 – Recommendations (including identification of data gaps and recommendations for collecting additional information).
- Section 5.0 – References

1.2 DESCRIPTION OF THE PILOT PTW

A detailed engineering description of the construction of the pilot PTW is provided in the companion *Engineering Report*, however, a general description of the system, based on existing reports provided by WVNS, is as follows. The pilot PTW was installed as an approximately 30-foot long by 26-foot deep by 7-foot thick "continuous" PTW (i.e., lateral hydraulic barriers were not installed to direct groundwater flow into the PTW) in an area characterized as the leading edge of the 2nd lobe of the Sr-90 plume. A general cross-section of the pilot PTW is provided as Figure 1-2. The PTW was constructed using conventional trench and fill techniques where the PTW trench was stabilized using sealed sheet piles to create a cofferdam-type structure prior to excavating native soil from the interior of the sheets. The sheet piles were installed to a depth of approximately 36 feet below ground surface (bgs) or approximately 10 to 12 feet below the anticipated contact between the upper water-bearing material and the underlying low permeability till. The native material within the cofferdam was dewatered prior to excavation using 8-inch dewatering wells installed prior to the excavation, and was kept dry during placement of the treatment material. Unmixed zeolitic material (i.e., 100 percent clinoptilolite as delivered) was placed to fill the cofferdam to near ground surface with the exception of an approximately 1.5 foot zone of gravel ("1-inch roundstone") that was placed at the upgradient front (south) of the pilot PTW. A horizontal drainpipe was placed at the bottom of the gravel section; the connecting riser pipe with pump assembly is located at the eastern end of the gravel section. Once the excavation was filled, the sealed sheet piles were removed starting at the west end of the pilot PTW. The sheet piles were installed in August 1999 and removed in November 1999.

Settlement of approximately four feet within the PTW material was reported by WVNS to have occurred following removal of the sheet piles. Following "topping off" with additional zeolite material, an approximately 2-foot thick low permeability clay cap was placed on top of the PTW to aid in preventing surface water infiltration to the system. A hardstand area was placed around the PTW to provide a working surface for heavy equipment during the PTW installation. A surface water cutoff drain was installed at ground surface in mid-October 2000, approximately 10 months following installation of the PTW, to limit the potential for surface water runoff (partially via the hardstand configuration) to infiltrate into the PTW.

The hydraulic monitoring system of the pilot PTW locally consists of a series of well points, monitoring wells, and piezometers located around and within the PTW; most of these devices were installed after the PTW construction. Locations and construction details (depth and screen intervals) of these water level monitoring points are discussed in greater detail in Section 3.

1.3 SUMMARY OF PREVIOUS PTW TECHNICAL PERFORMANCE ASSESSMENT REPORTS

As part of this assessment, we have reviewed and considered opinions rendered in previously published technical documents on the pilot PTW performance. Specifically, the reviewed assessment documents include:

- "Summary Information from PTW Evaluation and Assessment Activities," dated February 6, 2001, prepared by WVNS.

- "Technical Peer Review/Evaluation of the West Valley Pilot Permeable Treatment Wall," dated October 11, 2000, prepared by J. Moylan, URS Corporation.

- "Preliminary Operational Assessment for the Pilot Permeable Treatment Wall," dated May 12, 2000, prepared by WVNS.

- "Review of Preliminary Operational Assessment Report for the Pilot Permeable Treatment Wall (Draft), West Valley Demonstration Project," dated May 5, 2000, prepared by E. Berkey, Ph.D.

Although the purpose of this report (and section) is not to reiterate the detailed opinions and comments of each of the above documents, some common opinions include the following:

1. Uncertainty exists as to the direction of the local lateral hydraulic gradient prior to construction (the regional direction was generally observed to be toward the north; though some data indicated a northeasterly direction).

2. Water levels in well WP-25 control the interpretation of the local groundwater gradient direction; water levels increased following sheet pile installation and remain high. The direction of groundwater flow appears to flow directly eastward through the PTW based in part on well WP-25 data.

3. Surface water infiltration was considered to strongly contribute to hydraulic mounding within the PTW.

4. Uncertainty exists as to the nature and location of a "skin" effect and its influence on hydraulic connections between the PTW and the native aquifer; though results from long-term pumping does indicate reasonable hydraulic connection that appeared to improve following long-term pumping from within the PTW.

5. There is considerable variability in the distribution of hydraulic conductivity in the upper water-bearing unit.

6. Slug test results may not fully represent actual hydraulic conditions.

7. The activity of Sr-90 in groundwater sampled from within the PTW is very low. Activity of Sr-90 in groundwater sampled from west of the PTW is low; activity of Sr-90 in groundwater southeast, east, and north of the PTW is high and appears to have increased in several locations since installation of the PTW.

From the previous assessments, multiple opinions exist as to what the most appropriate modification to the PTW consists of (e.g., additional development, installation of "wing" or lateral hydraulic barriers to route groundwater through the PTW, etc.). Generally, however, certain data gaps exist which specific additional field characterization activities should adequately address and result in developing an engineering solution with high potential for success.

2.0 HYDROGEOLOGIC CONDITIONS AT THE NORTH PLATEAU

This section provides an interpretation of regional hydrogeologic conditions of the North Plateau. Emphasis is placed on interpreting the general hydrostratigraphic sequence and groundwater flow conditions regionally to assist in understanding the local conditions in and near the pilot PTW that contribute to the current hydraulic performance. Information that supports the following interpretation was obtained from existing reports for the site published between 1995 and the present, including WVNS (1995), Hemann, et al. (1998), and WVNS (2000), and unpublished information provided by WVNS. A map illustrating features of the site and showing the alignment of hydrostratigraphic cross-section, as well as showing the regional distribution of Sr-90 in groundwater is provided as Figure 2.1.

The underlying stratigraphy of the North Plateau was evaluated through examination of borehole logs collected during characterization activities performed in 1993, 1994, 1995 and 1997, as well as data collected during the PTW design activities. The comprehensive record of borings yields insight into the varying spatial and compositional distribution of geologic materials beneath the North Plateau.

Hydraulic head data from 1992 to the present were provided by WVNS for this assessment. Our analysis focused on four separate measurement dates to provide some representation of temporal and spatial variability in groundwater conditions across the plateau.

Data from the 1997 Geoprobe sampling event (Hemann, et al., 1998) were reviewed and integrated with stratigraphic cross-sections developed for this assessment to better interpret the vertical distribution of Sr-90 and identify preferential flow paths important to understanding the chemical migration pathway in the vicinity of the pilot PTW. Concentration data from more recent groundwater sampling events also were reviewed in this assessment.

Details of our findings of the regional conditions are provided in the following subsections.

2.1 SITE HYDROSTRATIGRAPHY

Two distinct hydrostratigraphic units are interpreted to exist beneath the North Plateau:

- A basal confining aquitard (or low permeability zone) consisting of lacustrine clay and silt, and Lavery Till;
- The Shallow Water-Bearing Zone (SWBZ), consisting of alluvial sand and gravel (AS&G), and the Slackwater Sequence (SWS).

The SWBZ may be further characterized as a heterogeneous unit representing a composite of the both laterally continuous and discontinuous layers representative of both the AS&G and the SWS as described in further detail henceforth the SWBZ exists approximately five feet below ground surface in the vicinity of the PTW, and extends downward to the basal confining unit. Unconfined conditions generally appear to exist in the upper portion of the SWBZ within the AS&G, however, logs for Geoprobe® and soil boring data show several layers of clay and silty clay present within the SWS and likely contribute to the development of semi-confined conditions to develop within the lower portion of the SWBZ.

In the western portion of the study area, the SWBZ is composed only of the coarse, unconfined deposits of the overlying AS&G unit, which extends from the ground surface downward to the basal confining unit. The thickness of the AS&G unit in the western portion of the study area varies from 5 to 15 feet, depending on the topography of the basal confining. The water table in AS&G unit is approximately five feet below ground surface.

Three cross-sections as shown on Figure 2.1 illustrate the hydrostratigraphy beneath the site (Figures 2.2, 2.3, and 2.4). Cross section A-A' (Figure 2.2) crosses both lobes of the Sr-90 plume from northeast of the PTW to southwest of the groundwater recovery system installed in the western lobe of the plume. The lower-most unit encountered in the boring logs along A-A' is the Lavery Till, defined in bore logs as a laterally continuous stiff, unsorted sequence of sand, silt, clay and gravel. The hydraulic conductivity of the Lavery till is reported to be low (less than 1×10^{-7}cm/s) producing a basal unit with variable elevation beneath the shallow water-bearing zone in the area of interest around the Sr-90 plume.

A sequence of lacustrine clay with silt overlies the Lavery Till across much of the study area, with the exception of the extreme northeast corner, where it is replaced with a fine sand and silt unit, identified in the boring for well #8603. The thickness of the clay with silt unit varies considerably from east to west, with the thickest section described in the boring log for well #0115 and boring B-94-11. This thick clay and silt unit has been interpreted as the Lavery Till in previous borings, however based on the evidence of an alternating sequence of clayey silt and silty clay in the bore logs, it is probable that it is the result of a lacustrine depositional environment. The hydraulic conductivity of the clay with silt unit was found to be $4x10^{-8}$ cm/s by hydraulic testing performed on well #0115, which is screened entirely in the unit.

The lacustrine clay with silt unit is overlain by a thick sequence of water-lain deposits composing the SWS. Sediments associated with the SWS are comprised of thin, well-sorted beds of gravel, fine sand and silt and fills a wide, channel-like depression in the eastern half of the study area (Figure 2.5). Locally, significant variations of the grain size of geologic materials comprising the SWS exist. The bore log for well #8604 on cross-section C-C' (Figure 2.4) identifies a two-feet thick silty clay lens within the surrounding gravel and fine sand. The interbedded nature of coarse and fine-grained deposits causes confining groundwater conditions to exist within the water-bearing deposits of the SWS. Variations in the stratigraphy (i.e., thickness and lateral continuity of water-bearing deposits) may significantly change the local hydraulic gradient and flow directions, which cannot be accounted for on site-wide maps.

The SWS is overlain by alluvial sand and gravel (AS&G) in the northern half of the study area. This unit is distinguished from the SWS by a finer sequence of silt and sand, with less gravel and less stratification than the underlying unit. The unit coarsens to the west, with the disappearance of much of the finest-grained material, and is described as alluvial gravel and fine sand (Cross section A-A').

The uppermost stratigraphic unit in the North Plateau is a silty clay unit, which overlies the finer grained alluvial silt and sand with gravel layer. Bore-logs along cross- sections A-A' and C-C' indicate that the silty clay is consistently 2 to 3 feet thick across the northern half of the study area. This unit may serve as a confining layer to the underlying water-bearing units, and may limit vertical recharge from surface infiltration by precipitation where it is continuous.

2.2 DISTRIBUTION OF HYDRAULIC CONDUCTIVITY

The variability of geologic materials across the North Plateau study-area contributes to variable hydraulic conductivity. The highest hydraulic conductivity values reported for the SWBZ exist in the western lobe of contamination near the groundwater extraction system (GWES) ($3x10^{-3}$ cm/s; WVNS, verbal communication). As the grain size composition of the AS&G becomes finer in an eastward direction toward the PTW, conductivity values decrease to $1x10^{-5}$ cm/s within the AS&G.

Well #0116 is screened west of the facies change in the AS&G unit west of the SWS, and yields a hydraulic conductivity value of $6x10^{-5}$ cm/s. The recovery wells (RW-01 through RW-

03) associated with the GWES in the western lobe of contamination are screened in the coarser gravel and fine sand of the AS&G. Hydraulic testing associated with these wells yields a significantly higher average hydraulic conductivity value of $3x10^{-3}$ cm/s.

Further to the east, in the vicinity of the PTW where the SWBZ consists of both the AS&G and the SWS, hydraulic conductivity values are reported to be in the range of $1x10^{-3}$ to $1x10^{-4}$ cm/s. The fully penetrating nature of wells and piezometers installed in the SWBZ yields *average* hydraulic conductivity values, and higher conductivity is possible in discrete, continuous sand and gravel layers associated with the SWS. Alternating silt and clay layers with sand and gravel layers contributes to anisotropic conditions in the SWBZ and localized confined conditions may develop within the sequence as a result of the decreased vertical hydraulic conductivity produced by the clay and silt layers.

2.3 REGIONAL GROUNDWATER CONDITIONS

The regional groundwater flow patterns in the SWBZ were evaluated for four separate groundwater flow conditions at the North Plateau. The first two scenarios were constructed to define the ambient regional groundwater flow patterns under period of low groundwater elevation (August 1998) and higher groundwater elevation (February 1997) and prior to any construction activities associated with the PTW. Potentiometric maps were constructed to show the site-wide head distribution before and after short term pumping (i.e., hydraulic development) of the PTW.

Low Groundwater Elevation Flow Pattern (August 1998): The groundwater flow pattern in the vicinity of the Sr-90 impacted zone was evaluated for a low-groundwater elevation condition that existed in August 1998 (Figure 2.6). The highest groundwater elevation was found southwest of the vitrification test facility, leading to a general groundwater flow direction to the north-northeast. A uniform hydraulic gradient existed throughout much of the study area, with the exception of the northeast corner, in the vicinity of wells #0105 and #8603 where the gradient increased, along with a change in flow direction directly eastward toward well #105. Superposition of the August 1998 groundwater condition surface onto current site maps suggests that the groundwater flow direction before construction of the PTW was virtually perpendicular to the current long-axis of the PTW.

High Groundwater Flow Pattern (February 1997): The geometry of ambient groundwater flow pattern for the high water table conditions in February 1997 is illustrated in Figure 2.7. The potentiometric surface under these conditions closely resembles that of the low groundwater flow pattern. The general flow direction toward the north-northeast, however, became more eastward in the vicinity of wells #105 and #8603. The hydraulic gradient between wells #105 and #8603 is interpreted to be higher than that associated with the lower water table conditions. The anomalously high head value in well #8603, which directly influenced the magnitude and direction of groundwater flow in the northeast quadrant of the study area, may be explained by a semi-confined condition for the water-bearing zone in the vicinity of well #8603. The groundwater flow direction was nearly perpendicular to the current PTW during the high groundwater condition.

Post Construction Pre-Development Groundwater Flow Pattern (May 2000): The groundwater flow pattern for the conditions after construction of the PTW was also evaluated (Figure 2.8). The overall site-wide flow pattern was similar to historical patterns, however, the flow field was perturbed in the immediate vicinity of the PTW. Emplacement of the PTW caused groundwater mounding south and west of the wall, as well as within the wall, changing the flow geometry. Furthermore, an apparent flattening of the horizontal hydraulic gradient is observed to the northeast of the PTW. Unlike pre-PTW-construction flow conditions, the overall flow direction was due eastward in the vicinity of the PTW, and *parallel* to the long axis of the PTW (Figure 2.8). The horizontal hydraulic gradient is steeper and directed eastward in the vicinity of well #105.

Post Construction Post Development Groundwater Flow Pattern (November 2000): The PTW was subjected to pumping in July, August and September 2000 in order to increase the hydraulic conductivity in the material both in and adjacent to the wall. Prior to development, groundwater was mounded in and around the PTW, leading to preferential pathways around, rather than through the wall itself. Development of the PTW has decreased, but not eliminated the mounding of groundwater in and around the wall. The presence of the apparent very low lateral hydraulic gradient downgradient of the PTW (Figure 2.9), persisting after development, indicates that conductivity of some of the material within or around the PTW may be comparatively lower than the surrounding native deposits. The general groundwater flow direction in the vicinity of the PTW is oblique from the southwest, but changes to directly eastward at the face of the wall. At this scale, groundwater flow appears to flow parallel to, and directly through the long axis of the PTW. On a smaller scale, however, flow near the western portion of the wall may have a short southerly flow vector from the wall into the SWBZ, caused by minor mounding within the wall. Substantial reduction in Sr-90 activity in WP-28 supports the identification of this flow path.

2.4 REGIONAL DISTRIBUTION OF STRONTIUM-90

Sr-90 with a radioactive half-life of approximately 28 years is a common product of the fission of Uranium-235 and has a general chemistry similar to that of calcium. Solubility of Sr in natural groundwater typically is controlled by Sr carbonate and sulfate minerals such as strontionite and celestite, respectively, with strontionite less soluble than celestite. The ambient concentration of Sr in natural waters at near neutral pH is typically much less than solubility and normally less than about 0.15 milligrams per liter. This suggests that Sr can be considered relatively conservative and not likely to be significantly incorporated into secondary mineralization unless, similar to calcium, hydrochemical conditions (such as a change in pH) alter mineral solubility. However, strontium can replace calcium in like minerals (similar to the objective of using the zeolite clinoptilolite in promoting an ion exchange reaction within the pilot PTW); thus the presence of native calcium-based carbonates and other minerals in the geologic material beneath the North Plateau can promote local limited natural mitigation of the Sr-90 plume.

The distribution of the two lobes of the Sr-90 plume as interpreted from data collected in December 1999 and January 2000 (Figure 2.1) are consistent with the distribution of higher hydraulically conductive materials across the North Plateau. The zones of higher conductivity associated with the SWS in the eastern lobe and the AS&G unit in the western lobe contain the highest concentrations of Sr-90 in the study area. The apparent bifurcation of the Sr-90 plume can best be attributed to the existence of the large lower conductivity clay and silt unit identified in the boring logs for well #0115 and B-94-13. A thin section of the AS&G sequence overlies the thick clay and silt unit, however the path of least resistance for groundwater is around the clay and silt unit into the thicker water-bearing zones associated with the SWS and the AS&G.

The *vertical* distribution of Sr-90 follows the zones of highest hydraulic conductivity in the subsurface. The highest concentrations of Sr-90 in the eastern lobe (12,200 pCi/L, 1997 Geoprobe® data) are found in the lower portions of the SWS (Figure 2.2). The highest Sr-90 concentrations associated with the western lobe of the plume are found in the coarse deposits associated with the AS&G deposits. Sr-90 concentrations are often higher in the lower portions of the AS&G unit, near its contact with the clay with silt unit.

The most recent Sr-90 sampling data (February 28, 2001) has yielded data prompting a revised interpretation of the location of the 10,000 pCi/L isoconcentration line. Based on concentration data from monitoring wells PZ-04 (41,100 pCi/L) and PZ-02 (3,650 pCi/L), the width of the

>10,000 pCi/L zone may be more narrow than that shown on the plot of the 1997 Geoprobe® sampling data. Implications of the new data from PZ-01 through PZ-04 are further discussed in Section 3.3. It is possible however that the low Sr-90 activity measured in PZ-02 in February 2001 illustrates a local condition that is not laterally extensive. Strontium-90 activities measured in well #8603 prior to the installation of the PTW were actually higher than 10,000 pCi/L. Additional characterization in this area would be beneficial to assess the distribution of Sr-90 south of the pilot PTW.

2.5 CONCEPTUAL HYDROGEOLOGIC MODEL OF THE NORTH PLATEAU

The groundwater flow conditions that existed prior to any construction activities associated with the PTW were primarily influenced by spatially varying hydraulic conductivity across the site as well as undulations in the surface of the underlying till. The general groundwater flow direction was toward the north-northeast. Groundwater movement occurs primarily within the SWBZ, with the low conductivity till and clay with silt layer acting as a basal hydraulic barrier to the flow system. On the eastern portion of the mounded aquitard unit near boring #0116, a northeasterly flow component developed, potentially directing groundwater into the higher conductivity, stratigraphically lower SWS below the eastern lobe of Sr-90 contamination.

The large area of low-conductivity clay with silt had no recognizable effect on water levels in the surrounding wells (when viewed on a regional scale), but appears to separate the Sr-90 plume into two parts. This is best explained by the existence of thicker, more conductive units east and west of the clay with silt low conductivity zone. Preferential pathways within these units caused Sr-90 to become more highly concentrated in discrete zones lower in the sequence where hydraulic conductivity may be locally higher. The eastern lobe of the Sr-90 plume is subjected to a steeper apparent hydraulic gradient toward well #0105, causing a slightly eastward extension of part of the lobe. The overall groundwater flow direction at the current location of the pilot PTW is therefore interpreted to be nearly parallel (east to east-northeast) to the long axis of the pilot PTW.

The flow system in the eastern lobe of the Sr-90-impacted area may become semi-confined where the thick silt and clay unit caps the SWBZ. The steep hydraulic gradient between well #8603 and WP-11 (illustrated in Figure 2.6) may be coincident with a transition between semi-confined and unconfined conditions.

3.0 HYDRAULIC CONDITIONS IN PROXIMITY TO THE PILOT PTW

This section presents an assessment of hydrogeologic conditions and hydraulic performance within and specific to the vicinity of the pilot PTW. Existing data has been reviewed and an interpretation of the local hydrostratigraphy, groundwater head conditions, distribution of Sr-90 from both pre and post-construction periods, and PTW hydraulic development activities is provided. A conceptual model of the PTW with respect to its hydraulic performance is presented at the conclusion of this section.

3.1 HYDROSTRATIGRAPHY PROXIMAL TO THE PILOT PTW

Figure 3.1 shows the locations of borings, monitoring points and the location of cross-section D-D' in the pilot PTW area. The local stratigraphy is shown on Figure 3.2. Geologic logs are available from borings GP5 97, GP21 97, GP4 97, GP20 97, GP3 97, GP19 97, and GP2 97, advanced during the 1997 Geoprobe investigation. The centerline of the PTW is located approximately 8 feet to the south of the soil borings alignment.

Consistent with the regional hydrostratigraphy described in Section 2.1, the glaciolacustrine deposits encountered in the PTW area consist predominantly of silty clay (ground surface to approximately 6 feet bgs), silt and gravel (approximately 6 to 11 feet bgs), alternating layers of fine to coarse sand with gravel and silty clay with gravel characteristic of the SWS (approximately 11 to 25 and 30 feet bgs). The SWS is highly heterogeneous with layers of sand and gravel alternating with clay and silt layers approximately every 6 inches based on boring log reports. The lateral extent of each interbed appears to be highly variable. This alluvial sequence overlies the Lavery till which is encountered at depths of approximately 25 to 30 feet bgs. The top of the Lavery till unit is variable and appears to undulate in the vicinity of the pilot PTW.

Our review of stratigraphic detail from boring logs along cross section D-D' (Figure 3.1) in the vicinity of the PTW suggests that the central and eastern ends of the pilot PTW may not penetrate the Lavery Till. Figure 3.2 illustrates the interpreted setting of the PTW in the local stratigraphic sequence. That portion of the PTW that lies above the top of the Lavery Till is considered to be a "hanging" PTW.

The SWS constitutes most of the shallow water-bearing zone in the vicinity of the PTW. The presence of the silty clay unit and the alternating beds of silt and clay within the SWS contribute to the development of semi-confined or confined groundwater flow conditions.

3.2 GROUNDWATER ELEVATION CONDITIONS

Figure 3.3 presents groundwater elevation contour maps representing conditions prior to the PTW construction, after construction but before the PTW drain and riser pipe were pumped (referred to as the "PTW development"), and after PTW development. Figure 3.4 shows hydrographs for select monitoring points in the PTW Area.

3.2.1 Pre-Construction Groundwater Elevations

Pre-construction groundwater elevations are available from well points WP-25, WP-26 and WP-27 starting in July 1999 (Table 2). Well points installed in the PTW area generally consist of 1-inch PVC casing perforated from 7 to 22 feet bgs or approximately 1363 to 1378 feet asl, wells PZ-08, -09 and -10 are made of 1.25-inch galvanized steel and have 3-foot long screens; this depth interval places most of the well screens completely within the SWS sediments (Figure 3.2) but penetrating both the silt and gravel (6 to 11 ft bgs) and lower alternating sand and gravel and silty clay sequence. Depth to groundwater prior to construction is approximately 6.5 to 6.7 feet bgs. Groundwater elevation data available for the month of July 1999 (Figure 3.3) indicate that WP-25, located to the west of the PTW has slightly higher groundwater elevations than WP-26 and WP-27, located to the south and east of the PTW, respectively. These data suggest a fairly flat gradient but exhibits an eastward groundwater flow component in the PTW area, which departs from the regional northward groundwater flow direction interpretation for the North Plateau (Figures 2.6 and 2.7). The absence of lithologic information from monitoring points limits our ability to interpret whether the measurements made from these points are representative of the same flow, or subflow, zone. It is likely that stratigraphic heterogeneities affect the local flow directions.

3.2.2 Post-Construction Groundwater Elevations

Post-construction groundwater elevations are available from a greater number of well points (WP-25 through WP-40 and PZ-01 through PZ-10) than during the pre-construction period. Well points WP-28 through WP-40 and PZ-01 through PZ-07, screened between 7 and 22 feet bgs, were installed by December 1999. Well points PZ-08 (screened between 15 and 18 feet bgs) and PZ-09 (screened between 15.5 and 18 feet bgs), were installed on August 24, 2000, and PZ-10, screened between 9 and 12 feet bgs, was installed on October 30, 2000.

The groundwater elevation contour map, hydrographs from July 1999 to February 2001, and hydrographs for February 2001 (Figures 3.3, 3.4 and 3.5) show the following observations:

- Groundwater elevation in WP-25 rose by approximately 2 feet above background (well #8603) after installing the sheet piles and remained elevated.

- Water levels measured from wells located in the PTW are consistently higher than measurements from well points screened in the native sediments with the exception of water levels measured in WP-25;

- The groundwater elevations measured from well points located inside the PTW are practically identical, indicating a near zero horizontal hydraulic gradient;

- Groundwater elevation contours indicate that groundwater is flowing towards the east northeast through the PTW and radiates away from the PTW on the south, east, and north faces indicating hydraulic mounding conditions;

- The horizontal hydraulic gradient between the PTW and the native material decreased following PTW development;

- Groundwater elevations fluctuations due to precipitation or snow melt are generally similar in all well points inside or outside the PTW; well points located inside the PTW have greater fluctuations;

- Groundwater elevations inside the PTW are higher than outside, except for elevations measured at well WP-25.

The current monitoring network does not allow the quantitative evaluation of vertical hydraulic gradients. The closest well pair is WP-25 and PZ-10 and groundwater elevations measured show an upward gradient.

The difference in groundwater elevations between well point WP-29 (inside the PTW) and other well points located outside the PTW at times: (1) before the PTW development (December 1999 to June 2000); (2) after the PTW development but before the surface water drain installation (September to Mid-October 2000), and (3) after surface water drain installation is shown below:

Groundwater Elevation Differences (feet)

	North			South		East	West
	WP-29–WP-30	WP-29–WP-34	WP-29–WP-40	WP-29–WP-28	WP-29–WP-36	WP-29–WP-27	WP-29–WP-25
Before Development	1.44	1.53	1.41	0.72	1.20	1.99	-0.57
After Development, Before Drain Installation	0.85	0.83	0.85	-0.03	0.41	1.18	-0.84
After Development and Drain Installation	0.83	0.81	0.83	-0.02	0.37	1.14	-0.81

The elevation difference calculations indicate that the elevation inside the PTW is consistently higher than the surroundings, except when compared to well point WP-25, which is between 0.2 to 1.2 feet higher than inside the PTW. The average groundwater elevation differences calculated after the installation of the surface drain indicates that the drain has slightly impacted groundwater elevations by further decreasing the elevation difference.

Figure 3.5 shows hydrographs and precipitation for February 2000 and 2001. Comparison of the two series of hydrographs show that the PTW development and the addition of the surface water drain has helped attenuate the horizontal hydraulic gradient across the PTW area. Ground water level increases within the wall in February 2001 are slightly larger than in well points outside the PTW but the increase is much smaller than during February 2000 precipitation events. Groundwater elevations in WP-29 did not exceed the elevation measured in WP-25, as observed in February 2000. In both February 2000 and February 2001, however, the magnitude of the rise in head was greater within the PTW than outside the PTW. This suggests that: (1) surface water infiltration still influences water levels within the PTW though less than before installation of the surface drain; (2) pressure response differences between the semi-confined aquifer system and the unconfined PTW system give rise to temporal mounding; and/or (3) both surface water drainage and pressure differentials influence the head response. These data suggest that both the PTW development and the surface water drain have been effective in reducing, but not completely eliminating surface infiltration into the PTW.

3.3 DISTRIBUTION OF SR-90

Assessment of the spatial and temporal variation in Sr-90 distribution in groundwater near and within the PTW provides insight toward understanding the influence of the constructed system on the ambient flow field. Defined metrics (WVDP-350, June 1999) for assessing the performance of the pilot PTW include: (1) the ability of the PTW's treatment matrix to remove Sr-90 from affected groundwater, and (2) the change in activity/concentration of Sr-90 downgradient of the pilot PTW. Therefore, the effect of the pilot PTW on the Sr-90 distribution is critical to assessing the performance of the pilot PTW.

3.3.1 Pre-Construction Distribution of Sr-90

The regional distribution of Sr-90 prior to construction of the pilot PTW was summarized in Section 2.4 and is indicated on Figure 2.1. The pilot PTW was installed in an area where the distribution of Sr-90 was represented by the leading edge of a lobe of Sr-90 with an activity of approximately 10,000 pCi/L as interpreted from a coarse network of Geoprobe and well installation information. A relatively steep activity gradient from greater than 10,000 pCi/L to less than 1,000 pCi/L was apparent from the central portion of this lobe (or the approximate central point of the then proposed PTW alignment) westward toward well #115 and the defined region of low hydraulic conductivity within the North Plateau.

Installation and sampling from new well points, WP-25 located on the west side of the proposed PTW alignment, WP-26 located on the south, and presumed hydraulic upgradient side of the proposed PTW, and WP-27, located immediately east of the proposed PTW, confirmed the location of the western edge of 2nd lobe of the Sr-90 regional plume. Analysis of groundwater samples from these well points also indicated that Sr-90 activities as high as 40,000 pCi/L existed immediately south, or presumed upgradient, from the proposed PTW alignment (WP-26) while the activity of Sr-90 was less than 500 pCi/L immediately adjacent to the western end of the PTW alignment (WP-25). Each of the well points have similar screen intervals (7 to 22 feet bgs) that cut across the alternating sequence of low and high hydraulic conductivity zones in the vicinity of the PTW. Inference from regional Geoprobe data as described previously, as well as the results of a scan of Beta activity in soil collected from the pre-emplacement PTW dewatering well borings suggest that the higher activity groundwater has a greater likelihood of existing in the lower half (i.e. depths greater than about 15 feet bgs) of this shallow water bearing system.

3.3.2 Post-Construction Distribution of Sr-90

Figures 3.6 and 3.7 illustrate the spatial and temporal variability in the distribution of Sr-90 following installation of the pilot PTW. Figure 3.6 depicts the current distribution (January/February 2001) of Sr-90 activity in groundwater interpreted from samples collected in the three initial well points (WP-25, WP-26, and WP-27) and the additional well points and piezometers installed following construction of the pilot PTW. Trends in Sr-90 activity over time since installation of the pilot PTW also are shown on the Figure. Figure 3.7 compares the historical trends in Sr-90 activity for samples collected from wells representing areas adjacent to each side of the pilot PTW: WP-25 on the west side, WP-26 on the south side, WP-27 on the east side, and WP-30 on the north side. Historical trends prior to February 2001 for Sr-90 activity in samples collected from PZ-01, PZ-02, PZ-03, and PZ-04 located south of the pilot PTW are not available as these points were sampled only in February 2001.

Certain similarities in the distribution of Sr-90 activity exist between the current condition and the interpreted condition prior to deployment of the PTW: Sr-90 activity west of the PTW remains low; the greatest Sr-90 activity occurs along the eastern tow-thirds of the south and north faces, and at the eastern end of the alignment. This confirms speculation that while the pilot PTW may have been located close to a leading downgradient front of the 2nd lobe (as designed), it also was placed at the western fringe of the lobe where the Sr-90 activity increases greatly over a short lateral distance and along the length of the pilot PTW. The general distribution tends to confirm a regional north-northeast migration pattern. However, other significant observations of the Sr-90 activity distribution and trends are important to assessing the effect of the PTW on the local migration pathway:

- Sr-90 activities in groundwater samples collected from within the zeolitic material of the pilot PTW are low to negligible indicating a combination of near complete treatment of affected groundwater (removal of Sr-90 due to ion exchange processes) and the influence of lower activity source water (area around WP-25);

- The high Sr-90 activity source water is located to the south-southwest of the PTW based on regional data and recent analysis of groundwater samples collected from PZ-01, PZ-03, and PZ-04 (the Sr-90 activity in the sample collected from PZ-02 was lower). A recent increasing trend in the activity of groundwater samples collected from WP-26 could indicate flux of a higher activity upgradient source moving toward the PTW. The lower Sr-90 activity in samples from WP-28 and WP-36, located within approximately 5 feet of the south side of the PTW, could indicate the flux of low activity [possibly treated] water outward from the PTW, or for the case of WP-28, the effect of lower activity groundwater from the west. The high Sr-90 activity in groundwater samples from PZ-09 located within the upgradient gravel section may represent direct influx to the PTW. The activity level in samples from PZ-09 also suggests that zeolitic fines capable of removing Sr-90 from the groundwater have not mixed significantly into this portion of the gravel section although the short screen interval of PZ-09 may not be indicative of the full vertical thickness of the gravel section.

- Activity in WP-27 (immediately east of the PTW) increased moderately (50 percent) from its baseline of about 10,000 pCi/L after the sheet piles were installed and prior to their removal, and increased more than 400 percent above its baseline activity immediately after removal of the sheets. The activity has remained consistent at or about 40,000 pCi/L since the sheets were removed in November 1999. The trends indicate that the installation and removal of the sheets had profound affects on the flow system. While the sheets were in place, flow of slightly higher than baseline (10,000 pCi/L) activity water was diverted to the WP-27 location from the southwest; following sheet removal, one explanation for the immediate 300 percent increase in activity might have been the hydrodynamic pressure effects on the flow system associated with sheet removal activities; however, the sustained high activity at WP-27 suggests an altered flow pattern, compared to pre-sheet emplacement, that is connected to a high activity source area. The exact physical cause of this condition is not fully explained and remains under consideration.

- Trends in activity levels in groundwater samples collected from wells located north of the PTW show variability depending on proximity to the pilot PTW:

  1. Sr-90 activity in groundwater samples from WP-30 is consistently low (<5,000 pCi/L) relative to other wells. This may be due to both to its location within a path of lower activity groundwater from the west and by the flux of low-activity (possibly treated) water outward from the PTW;

  2. Sr-90 activity in groundwater samples collected from WP-34 located within approximately five feet of the north face of the PTW generally has increased at an average rate of approximately 70 pCi/day during the past 12 months to greater than 30,000 pCi/L. This increasing trend follows an approximately three-month period beginning with removal of the sheet piles where the activity Sr-90 in groundwater samples decreased from approximately 15,000 pCi/L to about 6,000 pCi/L. The cause of the initial decreasing Sr-90 activity trend followed by the increasing activity trend is unknown. With respect to the initial decreasing trend, there may be a relation between an initial flushing of dilute low activity water through this area following sheet pile removal and local mitigation of Sr-90 water by ion exchange with zeolite fines that could have entered the formation near WP-35. The increasing trend, which appears to be consistent with a general site-wide increase in Sr-90 activity in groundwater, may be related to the possible underflow of high Sr-90 activity in the central and eastern portions of the pilot PTW. The approximately one foot head difference between wells WP-26 located immediately south of the pilot PTW and well WP-34 located immediately north of the pilot PTW suggests the presence of a northward lateral hydraulic gradient that may be indicative of continued groundwater flow beneath the pilot PTW. The start of the increasing trend in Sr-90 activity at WP-34 seems consistent with the start of an increasing trend in Sr-90 activity at WP-26 suggesting that both wells may be along a similar flow path even though they are on opposite sides of the pilot PTW.

  3. Sr-90 activities in WP-35 located approximately 20 feet north of the PTW has shown a similar rate of activity increase as WP-34 indicating migration of the Sr-90 lobe to the north-northeast.

  4. Sr-90 activities in groundwater samples collected from WP-40, after first increasing from approximately 10,000 pCi/L to approximately 22,000 pCi/L has decreased by about two times since November 2000. The decrease may indicate influence from a front of slowly migrating low activity (possibly treated) groundwater emanating from the northeastern end of the PTW toward the well location..

3.4 ASSESSMENT OF PTW DEVELOPMENT AND HYDRAULIC TESTING

Selected data collected during development and hydraulic testing of the PTW were used for qualitative analysis of hydraulic conditions around the PTW. PTW development was conducted evaluation by WVNS in July, August, September 2000, and January 2001. Hydraulic testing was performed by WVNS and consisted of "mini-pump tests" at WP-25 in January and April 2000, slug tests in several well points performed in April and October 2000 (Table 1). Figures 3.8 through 3.10 illustrate the results of selected test analyses. The main objective of our review is to discern whether the observed hydraulic pressure response due to the testing indicates whether a lower hydraulic conductivity skin exists along the interface of the PTW with the native aquifer.

3.4.1 Qualitative Pumping Test Analysis

Figure 3.8 shows the drawdown response in selected well points to pumping the PTW riser pipe during development activities in January 2001. The data is provided for the following well points: WP-29 located inside the PTW, WP-34 located to the north, PZ-06 located to the northwest, WP-26 located to the south, WP-27 located to the east, and WP-25 located to the west of the PTW. As reported by WVNS, the purpose of the development was to attempt to mobilize and remove any low permeability skin that may have developed due to PTW emplacement activities, and to qualitatively evaluate hydraulic response in the PTW area during longer duration pumping. As noted by WVNS during the initial development event in July 2000, several feet of fine sediment resembling zeolitic material was observed within the PTW riser pipe.

Qualitative observation indicates that there is good hydraulic connection between the PTW riser pipe and well WP-29, as indicated by the drawdown response of approximately 6 feet in WP-29. The next best response is encountered at well WP-25, followed closely by PZ-06 and WP-26 (drawdown of approximately 3 feet). The drawdown response from wells WP-34 (north of PTW) and WP-27 (east of the PTW) is approximately 2.5 feet. The drawdown responses indicate better hydraulic connection between the PTW and the western end of the PTW as seen at WP-25 and PZ-06, which are located farthest from PTW riser pipe, than between the eastern end as seen at WP-27, which is closest to the PTW riser pipe. These results suggest that zones of lower hydraulic conductivity exist between the PTW and the east, south, and north edges of the PTW.

Figure 3.9 shows the drawdown response to pumping from well point WP-25 in January 2000. What is most noticeable from the drawdown responses is the delayed response in WP-29 located inside the PTW. This observation strongly indicates the importance of storage (drainage) at WP-29 suggesting that unconfined, or less confining groundwater flow conditions exist within the PTW. Qualitative observations indicate similar behavior as noted above with the lowest drawdown response observed in WP-27. The decrease in degree of confinement between the native sediments and the PTW may partially account for the higher groundwater elevation distribution observed in the PTW during periods of transient head fluctuation. It is possible that the response to recharge in the PTW may result in an apparent mounding due to hysteresis of the drainage and imbibition process of the unconfined system, or due to a transient contrast in pressure response between the confined native sediments and the unconfined PTW. The magnitude of influence from these effects requires further evaluation using a numerical model.

3.4.2 Review of Slug Test Analyses

Figure 3.10 shows the hydraulic conductivity values obtained from slug testing. Generally, slug test results are not considered reliable for the quantitative assessment of hydraulic conductivity as the results may be highly dependent on well construction (i.e. sand pack. skin, and development) but they still can provide qualitative information useful for assessing approximate conditions. Reported results from the slug tests range from $1x10^{-5}$ cm/s in well point PZ-05 located to the east of the PTW to $2x10^{-2}$ cm/s in well point PZ-08 located within the "roundstone" section of the PTW. All slug test results obtained within the clinoptilolite portion of the PTW are in the $1x10^{-3}$ cm/s range. Hydraulic conductivity measured in PZ-09, the second well point screened in the gravel roundstone portion of the PTW, is $1.5x10^{-4}$ cm/s, which is much lower than in PZ-08. It should be noted that the construction of well points PZ-8, PZ-9, and PZ-10 is less conducive to slug testing than the other PTW well points and piezometers as the screens are much shorter (2.5 to 3 feet).

The slug test results appear consistent with the results of permeameter tests on the zeolite material performed at UB (Rabideau, et al, 1999). The reported results from the permeameter testing indicated a hydraulic conductivity for the uncompacted loose clinoptilolite to be $1.2x10^{-1}$ cm/s. When compacted, the hydraulic conductivity reduced to $4x10^{-3}$ cm/s, which is within the general range of the slug test results. However, these results are based on laboratory conditions and although strongly compacted, do not account for the likely sorting, and high degree of pulverizing and creation of fines that may have occurred during the PTW emplacement.

Review of the hydraulic responses and hydraulic conductivity values obtained from various types of hydraulic tests indicates that the PTW area is highly heterogeneous. The results of pumping test analysis suggest that the hydraulic conductivity of the PTW may be lower than first anticipated. A systematic program of step, constant-rate, and recovery testing is recommended to obtain representative hydraulic information that can be used for engineering design/modification activities.

3.5 Conceptual Model of the Pilot PTW Hydraulic Performance

A conceptual model with respect to hydraulic performance of the pilot PTW is presented in this section. The conceptual model provides insight into possible scenarios and influences that have lead to current conditions. The conceptual model is not without uncertainty, and is to be used as a tool by which to test certain hypotheses. The goal of the conceptual model is to provide direction by which to develop engineering solutions to either restore the intended hydraulic performance of the system, or modify the design to promote the desired treatment of affected groundwater in the site vicinity.

As part of the assessment of hydraulic conditions and development of this conceptual model, we have coordinated with Dr. Alan Rabideau and staff at UB who have been commissioned by WVNS to develop a model of hydraulic conditions in the vicinity of the PTW. The model being developed by UB is based on an analytical approach that approximates two-dimensional groundwater flow conditions (Rabideau, 2000, 2001). The analytical model does not account for vertical flow, or differences in hydraulic pressure regimes that, from a review of model results and site hydrogeologic conditions, may be important for developing the detail for engineering design purposes. The following description of the conceptual model, however, considers preliminary results from the UB modeling but does not detail those results. Separate reports have been and are being prepared by UB to document the analytical model development.

The conceptual model of the flow system in the vicinity of the pilot PTW consists of the following components, which are generally described in Figure 3.2:

1. A shallow, generally unconfined to semi-confined groundwater system that consists of a heterogeneous sequence of silty clay, silt and gravel, and alternating layers of fine to coarse sand with gravel, overlying a low permeability till. The contact between the till and the overlying water-bearing sediments undulates across the project area.

2. Hydraulic conductivity of the aquifer system in a bulk sense is approximately 10^{-4} to 10^{-3} cm/s though individual layers can range up or down by an additional one to two orders of magnitude. The bulk hydraulic conductivity of the underlying till is believed to be approximately 3 to 4 orders of magnitude lower.

3. The regional direction of the lateral hydraulic gradient generally is north to northeast with a magnitude of 0.03 to 0.05 ft/ft; however, the direction of the ambient (i.e., pre-construction) lateral hydraulic gradient appears to be relatively flat locally with a shift toward a more easterly direction in the vicinity of the PTW, possibly in response to the low permeability clay with silt sequence located immediately west of the PTW. The west end of the PTW may abut a portion of this low permeability zone. The interpretation of the more easterly gradient direction is controlled, in part, by measured water levels at wells WP-25 and WP-27. A review of limited head data suggests that an east-northeasterly direction existed prior to construction of the pilot PTW.

4. The activity of Sr-90 is greatest in the lower portions of the SWS, and exists as a narrow, but concentrated zone that migrates northeasterly through the location of the PTW; the activity generally increases from west to east in the local vicinity.

5. The pilot PTW straddles the western edge or fringe of the 2nd lobe of Sr-90. The pilot PTW may not fully penetrate the SWBZ down to the underlying Lavery Till in its central and eastern portion. This "hanging" PTW may allow underflow of high Sr-90 activity groundwater as suggested by increasing concentrations of Sr-90 along portions of the north side of the PTW.

6. The high stress of sheet pile installation likely modified hydrostratigraphic pathways near the sheet pile alignment; major flow condition changes are observed at wells WP-25 and WP-27, both located within approximately 5 feet of the pilot PTW. A discontinuous skin of fine zeolitic material likely exists at the contact between the zeolite and the native aquifer on the east and north sides of the PTW. The skin may have developed due to the creation of fines during emplacement and sheet pile movement, and may have migrated partially through the more permeable zeolite material to the interface with the native aquifer during sheet pile removal and the relatively quick "flooding" of the PTW by groundwater. The skin effect along the south side likely is a result of fines migrating into the gravel as a result of PTW development activities and smearing of fine aquifer material across more permeable zones adjacent to the sheet pile alignment. The interior of the PTW likely is more heterogeneous than designed due to the creation of fines and sorting of the zeolite from installation activities.

7. The hydraulic conductivity inside the PTW appears to be highly variable and is likely controlled by the distribution of fine zeolitic materials in the periphery of the PTW (i.e., "skin") and within the PTW.

8. Hydraulic testing and development activities suggest that the entire hydraulic system both within and outside the pilot PTW is connected. However, hydraulic connection does not necessarily indicate Sr-90 migration pathways. Local heterogeneity is more responsible for the Sr-90 migration pathway; some degree of anisotropy between the direction of transport and the direction of the hydraulic gradient likely exists.

9. The flow system change from a semi-confined native aquifer to an unconfined permeable treatment zone likely contributes to hydraulic mounding within the pilot PTW due to a contrasting transient response to recharge events between the native aquifer and the PTW. The magnitude of this effect could further be assured using a numerical model.

10. Surface water infiltration into the PTW, although reduced by construction of an upgradient drainage system, continues to contribute to head fluctuating within the PTW.

To summarize, the overall hydraulic performance of the pilot PTW likely is controlled by the following conditions:

- a predominantly more eastward groundwater flow direction than initially anticipated (the PTW was oriented for predominantly northward flow and did not include lateral hydraulic controls to direct flow into the PTW);

- a highly heterogeneous aquifer sequence of fine and coarse sediments;

- a relatively narrow flow zone of high activity Sr-90 water that exists low in the aquifer and that is low at the west end of the PTW and high at the east end; this flow path is partially diverted around the east end of the PTW;

- a hanging central and eastern portion of the pilot PTW which allows some underflow of high Sr-90 activity groundwater;

- a discontinuous skin of fine zeolitic material at the contact with the native aquifer material and heterogeneity with the PTW treatment material resulting from installation activities;

- slow discharge of dilute, low Sr-90 activity water from portions of the PTW; this water appears to dilute higher Sr-90 activity in some wells located close to the PTW; and,

- continuing, although reduced, direct surface water infiltration into the PTW.

Though not a specific physical condition, the scale of the test also influences the outcome and contributes to the observed performance of this pilot test. The pilot PTW system is a relatively small-scale test and cannot absorb the influences of complexity associated with a high degree of heterogeneity (in aquifer material and the direction of the hydraulic gradient) that characterizes the local system. Additional aspects of the effect of scale on development of the engineered solution are addressed in the complimentary engineering assessment report.

4.0 RECOMMENDATIONS

Although current conditions at the pilot PTW indicate that the system is not performing from a hydraulic perspective as intended, this pilot test successfully identified specific technical issues that can be addressed and designed for prior to deploying an effective full-scale system at this and/or other locations at the North Plateau. Prior to designing such a full-scale system, the identified technical issues that likely limit the hydraulic performance of the pilot PTW should be further evaluated so that proper remedies to these issues can be appropriately engineered.

Specifically, we recommend a follow-up phase to this assessment that integrates the gathering of additional specific field data with the development of a three-dimensional numerical groundwater model. The development of an engineering solution to either modify the existing PTW, alter its orientation, or design a new PTW under the unique site specific conditions in the same or different portions of the North Plateau, relies on the ability to appropriately represent current conditions. Although we have developed hypotheses as to how the current hydraulic conditions developed, there remains significant uncertainty for which performing a supplemental and cost-efficient field investigation would be beneficial to decrease the degree of uncertainty and increase the potential that an effective engineering solution can be developed and implemented.

We therefore recommend the following (in order of importance):

1. Install two new wells and collect stratigraphic information in the vicinity of WP-25 (west end) and WP-27 (east end) to confirm the water level conditions that appear to provide major control on the assumed groundwater flow directions in the vicinity of the PTW. We recommend that these wells be installed by conventional drilling methods and not direct-push methods.

2. Drill at least three additional soil borings each adjacent to the north, south, and eastern face of the PTW that are logged for stratigraphic detail to confirm whether the pilot PTW penetrates the underlying till or is "hanging" within the SWBZ. One additional soil boring for stratigraphy should be drilled toward the western end of the PTW, and one additional boring should be drilled near the south face of the PTW. These borings should be converted to 2-inch monitoring wells to facilitate focused aquifer testing (see Recommendation No. 4).

3. Perform a long-term (72 hours to one week) aquifer testing program consisting of a series of step-test periods, a constant-discharge period, and recovery period. The test could be performed using the vertical riser drainpipe installed within the upgradient gravel section of the pilot PTW with observation wells monitored using

downhole pressure transducers. Short-term (i.e., 4 to 8 hour) step-tests also should be performed in the new wells recommended in No. 2 above.

4. Perform one or more tracer tests to provide field evidence of the flow field that predominates in vicinity of the pilot PTW.

Other field measures that could provide additional insight into the performance of the PTW (including coring and determination of Sr-90 activity in zeolite cores at various locations in the PTW; and assessing the ion exchange ratio between Sr-90 and calcium concentrations in water samples) are available but are not considered to be as of high value for the task of developing an engineering solution to the PTW performance.

The data collected from the supplemental investigation should be integrated into the development of a three-dimensional numerical flow model to better assess and predict the observed hydraulic conditions. The modeling would be performed by UB with consultation from Geomatrix and WVNS.

We believe that the collection and assessment of the supplemental data, along with development of the three-dimensional model will greatly improve the ability to develop an engineering solution that achieves the level of operational success required by the project stakeholders.

We believe that the pilot PTW test is successfully meeting its objectives to provide information critical to designing and deploying a full-scale PTW at WVDP.

Additional assessment and an initial survey of possible engineering solutions will be provided in the complimentary engineering assessment report and the PTW modifications report.

5.0 REFERENCES

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GEOMATRIX

TABLE 1

CHRONOLOGY OF ACTIVITIES RELATED TO THE PTW PILOT TEST

West Valley Nuclear Services
West Valley, New York

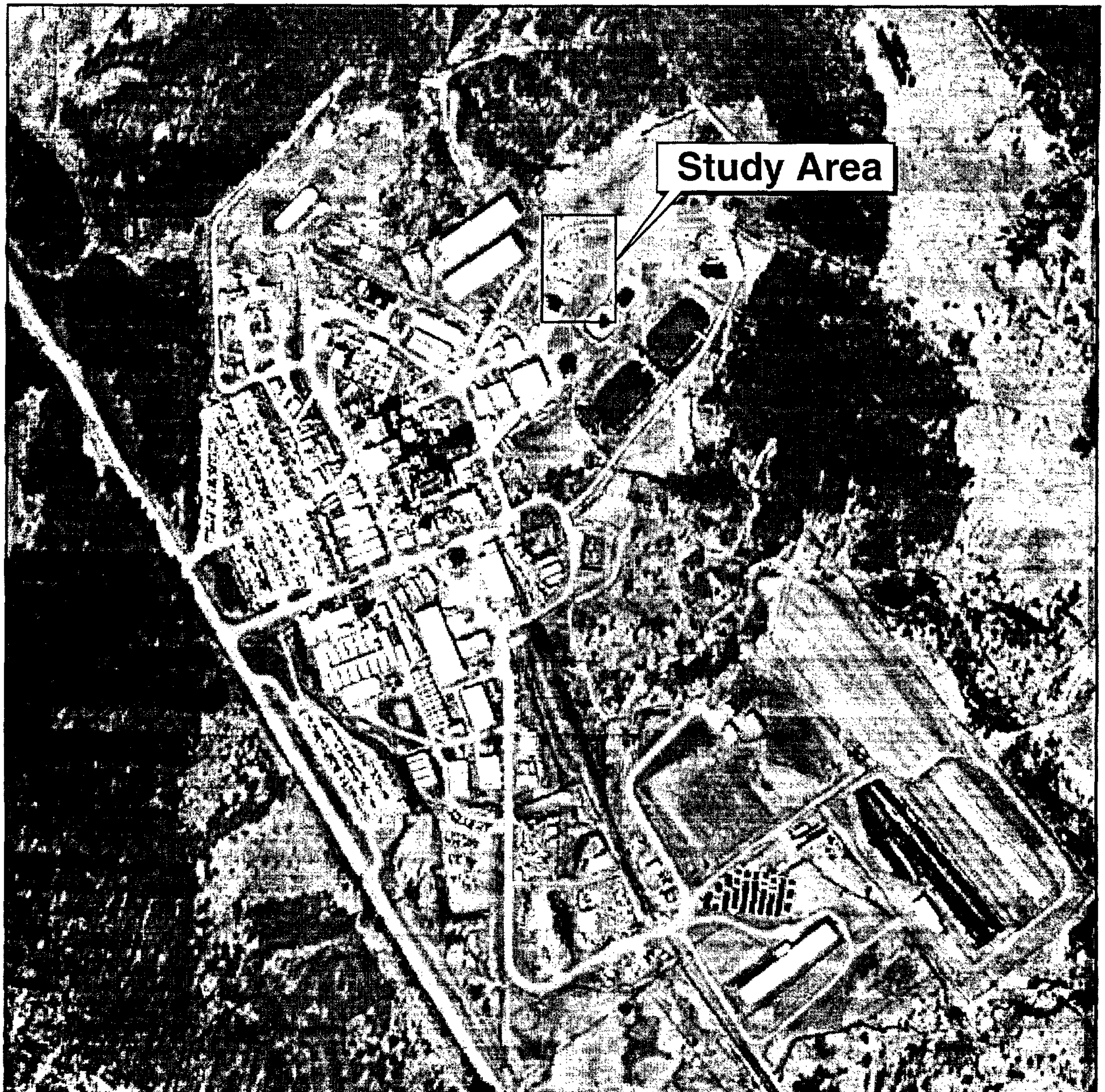
Year	Activity	Date
1999	• Installation of WP-25, -26, -27	July
	• PTW sheet piles installed	August
	• PTW sheet piles pulled	November 8 - 12
	• Installation of WP-28 through WP-40 and PZ-1 through PZ-7	Completed December 13
2000	• First mini pumping test from WP-25	January 31
	• Second mini pumping test from WP-25	April 19
	• Slug testing of selected PTW WPs and PZs	April
	• Peer review of PTW Preliminary Op. Assess. Report	May 5
	• Preliminary Operational Assessment Report	May 19
	• PTW development	July, August, and September 8-9
	• Installation of gravel zone PZs (PZ-08 and PZ-09)	August 24
	• URS (@ WVDP) review and evaluation of PTW conditions	September 15 - 18
	• Slug testing of PTW WPs and PZs not previously tested	September 26 - October 10 & 17
	• Head changes in PZ-01, -04, -05, -06	September 25 - October 2
	• Moylan peer review/evaluation and report	October 9 - 11 and 31
	• URS simplified groundwater modeling of PTW area	October 17 - 27; December 6 - 11
	• Completion of surface water drain around PTW	mid-October
	• Installation of PZ-10	October 30
	• WVNS management briefing on Pilot PTW path forward	November 29
2001	• Temporary pumping of PTW riser	
	– 2-hour test and recovery	January 19
	– 1-week pumping and recovery	January 24 - 31
	– 2-week pumping and recovery	February 5 - 19
	• SUNY Buffalo groundwater modeling	
	– Start of modeling effort	January 5
	– Preliminary report	February 21
	– Final report	March 9
	• Geomatrix PTW evaluation	
	– Kickoff meeting	February 6 - 7
	– PTW chapter reports (draft and final)	March and April
	– Final report	April 25
	• WVNS Report submitted to DOE	June

GEOMATRIX

TABLE 2

MONITORING POINT CONSTRUCTION
West Valley Nuclear Services
West Valley, New York

Monitoring Point ID	Monitoring Point Diameter	Top of Screen (feet bgs)	Bottom of Screen
PZ-01	1¼-inch	7	22
PZ-02	1¼-inch	7	22
PZ-03	1¼-inch	7	22
PZ-04	1¼-inch	7	22
PZ-05	1¼-inch	7	22
PZ-06	1¼-inch	7	22
PZ-07	1¼-inch	7	22
PZ-08	1¼-inch	15.5	18
PZ-09	1¼-inch	15.5	18
PZ-10	1¼-inch	9	12
WP-25	1-inch	7	22
WP-26	1-inch	7	22
WP-27	1-inch	7	22
WP-28	1-inch	7	22
WP-29	1-inch	7	22
WP-30	1-inch	7	22
WP-31	1-inch	7	22
WP-32	1-inch	7	22
WP-33	1-inch	7	22
WP-34	1-inch	7	22
WP-35	1-inch	7	22
WP-36	1-inch	7	22
WP-37	1-inch	7	22
WP-38	1-inch	7	22
WP-39	1-inch	7	22
WP-40	1-inch	7	22



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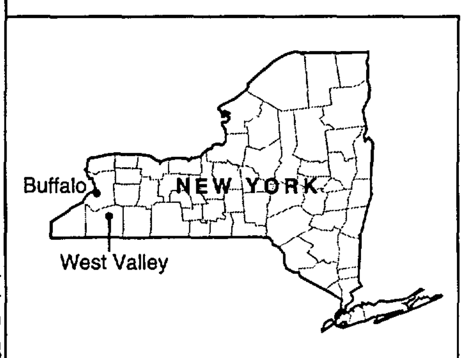

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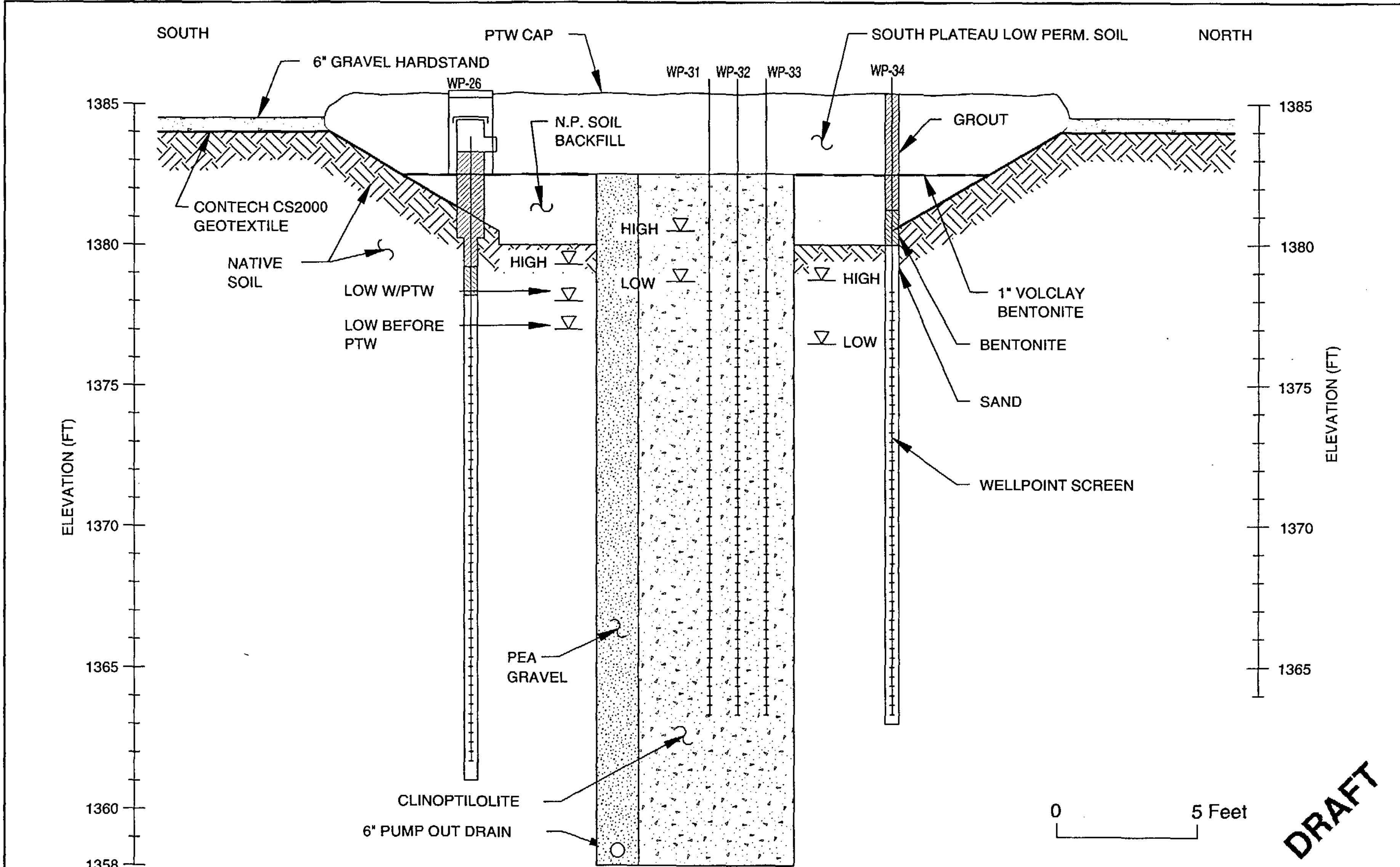
GEOMATRIX

SITE LOCATION MAP
West Valley Nuclear Services
West Valley, New York

Project No.
7061

Figure
1.1

S:\7000\7061\hr0301_fig_01-02.ai

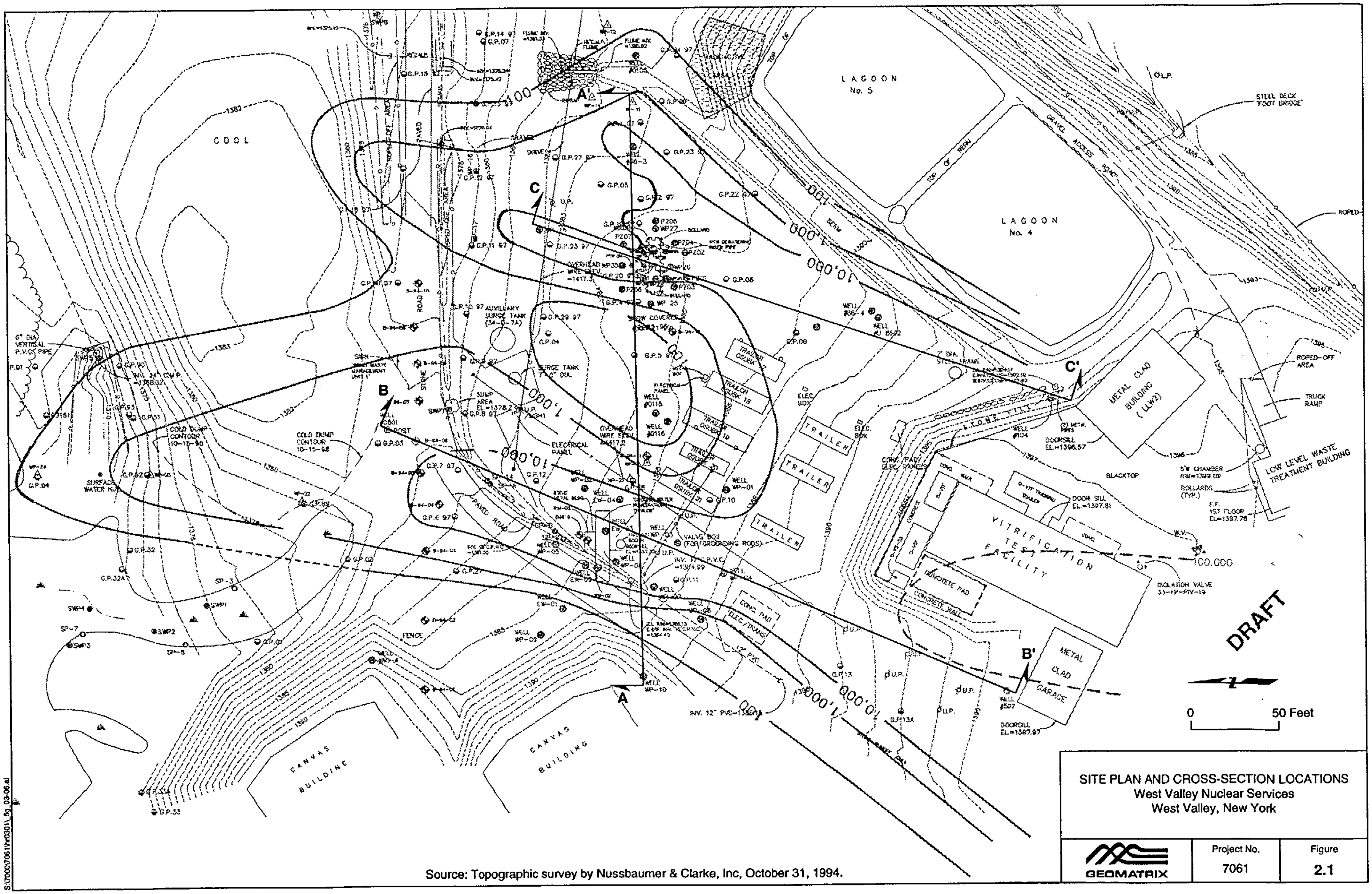


GEOMATRIX

PERMEABLE TREATMENT WALL CROSS-SECTION
West Valley Nuclear Services
West Valley, New York

Project No.
7061

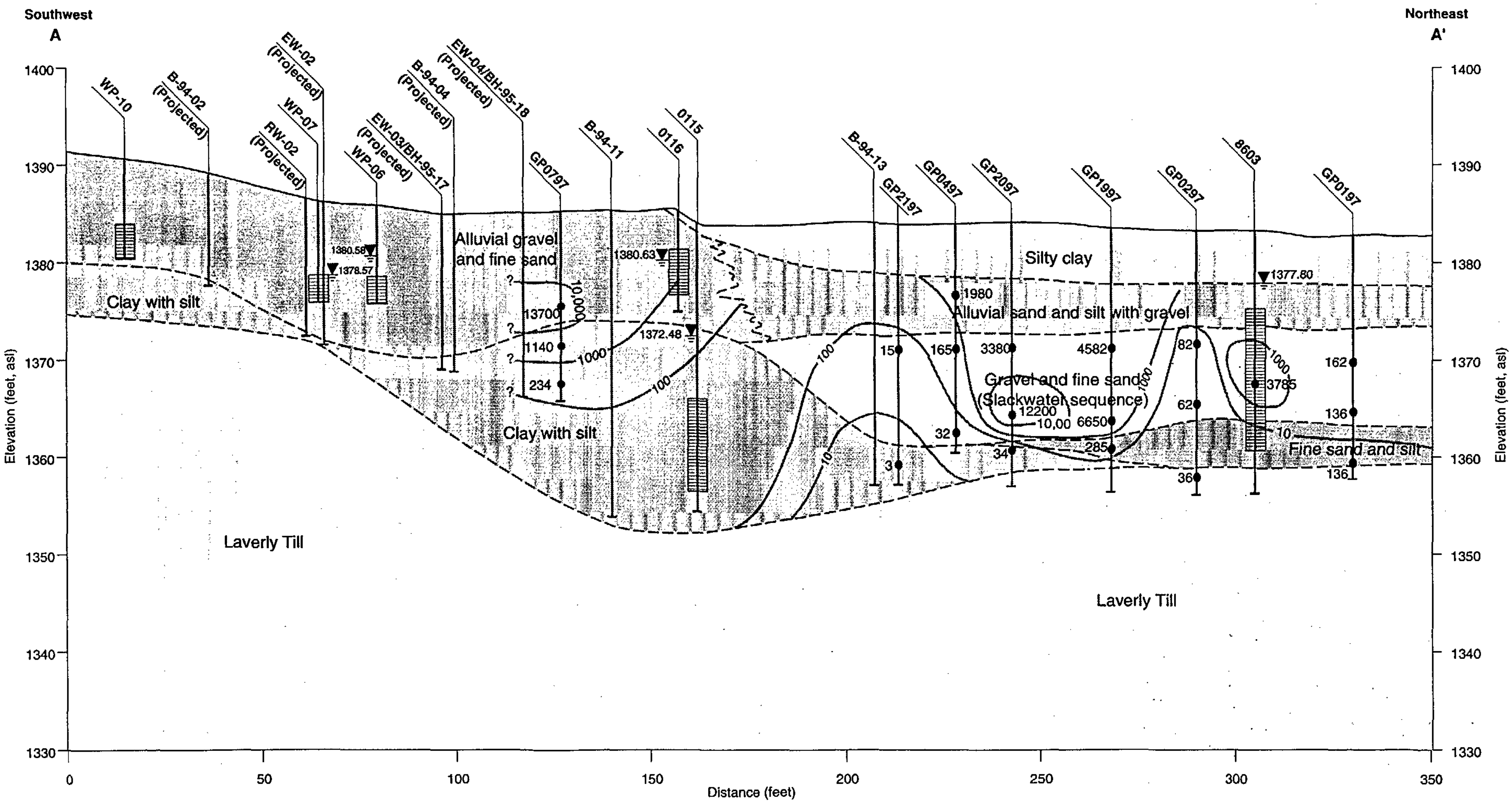
Figure
1.2



Source: Topographic survey by Nussbaumer & Clarke, Inc, October 31, 1994.

SITE PLAN AND CROSS-SECTION LOCATIONS
West Valley Nuclear Services
West Valley, New York

GEOMATRIX	Project No.	Figure
	7061	2.1



EXPLANATION

Boring

162 ← SR-90 groundwater activies, pCi/l

130

—100— Activity contour (pCi/l)

Source: Data provided by West Valley Nuclear Services

Notes:

1) Water levels taken 5/20 to 5/27 1999.

2) Source: Data provided by West Valley Nuclear Services

0 25 Feet

0

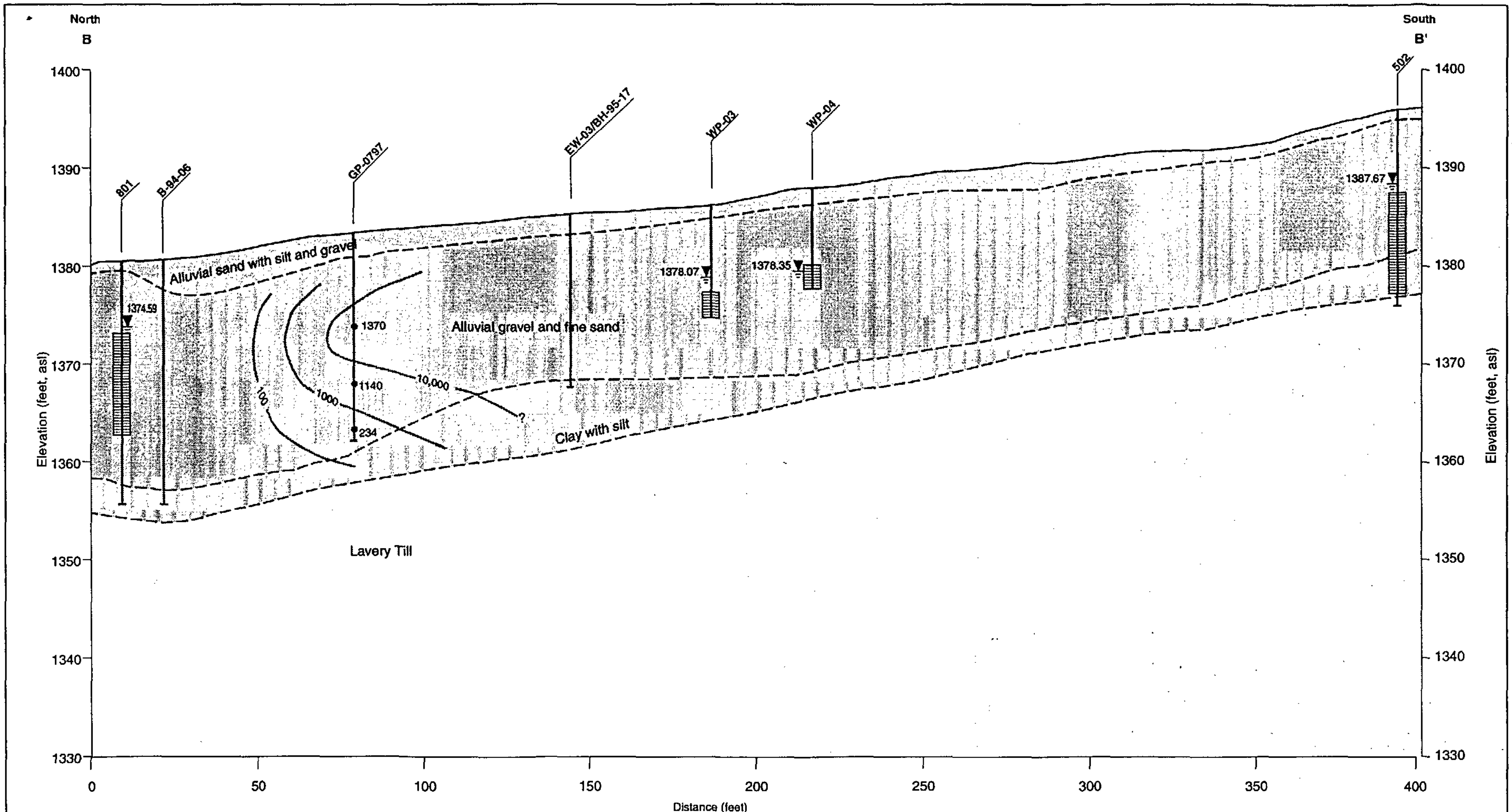
10 Feet

Vertical Exaggeration 2.5X

CROSS SECTION A-A'
West Valley Nuclear Services Inc.
West Valley, New York

GEOMATRIX

Project No.	Figure
7061	2.2



EXPLANATION

- Boring
- 162 — SR-90 groundwater activies, pCi/l
- 130
- —100— Activity contour (pCi/l)

Source: Data provided by West Valley Nuclear Services

Notes:

1) Water levels taken 5/20 to 5/27 1999.
2) Source: Data provided by West Valley Nuclear Services

0 — 25 Feet

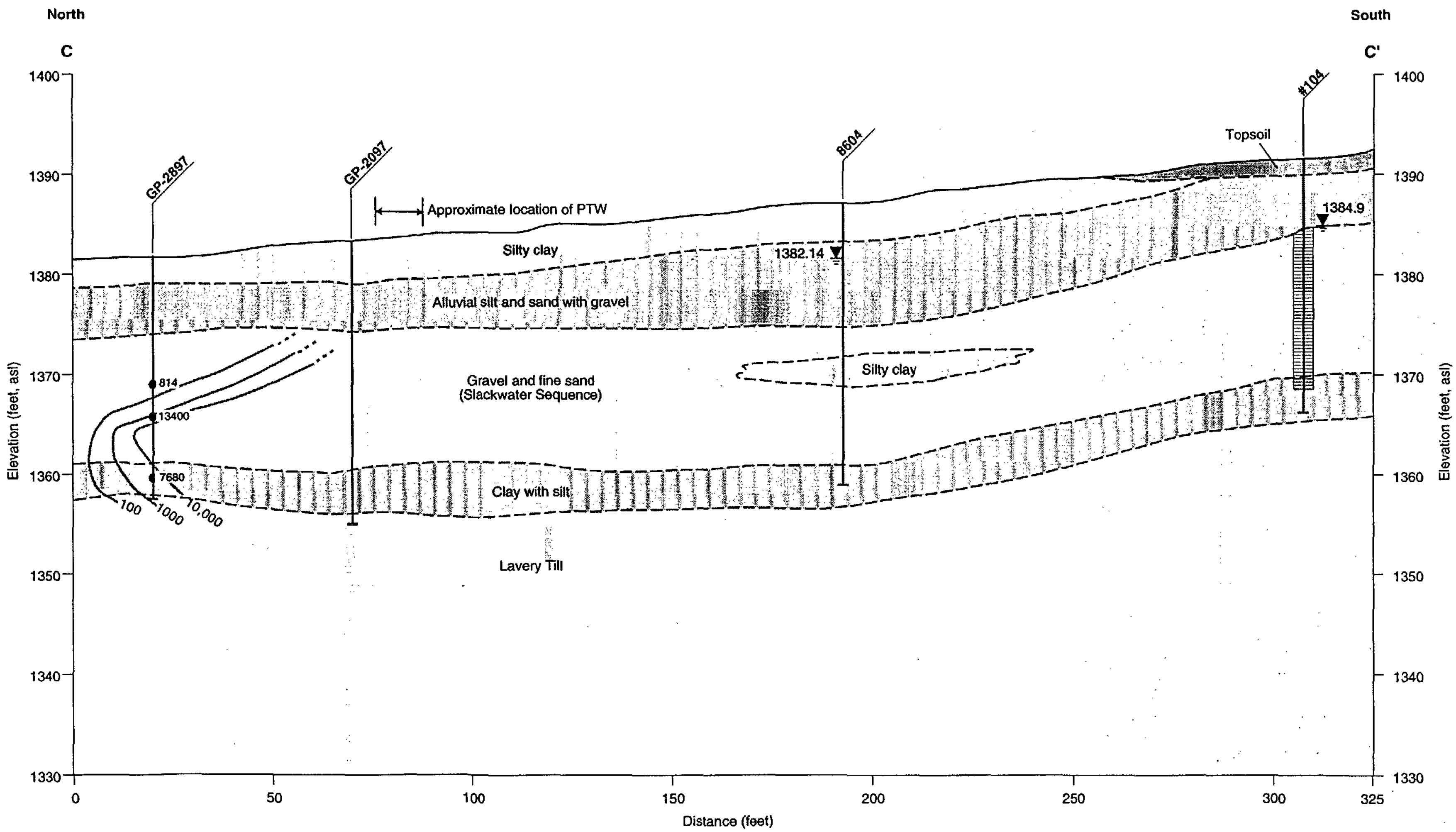
0 — 10 Feet

Vertical Exaggeration 2.5X

CROSS SECTION B-B'
West Valley Nuclear Services Inc.
West Valley, New York

GEOMATRIX

Project No.	Figure
7061	2.3



EXPLANATION

Boring

162 — SR-90 groundwater activies, pCi/l

130

—100— Activity contour (pCi/l)

Source: Data provided by West Valley Nuclear Services

Notes:

1) Water levels taken 5/20 to 5/27 1999.

2) Source: Data provided by West Valley Nuclear Services

0 — 25 Feet

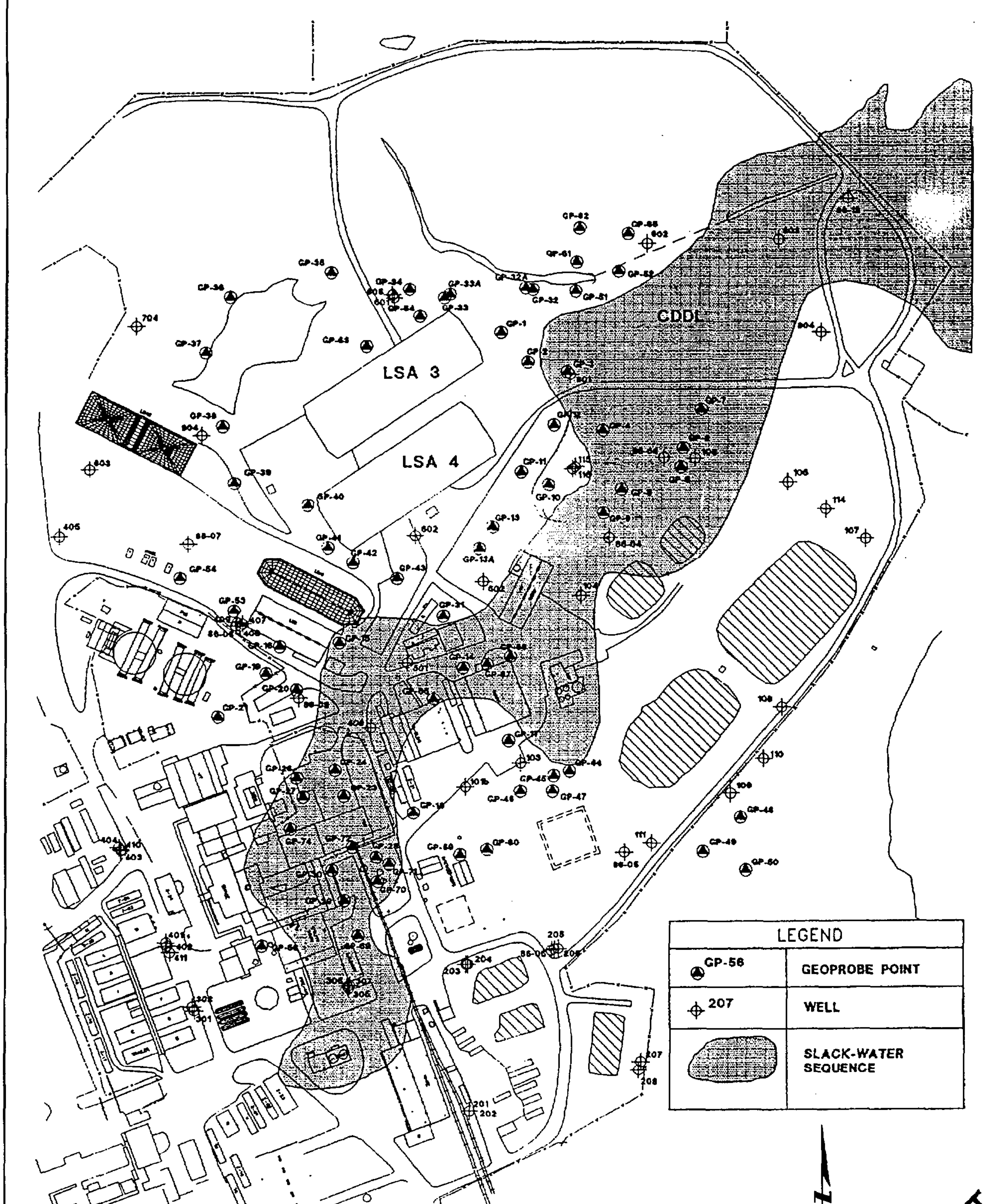
0 — 10 Feet

Vertical Exaggeration 2.5X

CROSS SECTION C-C'
West Valley Nuclear Services Inc.
West Valley, New York

GEOMATRIX

Project No.	Figure
7061	2.4



Dames & Moore 1994 Geoprobe Data

DRAFT

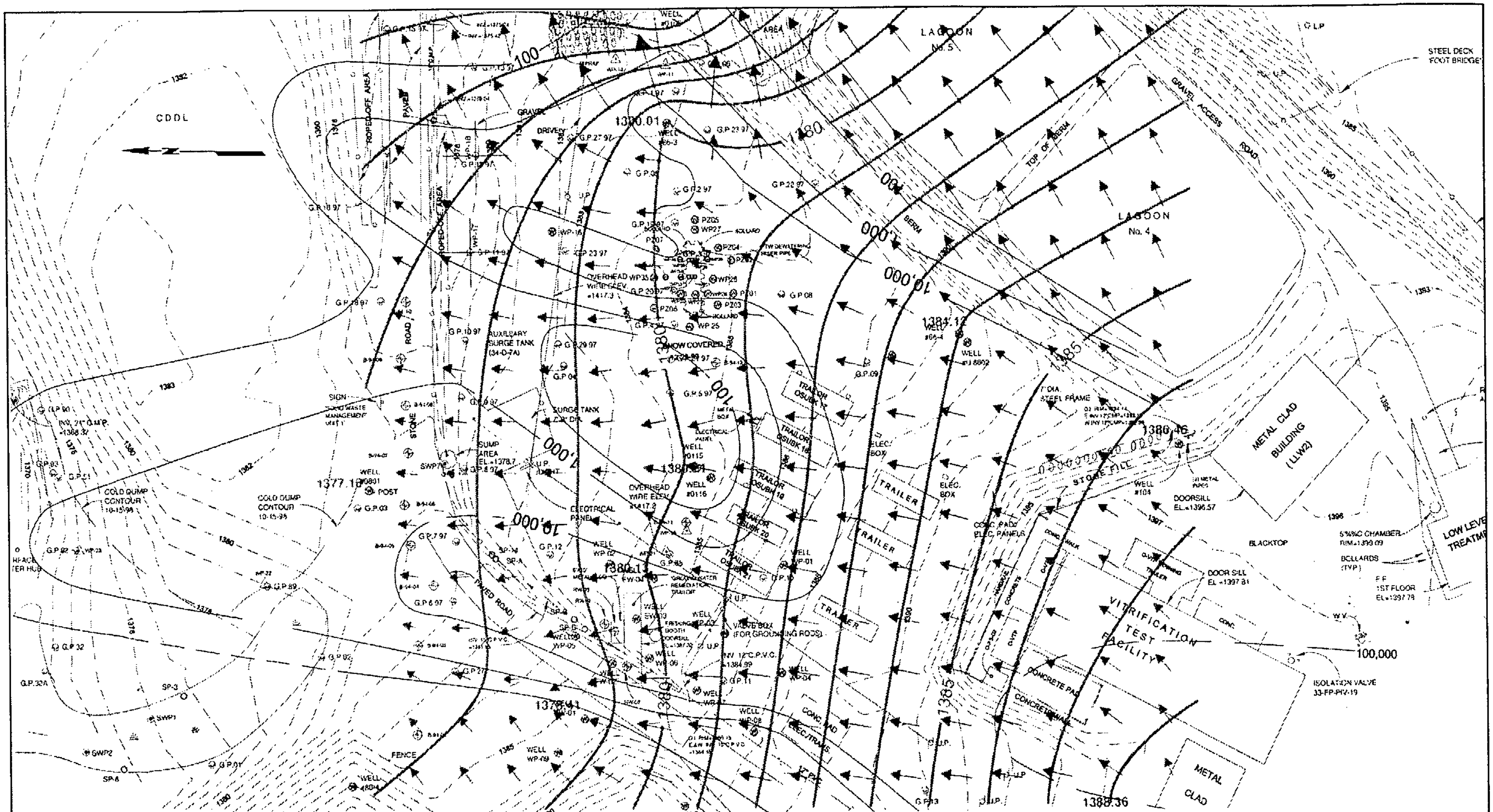
S:\7000\7061\hr0301_fig_02_05(03).ai

GEOMATRIX

AREAL EXTENT OF THE SLACK-WATER SEQUENCE
West Valley Nuclear Services
West Valley, New York

Project No.
7061

Figure
2.5



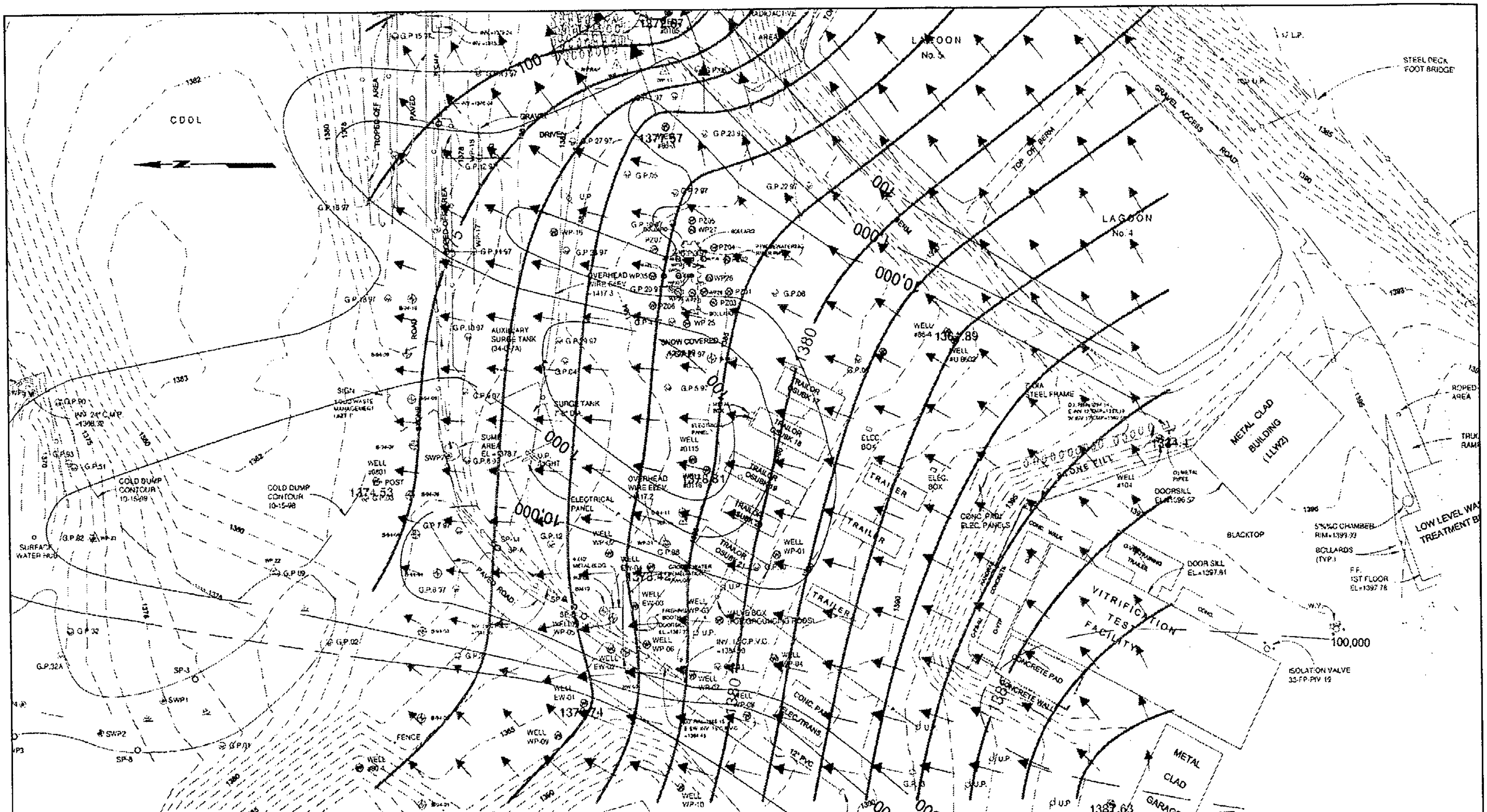
REGIONAL GROUNDWATER ELEVATIONS -
HIGH WATER LEVEL
West Valley Nuclear Services
West Valley, New York

GEOMATRIX	Project No.	Figure
	7061	2.6

SOURCE:
Topographic Survey for the North Plateau Barrier Wall, Nussbaumer & Clarke, Inc., October 31, 1994.

Note: Water levels were measured on February 26 and 27, 1997; data provided by West Valley Nuclear Services.

S:\7000\7061\0403011_fig_02-06(05).ai



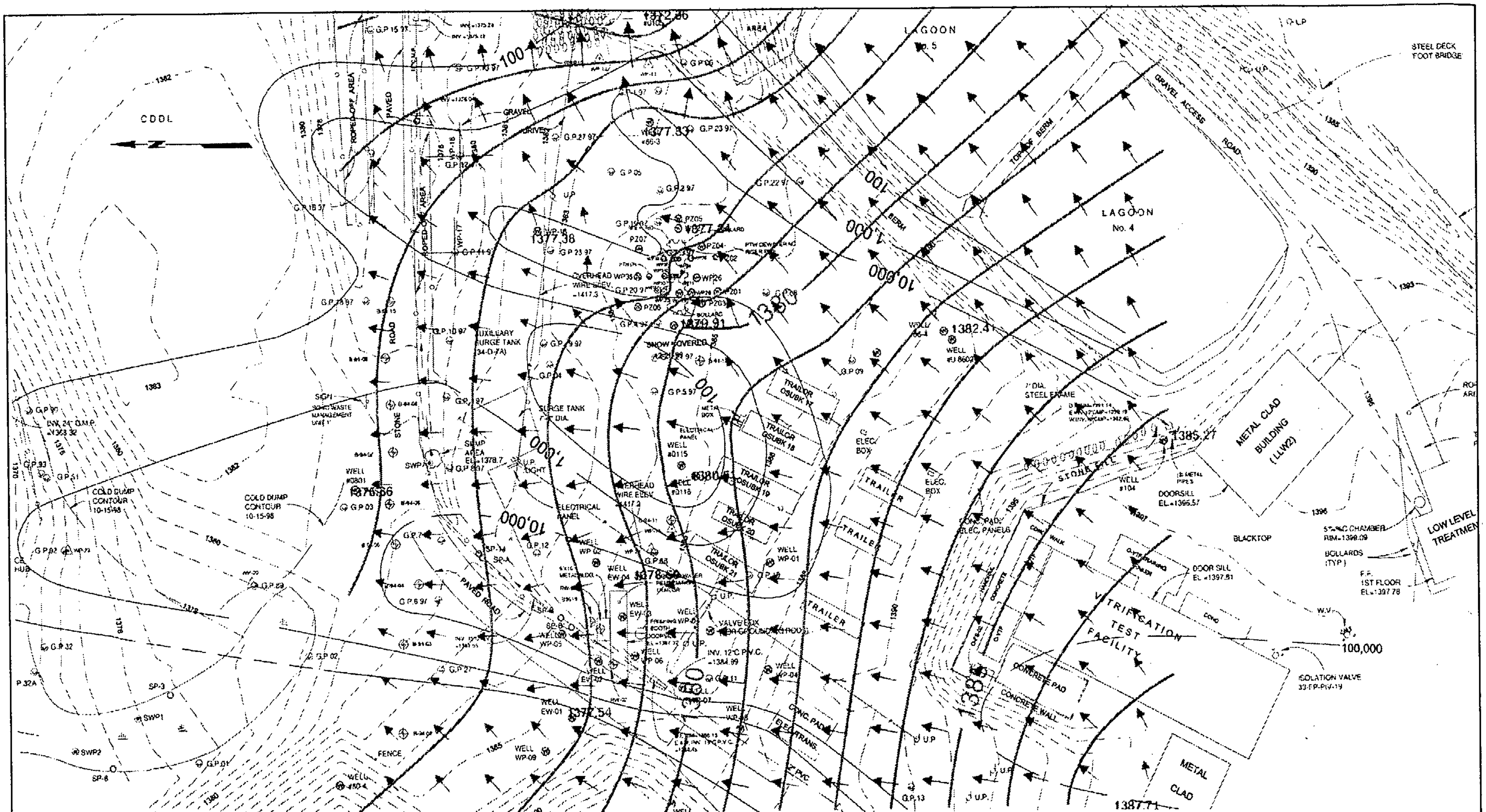
REGIONAL GROUNDWATER ELEVATIONS - LOW WATER LEVEL
West Valley Nuclear Services
West Valley, New York

GEOMATRIX	Project No.	Figure
	7061	2.7

SOURCE:
Topographic Survey for the North Plateau Barrier Wall,
Nussbaumer & Clarke, Inc., October 31, 1994.

Note: Water levels were measured on August 27, 1998;
data provided by West Valley Nuclear Services.

S:\7000\7061\Wr0301_fig_02-07(04).ai



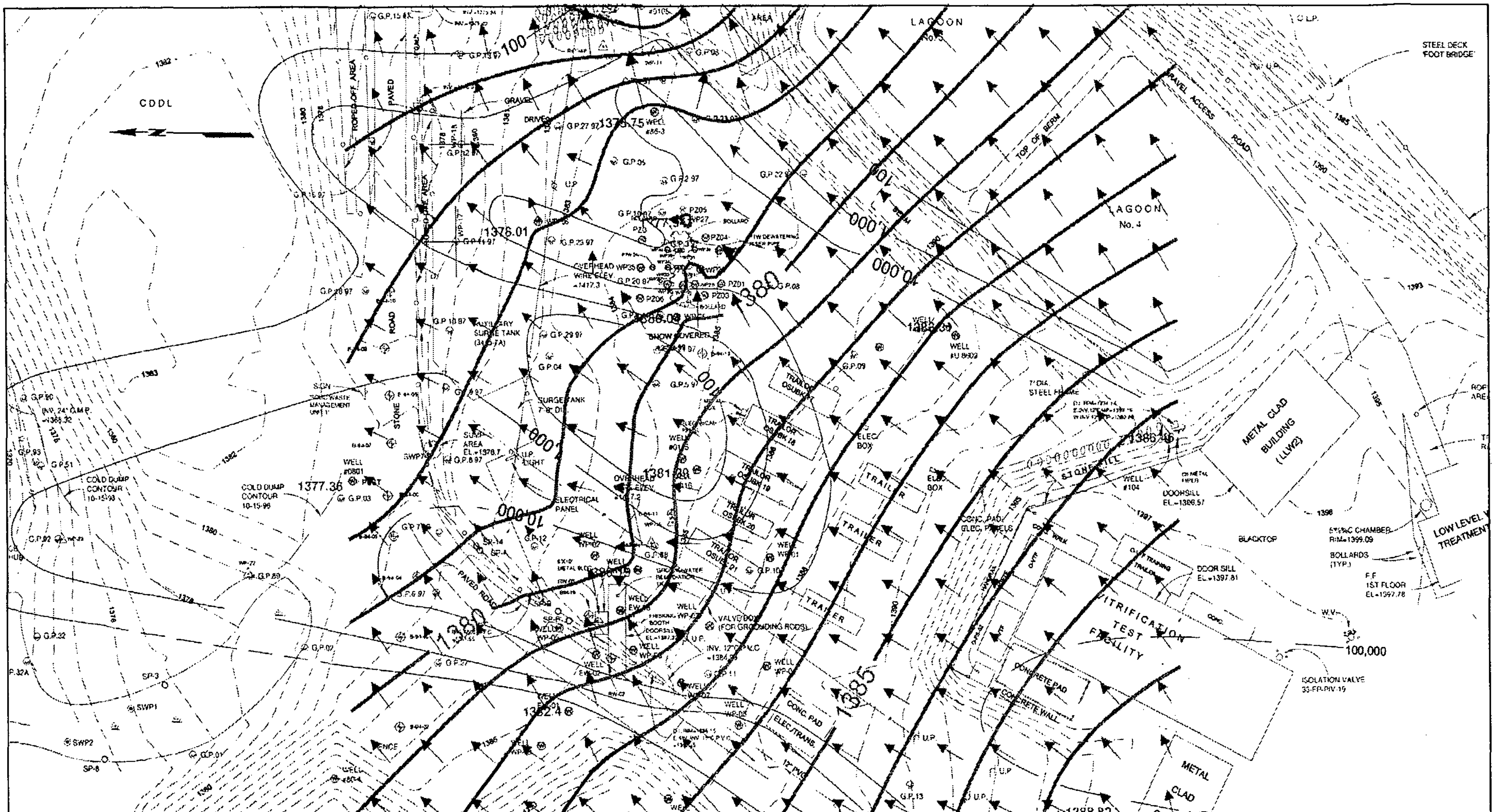
SOURCE:
Topographic Survey for the North Plateau Barrier Wall,
Nussbaumer & Clarke, Inc., October 31, 1994.

Note: Water levels were measured on May 30, 2000;
data provided by West Valley Nuclear Services.

REGIONAL GROUNDWATER ELEVATIONS -
POST-CONSTRUCTION, PRE-DEVELOPMENT
West Valley Nuclear Services
West Valley, New York

GEOMATRIX

Project No.	Figure
7061	2.8



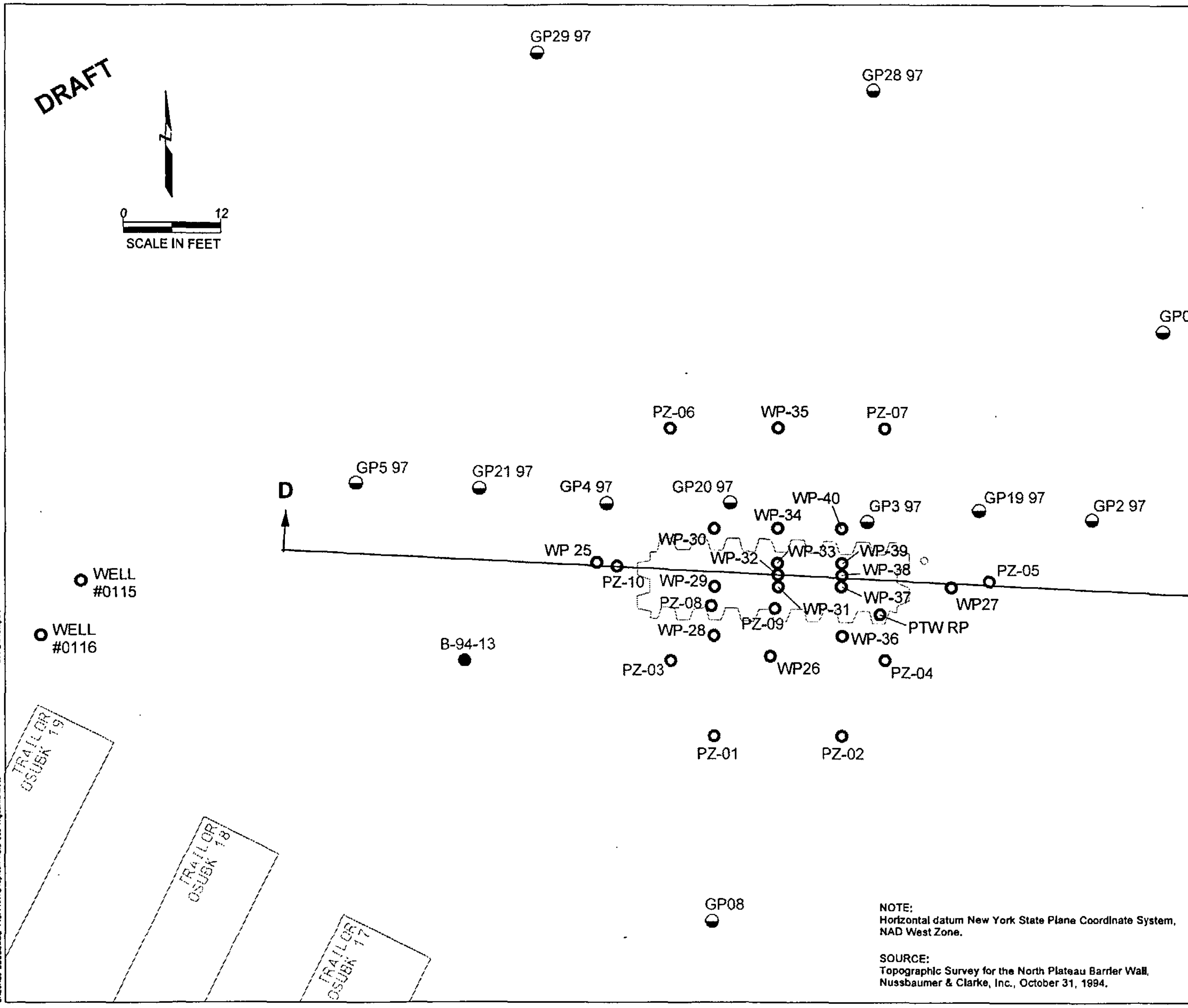
SOURCE:
Topographic Survey for the North Plateau Barrier Wall, Nussbaumer & Clarke, Inc., October 31, 1994.

Note: Water levels were measured on November 30, 2000; data provided by West Valley Nuclear Services.

REGIONAL GROUNDWATER ELEVATIONS - POST CONSTRUCTION, POST-DEVELOPMENT
West Valley Nuclear Services
West Valley, New York

GEOMATRIX

Project No.	Figure
7061	2.9



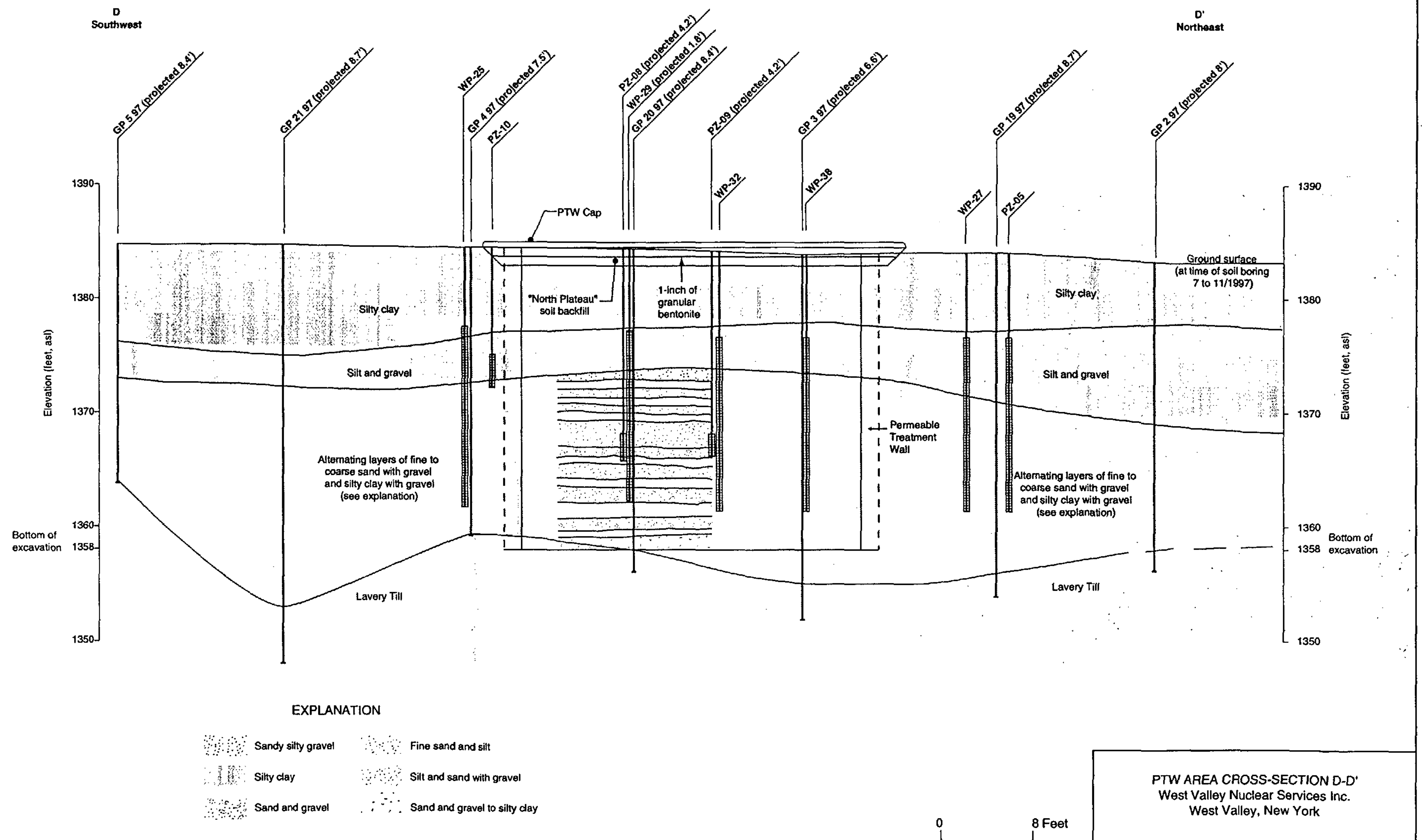
NOTE:
Horizontal datum New York State Plane Coordinate System, NAD West Zone.

SOURCE:
Topographic Survey for the North Plateau Barrier Wall, Nussbaumer & Clarke, Inc., October 31, 1994.

PTW AREA AND CROSS-SECTION D-D' LOCATION
West Valley Nuclear Services
West Valley, New York

GEOMATRIX

Project No.	Figure
7061	3.1



Source: Data provided by West Valley Nuclear Services

PTW AREA CROSS-SECTION D-D'
West Valley Nuclear Services Inc.
West Valley, New York

GEOMATRIX	Project No.	Figure
	7061	3.2

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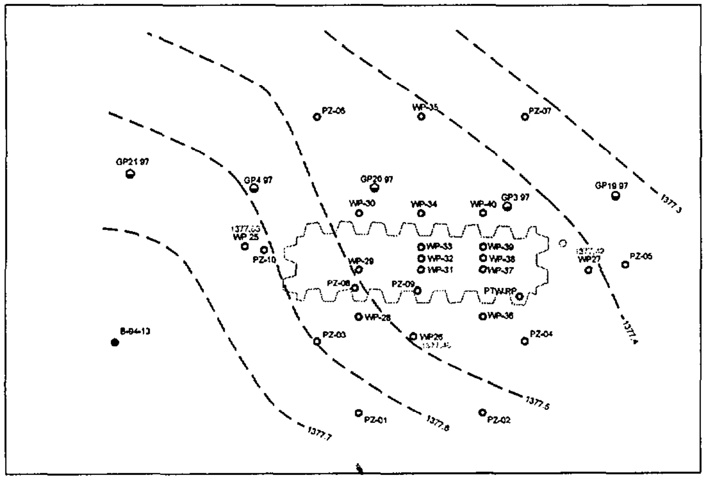
09-MAR-2001 11:43 lshkolnikova
s:\7000\7061\hr030_fig_3.3.dgn
CHECKED:________ \\SFPRNT2\Splash Hold Job-Nogdolmlrx.ctb MAP_WT3X.pen

PRE-PTW CONSTRUCTION
JULY 1, 1999



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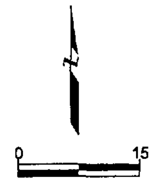
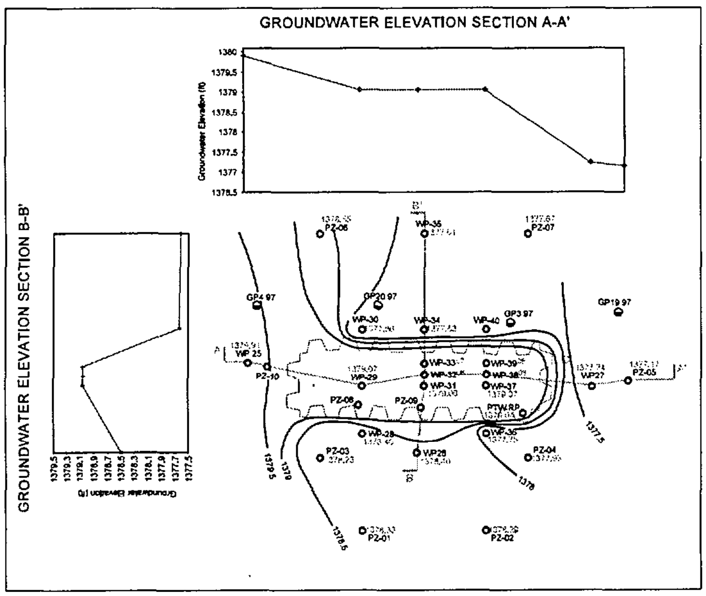


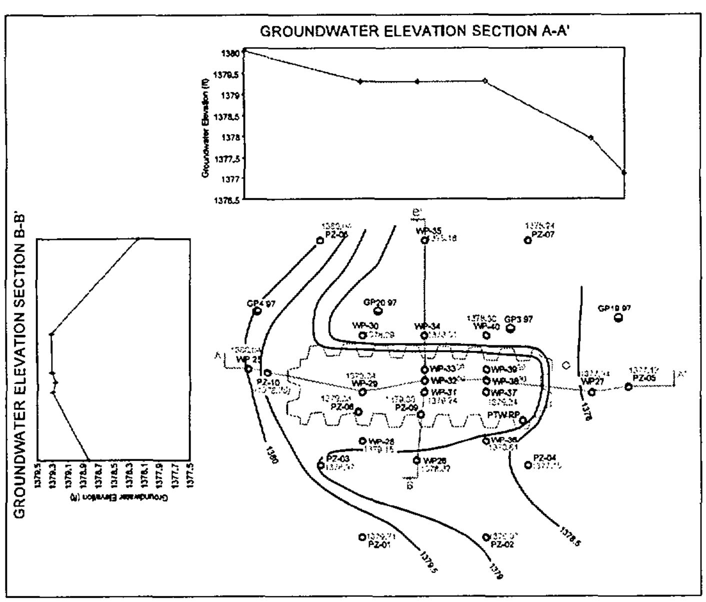
POST-PTW CONSTRUCTION, PRE-DEVELOPMENT
30 MAY 2000



EXPLANATION

- GEOPROBE SAMPLING LOCATION
- WELL POINT, OR PIEZOMETER LOCATION
- BORING WELL LOCATION
- GROUNDWATER ELEVATION NOT USED IN CONTOURING; PZ-10 IS SCREENED FROM 9 TO 12 FEET BGS
- 1378 —— LINE OF EQUAL GROUNDWATER ELEVATION (CONTOUR INTERVAL = 0.5 FOOT)
- 1377.1 – – ASSUMED LINE OF EQUAL GROUNDWATER ELEVATION (CONTOUR INTERVAL = 0.1 FOOT)

POST-PTW CONSTRUCTION, POST DEVELOPMENT
27 NOVEMBER 2000



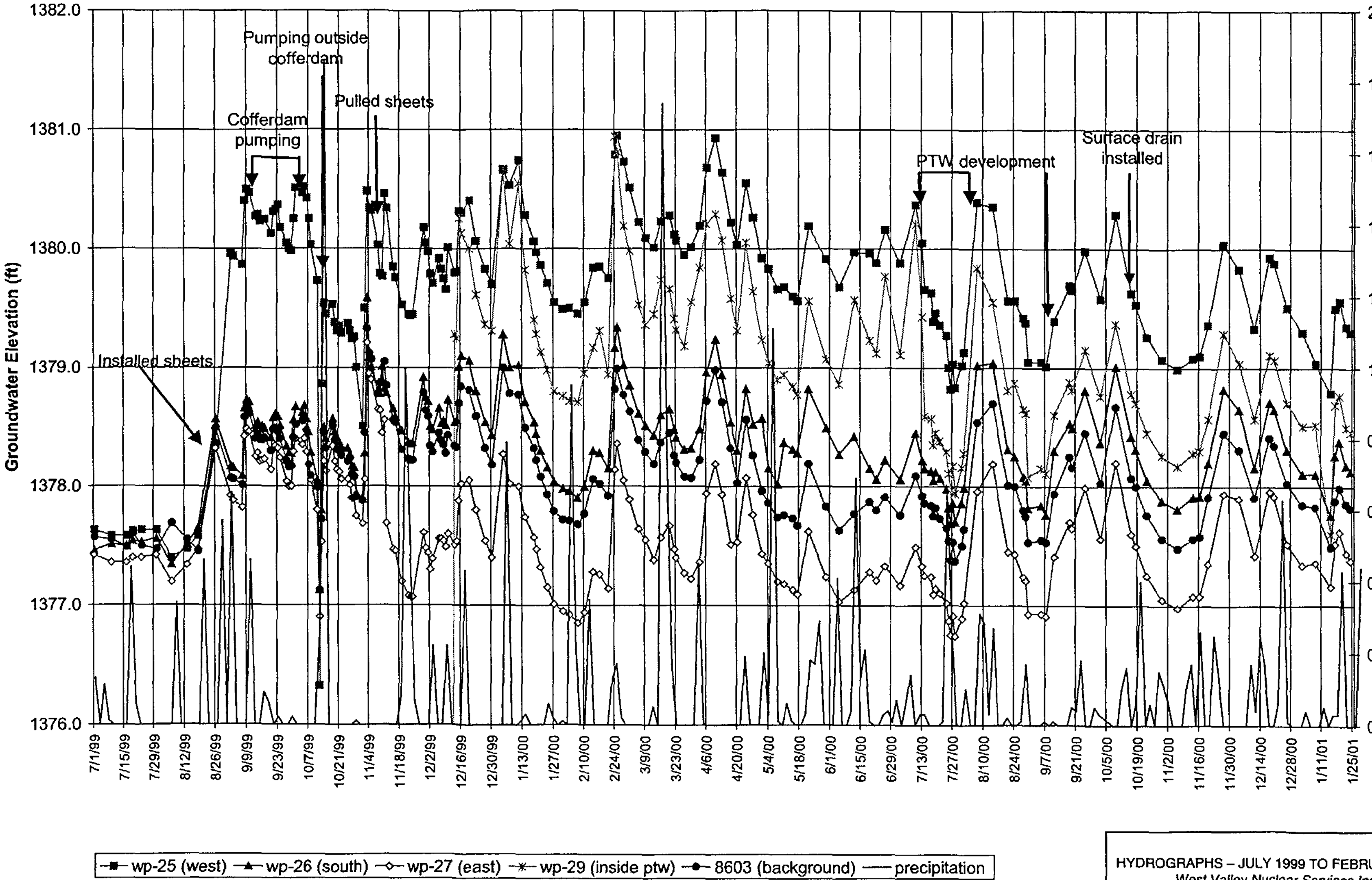
NOTE:

1. *Contour of equal elevation of potentiometric surface* are shown as solid lines for clarity and are not imply certainty.
2. Data provided by West Valley Nuclear Services.

SOURCE:
Topographic Survey for the North Plateau Barrier Wall, Nussbaumer & Clarke, Inc., October 31, 1994. Horizontal datum New York State Plane Coordinate System, NAD West Zone.

GROUNDWATER ELEVATIONS PRE- AND POST-CONSTURCTION
West Valley Nuclear Services
West Valley, New York

GEOMATRIX	Project No. 7061	Figure 3.3



DRAFT

HYDROGRAPHS – JULY 1999 TO FEBRUARY 2001
West Valley Nuclear Services Inc.
West Valley, New York

GEOMATRIX

Project No.	Figure
7061	3.4

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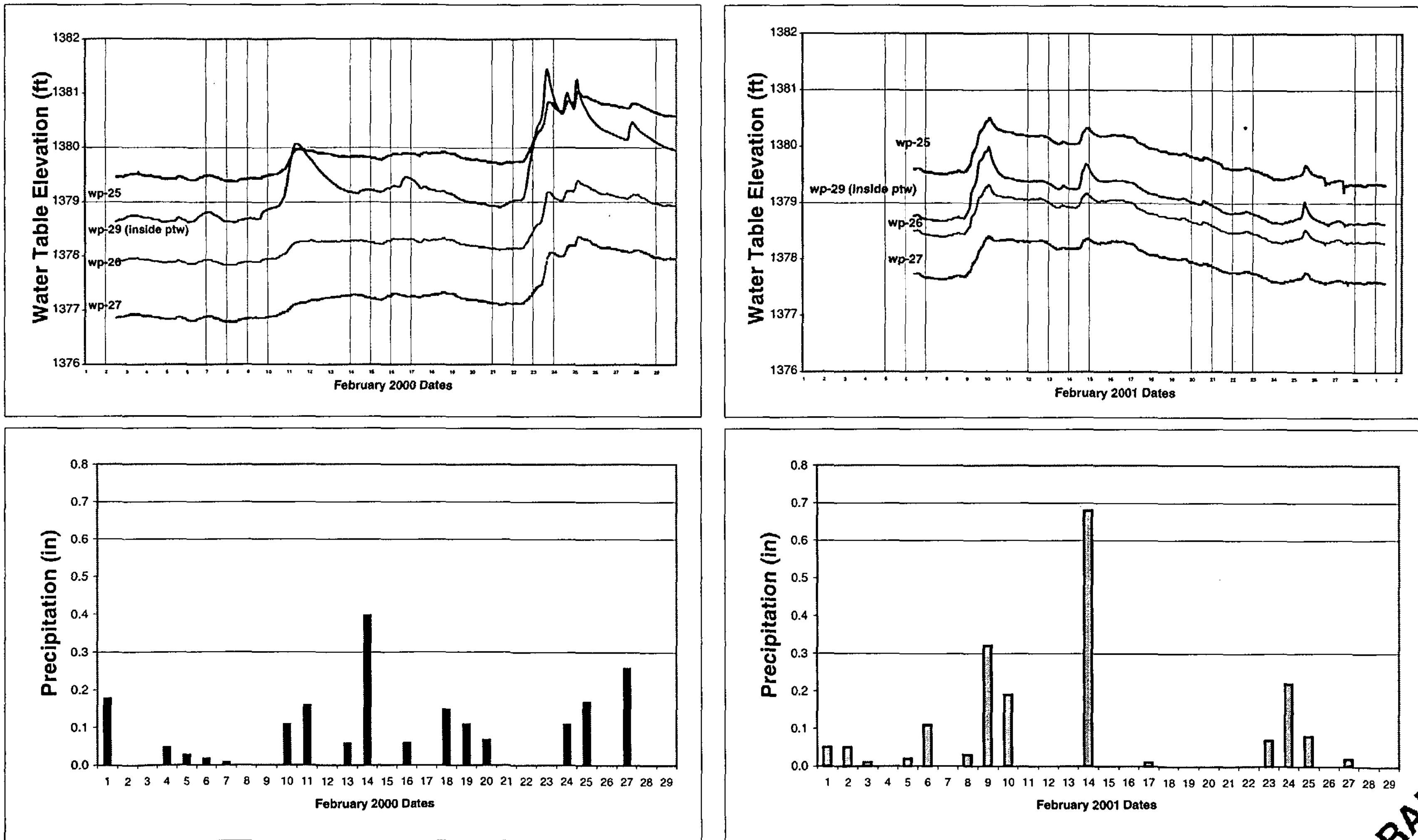
S:\7000\7061\hr0301_fig_03-05.ai



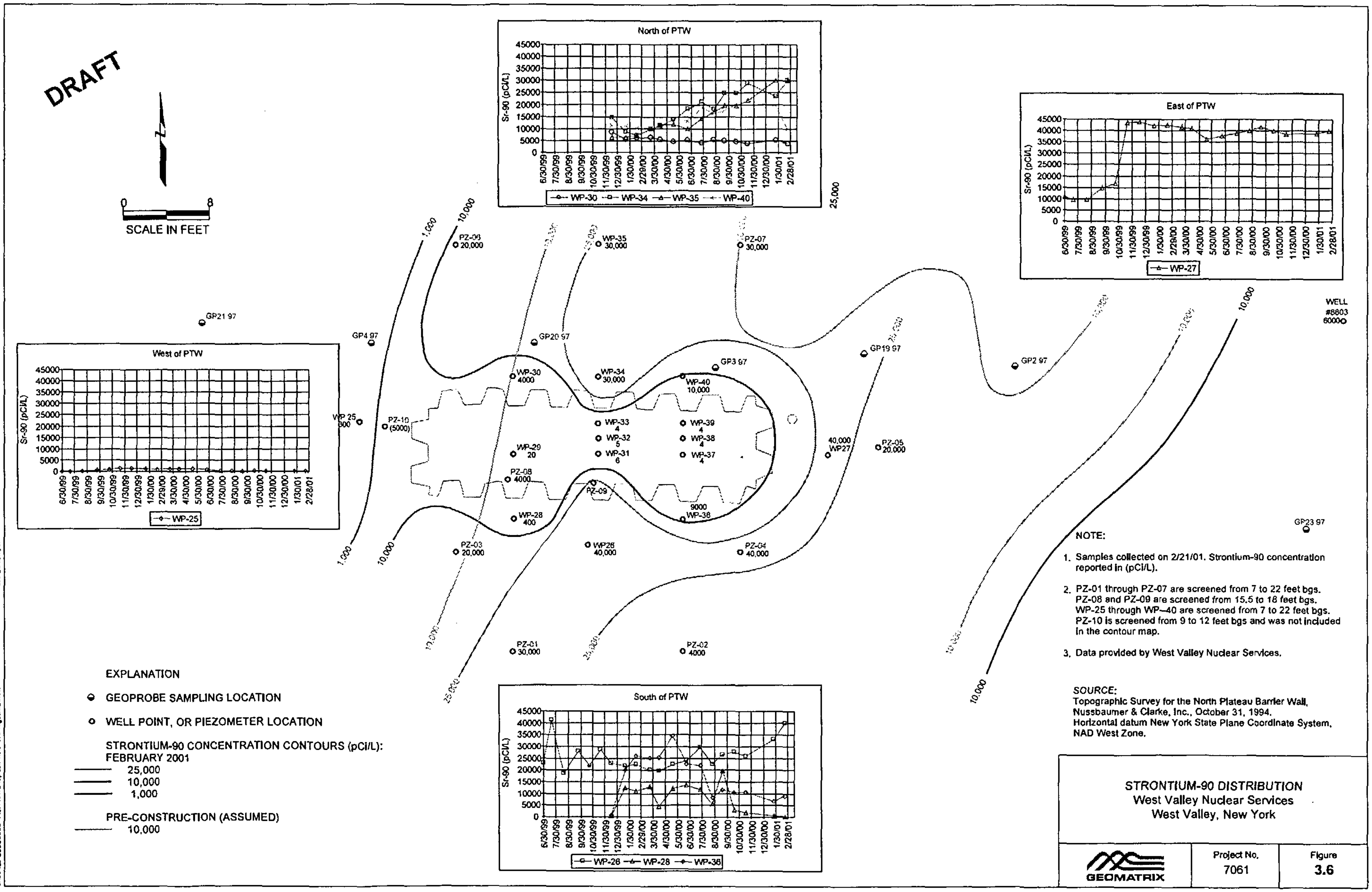
DRAFT

GEOMATRIX

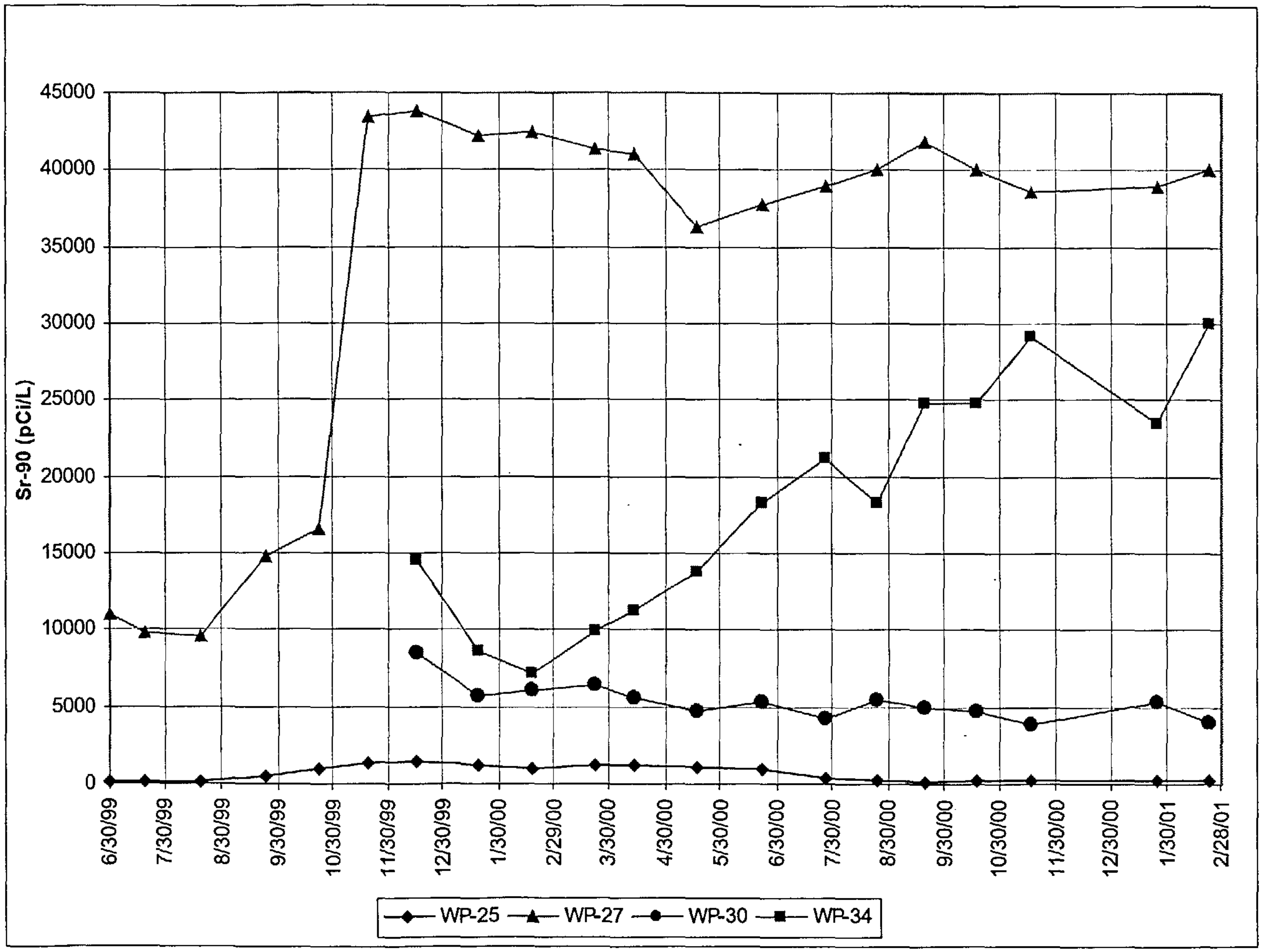
HYDROGRAPHS AND PRECIPITATION - FEBRUARY 2000 AND 2001
West Valley Nuclear Services
West Valley, New York

Project No. 7061

Figure 3.5


DRAFT
SCALE IN FEET
North of PTW
Sr-90 (pCi/L)
WP-30
WP-34
WP-35
WP-40
East of PTW
WP-27
West of PTW
WP-25
South of PTW
WP-26
WP-28
WP-36
GP21 97
GP4 97
GP20 97
GP3 97
GP19 97
GP2 97
GP23 97
WELL
#8603
6000
PZ-06
20,000
WP-35
30,000
PZ-07
30,000
WP-30
4000
WP-34
30,000
WP-40
10,000
WP 25
300
PZ-10
(5000)
WP-33
4
WP-32
5
WP-31
6
WP-39
4
WP-38
4
WP-37
4
WP-29
20
PZ-08
4000
PZ-09
WP-28
400
9000
WP-36
40,000
WP27
PZ-05
20,000
PZ-03
20,000
WP26
40,000
PZ-04
40,000
PZ-01
30,000
PZ-02
4000
1,000
10,000
25,000
EXPLANATION
GEOPROBE SAMPLING LOCATION
WELL POINT, OR PIEZOMETER LOCATION
STRONTIUM-90 CONCENTRATION CONTOURS (pCi/L):
FEBRUARY 2001
25,000
10,000
1,000
PRE-CONSTRUCTION (ASSUMED)
10,000
NOTE:
1. Samples collected on 2/21/01. Strontium-90 concentration reported in (pCi/L).
2. PZ-01 through PZ-07 are screened from 7 to 22 feet bgs. PZ-08 and PZ-09 are screened from 15.5 to 18 feet bgs. WP-25 through WP-40 are screened from 7 to 22 feet bgs. PZ-10 is screened from 9 to 12 feet bgs and was not included in the contour map.
3. Data provided by West Valley Nuclear Services.
SOURCE:
Topographic Survey for the North Plateau Barrier Wall, Nussbaumer & Clarke, Inc., October 31, 1994. Horizontal datum New York State Plane Coordinate System, NAD West Zone.
STRONTIUM-90 DISTRIBUTION
West Valley Nuclear Services
West Valley, New York
GEOMATRIX
Project No.
7061
Figure
3.6

S:\7000\7061\Fig_03_07.ai



DRAFT

GEOMATRIX

STRONTIUM-90 CONCENTRATION VS. TIME
West Valley Nuclear Services
West Valley, New York

Project No.
7061

Figure
3.7

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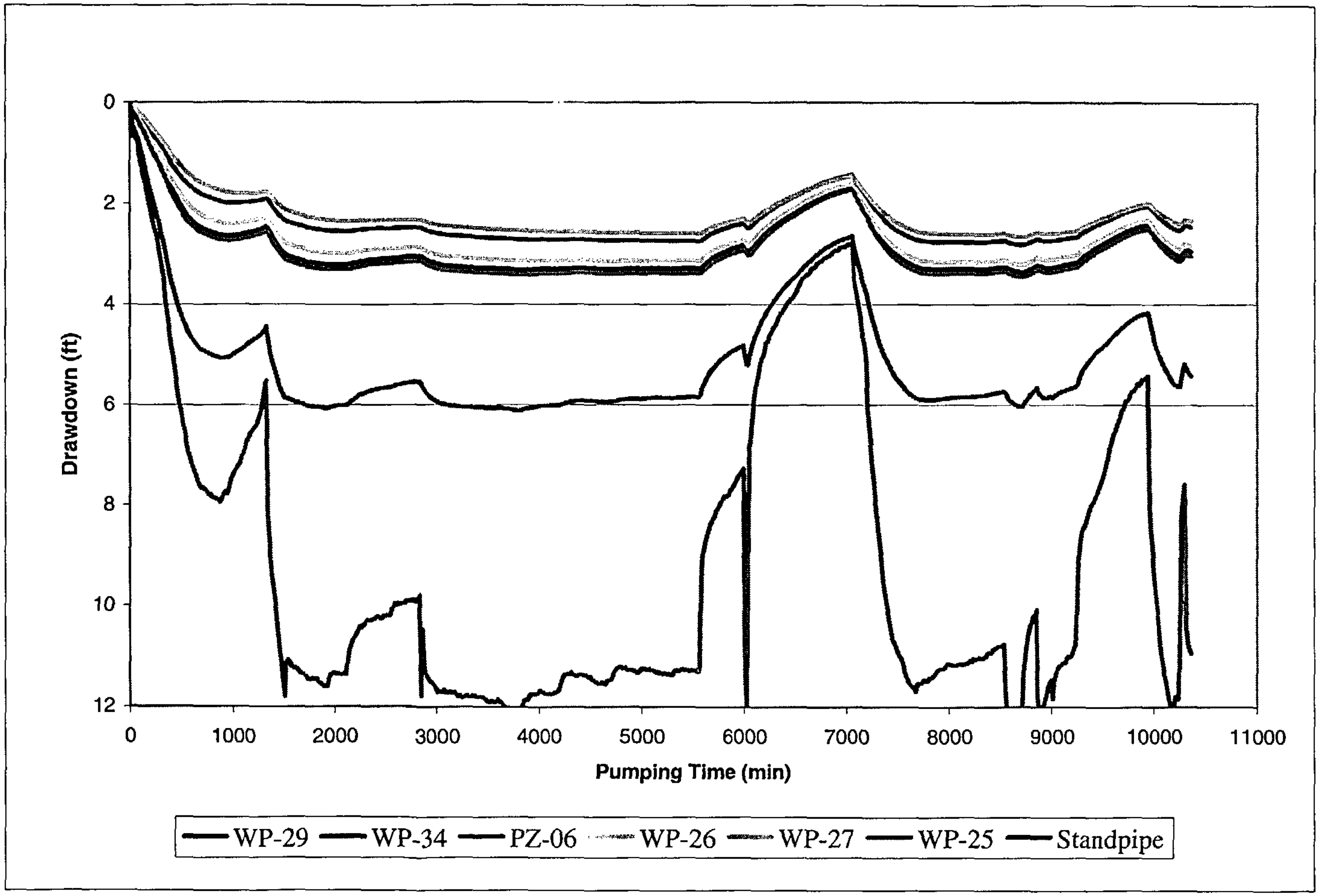


Note: Data from the January 2001 PTW riser pipe pumping test
Source: Data provided by West Valley Nuclear Services

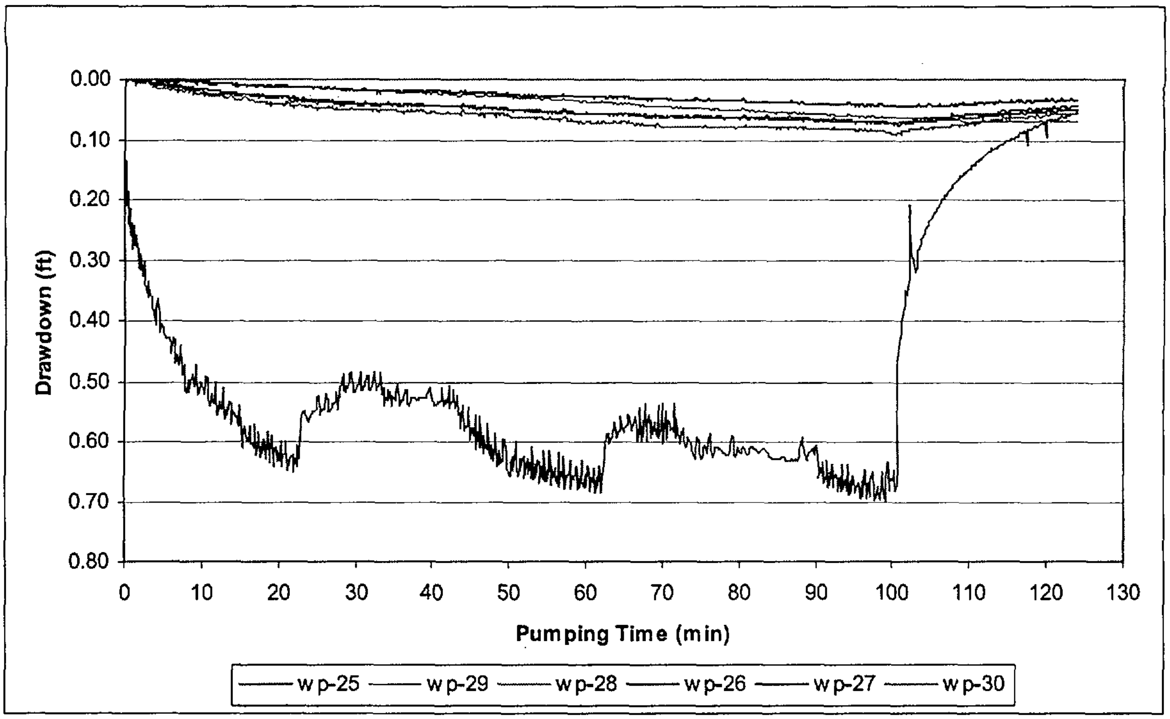
DRAFT

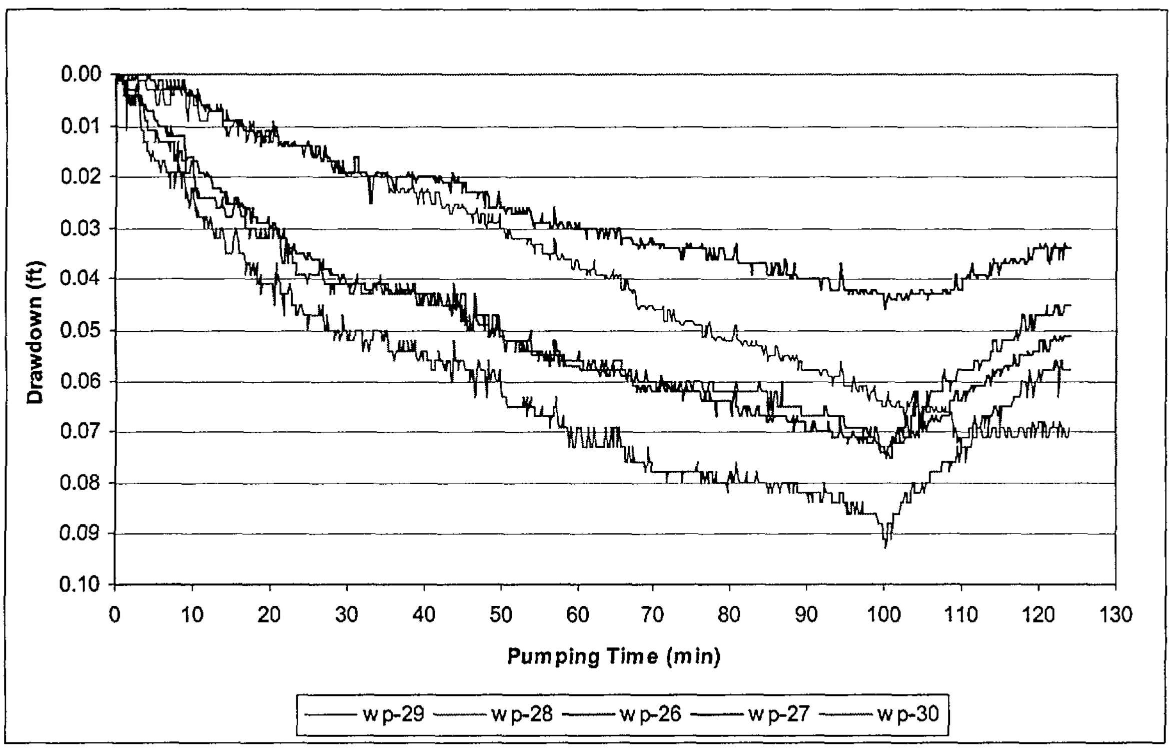
GEOMATRIX

SELECTED PTW STANDPIPE PUMPING TEST RESPONSES
West Valley Nuclear Services
West Valley, New York

Project No.
7061

Figure
3.8





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GEOMATRIX

WP-25 PUMPING TEST RESPONSES
West Valley Nuclear Services
West Valley, New York

Project No.
7061

Figure
3.9

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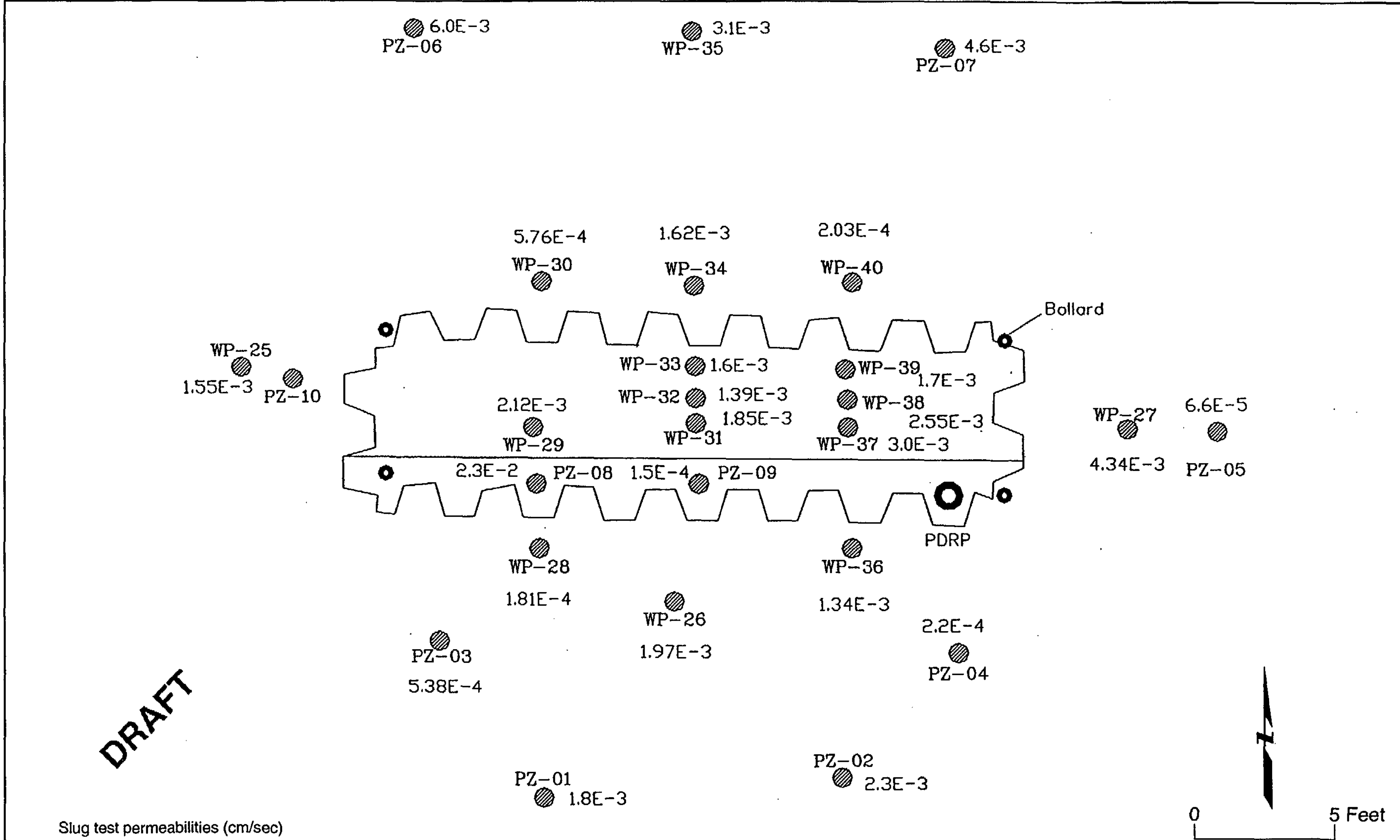

Slug test permeabilities (cm/sec)

GEOMATRIX

SLUG TEST RESULTS
West Valley Nuclear Services
West Valley, New York

Project No.
7061

Figure
3.10

APPENDIX 11
GEOMATRIX REPORT: PILOT PERMEABLE TREATMENT WALL HYDRAULIC ENGINEERING REPORT

GEOMATRIX

Pilot Permeable Treatment Wall Engineering Evaluation Report

West Valley Nuclear Services, LLC
West Valley, New York

Prepared for:

West Valley Nuclear Services, LLC
10282 Rock Springs Road
West Valley, New York 14171-9799

March 2001

Project No. 7061.000

Geomatrix Consultants, Inc.

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(Continued)

FIGURES

PILOT PERMEABLE TREATMENT WALL
ENGINEERING EVALUATION REPORT
West Valley Nuclear Services, LLC
West Valley, New York

1.0 INTRODUCTION AND SCOPE OF REPORT

This *Pilot Permeable Treatment Wall, Engineering Evaluation Report* was prepared by Geomatrix Consultants, Inc. (Geomatrix) at the request of West Valley Nuclear Services, LLC (WVNS). The report was commissioned by WVNS (Project 19-098745-C-JK) to assist in assessing the performance of a pilot permeable treatment wall (PTW) designed to remediate groundwater affected by radioactive Strontium-90 (Sr-90) beneath a portion of the West Valley Demonstration Project (WVDP) located in western New York state (Figure 1.1). A companion report, the *Pilot Permeable Treatment Wall Hydraulic Evaluation Report,* (the Hydraulic Evaluation Report) was also prepared by Geomatrix for WVNS. Both of these reports support preparation of the *Pilot Permeable Treatment Wall Modification Report*, to be submitted to WVNS by April 25, 2001.

A pilot PTW was installed by WVNS at the WVDP in Fall 1999 to assess the ability of the technology to passively and effectively reduce the concentration of Sr-90 affected groundwater. The pilot PTW was installed to treat a portion of the "2nd lobe" of the Sr-90 plume beneath the North Plateau of the site (Figure 1.2). The "1st lobe" of the Sr-90 plume to the west currently is being remediated by a groundwater recovery and aboveground ion exchange treatment system ("pump-and-treat") that was installed in 1995. While the pump-and-treat remedy reportedly reduces local migration of the Sr-90 plume, WVNS does not consider it capable of completely capturing and remediating the affected groundwater beneath the North Plateau. Thus WVNS identified PTW technology as a method potentially capable of effectively mitigating further migration of Sr-90 affected groundwater.

Monitoring data collected by WVNS indicates that the PTW may not be functioning as designed, specifically groundwater from south, and presumed up hydraulic gradient side of the PTW may not be flowing northward through the PTW. The general purpose of this report is to evaluate the engineering design and construction of the pilot PTW and, in conjunction with the companion Hydraulic Evaluation Report, to better understand the monitoring data collected to date.

This engineering evaluation has focussed on four tasks:

- A description of the design of the PTW
- A narrative of the construction of the PTW
- Potential causes of limited or no flow through the PTW
- Potential causes of raised water levels measured in the PTW relative to surrounding water levels.

A brief discussion of lessons learned from PTW installations at other sites, and how such lessons relate to the Pilot PTW at WVDP also is presented.

1.1 ORGANIZATION OF REPORT

The organization of this report follows the approach we have taken in this evaluation. Following this introductory section this reports consists of the following Sections:

- Section 2.0 Pilot PTW Construction and Installation Methods (including discussions of engineering design of the PTW, and reported as-built construction of the PTW).
- Section 3.0 Potential Causes of Inferred Limited or No Groundwater Flow Through the Pilot PTW (including data gaps for the causes)
- Section 4.0 Potential Causes of Higher Hydraulic Head Measured in the Pilot PTW (including data gaps for the causes).
- Section 5.0 Conclusions
- Section 6.0 Recommendations
- Section 7.0 Lessons Learned

1.3 SUMMARY OF HYDRAULIC EVALUATION REPORT

1.3.1 Summary of Previous Reports

The Hydraulic Evaluation Report summarized pertinent results from previous evaluation reports on the PTW performance, specifically:

- "Summary Information from PTW Evaluation and Assessment Activities," dated February 6, 2001, prepared by WVNS.

- "Technical Peer Review/Evaluation of the West Valley Pilot Permeable Treatment Wall," dated October 11, 2000, prepared by J. Moylan, URS Corporation.

- "Preliminary Operational Assessment for the Pilot Permeable Treatment Wall," dated May 12, 2000, prepared by WVNS.

- "Review of Preliminary Operational Assessment Report for the Pilot Permeable Treatment Wall (Draft), West Valley Demonstration Project," dated May 5, 2000, prepared by E. Berkey, Ph.D.

Common opinions from these reports include:

1. Uncertainty exists as to the direction of the local horizontal hydraulic gradient prior to PTW construction (the regional direction was generally observed to be toward the north, though some data indicated a northeasterly direction).

2. Water levels in well WP-25, directly west of the PTW, and well WP-27, directly east of the PTW, control the interpretation of the local groundwater gradient direction; water levels increased in well WP-25 and decreased slightly in well WP-27 following sheet pile installation. The direction of groundwater flow appears to flow directly eastward through the PTW based in part on well WP-25 and well WP-27 data. Water levels also are generally higher on the south side of the PTW compared to the north side of the PTW.

3. Surface water infiltration was considered to be a significant contributing factor to hydraulic mounding within the PTW.

4. There is potentially a "skin" around the subsurface PTW caused by the construction method. Uncertainty exists as to the nature and location of the "skin" and its influence on the hydraulic connection between the PTW and the aquifer, although water level data indicates that the hydraulic connection appeared to improve following long-term pumping from within the PTW.

5. The activity of Sr-90 in groundwater sampled from within the PTW is very low. Activity of Sr-90 in groundwater sampled from west of the PTW is low; activity of Sr-90 in groundwater southeast, east, and north of the PTW is high and appears to have increased in several locations since installation of the PTW.

6. From the previous assessments, multiple opinions exist as to what the most appropriate modification to the PTW consists of (e.g., additional pumping and development, installation of "wing" or lateral hydraulic barriers to route groundwater through the PTW, etc.).

1.3.2 Site Hydrostratigraphy

The following information on site hydrostratigraphy generally is summarized from the companion Hydraulics Report.

Two distinct hydrostratigraphic units are interpreted to exist beneath the North Plateau:

- A basal confining aquitard (or low permeability zone) consisting of lacustrine clay and silt, and Lavery Till
- The Shallow Water-Bearing Zone, consisting of alluvial sand and gravel, and the Slackwater Sequence

Figure 1.3 shows the borings, well-points and piezometers in the vicinity of the PTW. Cross-section D-D' (Figure1.4) shows the stratigraphy through the PTW indicated by the boring logs. (Cross-sections A-A', B-B', and C-C' are provided in the companion Hydraulics Report but are not repeated in this report). In the vicinity of the PTW, the Shallow Water-Bearing Zone generally consists of silty clay (ground surface to approximately 6 feet bgs), silt and gravel (approximately 6 to 11 feet bgs), and alternating layers of fine to coarse sand with gravel and silty clay with gravel characteristic of the Slackwater Sequence (approximately 11 to 25 and 30 feet bgs). The Shallow Water-Bearing Zone overlies the Lavery till which is encountered at depths of approximately 25 to 30 feet bgs. The top of the Lavery till unit is variable and appears to undulate in the vicinity of the pilot PTW.

1.3.3 Pre- and Post-Construction Groundwater Elevations

Three well-points (WP-25, WP-26 and WP-27) were installed in July 1999, prior to construction. Twenty-three additional well-points and piezometers were installed in and around the PTW following construction. Comparison of the pre-and post-construction data and review of post-construction data indicated the following:

- Water levels measured from wells located in the PTW are consistently higher than measurements from well points screened in the native sediments with the exception of water levels measured in WP-25;
- Following installation of the sheet piles, water levels in well WP-25 increased above ambient conditions and have remained high; following removal of the sheet piles, water levels in WP-27 decreased slightly;
- The groundwater elevations measured from well points located inside the PTW are practically identical, indicating a near zero horizontal hydraulic gradient;

- Groundwater elevation contours indicate that groundwater is flowing towards the east through the PTW and radiates away from the PTW on the south, east, and north faces indicating hydraulic mounding conditions; however, water levels on the south are also generally higher than water levels to the north and east sides of the PTW;

- The horizontal hydraulic gradient between the PTW and the native material decreased following PTW development (as suggested by water levels measured in wells WP-29 and WP-26,respectively, and shown in Figure 3.4 of the Hydraulic Evaluation Report);

- Groundwater elevation fluctuations due to precipitation or snow melt are generally similar in all well points inside or outside the PTW; well points located inside the PTW have greater fluctuations;

- Groundwater elevations inside the PTW are always higher than those measured outside the PTW, , except during precipitation and snowmelt when elevations measured in WP-25 are higher;

- Construction of the surface water drain in October 2000 has been effective in reducing surface water infiltration into the PTW.

1.3.4 Hydraulic Testing in and near PTW

In summer 2000, the PTW was "developed" by pumping water out of the drain pipe in the roundstone zone of the PTW. The purpose of the development was to attempt to mobilize and remove any low permeability "skin" that may have been present around the PTW. Several feet of fine sediment, presumably clinoptilolite, was reportedly observed within the 10-inch riser pipe within the roundstone zone.

Water levels were observed in a number of well-points and piezometers in and around the PTW during pumping. Water levels indicated there was some hydraulic connection between the PTW and the surrounding aquifer. Evaluation of the water levels indicated that zones of lower hydraulic conductivity may exist at the east, north,and south sides of the PTW.

1.3.5 Conclusions

The Hydraulic Evaluation concluded that the performance of the PTW is probably controlled by:

- a more eastward groundwater flow direction than initially anticipated (the PTW was oriented for northward flow);

- a highly heterogeneous aquifer sequence of fine and coarse sediments;

- a narrow flow zone of high activity Sr-90 water near the base of the aquifer and that is lower in activity at the west end and higher in activity at the east end of the PTW; this flow appears to be partially diverted around the east end of the PTW;

- the PTW may not have fully penetrated the aquifer, and the "hanging" central and eastern portion of the PTW may allow underflow of high Sr-90 activity groundwater;

- a discontinuous skin of lower permeability material at the contact of the PTW materials with the native aquifer material and heterogeneity within the PTW treatment material resulting from installation activities; and,

- slow discharge of treated low Sr-90 activity water from portions of the PTW; this water appears to reduce Sr-90 activity in some wells located close to the PTW.

1.3.6 Recommendations for Collecting Additional Data

The Hydraulic Evaluation Report recommended the following data collection activities to assess the possible causes of poor PTW performance:

1. Install two or more new wells and collect stratigraphic information in the vicinity of WP-25 (west end) and WP-27 (east end) to confirm the water level conditions that appear to provide major control on the assumed groundwater flow directions in the vicinity of the PTW.

2. Drill at least three additional soil borings around the eastern end of the pilot PTW (one each adjacent to the north, south, and eastern face) that are logged for stratigraphic detail to confirm whether the pilot PTW penetrates the underlying till or is "hanging" within the aquifer. One additional soil boring for stratigraphy should be drilled toward the western end of the PTW, and one additional boring should be drilled near the south face of the PTW. These borings should be converted to 2-inch monitoring wells for subsequent aquifer testing.

3. Perform one or more tracer tests to provide field evidence of the flow field that has been interpreted from water levels in the vicinity of the pilot PTW.

4. Perform a long-term aquifer test consisting of a series of step-test periods, a constant-discharge period, and recovery period, to provide additional data on the hydraulic connection of the PTW to the surrounding aquifer.

2.0 PILOT PTW CONSTRUCTION AND INSTALLATION METHODS

2.1 ENGINEERING DESIGN OF PILOT PTW

The engineering design of the PTW was evaluated from the design drawings supplied to Geomatrix. These drawings were:

- North Plateau Permeable Treatment Wall, Drawing no. 900D-7867, sheets 1 through 8 of 8.
- North Plateau Permeable Treatment Wall Cofferdam, Drawing no. 900D-7857, sheet 1 through 4 of 4, and
- Site, North Plateau Area, Topography and Underground Piping, Drawing no. 900D-6743, sheet 1 of 1.

The following presents a narrative of the design of the PTW.

2.1.1 Placement and Assembly of Cofferdam

The PTW was designed as a rectangular cofferdam, measuring 7 feet wide by 30.5 feet long. The design specified 42 sheets of Arbed AZ48 sheet piles to be driven with a vibratory hammer to the approximate contact with the Lavery Till at 28 feet below original ground surface along the alignment of the treatment wall. The design depth is shown as 1356 feet above mean sea level (msl) on CAD drawing #900D-7857 sheet 1. The design then specified the sheet piles be driven an additional ten feet (to 1346 msl) into the Lavery Till with an impact hammer. The design also specified a contingency plan, allowing the contractor to drive the sheet piles to a lesser depth and cut off the tops to a uniform height, should penetration of the Till be difficult. Prior to driving, Adeka Ultraseal #50A was to be applied to each sheet pile interlock, to minimize leakage of groundwater into the excavation during construction.

The design required the cofferdam to be supported by a single level of external bracing that consisted of 2-W36x182 wales with restraint brackets mounted on 7'7" centers along the length of the cofferdam , and of two W18x86 wales along the width. The wales were to be installed horizontally around the outer perimeter of the cofferdam beneath the existing ground surface (1384 feet msl), with the horizontal axis at 1382 feet msl. On the south side of the cofferdam, flowable fill was to be placed under the wale system to provide support for equipment.

After completion of the structural elements of the cofferdam, the volume of soil within the cofferdam was to be dewatered with four dedicated pumps (DW-1, -2, -3, -4) installed at equal intervals directly beneath the alignment of the PTW. Each dewatering well was to be activated by a pressure switch designed to engage the pump whenever the groundwater rises in the well 12 inches above each individual pump. Groundwater pumped from these wells was to be pumped to the surface, conveyed in 2-inch polyethylene pipe and discharged to a 1000 gallon holding tank. From the tank, the treated water was to be pumped by a sump pump, activated by a float switch, to Lagoon #2.

2.1.2 Excavation of Soil and Placement of the Treatment Media

The soil within the cofferdam was to be excavated to 1356 feet msl or approximately 28 feet below original ground surface (bgs). The drawings do not indicate any slope of the bottom of the PTW. Prior to any backfilling, the drawings specify placement of a 6 inch perforated PVC pipe, wrapped in non-woven geotextile or a sock-type geosynthetic pipe sleeve, attached to a 10 inch diameter perforated PVC standpipe to be placed in the east end of the roundstone zone of the PTW.

A divider system, consisting of a long steel plate, suspended by cables from the surface, was to be constructed and installed to maintain physical separation of a 18-inch wide vertical layer of #1 stone (roundstone) adjacent to a 66-inch wide vertical layer of clinoptilolite, the zeolitic treatment media. The separator plate was to be raised stepwise in increments of 2.75 feet always leaving a minimum of 8 inches of the plate buried between the stone and the clinoptilolite. This buffer would be maintained by visual inspection. The roundstone was to be poured in 12 inch lifts on the southwest side of the divider system into a 42x42 inch hopper with a flexible 8 inch PVC "elephant trunk" attached to its discharge port. The purpose of the flexible hose was to channel and control the flow of the roundstone, as the hopper, free to roll on steel I-beam rails at the surface, was guided forward and backward dispensing the stone in a uniform fashion. Other than the 12-inch lift limitation, alternating with lifts of stone, the method of clinoptilolite placement was not specified.

Following placement of the roundstone and clinoptilolite backfill to within 2 feet of finished grade of elevation 1383.2, the wales and backfilling system were to be removed. The area behind the sheet piles where the wales were located to be filled with previously excavated, non-contaminated material, and the clinoptilolite and roundstone were to be brought up to elevation 1383.2. The design called for a one inch layer of granular sodium montmorillonite (Volclay CG50 Bentonite) to be placed over the roundstone, clinoptilolite, and backfill placed around the outside of the sheet piles, mounded to 6 inches high on the inside and outside of the sheet piles. We understand that this layer of bentonite may have been placed after the sheet piles were extracted, although this conflicts with the as-built drawings. The sheet piles then were extracted and the surface completion installed.

The surface completion was to consist of clay fill approximately 1.4 to 1.9 feet thick constructed over the PTW. The clay fill was to match existing grades (El. 1384.6) at the edge of the excavation and be mounded 6-inches higher in the middle of the PTW (El. 1385.1). Protective 3.5-inch diameter bumper posts were to be placed around the perimeter of the PTW at each of the four corners. No compaction requirements or select backfill specifications are indicated by the drawings.

We understand that quality assurance testing of the clinoptilolite before delivery showed less than 4 percent fines in the material. Thedesign did not prescribe any specific procedures for transportation, storage, handling or inspection of the clinoptilolite at the site prior to placement in the PTW.

2.2 AS-BUILT CONSTRUCTION OF PILOT PTW

The as-built construction of the PTW was described in the as-built construction drawings and in WVNS's response to questions submitted by Geomatrix on February 14, 2001. Jim Woodworth, the Cognizant Engineer for the PTW work, and Mark Hemann of WVNS provided responses to Geomatrix's questions. The following narrative was developed from their answers and review of the as-built drawings.

The pilot PTW was installed to treat a portion of the "2nd lobe" of the Sr-90 plume beneath the North Plateau of the site (Figure 1.2). The approximately 100 feet by 100 feet area of the PTW is located to the north west of Lagoons No. 4 and No. 5. The ground surface generally slopes down at about 3 percent from south to north in the area of the PTW.

The 100 feet by 100 feet area around the PTW was first prepared by laying down a working surface consisting of a 7-inch thick layer of crushed stone placed on geotextile (the "hardstand"). The PTW dewatering system of four wells was then installed, and the well discharge piped to the 1000-gallon holding tank then to Lagoon #2. The electrical line to the wells was installed in a conduit that was placed in a trench about 2 feet wide and 2 feet deep.

In the area of the PTW, the hardstand layer was scraped back and the geotextile cut to allow installation of the sheet piles. The PTW cofferdam was then laid out and the sheet piles driven to create the structure. The sheet piles were driven on the week of August 23, 1999.

The sheet piles were driven with a vibratory hammer to approximately elevation 1358 msl, which is approximately the top of the Lavery Till; the sheet piles then were driven to grade with an impact hammer. The top of the Till was encountered at approximately elevation 1358 instead of the design elevation of 1356. The sheet piles thus were driven 12 feet into the Till, instead of the designed 10 feet penetration. The hardstand and native soil were excavated around the outside of the sheet piles and the geotextile was cut back to provide room to place the external wale system. Two inclinometers were installed at the same time as the four dewatering wells about 8 feet from the sheet piles on the north and south sides of the cofferdam; these inclinometers were monitored during excavation and backfilling of the PTW, including extraction of the sheet piles.

The interior of the PTW was dewatered by pumping the 4 wells within the cofferdam. Excavation began to remove the material inside the cofferdam. The excavation was dewatered by the 4 wells until the excavation reached about 15 feet bgs. Below this depth sump pumps were dropped into the excavation to dewater the top of the excavation as it proceeded, and the dedicated dewatering wells were removed by the excavation. The sheet piles had Adeka sealant on the interlocks to reduce seepage into the cofferdam excavation. This sealant is designed to swell in contact with water and plug the space in the interlock. However some water continued to flow into the excavation; this water was removed by the sump pumps.

The base of the excavation was at approximately elevation 1358 msl at the east end, sloping up to approximately 1358 msl at the west end. A base elevation of1356 msl was depicted on the design drawings. The base of the excavation sloped from east to west, and the west end was about 1 foot higher than the east. Based on pre-excavation boring data, portions of the excavation did not extend to the top of the Till (see Figure 1.4). The inclinometers indicated a maximum of about 1 inch of movement of the top of the excavation after it was complete.

A divider system was installed within the cofferdam to keep the clinoptilolite and roundstone separate during backfilling. A 6-inch diameter perforated PVC pipe was placed along the base of the excavation within the roundstone backfill area, with a 10-inch PVC riser pipe attached to the lower east end of the drainage pipe. The cofferdam was backfilled by emptying supersacks of clinoptilolite into the excavation from the surface, and placing roundstone through a PVC "elephant trunk". Lift thicknesses were approximately 1 foot. The level of the clinoptilolite and roundstone backfills were maintained a maximum1 foot apart during backfilling by observation. Backfill was placed in a dry excavation. This method of backfilling the excavation was reportedly very effective.

The cofferdam was backfilled with clinoptilolite and roundstone to approximately elevation 1382 when the divider system was removed. The clinoptilolite and roundstone was then brought up to design elevation of 1383.2. By this time the wales had been removed, and the zone outside the cofferdam where the wales had been was backfilled with previously excavated material. The geotextile underneath the hardstand was cut back to where it daylighted into the excavation sidewalls. The as-built drawings indicate that the 1-inch thick layer of Volclay CG-50 (a granular bentonite with a grain size similar to a sand) was placed over the excavation at 1383.2, with additional mounds of CG-50 placed on both sides of the sheet piles; however we understand that no bentonite was placed until the sheet piles were extracted. The purpose of the CG-50 was to seal the surface of the treatment zone.

From November 8 through 12, 1999 the sheet piles were withdrawn from the ground. The sheet piles were removed with a vibratory hammer, starting at the west side of the cofferdam near WP-25. Any material stuck to the sheet piles was scraped off; reportedly this material amounted to less than 2 wheelbarrow loads per sheet pile. As the sheet piles were withdrawn, the surface of the clinoptilolite settled; once all of the sheet piles had been withdrawn, the surface of the clinoptilolite had settled approximately 4 feet. The inclinometers indicated lateral movement into the excavation of about 7 inches on the south side, and 3 inches on the north side. Well WP-26 directly south of the cofferdam settled about 0.36 feet.

Additional clinoptilolite was added to the existing material to bring it back up to grade at 1383.2. A one-inch layer of CG-50 was placed over the roundstone and clinoptilolite surface, and clay fill was placed in the excavation to bring the surface up to final elevation of 1384.6, mounded to 1385.1 in the middle of the PTW. The clay fill was not compacted so that further settling of the clinoptilolite would not occur. Hardstand stone was then reportedly raked over the filled area to provide a good working surface.

Four bumper posts were placed near the corners of the PTW. The bumper posts were initially installed as the cofferdam was backfilled, but they were in the wrong location so they were removed and reinstalled. The bumper posts are 7 feet long, and about 3.5 feet of each post is embedded in the ground. The posts were installed by rotating them into the ground. A helical screw at the base of each post secures it into the ground. Based on the as-built location of the bumper posts and the PTW, it appears that the at least one and possibly two of the posts were installed directly over the clinoptilolite or roundstone zone; the other two posts are within 6-inches of the clinoptilolite and roundstone zone. The top of the filled area was at elevation 1384.6 to 1385.1, so the helical screw is at 1381.1 to 1381.6, or about 1.5 to 2 feet below the top of the clinoptilolite and roundstone.

A surface water drain was installed in October 2000. This drain consisted of a 1-foot deep ditch lined with HDPE on the base and downstream side and filled with surge stone. The drain was installed about 12 feet south and slightly uphill from the PTW, and it directed flow north and away from the PTW area. Flow has been observed in the drain, and the sharp water level increases within the PTW that were observed following rain or snowmelt have not been observed since the drain was installed.

3.0 POTENTIAL CAUSES OF LIMITED GROUNDWATER FLOW THROUGH THE PILOT PTW

Water levels were measured in well points and piezometers installed in and around the PTW before and after construction of the PTW. Interpretation of these water levels indicates that groundwater from the south and expected regional upgradient direction likely is flowing around the PTW to the east; with groundwater from the west entering the PTW. This section discusses potential causes of limited flow through the PTW that could have resulted from the design and construction of the PTW.

3.1 HIGH GROUNDWATER LEVEL

Measured water levels within the PTW are higher than water levels on the outside except at WP-25. Higher water levels within the PTW would prevent the Sr-90 plume to the south from flowing through the PTW, and no other effects are necessary to explain the observed hydraulic condition. . Potential causes of higher water levels within the PTW are discussed in Section 4 of this report.

3.2 SMEARING OF PTW SIDEWALLS

3.2.1 Description of Mechanism

Driving and extraction of sheet piles will "smear" materials along the interface between the sheet piles and the native soil, creating a skin of lower permeability material around the PTW. As described in the Hydraulic Evaluation report, the Slackwater Sequence penetrated by the PTW consists of interlayered fine-grained and coarse-grained units, and this sequence is probably the most susceptible to smearing as the finer-grained units are smeared across the coarse-grained water bearing units. Portions of the PTW sidewalls are probably smeared, reducing flow through these portions of the aquifer.

However, some flow was observed coming in to the excavation through the sealed sheet piles as the PTW was constructed. This indicates that groundwater flow through portions of the aquifer was probably not affected by smearing, at least when the sheet piles were installed. In addition, an hydraulic connection between the PTW and the surrounding aquifer has been observed in various pumping tests in the area.

We believe that smearing may have cut off some of the thinner water bearing zones, but has only marginally affected thicker, coarser-grained water bearing zones. If hydraulic heads are higher in these coarser units, then higher water levels would be observed in the PTW than are observed in well-points and piezometers outside the PTW that are screened across both coarse-grained and fine-grained water bearing units. Smearing may also reduce the effect of underflow beneath the PTW, although this cannot be quantified with available data.

The magnitude of smearing caused by driving and extracting the sheet piles, and its effect on the local hydraulic conductivity of native materials has not been extensively studied, to our knowledge. We expect that the magnitude of this effect depends on soil strength and plasticity, the thickness of the coarse- and fine-grained units, the thickness of the sheet piles and the length of time the sheet piles are in the ground.

3.2.2 Collecting Data to Support Mechanism

Careful installation of monitoring wells screened only in candidate coarse-grained units near the PTW can indicate whether piezometric water levels are higher in these units than in finer-grained units. These monitoring wells should be installed in addition to those recommended in the Hydraulic Evaluation report. If water levels in these wells are similar to water levels measured in the PTW, then smearing could be the sole cause of the observed hydraulic regime.

Additional monitoring wells could also be installed through the PTW and screened in the potential underflow zone to evaluate groundwater flow under the PTW.

3.3 CONSOLIDATION OF CLINOPTILOLITE

3.3.1 Description of Mechanism

As the sheet piles were withdrawn, the surface of the clinoptilolite settled about 4 feet, indicating significant consolidation and densification of the material; for a 30 feet long and 8.5 feet wide PTW, we estimate a volume loss of about 1000 cubic feet. In addition, the inclinometers showed up between 2 and 6 inches of movement into the excavation; we estimate this volume loss to be about 85 cubic feet to the north and 113 cubic feet to the south of the PTW. We therefore estimate that there was a loss of volume of about 1200 cubic feet during sheet pile extraction. This volume loss probably resulted from four mechanisms; the volume of the extracted sheet piles, consolidation of the clinoptilolite, crushing of the clinoptilolite, and movement of the clinoptilolite into the roundstonè zone. The volume of the extracted sheet piles was approximately 200 cubic feet (cross sectional area of 0.192 square feet per sheet pile, 40 sheet piles and an average depth of about 26 feet). Thus about 1000 cubic feet of volume loss probably occurred due to consolidation, crushing or movement of the clinoptilolite. We believe that about 80 to 90 percent of this volume loss occurred because of crushing of the clinoptilolite and movement of the clinoptilolite into the roundstone zone (see following sections of this report). Thus we estimate that approximately 100 to 200 cubic feet of lost volume resulted from consolidation of the clinoptilolite.

Laboratory tests conducted by the University of Buffalo (UB) (Rabideau, 2000) on loose samples of clinoptilolite indicate a hydraulic conductivity of about 1.2×10^{-1} centimeters per second (cm/sec). We believe that consolidation of the material without crushing the grains (see below) would not sufficiently reduce hydraulic conductivity to prevent groundwater flow through the PTW.

3.4 CRUSHING OF CLINOPTILOLITE

3.4.1 Description of Mechanism

Although the clinoptilolite was manufactured as a 14x50 mesh size, clinoptilolite can be crushed to dust by finger pressure. Material testing performed at UB showed an hydraulic conductivity of 1.2×10^{-1} cm/sec for loose clinoptilolite, and an hydraulic conductivity of 4×10^{-3} cm/sec for compacted clinoptilolite. Clearly any significant mechanical disturbance of the clinoptilolite will crush some of the grains and reduce the hydraulic conductivity of the material. Mechanical disturbance of the clinoptilolite would have occurred at least three times during construction of the PTW: transportation, placing clinoptilolite in the PTW, and extraction of the sheet piles.

The clinoptilolite was delivered to WVNS in supersacks that were transported by truck from the source in Oregon. Jostling and vibration during transportation and delivery probably resulted in some grain breakage, but we expect that the limited amount of fines generated would not have a significant impact on the hydraulic conductivity of the material. Grain size analysis of delivered clinoptilolite would have been a useful quality assurance check.

The clinoptilolite was dropped into the excavation to backfill it. The high drop probably crushed more clinoptilolite grains, resulting in more fines generation. We cannot quantify the effect of this placement method on the grain size of the clinoptilolite.

The clinoptilolite was subjected to the most intense disturbance when the sheet piles were withdrawn with a vibratory hammer. The clinoptilolite next to the sheet piles would have been most affected, with many of these grains almost certainly crushed. The clinoptilolite near the bottom of the PTW would be subject to the most severe crushing because this material is subject to additional stresses from the depth of burial. Crushing of the clinoptilolite grains probably dissipates with lateral distance from the sheet piles, and we estimate that clinoptilolite grains more than about 2 feet away suffered only minor breakage. Along the south side of the PTW, the roundstone was adjacent to the sheet piles, so we would not expect much grain breakage in the clinoptilolite zone 2 feet away from the sheet piles. In addition, if water levels within the PTW rose significantly during sheet pile extraction, some of the vibratory energy would be dissipated into the water and the amount of grain crushing would be less in areas where the sheet piles were withdrawn after water levels rose. If this model is correct, crushed clinoptilolite grains were present within about 2 feet of the north, east and west sides of the PTW, and the grains were more severely crushed at the base of the PTW. Inflow of water during and possibly after the sheet piles were withdrawn may have transported fines around the PTW and affected this distribution.

As discussed above, we estimated the total volume loss of the clinoptilolite zone to be about 1000 cubic feet. We estimated volume loss due to consolidation of about 150 cubic feet, and that due to movement into the roundstone of about 530 cubic feet (see below). Thus we estimate the volume loss due to grain crushing to be about 320 cubic feet, or about 6 to 7 percent of the estimated clinoptilolite volume of 4875 cubic feet. We note that this corresponds very closely to the volume reduction noted by Rabideau (2000) in compacting clinoptilolite, where the porosity reduced from 0.51 to 0.48, or a volume reduction of 6 percent.

The inclinometer readings indicated that the north side of the excavation moved inward about 2 inches after the sheet piles were pulled, while the south side, adjacent to the roundstone zone, moved inward about 6inches. Because the roundstone probably compacts less than the clinoptilolite, and it would not be subject to grain breakage, the greater inward movement on the south side indicates that the roundstone/clinoptilolite interface was probably more unstable than the clinoptilolite on the north side during sheet pile extraction. Because we do not expect much grain breakage along the roundstone/clinoptilolite interface, this would indicate that the clinoptilolite penetrated into voids within the roundstone. This conclusion is also supported by the clinoptilolite fines that were found in the PVC riser pipe in the roundstone zone.

In conclusion we believe that the clinoptilolite next to the sheet piles on the north, east, and west sides of the PTW, and at the base of the PTW, probably suffered significant grain breakage and inter-grain abrasion as the sheet piles were extracted. Along the north side, the clinoptilolite suffered much less grain breakage, but the sides of the excavation moved more as the clinoptilolite penetrated voids within the 2-feet thick roundstone zone, as discussed in the next section of the report.

3.4.2 Collecting Data to Support Mechanism

Slug test results discussed in the Hydraulic Evaluation Report indicate a hydraulic conductivity of about $1x10^{-3}$ cm/sec throughout the clinoptilolite zone. It appears that the in situ hydraulic conductivity of the clinoptilolite is significantly less than $1.2x10^{-1}$ cm/sec determined for loose clinoptilolite, and is in the range of $4x10^{-3}$ cm/sec that was determined in laboratory tests on compacted clinoptilolite in which fines were generated within the clinoptilolite by the compaction effort (Rabideau, 2000).Review of the hydraulic responses and distribution of hydraulic conductivity values from various types of hydraulic tests in and around the PTW (see the Hydraulic Evaluation Report, Section 3.4.2)indicates that the hydraulic conductivity likely is highly variable within the PTW.

Collecting samples of clinoptilolite along the north, east, and west sides of the PTW and performing grain size analyses on these samples may provide additional support for the grain-crushing hypothesis. However Rabideau noted only 23 percent fines (passing no. 200 sieve) in the compacted clinoptilolite (compared to 2 percent fines in the uncompacted clinoptilolite), which was sufficient to reduce the hydraulic conductivity by two orders of magnitude. The process used to sample the clinoptilolite may also generate enough fines to mask those already present and make evaluation of in situ fines difficult. Therefore we do not recommend this course of action because the additional data may be misleading.

3.5 CLINOPTILOLITE PLUGGING VOID SPACES IN ROUNDSTONE

3.5.1 Description of Mechanism

The grain size of the roundstone was 90-100% smaller than 0.5 inches, and 0-15% smaller than 0.25 inches, while the grain size of the clinoptilolite was between about 0.055 and 0.017inches. With an average grain size of about one tenth that of the roundstone, the clinoptilolite could easily penetrate the void spaces within the roundstone. As described in Section 3.4, interpretation of inclinometer data indicate that the clinoptilolite penetrated into voids within the roundstone on the south side of the PTW. This conclusion is also supported by the clinoptilolite fines that were found in the PVC riser pipe in the roundstone zone.

The presence of clinoptilolite within the voids of the roundstone reduces the hydraulic conductivity of the roundstone. As discussed in the Hydraulic Evaluation report, results of slug test in the two piezometers (PZ-08 and PZ-09) in the roundstone zone of the PTW indicate hydraulic conductivities of $2x10^{-2}$ and $1.5x10^{-4}$ cm/sec. Although these slug test results may be biased from the direct push installation and construction of the piezometers, these data indicate variable hydraulic conductivity in the roundstone, with areas of reduced hydraulic conductivity due to the presence of clinoptilolite particles and fines. If the clinoptilolite filled all of the voids of the roundstone (estimated to be 30 percent voids in a volume 30 feet long, by 26 feet deep by 2.25 feet wide), we estimate that this accounts for about 530 cubic feet of volume loss in the clinoptilolite.

3.5.2 Collecting Data to Support Mechanism

It is very likely that clinoptilolite entered the roundstone zone. We do not think there is any merit in obtaining additional data to support this hypothesis.

3.6 Fines Movement as PTW Filled with Groundwater

3.6.1 Description of Mechanism

The PTW was not filled with water before the cofferdam was removed. Thus groundwater flowed into the PTW as the sheet piles were extracted. At the base of the excavation there was approximately 20 feet of hydraulic head forcing the groundwater into the PTW. Significant amounts of clinoptilolite fines could have been transported through the PTW as the turbulent flow of water entered the PTW and filled it. Assuming a head potential of approximately 20 feet between the interior of the dry cofferdam and the ambient potentiometric surface, a hydraulic conductivity of 1×10^{-2} cm/sec (or 3×10^{-3} ft/sec) based on estimates for the uncompacted loose clinoptilolite, and a porosity of 0.3, the initial velocity of water flowing into the cofferdam as the first sheet piles were removed, according to the equation:

$v = Ki/n$

where,

v = velocity (ft/sec), K = hydraulic conductivity (ft/sec), i. = gradient (ft/ft) and n = porosity

would be approximately 0.2 feet per second. This should be considered an upper bound approximate estimate; the actual inflow rate likely was somewhat less as the clinoptilolite at the bottom of the cofferdam would have been somewhat compacted from the weight of the overlying clinoptilolite . As the PTW filled with water, the flow rate would have reduced and eventually ceased when the water level reached equilibrium with adjacent groundwater levels.

The approximately 10,000 gallon void space of the dry PTW likely was on the order of tens of hours, or about one to two days .

Sheet piles were first extracted at the west side of the PTW, near WP-25 where elevated groundwater elevations are almost the same as current groundwater elevations in the PTW. Any fines in this area would have been flushed to the south, east and north sides of the PTW as the groundwater flowed in. We cannot calculate how fast the PTW would fill up with groundwater (the distribution of hydraulic conductivity is not well defined in the PTW), nor do we know exactly how long it took to remove the sheet piles. However, it seems likely that the major portion of the inflow occurred as the sheet piles at the west end of the PTW were removed. Thus, fines generated in this area from sheet pile extraction could have been flushed into the PTW, and could conceivably contribute to the "skin" of low permeability material that is thought to be present around the PTW. Additional fines generated when extracting sheet piles along the south, east and north sides of the PTW would then contribute to this "skin".

3.6.2 Collecting Data to Support Mechanism

This was a transient effect that occurred when the sheet piles were withdrawn. The mechanism cannot easily be replicated, and other data that could be collected to support it may prove inconclusive.

3.7 GROUNDWATER FLOW UNDER PTW

3.7.1 Description of Mechanism

As described in the Hydraulic Evaluation report, as-built information indicates that the PTW was not excavated to the top of the Lavery Till, and groundwater may be flowing under the east side of the PTW.

3.7.2 Collecting Data to Support Mechanism

The Hydraulic Evaluation Report recommended drilling at least three additional soil borings to confirm whether the pilot PTW penetrates the underlying till or is "hanging" within the aquifer.

4.0 POTENTIAL CAUSES OF HIGHER HYDRAULIC HEAD MEASURED IN THE PILOT PTW

This section discusses potential causes of higher hydraulic head measured in the PTW. This discussion does not address potential causes associated with site stratigraphy or hydrogeology; these issues are discussed in the Hydraulic Evaluation report. This section describes potential conduits for surface water to infiltrate into the PTW. Each of the potential conduits described below is illustrated in Figure 4.1

4.1 POTENTIAL CONDUITS INTO THE PTW

4.1.1 Description of potential conduits

Water level data indicate that water levels within the PTW rise during rainfall and snow events. Thus there appears to be a hydraulic connection between surface water and groundwater within the PTW.

A potential source of surface water that could enter the PTW is the hardstand surface layer that is present over the area of the PTW, as noted in previous evaluation reports. The 7-inch thick layer over the 100 by 100 feet area of the PTW could hold up to 13,000 gallons of water when fully saturated. Surface grades over this area slope down to the north, so water can flow to the PTW. A surface drain was constructed in October 2000 to divert surface water around the southern side of the PTW. Since installation of the drain, sharp rises in water levels within the PTW have not been observed during rainfall and snow events. Therefore the volume of flow into the PTW has been reduced by the construction of the surface drain.However, the data still indicate that the hardstand layer is hydraulically connected to the PTW.

Drawing 900D-7867 sheet 1 shows two buried conduits that enter the area of the PTW from the west (the area of elevated groundwater around WP-25). The trenches in which these conduits are buried could divert surface water into the PTW. The cross-section of the trenches in which the conduits are buried is reportedly 2-feet deep and 2 feet wide and probably filled with granular material.

The as-built locations of some of the bumper posts are on top of the PTW. The locations of the remaining bumper posts are almost certainly on top the cap over the PTW. We do not know where the bumpers were first installed before they were moved to their current position, but some of these locations may also have been over the PTW. The bumper posts are screwed 3.5 feet into the ground, which would penetrate the cap over the PTW. These bumpers therefore also provide an entry point for surface water to enter the PTW.

4.2 PERMEABLE CAP OVER PTW

The cap over the PTW was designed to have a low hydraulic conductivity to prevent infiltration of surface water into the PTW. There are issues with both the design and the construction of the cap that have probably rendered it ineffective in preventing surface water infiltration. The

1-inch thick bentonite layer placed over the clinoptilolite and roundstone zone was Volclay CG-50 which has a grain size of 99 percent smaller than 0.008 inches and 15 percent less than 0.0003 inches, or about one tenth that of the clinoptilolite, and one hundredth that of the roundstone. This material could easily fall into the void spaces of the PTW materials. We also suggest that it would be very difficult to maintain the 1-inch thickness during placement of the overlying clay material, and that the bentonite could easily have been displaced either laterally or into the PTW materials as the clay fill was placed, thus compromising the integrity of the low permeability layer.

We understand that the clay fill has a hydraulic conductivity of about 1×10^{-7} cm/sec when compacted. However the clay was not compacted when placed over the PTW for fear of further crushing the clinoptilolite. There are almost certainly flow paths through this uncompacted clay that would allow surface infiltration to enter the PTW. While the cap limits the volume of surface water that directly enters the PTW from above, some bypass likely does occur. Hydrographs shown as Figures 3.4.and 3.5 of the Hydraulic Evaluation Report indicate that the magnitude of water level increases within the PTW is greater than the water level rise in the native aquifer during and after rainfall and snow events. This response appeared to become less pronounced, although not eliminated, after the installation of an upgradient surface drain.

The construction detail of how the clay cap is tied into the adjacent hardstand layer is critical in evaluating whether water in the hardstand can enter the PTW. No details were shown in the set of construction drawings we reviewed. According to Woodworth, the connection consisted of sprinkling bentonite over the exposed native soil surfaces then placing the clay cap over the PTW; once at grade, the hardstand stone was raked over the clay to provide a good walking/working surface. There is a high probability that surface water can leak through this connection.

5.0 CONCLUSIONS

The pilot PTW has yielded significant information that can be used in the design of full-scale permeable reactive barriers at the WVDP site. The conceptual design of the PTW was based on a sound evaluation of the clinoptilolite to treat Sr-90-affected water and a reasonably good understanding of geologic and hydrogeologic conditions in the area of the installation. The engineering design relied on traditional construction methods that have been used successfully at other sites. Evaluation of the hydraulics in and around the PTW contained in the Hydraulic Evaluation Report indicates that the hydraulic head measured in the PTW is higher than the surrounding aquifer, and there may be only limited groundwater flow through the PTW. This Engineering Evaluation Report has assessed the design and construction of the PTW and has postulated potential causes of the observed hydraulic conditions in and around the PTW. The following conclusions are drawn from this assessment:

1. The sides of the PTW may have a lower than expected hydraulic conductivity because the native soils may have been "smeared" during pile driving and extraction. However, water flow through the sheet pile joints during excavation of the PTW indicates that smearing probably did not eliminate or significantly reduce water flow across the entire interface. Smearing was most likely in the interlayered Slack Water Sequence, where thin layers of fine-grained material smeared across thin layers of coarse grained material.

2. During sheet pile extraction, the volume of the PTW reduced by about 18 percent. We believe this volume loss resulted from the volume of the sheet piles, consolidation and particle breakage of the clinoptilolite, and movement of the clinoptilolite into the roundstone zone. We estimate that the clinoptilolite lost about 7 percent of its volume due to particle breakage, which is a similar volume reduction noted by Rabideau (2000) when samples of clinoptilolite were compacted. Slug tests indicate a hydraulic conductivity of about $1x10^{-3}$ cm/sec throughout the clinoptilolite. Slug tests results for piezometers within the roundstone may be biased because of the piezometer installation methods, however, the hydraulic conductivity in this zone likely is variable and lower in a bulk sense due to clinoptilolite and fines infilling pore spaces in the gravel. The field values are close to the hydraulic conductivity of $4x10^{-3}$ cm/sec measured by Rabideau for compacted clinoptilolite, and are at least two orders of magnitude lower than the hydraulic conductivity of $1.2x10^{-1}$ cm/sec measured for uncompacted clinoptilolite.

3. The PTW was not filled with water prior to extracting the sheet piles. The groundwater would have been under approximately 20 feet of head as the first sheet was extracted. Significant amounts of fines, if present, could have been flushed (at a flow velocity initially several orders of magnitude greater than likely ambient flow velocity) to the sides of the PTW as it filled up. The first sheets were pulled at the west end of the PTW, so the fines would have been flushed to the north, south and east sides. However, we expect that only a limited amount of fines would be present (the fines that were in the delivered product, and the fines generated as the clinoptilolite was dumped in the PTW) as the first sheet pile was extracted, so it is unclear what contribution this mechanism had on the formation of a "skin" around the three sides of the PTW.

4. Comparison of reported as-built conditions and Geoprobe borings performed before PTW construction indicate that the PTW may not extend to the top of the Lavery

Till at the east side of the PTW. Groundwater may be flowing under the PTW in this area.

5. The hardstand surface layer has been recognized as a potential reservoir of surface water that is available to flow into the PTW if a suitable flow path exists. Construction of a surface drain in October 2000 upgradient of the PTW reduced the hydraulic response to rain and snowmelt, indicating that the hardstand is connected hydraulically to the PTW.

6. Potential flow paths exist linking the hardstand surface layer with the PTW. These include an existing conduit trench, the uncompacted clay cap, the connection of the cap to the sides of the PTW excavation and to the PTW materials, and the installation of bumpers through the surface cap into the PTW. Any or all of these flow paths may be contributing to the higher water levels observed in the PTW than in the surrounding aquifer.

This report therefore concludes that the clinoptilolite and roundstone zones within the PTW were probably transformed during sheet pile extraction. The resulting PTW probably contains a comingled zone of roundstone and clinoptilolite, and zones of clinoptilolite around the north and east edges that contain a significant amount of crushed clinoptilolite particles. The hydraulic conductivity of the material is two or three orders of magnitude less than that for uncrushed clinoptilolite. The hardstand is hydraulically connected to the PTW, and numerous potential flow paths have been identified through the cap and other surface features. The PTW does not appear to be fully penetrating through the upper water bearing zone and likely "hangs" in its central and eastern portions above the top of the Lavery Till potentially allowing underflow.

Future permeable reactive barrier (PRB) installations at the WVDP in similar native soils should include consideration of these factors in design and construction.

- Sheet pile cofferdams are probably the most effective means of constructing the PRB, but driving and extraction of the sheet piles will affect local hydraulic conductivity.

- Wing walls should be considered in design to create a remedy that will direct the flow of water through the PRB. Suitable dimensions of the wing wall sections can be developed from careful hydraulic modeling of all anticipated groundwater conditions.

- Geotechnical design should minimize sheet pile penetration of the Lavery Till to reduce the energy required to extract the sheet piles.

- A conservative cap design, using HDPE liners or other impermeable materials should be used to prevent surface water infiltration.

- All possible sources of surface water should be diverted away from the area of the PRB.

- More delicate placement techniques can be developed to place the clinoptilolite.

- The excavation should be filled with water before the sheet piles are removed.

- The divider system between the clinoptilolite and any gravel zones should be removed after the sheet piles are removed.

- Considerations for future designs also should consider using a coarser grain size distribution for the zeolite treatment media, or an aggregate that is less susceptible to grain breakage during construction of the PRB.

6.0 RECOMMENDATIONS

If clinoptilolite fines have lowered the in situ hydraulic conductivity of the PTW materials, a systematic program to further develop the PTW by aggressive pumping from the PVC riser pipe could be designed. However, we understand pumping rates are limited by the apparent plugging of the geotextile around the PVC riser pipe.

Methods could be developed to attempt to remove or penetrate the "skin" of lower permeability material around the PTW. This zone of material could be removed by augering, or a piping system and manifold could be constructed within the clinoptilolite zone to remove water more effectively. Other innovative methods that are being developed by practitioners in the field include *in situ* sonification and *in situ* fluidization which may merit consideration. However, both of these methods have significant limitations (sonification may create more fines; fluidization would require a method to collect the fluidized fines, such as a horizontal well installed near the top of the saturated zone within the PTW) and unpredictable results.We believe that many of these techniques will also generate additional fines, making evaluation of the modified PTW even more complex.

One passive method that could induce flow through the treatment media of the PTWwould be to lower the head within the PTW using a siphon by connecting a new well installed in the center of PTW to a lower head reservoir, such as a downgradient lower elevation ditch. This method, while hypothetically possible, may not be practical considering site logistics.

The potential flow paths connecting the hardstand layer and the PTW can be tested to see which is contributing to the higher water levels within the PTW. While monitoring wells within and near the PTW, fresh water can be introduced into the potential flow paths, such as onto the hardstand directly upgradient from the PTW, or into the conduit trench west of the PTW, or around the bumper posts, or directly onto the capped surface. Through controlled application of water and careful monitoring of water levels in the wells, the specific flow path or paths can be identified. Recommendations can then be developed to address the identified flow paths.

Alternatively, the surface completion over the PTW can be replaced with a carefully designed low permeability cap comprised of a suitable high-density polyethylene (HDPE) liner keyed into the adjacent surface clay layer, with sealed penetrations through the liner for the wells and PVC riser pipe. Bumper posts would be moved away from the PTW zone. A carefully designed and constructed cap should eliminate flow paths from the hardstand to the PTW. Subsurface conditions that could result in higher water levels within the PTW are addressed in the Hydraulic Evaluation report.

7.0 LESSONS LEARNED

This report has focused on the engineering detail of the PTW design and installation. The conclusions of this report will assist in the eventual development of engineering alternatives to enhancing the performance of the PTW. Based on the conclusions of this report and the companion Hydraulic Evaluation report, several "lessons learned" are identified that we believe are important to future PTW design work at the WVDP:

- Site characterization for PTW design purposes must focus on the location of the proposed installation and cannot rely solely on regional information
- PTW design work must include temporal and spatial data on the three-dimensional distribution of target contaminants in the proposed location
- Hydraulic head information should focus locally and include a sufficiently wide area to account for potential spatial (both lateral and vertical) and temporal changes to the direction and magnitude of the hydraulic gradient. This information should

be collected before and after PTW construction so that the effect of the PTW on the local hydraulic regime can be understood.

- Stratigraphic information must be sufficiently detailed in the vicinity of the proposed location to accurately design the PTW for proper vertical coverage, and/or penetration of the affected water bearing zone. This stratigraphic information must also be considered in the engineering designing of the excavation support for the PTW.

- Generally, the use of sheet piles to support the excavation for a PTW will modify the local stratigraphy and may affect discrete flow paths. Removal of sheet piles will consolidate any loose or uncompacted material in the PTW, and will allow materials with dissimilar grain sizes to co-mingle. Sheet pile removal may also generate high dynamic stresses within the PTW materials that can break fragile particles within the PTW.

- The hydraulic head within a PTW excavation should be maintained at the top of the emplaced material when the excavation support system is removed to reduce the potential for rapid inflow of water and turbulent flow conditions. These conditions may mobilize fines or other materials within the PTW during removal of the excavation support.

- The performance of a PTW can be affected by numerous external factors, such as surface water infiltration, utility trenches, etc., that must be addressed during the detailed engineering design. Given the high cost of installation, monitoring and correcting any performance problems, the engineering design should be conservative in addressing site-specific issues that could affect PTW performance. For example, an HDPE liner placed over the PTW treatment materials and appropriately keyed into the surrounding native material would be a more effective cap than the granular bentonite and uncompacted clay cap that was installed.

- If the PTW does not perform as-designed, accurate and well-documented as-built information is critical in understanding the problem and developing suitable remedies. When constructing a pilot PTW to evaluate the technology, this information is even more important.

The conditions and issues that have reduced the intended performance of the pilot PTW also have been problematic for other sites that have deployed PTW technology over the past 10 years. Generally, the remedial effectiveness of PTW technology, from a chemical standpoint, that is, destruction of organic compounds, immobilization of inorganic compounds, buffering of low pH conditions, has been demonstrated and is fairly well understood thanks to the thousands of laboratory-scale, hundreds of pilot tests, and nearly 50 full-scale PTW implementations that have been reported by groups such as the Remediation Technology Development Forum (e.g., see http://www.rtdf.org). We have not seen any reports of full-scale

PTW failures due to chemical treatment inadequacies although laboratory bench-tests have shown limitations to various chemical treatment processes. Issues regarding plugging or fouling of a PTW from chemical processes are being studied and monitored at both research and full-scale commercial sites; however, these processes are anticipated and have apparently not yet diminished the effectiveness of a PTW for a given site.

As is the case for the pilot PTW at WVDP, most difficulties with PTW operation to-date generally are due to unintended hydraulic performance with the specific performance inadequacies resulting in:

- incomplete capture of the affected groundwater (e.g., flow around or below, the PTW).
- design groundwater velocity not being achieved.
- non-uniform flow conditions within the PTW.

From our experience with PTW design and assessment, our participation in the RTDF and development of the U.S. Environmental Protection Agency training course on PTW technology, and our general review of the state-of-the practices, we have developed the following list of lessons learned:

- A comprehensive site characterization program that provides detailed stratigraphic information along and in the vicinity of the proposed PTW alignment is critical to developing a reliable hydraulic and geotechnical PTW design. Incomplete site characterization is the primary cause of hydraulic failure in existing PTW sites. Information on the depth and location of primary, and discrete groundwater flow paths, temporal and spatial variability in three-dimensional hydraulic head information, and reliable detail on the expected range of hydraulic conductivity values must be collected.
- A three-dimensional groundwater flow model that can reliably interpret the spatial and temporal variability in site conditions is critical to designing a reliable PTW. Temporal and spatial variability in hydraulic head, direction and magnitude of hydraulic gradients, and contaminant flowpaths can be addressed by a representative model
- The PTW deployment approach must take greater care to avoid potential "skin" effects and pulverization of treatment material, and creation of fines. Many conventional implementation methods (e.g., sheet piled excavation and fill; trenching machine) have the potential for creating a skin across preferential flow paths. Integration of the site characterization, which should identify the degree of heterogeneity with the aquifer system and potential for smearing due to clay and silt

seams in the subsurface, with the engineering design can help reduce the potential for a skin to develop and control the hydraulic performance of the PTW. Different deployment scenarios, including modifications of geometry, orientation, and grain size distribution of the treatment material with a PTW can be considered.

- PTWs that hang, or do not completely penetrate an underlying low hydraulic conductivity unit, over all or part of their alignment,generally have greater potential for unintended performance than fully-penetrating PTW designs.

- Continuous wall PTW designs (similar to the WVDP pilot) typically are less complicated to design and build than "funnel and gate" designs; however, the continuous wall must fully capture the affected groundwater, including during variations in the direction of the lateral hydraulic gradient. Assessing the performance of a continuous wall design is scale dependent; that is, short walls that do not fully cover the width of a plume may not be able to handle the inherent heterogeneity of an aquifer system and provide adequate treatment. Also, because any emplaced engineering structure such as a PTW changes the ambient flow field, shorter walls may have a tendency to result in greater relative changes to the flow field than longer walls.

- Trench and fill type PTWs always create unconfined head conditions; this must be taken into account in any PTW design as the creation of a unconfined trench in a semi- or completely confined aquifer can affect the hydraulic conditions and flow field within and around the PTW.

Examples and lessons learned from reports on specific sites are summarized in the following paragraphs. Generally, these summaries can be found courtesy of the Remediation Technologies Development Forum, Permeable Barriers Action Team website (of which Geomatrix is a member), http://www.rtdf.org/public/permbarr/prbsumms/default.cfm

U.S. Department of Energy, Kansas City Plant, Kansas City, Mo (information source RTDF) *(hydraulic head redistribution and diversion of groundwater flow)*

A PTW was installed in April 1998 at the U.S. Department of Energy's Kansas City Plant in Kansas City, MO. Contaminants of concern include 1,2-dichloroethylene (1,2-DCE) and vinyl chloride (VC). Maximum initial concentrations encountered at the site were 1,377 μg/L of 1,2-DCE and 291 μg/L of VC. The PTW was constructed as a continuous trench measuring 130 ft long. Sheet piles were driven into bedrock to support the side walls. The resulting excavation was 6 ft wide. The first 6 ft of the trench above bedrock was filled with 100% zero-valent iron. The remainder of the trench was filled with 2 ft of zero-valent iron and 4 ft of sand. These differing thicknesses were used to compensate for the increased flow-through thickness required for the basal gravel unit. Data evaluation indicated that flow around the wall's south end was caused by head redistribution. The wall acts somewhat like an equalization tank redistributing heads. Flow gradient into the north end of the wall is approximately four times higher than at the south end. Therefore some of the groundwater flow at the south end is redistributed around the wall. Potential remedies to treat flow around the wall's south end include: 1) a cut-off wall across the permeable barrier to prevent head redistribution, 2) a cut-off wall or permeable barrier at the south end to direct groundwater flow back into the wall, or 3) extension of the iron treatment wall to the south. Lesson learned: Installation of the continuous permeable barrier can cause a redistribution of heads and a partial change in plume direction.

Former Manufacturing Facility, Sunnyvale, California (information source, Geomatrix)

A PTW was installed in November 1994 at a former semi-conductor manufacturing site in northern California. The PTW, composed of zero-valent iron sandwiched between up and downgradient pea gravel sections, is successfully treating chlorinated VOCs. The site replaced a former pump and treat remedy. The system includes a 38 foot long by 22 foot deep by 8 foot wide PTW cell, with lateral low permeability barriers extending upgradient more than 250 feet on either side of the PTW to direct groundwater flow and reduce affects from changing hydraulic gradient directions. A short (20 foot) downgradient sheet pile on one side of the PTW reduces potential non-uniform flow conditions associated with the variable ambient flow direction. Hydraulic mounding has occurred following extended precipitation events; the mounding is temporal and dissipates following the rainy season. Transient response to regional and local precipitation and pressure head changes between the ambient semi-confined aquifer and the constructed unconfined PTW are believed to contribute to these conditions. Because these conditions are temporal and dissipate, modifications to the system are not currently required.

U.S. Coast Guard Support Center, Elizabeth City, NC (information source: RTDF and U.S. EPA National Risk Monitoring Laboratory, Ada, Oklahoma

A full-scale demonstration of a PTW to remediate ground water contaminated with chromium and chlorinated organic compounds was initiated at the U.S. Coast Guard Support Center site in Elizabeth City, NC, in 1995. The primary contaminants of concern are hexavalent chromium (Cr^{+6}) and trichloroethylene (TCE). Initial maximum concentrations were more than 4,320 μg/L for TCE and more than 3,430 μg/L for (Cr^{+6}). The contaminant plume was estimated to cover a 34,000-ft^2 area. The plume is adjacent to a former electroplating shop that operated for more than 30 years prior to 1984 when operations ceased. Ground water begins approximately 6 ft below ground surface, and a highly conductive zone is located 16-20 ft below the surface. This layer coincides with the highest aqueous concentrations of chromium and chlorinated organic compounds found on the site. A low-conductivity layer—clayey, fine sand to silty clay—is located at a depth of about 22 ft. This layer acts as an aquitard to the contaminants located immediately above. A continuous wall composed of 100% zero-valent iron (Fe^0) was installed in June 1996 using a trencher that was capable of installing the granular iron to a depth of 24 ft. The continuous trenching equipment used for the installation has a large cutting chain excavator system to remove native soil combined with a trench box and loading hopper to emplace the iron. The PTW is approximately 2 ft thick and about 150 ft long. Researchers are investigating the possibility that the TCE plume has dipped lower in the aquifer after the wall was installed and is now moving under the wall. A significant amount of recharge occurred into the reaction zone following installation due to removal of the concrete parking lot covering the site. This recharge may have driven the plume deeper than had previously been observed allowing some of the plume to move under the wall. Smearing at the interface between the PTW and the native material may have occurred during construction, however, there is little indication at this time that such smearing, if it does occur, has significantly affected performance of the PTW

U.S. Department of Energy, Fry Canyon Site, UT (source: RTDF and U.S. Geological Survey)

A field-scale demonstration of a PTW system is underway at an abandoned uranium upgrader site in Fry Canyon, UT. The U.S. Environmental Protection Agency (EPA) is the lead agency on the site. The ultimate goal of the demonstrations is to determine the technological and economic feasibility of using permeable chemical or biological obstacles, placed in the flow path, for removing dissolved metals and radionuclides from contaminated ground water. This project is testing the performance of three permeable reactive barriers at the Fry Canyon site. Anticipated results of the research for each of the PTW tested will include long-term removal efficiencies for uranium and an evaluation of the commercialization potential for each. Specific objectives of the field demonstration project include: (1) hydrologic and geochemical characterization of the site prior to emplacement of barriers; (2) design, installation, and operation of three PRBs; and (3) evaluation of barrier(s) performance and commercialization potential. At the Fry Canyon site, the water table is located approximately 8 ft to 9 ft below ground surface, and the underlying aquifer ranges from 1 ft to 6 ft deep. Estimated hydrologic properties and measured hydraulic gradients indicate that ground water in the alluvial aquifer moves at a rate of about 1.5 ft/day nearly parallel to the direction of stream flow.

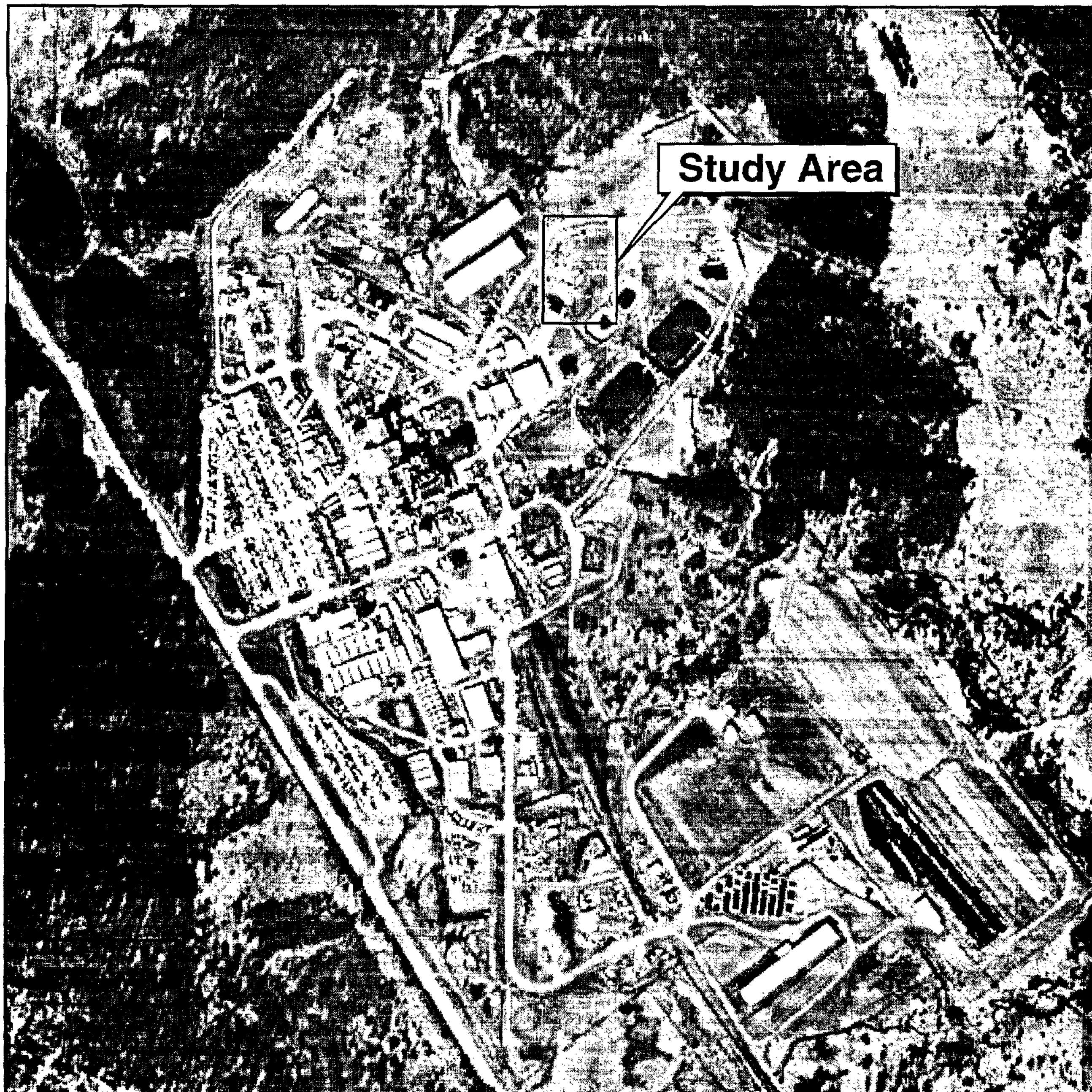
The system has successfully shown the removal efficacy of several reactive media. The following performance issues also are being assessed:

1) In a low-gradient system like Fry Canyon, it is difficult to estimate mass of treated water and, at times, whether there is even flow getting through some of the gate structures. This presents an unknown to regulators in estimating total mass of contaminant that will be cleaned up per unit of time since PTW deployment.
2) Seasonal changes are apparent in the PTWs' efficiency in removing uranium. The processes causing these changes need to be identified in order to effectively determine long-term clean-up goals.
3) PRBs that are placed adjacent to ephemeral channels could be destroyed or have their long-term function significantly compromised during intense thunderstorm events in the Fry Creek drainage basin without proper erosion control measures.
4) Ground settling could compromise the lack of visual impact that PRBs have in future remediation applications and could impact monitoring wells.

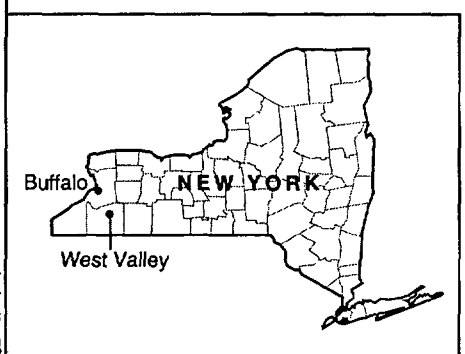
Other sites, including a pilot test of a landfill in the northeast U.S. and Federal Facility in

Colorado, have observed hydraulic effects due to the presence of skin (referenced reports not available). In each case, (one a pilot caisson installation, one a full-scale sheet-pile reactive gate installation) skin was speculated to divert flow or create mounding. The skin effect for the pilot test was apparently not remedied. A remedy for the full-scale reactive gate that involved siphoning water from the interior of the PTW reactive zone around the skin was apparently designed; the skin, in this case, was not removed.

The main point of these examples is that designing for hydraulic performance is critical to any PTW application. Comprehensive site characterization is key, and will more likely result in a reliable PTW design that becomes a cost-effective remedy for a given site.



Source: www.terraserver.microsoft.com



GEOMATRIX

SITE LOCATION MAP
West Valley Nuclear Services
West Valley, New York

Project No.
7061

Figure
1.1

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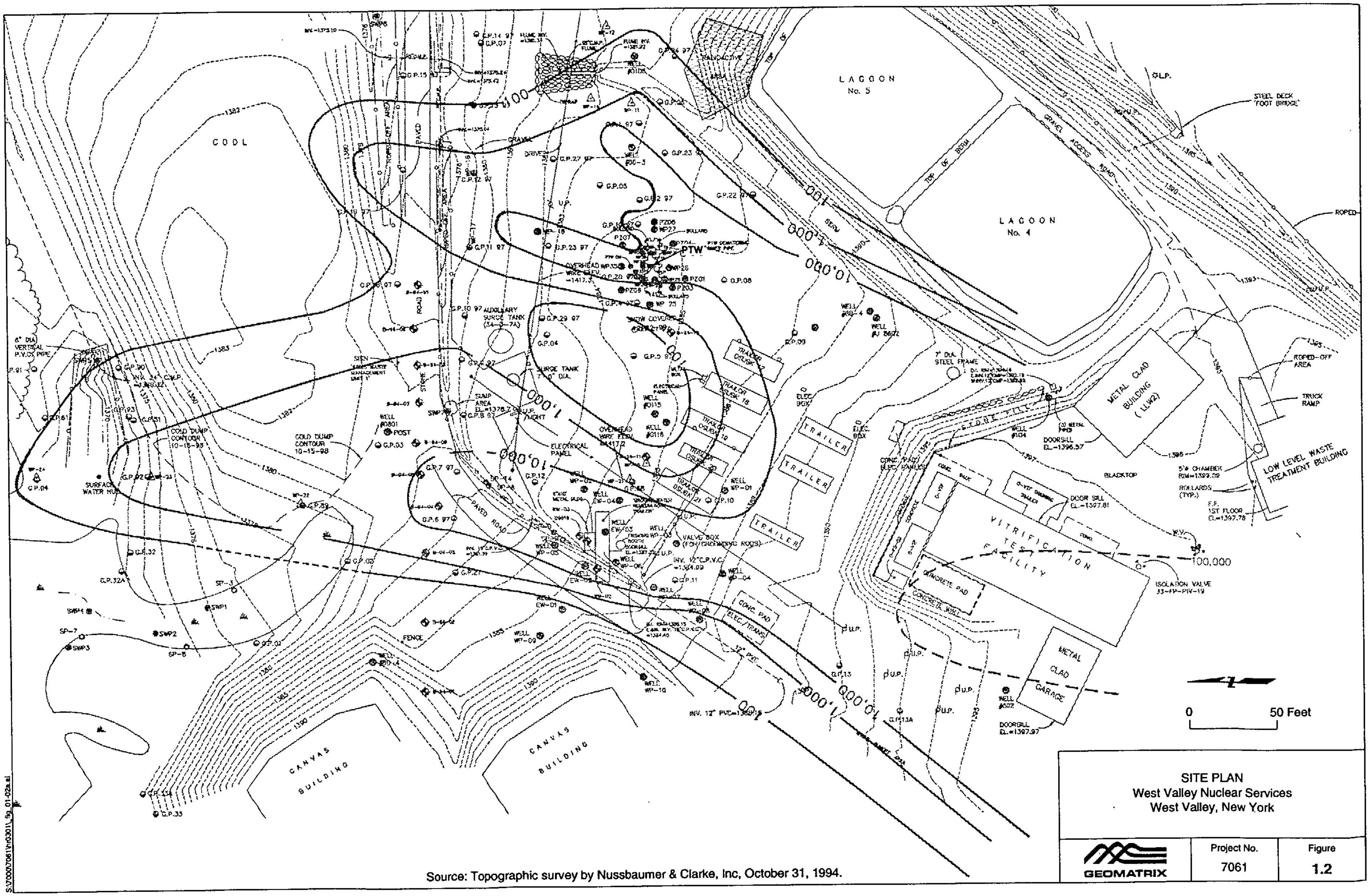
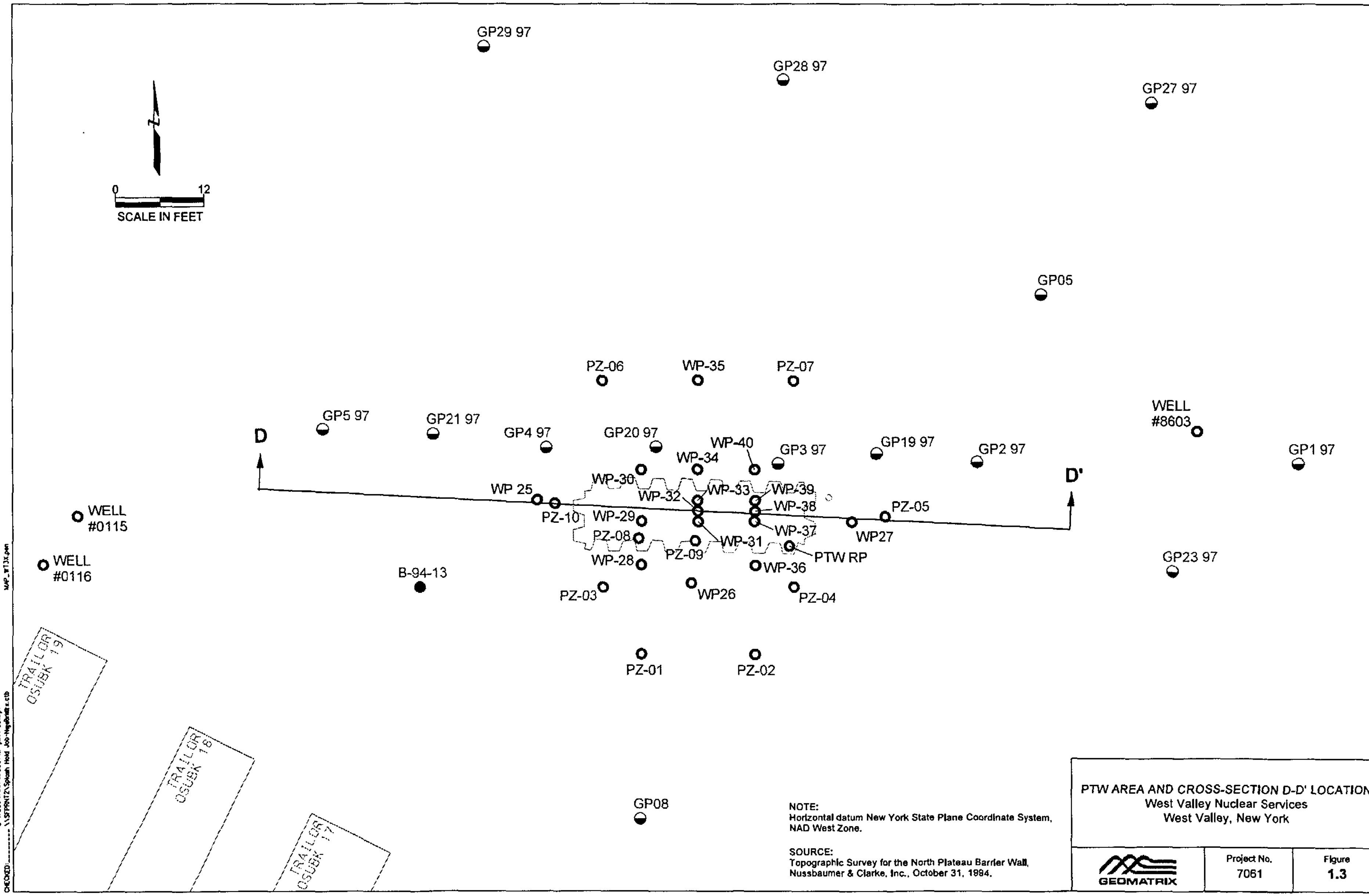


SITE PLAN
West Valley Nuclear Services
West Valley, New York

GEOMATRIX

Project No.	Figure
7061	1.2

Source: Topographic survey by Nussbaumer & Clarke, Inc, October 31, 1994.

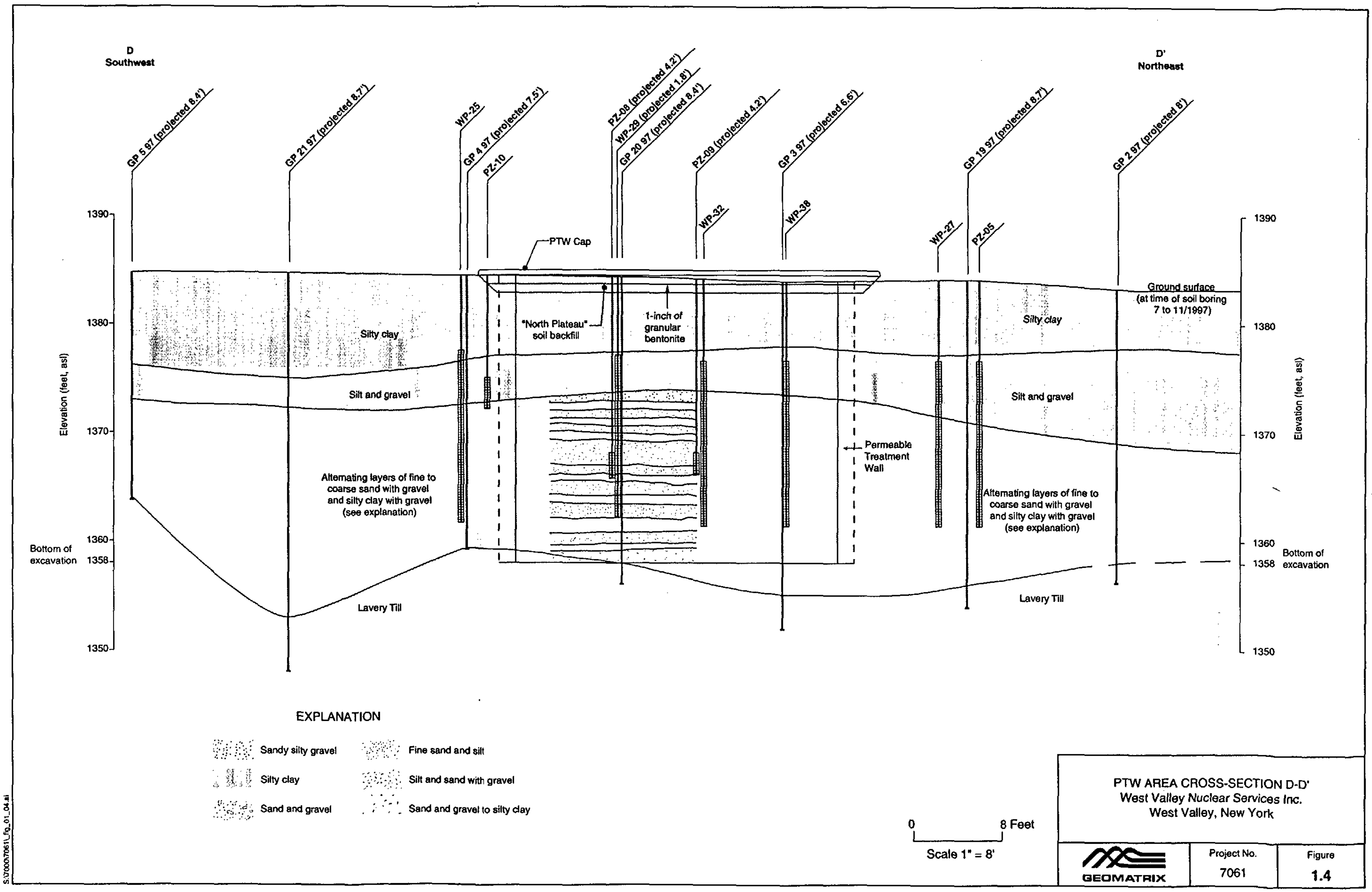


NOTE:
Horizontal datum New York State Plane Coordinate System, NAD West Zone.

SOURCE:
Topographic Survey for the North Plateau Barrier Wall, Nussbaumer & Clarke, Inc., October 31, 1994.

PTW AREA AND CROSS-SECTION D-D' LOCATION
West Valley Nuclear Services
West Valley, New York

GEOMATRIX	Project No. 7061	Figure 1.3



PTW AREA CROSS-SECTION D-D'
West Valley Nuclear Services Inc.
West Valley, New York

GEOMATRIX

Project No.	Figure
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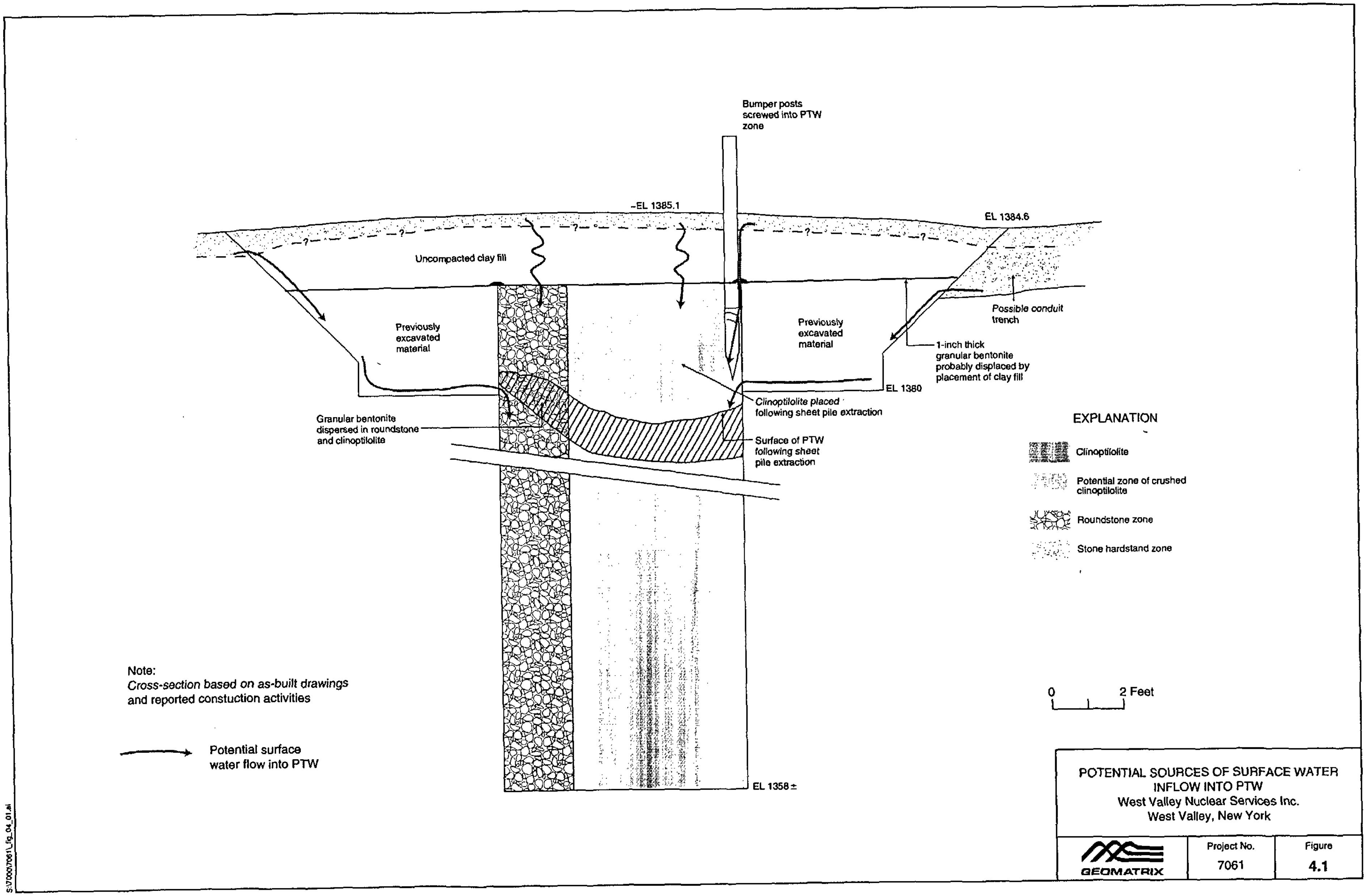

POTENTIAL SOURCES OF SURFACE WATER
INFLOW INTO PTW
West Valley Nuclear Services Inc.
West Valley, New York

GEOMATRIX	Project No.	Figure
	7061	4.1

APPENDIX 12
GEOMATRIX REPORT: PILOT PERMEABLE TREATMENT WALL MODIFICATION OPTIONS REPORT

GEOMATRIX

Pilot Permeable Treatment Wall Modification Options Report

West Valley Nuclear Services, LLC
West Valley, New York

Prepared for:

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April 2001

Project No. 7061.000

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PILOT PERMEABLE TREATMENT WALL MODIFICATION OPTIONS REPORT

West Valley Nuclear Services, LLC
West Valley, New York

1.0 INTRODUCTION

This *Pilot Permeable Treatment Wall, Modification Options Report* was prepared by Geomatrix Consultants, Inc. (Geomatrix) at the request of West Valley Nuclear Services, LLC (WVNS). The report was commissioned by WVNS (Project 19-098745-C-JK) to assist in recommending options for assessing and enhancing the performance of a pilot permeable treatment wall (PTW) designed to remediate groundwater affected by radioactive Strontium-90 (Sr-90) beneath a portion of the West Valley Demonstration Project (WVDP) located in western New York state (Figure 1.1). Two supporting reports, the *Pilot Permeable Treatment Wall Hydraulic Evaluation Report,* (Geomatrix, 2001a) (the Hydraulic Evaluation Report) and the *Pilot Permeable Treatment Wall Engineering Evaluation Report* (Geomatrix, 2001b) (the Engineering Evaluation Report) also were prepared by Geomatrix and previously submitted to WVNS.

Each of these reports, and the opinions and recommendations provided within, are based on our review and evaluation of data and other relevant documentation on the pilot PTW program at provided by WVNS, as well as technical discussions held with WVNS staff and their contractors.

A pilot PTW was installed by WVNS at WVDP in Fall 1999 to assess the ability of the technology to passively and effectively reduce the concentration of Sr-90 affected groundwater. The pilot PTW was installed to treat a portion of the "2nd lobe" of the Sr-90 plume beneath the North Plateau of the site (Figure 1.2). Figures 1.3 and 1.4 show the monitoring well network and a cross-section of the PTW area. The "1st lobe" of the Sr-90 plume to the west currently is being remediated by a groundwater recovery and aboveground ion exchange treatment system ("pump-and-treat") that was installed in 1995. While the pump-and-treat remedy reportedly reduces local migration of the Sr-90 plume, WVNS does not consider it capable of completely capturing and remediating the affected groundwater beneath the North Plateau. Thus, WVNS identified PTW technology as a method potentially capable of effectively mitigating further migration of Sr-90 affected groundwater.

Monitoring data collected and analyzed by WVNS indicates that the pilot PTW may not be functioning as designed; specifically affected groundwater from south, and presumed down hydraulic gradient side of the PTW may not be flowing through the pilot PTW. The Hydraulic Evaluation and Engineering Evaluation Reports assessed the hydraulic performance and construction methods of the pilot PTW using data and relevant information provided by WVNS. The analyses considered preliminary results of groundwater modeling of the system being performed by researchers at the State University of New York at Buffalo (UB). The reports also provided possible explanations for the observed performance of the pilot PTW as well as recommendations for collecting additional data to both confirm the explanations and provide information for developing an engineering solution.

1.1 PILOT TEST OBJECTIVES AND ASSESSMENT

We understand that the purpose of the PTW pilot test program at WVDP is to assess the feasibility and practicality of mitigating Sr-90 affected groundwater using PTW technology. Pilot tests are typically performed for proving the efficacy of a remediation method under field conditions if: (1) the technology relies on innovative treatment methods where performance data is not readily available for a wide variety of sites; (2) the installation methods proposed have not been tested elsewhere; and (3) unique field conditions require a pilot test for developing additional data designing the full-scale application. We believe that performing the pilot test at WVDP was well founded based on these considerations because no other full-scale PTW systems have been deployed, to our knowledge, to mitigate Sr-90 affected groundwater, although laboratory studies have been performed by several organizations, and at least one other pilot test is being performed in North America.

Data Quality Objectives (DQOs) were developed by WVNS (Report WVDP-350) in June 1999 for assessing the performance of the pilot PTW. These DQOs included: (1) establishing groundwater flow through the pilot PTW; and (2) providing treatment of the Sr-90 affected groundwater by the pilot PTW clinoptilolite (zeolite) treatment media. The specific criteria for assessing the performance of the pilot PTW based on the DQOs, therefore, include:

1. Determining if groundwater flows through the PTW and is not backed up or diverted around the pilot PTW.

2. Determining if Sr-90 activity in groundwater is reduced as the groundwater passes through the pilot PTW.

3. Comparing Sr-90 activities up and downgradient of the PTW to assess the potential for mitigation of downgradient groundwater (this activity was identified as possibly continuing beyond the initial pilot test performance program).

To assess these DQOs, specific criteria were developed by WVNS including:

- Contouring groundwater head data to evaluate the direction of groundwater flow near and within the pilot PTW
- Identifying whether Sr-90 activity in groundwater within the PTW is reduced to less than 1000 to 1500 picocuries per liter (pCi/L).

The evaluations performed by WVNS and others to-date indicate that only DQO #2 above may have been met, although there is no confirmation that the low Sr-90 activity groundwater sampled within the pilot PTW is the result of treating previously high Sr-90 activity groundwater from upgradient sources.

Beyond the DQOs specified by WVNS for the pilot PTW project, a pilot program also provides other useful information for designing and deploying a full-scale system, including:

- effects of construction activities on the native hydraulic system;
- details of the local hydrostratigraphic characteristics;
- geotechnical information;
- effects of recharge on hydraulic performance.

It is important to note that *the performance of a pilot test does not need to be perfect for it to provide useful information* for either making a go/no-go decision on future deployment, or for designing a successful full-scale system as long as the test: (1) identifies the specific and unique site and engineering characteristics that affect system performance, and (2) provides information useful for overcoming performance deficiencies.

DQO #1—hydraulic efficacy—is the most important metric that appears not to have been met by the pilot PTW thus far. From our analysis of the hydraulic and engineering information provided by the pilot PTW (as detailed in Geomatrix 2001a and 2001b and summarized in Sections 1.4 and 1.5 of this report), we conclude that the unintended hydraulic performance of the pilot PTW may be due to specific design and construction factors, including:

1. a short, continuous wall PTW that: (a) may not be oriented perpendicular to the local lateral hydraulic gradient direction, (b) has no lateral hydraulic control; and

(c) may not fully penetrate the complete thickness of the affected aquifer (i.e., the PTW may "hang" above the underlying till in its central and eastern portions);

2. recharge of surface water directly into the pilot PTW;

3. a discontinuous "skin effect" that prevents efficient groundwater flow through the PTW and may have developed due to specific construction activities.

We do not believe that a technically feasible and cost-effective method or approach exists that can completely eliminate these factors from the existing pilot PTW. We also do not believe that all shortcomings or potential causes of the poor PTW performance must be corrected to render the pilot test successful. However, we have developed recommendations for providing additional data and engineered modification alternatives to both confirm and enhance the hydraulic performance of the pilot PTW.

1.2 ORGANIZATION OF REPORT

Following this introductory section, which also includes summaries of pertinent information from the Hydraulic Evaluation and Engineering Evaluation Reports, this report consists of the following Sections:

- Section 2.0 Pilot PTW Modification Options (including a description of the alternative; the basis for selection, assumptions, rough cost estimate, assessment of future performance, waste generation, ease of implementation, and likelihood of success).

- Section 3.0 Conclusions and Recommendations (of the preferred alternative, and the recommended course of action).

1.3 PERTINENT ASPECTS OF THE HYDRAULIC EVALUATION REPORT

This section summarizes pertinent aspects of the Hydraulic Evaluation Report that are important for selecting an appropriate path forward in the pilot test program. The Hydraulic Evaluation concluded that the performance of the PTW likely is controlled by:

- a more eastward groundwater flow direction than initially anticipated (the PTW was oriented for northward flow);

- a highly heterogeneous aquifer sequence of fine and coarse sediments;

- a narrower than anticipated flow zone of high activity Sr-90 water near the base of the aquifer, with generally lower activity groundwater at the west end and higher activity groundwater at the east end of the PTW; this flow appears to be partially diverted around the east end of the PTW;

- the PTW may not have fully penetrated the aquifer, and the "hanging" central and eastern portion of the PTW may allow underflow of high Sr-90 activity groundwater;

- a discontinuous skin of material at the contact with the native aquifer material and heterogeneity within the PTW treatment material resulting from fines created during installation activities; and,

- reduced, though continuing (primarily during precipitation or runoff events), direct surface water infiltration into the pilot PTW.

1.4 Pertinent Aspects of the Engineering Evaluation Report

This section summarizes pertinent aspects of the Engineering Evaluation Report that are important for selecting an appropriate path forward in the pilot test program. The Engineering Evaluation Report concluded that the following construction-related activities and methods likely contributed to the observed performance of the pilot:

- the zeolite and roundstone zones within the pilot PTW were likely transformed during the construction activities to a more homogeneous mixture of roundstone and fine (some crushed) zeolite particles which lowered the effective hydraulic conductivity along the south face of the pilot PTW;

- the PTW does not appear to be fully penetrating through the upper water bearing zone and likely "hangs" in its central and eastern portions above the top of the Lavery Till potentially allowing underflow;

- the installation and removal of the sheet piles likely altered local but significant hydrostratigraphic zones and flow paths adjacent to the pilot PTW that control the migration of Sr-90 within the upper water bearing zone;

- both the orientation (at an angle to the local lateral hydraulic gradient) and the relatively short length of this "continuous wall" pilot PTW contribute to the unintended apparent "deflection" of groundwater around the system;

- the creation of a confined permeable trench within an otherwise semi-confined system may contribute to unintended transient hydraulic effects (including minor mounding) within the pilot PTW; and,

- the surface hardstand is hydraulically connected to the pilot PTW, and numerous potential flow paths have been identified through the cap and other surface features.

1.5 ADDITIONAL DATA NEEDS

The Hydraulic Evaluation and Engineering Evaluation Reports recommended the collection of additional data that would: (1) address the possible causes of the unintended performance of the pilot PTW and (2) be used to assist in designing a successful engineering solution for the pilot PTW. We emphasize the importance of collecting additional information to assess and confirm both the hydraulic performance of the pilot PTW, and to support the selection of an engineering alternative.

The following data collection activities were recommended in the Hydraulic Evaluation and Engineering Evaluation Reports.

1. Install two or more new wells and collect stratigraphic information in the vicinity of WP-25 (west end) and WP-27 (east end) to confirm the water level conditions that appear to provide major control on the assumed groundwater flow directions in the vicinity of the PTW.

2. Drill at least three additional soil borings around the eastern end of the pilot PTW (one each adjacent to the north, south, and eastern face) that are logged for stratigraphic detail to confirm whether the pilot PTW penetrates the underlying till or is "hanging" within the aquifer. One additional soil boring for stratigraphy should be drilled toward the western end of the PTW, and one additional boring should be drilled near the south face of the PTW. These borings should be converted to 2-inch monitoring wells for subsequent aquifer testing.

3. Perform a long-term aquifer test consisting of a series of step-test periods, a constant-discharge period, and recovery period, to provide additional data on the hydraulic connection of the PTW to the surrounding aquifer.

4. Perform one or more tracer tests to provide field evidence of the flow field that has been interpreted from water levels in the vicinity of the pilot PTW.

5. Develop a comprehensive three-dimensional numerical model that can best assess and predict the observed hydraulic conditions, and can thus be integrated into design activities for developing the engineering modification or alternative, and eventually, the full-scale design.

Prior to selecting and subsequently designing an engineered modification alternative, we recommend performing a focused data collection and analysis program. The basis for such a program is described in the following paragraphs.

Note that completion of the focused data collection program may show that the hydraulic communication is better than previously indicated. In this case, the overall pilot program may be sufficient to move to a full-scale design program without implementing an engineering solution to enhance the performance of the pilot PTW. If the results of the data collection program confirms that hydraulic communication is insufficient for making decisions regarding the feasibility of a full-scale system, an engineering alternative may be justified.

1.5.1 Description

The focused data collection and analysis program would be designed to: (1) confirm the hydraulic conditions in and adjacent to the pilot PTW; and (2) collect key hydrostratigraphic and hydraulic information necessary for determining the hydraulic effectiveness of the pilot PTW and for use in selecting, designing, and assessing the performance of engineering modifications. A specific objective of this program is to confirm the high head potential measured at well WP-25 and to determine whether groundwater flow from south of the pilot PTW is occurring. A second objective will be to confirm the elevation of the top of the Lavery Till to determine whether the pilot PTW is hanging in its central and eastern portions.

The components of this program include:

1. Borehole drilling and logging, monitoring well installation, and water level monitoring.
2. Systematic and focused hydraulic testing program.
3. Tracer test program.

Details of the components are provided in the following paragraphs.

Component 1: Borehole drilling, well installation, and water level monitoring program

This component will consist of a program to develop additional hydrostratigraphic information and to confirm water level and hydraulic gradient (lateral and vertical) conditions adjacent to the pilot PTW. Figure 1.5 indicates the approximate locations of the seven recommended borings. Detailed hydrostratigraphic boring logs will be produced for each boring, and the cross-sections developed for the Hydraulic Evaluation Report will be updated Although the specific drilling and well completion method would be specified in a work plan developed prior to conducting the field work, we recommend using coring for developing representative and detailed stratigraphic logs and we recommend completing the borings as 2-inch (or greater) diameter wells to assure reliable hydraulic communication with the native hydraulic system during water level monitoring and hydraulic testing. As a narrow diameter (i.e., 1 inch-diameter) well commonly is installed using direct push methods, the potential for compromising hydraulic communication with the native system is greater than for larger diameter wells due to a propensity for smearing of the narrow-diameter well screen and well damage.

Four of the boring/wells will be located along the western, southern and eastern sides of the pilot PTW. These wells (WV-1, WV-2, WV-4, and WV-5) will be completed similarly to other borings with approximately 15-foot well screens within the saturated zone. The three additional borings will be completed as shorter-screened wells (WV-3A, WV-3B, and WV-6) (5-foot well screen) and would be installed at various depths (WV-3A screened 8-13 ft bgs, WV-3B and WV-6 screened 17-22 ft bgs) as well pairs to provide vertical gradient information. Water levels and groundwater sampling for these new wells would be integrated into the network-wide monitoring program. We recommend a monthly water level and sampling program for a three-month period.

Component 2: Focused hydraulic testing program

This component will consist of a focused hydraulic testing program with two primary objectives: (1) to confirm the influence and distribution of skin at locations around the pilot PTW; and (2) to better assess the distribution of hydraulic conductivity in and around the pilot PTW. The testing will consist of a series of step-rate pumping tests and a longer constant rate test. Step-rate pumping tests are performed by pumping from a well at three successfully greater flow rates for relatively short durations (approximately 2 hours each); a water level recovery phase ensues immediately following the last step. Water levels are monitored in the pumping well and nearby observation wells. The step test, which provides specific capacity information, can provide empirical estimates of hydraulic conductivity and can stress the system in such a way that boundary effects may be observable. The step testing also provides information for selecting an appropriate pumping rate for the ensuing constant-rate test. The short duration of the testing (we recommend testing one well per day) allows broad coverage.

We recommend performing step testing in 2-inch diameter wells and at the location of wells WP-25, WP-28, WP-27 and WP-34 (or the new wells installed during Component No. 1).

A longer duration constant rate test, based in part on results from the step-testing, is recommended to provide critical information to assessing the presence of skin and boundary conditions in and near the pilot PTW. We recommend performing two constant rate tests lasting approximately 12 hours each followed by a 12 hour recovery period; one at the location of well WP-28, and one at the location of well WP-34.

We recommend instrumentation of nearby monitoring wells both inside and outside the pilot PTW with pressure transducers to continually monitor water level responses to pumping. We assume that up to 10 wells would be instrumented. Details for instrumenting the wells, and the sequence of the testing would be detailed in a workplan developed prior to implementing this program. A report detailing the testing and analysis would be developed following the field program.

Component 3: Tracer testing program

A limited tracer testing program will provide direct evidence of the flow pathways in and around the pilot PTW. We recommend introducing tracer into three wells simultaneously and monitoring observation wells using available field equipment, including ion specific meters.

The existing wells that should be considered for the testing program include WP-25 (west end); WP-28, south side, and WP-36, south side. Newly installed wells under Component No. 1 may also be considered for testing.

Tracer solutions that are conservative with respect to the Pilot PTW should be applied. Because the clinoptilolite material within the pilot PTW has a slightly negative charge and thus is prone to ion exchange reactions with positively charged ions, the tracer should consist of a solution that is anionic (such as bromide or chloride). Dyes may be considered as well, however, additional information as to their potential retardation within the zeolite media is required and can be included in a work plan developed for the testing program.

Assuming bromide and a dye are used as tracers, the tracer program would involve injecting a bromide tracer solution in Well WP-25 and well WP-26, and a dye in well WP-28. Field equipment, including ion specific electrodes and colorimeters would be used to analyze samples collected from a series of wells (we assume 10) in and around the pilot PTW on a regular schedule. Specifics of the testing program would be provided in a test workplan prepared in advance of commencing this program.

Analysis would consist of developing breakthrough curves for each of the observation wells to assess the travel path and migration rates of the tracer in the system. A report documenting the test would be developed following the program.

The results of this program could provide information to definitively determine the communication conditions between the pilot PTW and the native aquifer material, as well as indicating the areas likely affected by skin.

This focused data gathering program is recommended because of the importance of gathering data that can be used to confirm: (1) the high head at well WP-25 adjacent to the western portion of the pilot PTW; (2) whether the PTW is "hanging" near its central and eastern sections; (3) the hydraulic efficacy of the pilot PTW in its current state without deploying invasive engineering activities. We also understand, based on recent discussions with researchers at UB, a calibrated hydraulic model of the pilot PTW system has not yet been completed due in part to difficulty in accounting for apparent data inconsistencies. Therefore, this field testing and data gathering program would be critical to developing a representative model that could be used for further evaluating and selecting engineered pilot PTW modification alternatives, as well as being useful for designing a full-scale PTW system.

2.0 PILOT PTW MODIFICATION OPTIONS

Four alternatives for the pilot PTW program are presented in this section; one "no additional work" alternative, two engineering modification alternatives, and one alternative that comprises a complete rebuild of the pilot PTW. We emphasize that the objective of the two engineering modification alternatives is to modify the PTW so that sufficient information can be obtained from the installation to allow design of a full-scale PTW at WVDP. The objective is not to create a perfectly functioning pilot PTW.

We do not recommend attempting to remove the postulated low-permeability zone around two or more sides of the PTW (the "skin"), or to prevent potential underflow beneath the PTW in areas where the PTW may not extend to the underlying Lavery Till. WVNS has already attempted to dislodge the skin through aggressive pumping of water within the PTW, with limited success. Other, more invasive methods, such as disturbing the skin area with drilling or other tools, will crush additional clinoptilolite, generating more fines that will only add to the skin. Methods that involve excavation of the clinoptilolite below the water table (such as a passive piping and manifold system to move water through the skin-zone) require shoring that will again generate more fines as the sheet piles are driven and extracted. Similarly, any invasive methods that could reduce potential underflow, such as grouting, would require drilling or penetration through the clinoptilolite zone, creating more fines. We suggest that these methods will introduce additional complicating variables to an already complex system, and therefore we recommend implementing simple solutions that allow sufficient information to be obtained to complete a full-scale PTW design.

Each alternative is presented according to the requirements of the scope of work contained in the Geomatrix contract. A description of the alternative is first presented, followed by the basis for selecting the alternative. Assumptions made in developing the alternative and the associated cost estimate are then described. A general cost estimate based on REM IV guidelines is developed for each alternative; these cost estimates should be considered "order of magnitude" estimates because we are not familiar with WVNS contracting and procurement procedures for civil construction work. Any waste materials that may be generated are described, (though costs for managing potentially radioactive spoil material are not included) along with the relative ease of implementing the alternative. Finally the likelihood of success is evaluated.

2.1 CRITERIA FOR SELECTING PILOT PTW MODIFICATION OPTIONS

The criteria we have used in selecting potential alternatives for meeting the goals of the pilot PTW test are based on simple, relatively low cost strategies that can be used to provide the information necessary to assess the feasibility of a full-scale PTW at the site.

The basic criteria used for identifying each alternative are listed below.

- Does the alternative address and mitigate the possible high head potential at the west end of the pilot PTW?
- Does the alternative have a high likelihood of restoring sufficient hydraulic performance to the pilot PTW?
- Will the alternative be of sufficiently low invasiveness so as not to create more fines within the pilot PTW, or negatively divert ambient groundwater flow from the pilot PTW
- Is the alternative technically feasible?
- Is the alternative cost-effective (for this case, we have assumed that a cost-effective alternative is approximately 20 percent or less of the estimated cost of the design, installation, and assessment of the current pilot PTW system)

The four alternatives identified for consideration include:

Alternative 1	No Modification of Pilot PTW
Alternative 2	Install Lateral Hydraulic Barriers
Alternative 3	Install Pilot PTW Extension
Alternative 4	Install New Pilot PTW

Each of the alternatives listed in the following sections generally meet most or all of the above criteria in our professional judgement. As discussed previously, other "technically feasible" alternatives, such as invasive programs that attempt to "remove" or reduce potential skin effects, or that are not seen to be cost-effective at this time (such as deploying a completely new pilot PTW) are not considered here.

2.2 ALTERNATIVE 1 - NO MODIFICATION OF THE PILOT PTW

2.2.1 Description

This alternative consists of completing the assessment of the pilot PTW program and moving forward with making the decision of whether or not to design the full-scale PTW for the site. The purpose of pilot testing a remedial method is to collect field data necessary to design a full-scale system. We believe that the data collected and evaluated as part of the PTW pilot study, including the laboratory data evaluated by UB that assesses the ability of the zeolite treatment material to provide ion exchange-based mitigation of the Sr-90 in groundwater, provides data sufficient to develop a program for designing a full-scale remedy at WVDP. Figures 1.2 and 1.3 indicate the current geometry of the existing pilot PTW and, thus, this alternative.

2.3.1 Basis for Selection

This alternative is evaluated according to the selection criteria described in Section 2.1:

- *Does the alternative address and mitigate the possible high head potential at the west end of the pilot PTW?*

This alternative does not attempt to further assess the high head potential at the west end of the pilot PTW. Additional data collection performed as recommended in Section 1.5 would provide further data. However, we believe that modifying the pilot PTW is not necessary to implement a program to design a full-scale PTW at the site. A full-scale PTW, if deployed in this same area, would be designed to accommodate the field conditions.

- *Does the alternative have a high likelihood of restoring sufficient hydraulic performance to the pilot PTW?*

Based on experience and information collected during the pilot PTW program, a full-scale system could be designed to provide appropriate hydraulic performance.

- *Will the alternative be of sufficiently low invasiveness so as not to create more fines within the pilot PTW, or negatively divert ambient groundwater flow from the pilot PTW*

Based on experience and information collected during the pilot PTW program, a full-scale system could be designed to reduce the potential for a significant amount of fines to either be created, or to significantly impair the required hydraulic performance of the PTW system.

- *Is the alternative technically feasible?*

Based on existing data from the pilot PTW program, including laboratory chemical treatability tests, and knowledge of site conditions in other portions of the site, designing a full-scale PTW system is potentially feasible without first modifying the pilot PTW program. We assume that as part of a full-scale design program, comprehensive hydraulic, hydrostratigraphic and geotechnical information would be collected in the vicinity and along the proposed alignment of a full-scale PTW.

- *Is the alternative cost-effective (for this case, we have assumed that a cost-effective alternative is approximately 20 percent or less of the estimated cost of the design, installation, and assessment of the pilot PTW system)?*

This alternative would have zero costs associated with the pilot PTW program. No costs have been estimated for a full-scale system because this cost is dependent on field conditions along the proposed (unknown) alignment, which has not been determined and is beyond the scope of this work.

2.3.2 Assumptions

The purpose of a pilot program is to develop information necessary for designing and deploying a full-scale system. The pilot PTW program at WVDP, including previously conducted laboratory treatability testing, provided information, including hydraulic, hydrostratigraphic, and engineering data that can be used to successfully design a full-scale system at the site. Designing and applying engineering solutions to make the pilot PTW function perfectly are not considered necessary to successfully design a full-scale PTW.

2.3.4 General Cost Estimate

Zero future costs are associated with the pilot PTW program under this alternative. Costs for a full-scale system are beyond the scope of this report.

2.3.5 Waste Generation

No waste will be generated by this alternative.

2.3.6 Ease of Implementation

An evaluation of the pilot PTW has been performed. Thus, this alternative has been implemented.

2.3.7 Likelihood of Success

The pilot PTW program has successfully provided local hydrostrigraphic, hydraulic, and engineering information. This information forms a basis for implementing a full-scale PTW design program without completely modifying the pilot PTW for enhanced performance.

2.3 ALTERNATIVE 2 - INSTALL LATERAL HYDRAULIC BARRIERS

2.3.1 Description

This alternative consists of installing flow barriers at the west and, possibly, east ends of and perpendicular to the PTW, as shown on Figure 2.1. The purpose of the flow barriers is to hydraulically isolate the PTW from higher water levels at WP-25, and to redirect the flow of groundwater through the PTW.

If hydraulic mounding remains after deploying the lateral barriers, pumping from one or more nearby downgradient wells may be implemented to attempt to stimulate flow through the northern side of the pilot PTW. Pumping could be conducted from one or more of the proposed 2-inch (or larger) diameter wells recommended for the focused additional data collection program.

2.3.2 Basis for Selection

This alternative is evaluated according to the selection criteria described in Section 2.1:

- *Does the alternative address and mitigate the possible high head potential at the west end of the pilot PTW?*

The western sheet pile barrier wall is expected address the anomalously high water level in WP-25 that has been present since construction of the pilot PTW started.

- *Does the alternative have a high likelihood of restoring sufficient hydraulic performance to the pilot PTW?*

If the mounded groundwater in the PTW results from high hydraulic head or other hydrogeologic conditions near WP-25, then this solution will remove the influence of these local conditions on the hydraulic performance of the PTW. The eastern sheet pile barrier wall will also isolate the PTW from any anomalous conditions that may be present at the eastern end of the PTW. The two lateral hydraulic barrier walls will create an approximately two-dimensional flow regime where groundwater flow is orthogonal to the orientation of the PTW, allowing much easier interpretation of groundwater flow conditions through the PTW. Flow through the PTW will then be determined by the presence of the "skin" on the sides of the PTW.

- *Will the alternative be of sufficiently low invasiveness so as not to create more fines within the pilot PTW, or negatively divert ambient groundwater flow from the pilot PTW*

The sheet piles will be driven through the clinoptilolite zone, and so will create more fines. However, these fines should only be present near the sheet pile barrier and will only influence flow near the barrier. The groundwater flow regime away from the barriers will not be affected, except to the extent that the barriers redirect groundwater flow.

- *Is the alternative technically feasible?*

Sheet piles were driven at WVNS to construct the PTW, so construction of the barriers is technically feasible.

- *Is the alternative cost-effective (for this case, we have assumed that a cost-effective alternative is approximately 20 percent or less of the estimated cost of the design, installation, and assessment of the pilot PTW system)?*

As described below, the estimated cost for this alternative is $80,000 for installing just the western lateral hydraulic barrier, which we understand is less than 10 percent of the cost of the original installation.

2.3.3 Assumptions

The flow barriers will consist of sheet piles driven approximately 1 foot into the Lavery Till. The sheet piles will be driven in interlock to provide a continuous barrier; we do not recommend using sealable sheet piles for this application because the head differences across the sheet piles will be very low. The length of the barrier walls necessary to create the required groundwater flow conditions will be determined in detailed design; for the purposes of preparing a cost estimate, we assumed a length of 60 feet for both barrier walls.

2.3.4 General Cost Estimate

Table 1 provides general cost information for this alternative, including engineering design, construction oversight and bid and scope contingencies. WVNS purchased the sheet piles used to construct the PTW; these sheet piles can be reused to install the flow barriers. Costs are based on conventional activities, and do not include special circumstances, procedures, or handling of materials at the WVDP. Costs for managing spoils and other investigation derived waste is not included in this estimate. This cost does not include evaluation of the success or failure of this alternative to provide the necessary information to design a full-scale PTW.

The general cost estimate for this alternative is $80,000. If this alternative is implemented in phases, the additional cost will be about $50,000.

2.3.5 Waste Generation

Minimal waste will be generated from driving the sheet piles.

2.3.6 Ease of Implementation

The design of the sheet pile walls will require determination of the length and depth of the walls, the type of sheet piles, and the equipment used to drive the sheets. The sheet piles will be those that WVNS purchased to install the PTW, and the pile driving equipment will probably also be the same. The length of the sheet pile barrier walls will be determined form analytical modeling, probably using the model already developed at UB. The depth of the walls can be determined from existing subsurface data. Once the dimensions are specified, the walls can quickly be installed.

2.3.7 Likelihood of Success

The two sheet pile barrier walls will effectively eliminate the influence of hydraulic conditions east and west of the PTW. Evaluation of monitoring data will be greatly simplified because the system will be a quasi-two dimensional flow system with groundwater flow throughout the PTW. Low permeability zones within the PTW, such as the skin, will retard the flow of water through the PTW, we anticipate that some affected groundwater will flow through. Thus the effectiveness of the PTW technology can be evaluated based on the monitoring data.

2.4 ALTERNATIVE 3 - INSTALL EXTENSION TO PTW

2.4.1 Description

This alternative consists of installing an extension on the east side of the existing PTW, as shown on Figure 2.2. The purpose of the PTW extension is to capture the flow of groundwater that appears to be flowing around the eastern end of the existing PTW. The conceptual design of this alternative assumes an extension of approximately equal length (approximately 30 feet) and width to the existing pilot PTW. However, we strongly recommend using a groundwater flow model to design the final geometry and alignment of this alternative.

2.4.2 Basis for Selection

This alternative is evaluated according to the selection criteria described in Section 2.1:

- *Does the alternative address and mitigate the possible high head potential at the west end of the pilot PTW?*

The eastern extension of the PTW will not directly mitigate the high head potential at the west end of the existing PTW. By capturing additional flow that currently flows around the eastern end of the PTW, the effect of this high head may be reduced.

- *Does the alternative have a high likelihood of restoring sufficient hydraulic performance to the pilot PTW?*

The eastern extension of the PTW will be installed using construction techniques that have been modified based on the lessons learned from the original pilot PTW installation. Thus less fines and a less significant skin will be present in and around the eastern extension of the PTW. It is therefore anticipated that groundwater currently flowing around the PTW will flow through the PTW extension, providing much better hydraulic performance.

- *Will the alternative be of sufficiently low invasiveness so as not to create more fines within the pilot PTW, or negatively divert ambient groundwater flow from the pilot PTW*

The PTW extension will be installed using refined techniques, so fines generation will be much lower than in the original installation. Minimal fines may be created within the existing PTW in the area where the PTW extension connects to it, because sheet pile driven into the PTW will create fines locally. These fines are not expected to adversely affect the performance of the extended PTW.

- *Is the alternative technically feasible?*

Construction will be similar to the original PTW installation, so construction is technically feasible.

- *Is the alternative cost-effective (for this case, we have assumed that a cost-effective alternative is approximately 20 percent or less of the estimated cost of the design, installation, and assessment of the pilot PTW system)?*

As described below, the estimated cost for this alternative is about $400,000, which we understand is about 30 percent of the cost of the original installation. This alternative therefore does not meet the criteria for cost-effectiveness.

2.4.3 Assumptions

The PTW extension will be installed within a cofferdam similar to the construction of the original PTW. Bracing design will probably differ, allowing less sheet pile penetration of the Lavery Till and therefore less disruption during sheet pile extraction. The cofferdam will be flooded prior to removing the sheet piles to prevent fines being redistributed within the PTW. A comprehensive cap will also be installed over the PTW.

2.4.4 General Cost Estimate

Table 2 provides general cost information for this alternative, including engineering design, construction oversight and scope contingencies. WVNS purchased the sheet piles used to construct the original PTW; these sheet piles can be reused to install the PTW extension.

Costs are based on conventional activities, and do not include special circumstances, procedures, or handling of materials at the WVDP. Costs for managing spoils and other investigation derived waste is not included in this estimate. This cost does not include evaluation of the success or failure of this alternative to provide the necessary information to design a full-scale PTW

The general cost estimate for this alternative is $395,000.

2.4.5 Waste Generation

Significant waste will be generated from construction of the PTW extension, similar to that generated from the construction of the original PTW. Dewatering the cofferdam will generate significant quantities of water containing Sr-90, and excavation will generate significant quantities of spoils. Handling of these wastes is not included in the cost estimate for this alternative.

2.4.6 Ease of Implementation

Construction will be similar to construction of the original PTW, and should be easier based on lessons learned from that experience. The design of the PTW extension will be slightly different to accommodate lessons learned from the performance of the PTW.

2.5.7 Likelihood of Success

The PTW extension should capture flow that is currently going around the existing PTW if designed based on the current knowledge of site hydrogeologic conditions in the vicinity of the pilot PTW. However, evaluating this alternative with a groundwater flow model is necessary for finalizing the design.

2.5 ALTERNATIVE 4 - INSTALL A NEW PILOT PTW

2.5.1 Description

This alternative consists of installing a new pilot PTW at a suitable location at WVDP. Additional soil and groundwater investigation will be performed to locate and design the new pilot PTW, and the new PTW will be constructed in a manner that incorporates the lessons learned from the design, construction and monitoring of the existing PTW. A representative groundwater model is necessary for designing this alternative.

2.5.2 Basis for Selection

This alternative is evaluated according to the selection criteria described in Section 2.1:

- *Does the alternative address and mitigate the possible high head potential at the west end of the pilot PTW?*

This alternative mitigates this by installing a new pilot PTW.

- *Does the alternative have a high likelihood of restoring sufficient hydraulic performance to the pilot PTW?*

No, this alternative relies on an adequate design, installation and monitoring of a new pilot PTW that has sufficient hydraulic performance so that the effectiveness of the PTW technology can be evaluated.

- *Will the alternative be of sufficiently low invasiveness so as not to create more fines within the pilot PTW, or negatively divert ambient groundwater flow from the pilot PTW*

No, this alternative relies on an adequate design, installation and monitoring of a new pilot PTW that has sufficient hydraulic performance so that the effectiveness of the PTW technology can be evaluated.

- *Is the alternative technically feasible?*

Installation of the new pilot PTW will use design, construction and monitoring techniques similar to those employed for the original pilot PTW, so the alternative is technically feasible.

- *Is the alternative cost-effective (for this case, we have assumed that a cost-effective alternative is approximately 20 percent or less of the estimated cost of the design, installation, and assessment of the pilot PTW system)?*

As described below, the estimated cost for this alternative is about $720,000, which we understand is about 50 percent of the cost of the original installation. Therefore this alternative does not meet the criteria for cost-effectiveness.

2.5.3 Assumptions

The new pilot PTW will have similar dimensions to the existing pilot PTW, and will be installed using a similar cofferdam design, except that the bracing will be designed to allow less sheet pile penetration of the Lavery Till. WVNS will be able to reuse the sheet piles that were purchased to install the original pilot PTW.

2.5.4 General Cost Estimate

Table 3 provides general cost information for this alternative, including engineering design, construction oversight and scope contingencies. WVNS purchased the sheet piles used to construct the PTW; these sheet piles can be reused to install the new pilot PTW.

This cost estimate also includes soil and groundwater investigation and an engineering design for the PTW. All costs are based on conventional activities, and do not include special circumstances, procedures, or handling of materials at the WVDP. Costs for managing spoils and other investigation derived waste is not included in this estimate. This cost does not include evaluation of the success or failure of this alternative to provide the necessary information to design a full-scale PTW.

The general cost estimate for this alternative is $720,000.

2.5.5 Waste Generation

Significant waste will be generated from construction of the new pilot PTW, similar to that generated from the construction of the original PTW. Dewatering the cofferdam will generate significant quantities of water containing Sr-90, and excavation will generate significant quantities of spoils. Handling of these wastes is not included in the cost estimate for this alternative.

2.5.6 Ease of Implementation

A suitable location for the new pilot PTW at WVDP will need to be identified, and a comprehensive soil and groundwater investigation completed in the area. A new PTW design will be completed to incorporate this investigation data and the lessons learned from construction of the original pilot PTW. Construction will be similar to construction of the original PTW, and should be easier based on lessons learned from that experience.

2.5.7 Likelihood of Success

The success of the new pilot PTW will depend on the adequacy of the additional soil and groundwater investigation and the new engineering design, including groundwater modeling. Site-specific parameters will need to be evaluated and carefully considered in the implementation of the new pilot PTW.

3.0 CONCLUSIONS AND RECOMENDATIONS

This section provides our recommendations for the preferred alternative and course of action.

3.1 PREFERRED ALTERNATIVE

The final engineering modification alternative is dependent on testing the selected alternates with the groundwater modeling tool currently being developed by UB. However, based on the general objectives for performing a pilot test of a remedial technology, and the data collected during the WVDP pilot PTW and feasibility testing program, we recommend that Alternative 1 be formally implemented, that the pilot PTW program be concluded, and that the full-scale PTW design program begin. The full-scale PTW design program might include methods described in focused additional data collection program described in Section 1.5, but would not necessarily include the specific data collection activities recommended for assessing performance of the existing pilot PTW.

3.2 RECOMMENDED COURSE OF ACTION

If an engineered modification alternative is selected, we recommend and emphasize the importance of completing Components 1 and 2 of the focused additional data collection program described in Section 1.5, and completing the modeling study by UB (the results from the focused additional data collection program may be critical for developing a representative groundwater model). These actions will increase the confidence that a reliable solution to the pilot PTW's current performance can be selected, designed, and implemented and can be designed with a high probability of success.

4.0 REFERENCES

Geomatrix Consultants, Inc., 2000a, Pilot Permeable Treatment Wall Hydraulic Evaluation Report, prepared for West Valley Nuclear Services, LLC, West Valley, New York, March.

Geomatrix Consultants, Inc., 2000a, Pilot Permeable Treatment Wall Engineering Evaluation Report, prepared for West Valley Nuclear Services, LLC, West Valley, New York, March.

TABLES

GEOMATRIX

TABLE 1

ESTIMATED COST
CONCEPTUAL ALTERNATIVE NO. 2
West Valley Nuclear Services, LLC
West Valley, New York

Conceptual Alternative 2: Install Two Sheetpile Barriers (60 feet long by 30 feet deep) Adjacent to West and East Edges of PTW

1	**CONSTRUCTION COSTS**					
1.1	**Contractor and Equipment Costs - Site Preparation**					**SOURCE**
1.1a	Surveying - site preparation	1	lump sum	$2,500	$2,500	Geomatrix estimate
		Estimated Contractor Costs - Site Prep.			**$2,500**	
1.2	***Contractor and Equipment Costs - Sheetpile Barrier Installation***					
1.2a	Mobilization/demobilization	1	lump sum	$10,000	$10,000	Geomatrix estimate
1.2b	Purchase sheetpiles	1	lump sum	$0	$0	Sheets already on site
1.2c	Install two sheetpile barriers (each 60 feet long by 30 feet deep)	3,600	square feet	$12	$43,200	Geomatrix estimate
		Est. Cont. Costs -Sheetpile Barrier Installation			**$53,200**	
		Estimated Contractor Costs			**$55,700**	
1.3	**Contingency, Supervision and Design Costs (CSD)**					
1.3a	Scope contingency (20% est. contractor costs)	1	lump sum	$11,140	$11,140	REM IV
1.3b	Construction management (10% est. contractor costs incl. cont.)	1	lump sum	$6,684	$6,684	REM IV
1.3c	Engineering design costs (10% est. contractor costs incl. cont.)	1	lump sum	$6,684	$6,684	Geomatrix estimate
		Estimated CSD Costs			**$24,508**	
		ESTIMATE TOTAL COSTS			**$80,208**	

Note:
Estimate does not include the cost of disposal of soil and water or health and safety issues related to the presence of Sr-90 at the site.

GEOMATRIX

TABLE 2

ESTIMATED COST
CONCEPTUAL ALTERNATIVE NO. 3
West Valley Nuclear Services, LLC
West Valley, New York

Conceptual Alternative 3: Install Extension of PTW

1	**CONSTRUCTION COSTS**						
1.1	**Contractor and Equipment Costs - Site Preparation**						**SOURCE**
1.1a	Surveying	1	lump sum	$2,500	$2,500		Geomatrix estimate
1.1b	Site Preparation	1	lump sum	$8,000	$8,000		Geomatrix estimate
		Estimated Contractor Costs - Site Prep.			**$10,500**		
1.2	**Contractor and Equipment Costs - PTW Extension Installation**						
1.2a	Mobilization/demobilization	1	lump sum	$20,000	$20,000		Geomatrix estimate
1.2b	Purchase sheet piles	1	lump sum	$0	$0		Sheets already on site
1.2c	Install sheetpile cofferdam	1	lump sum	$120,000	$120,000		Geomatrix estimate
1.2d	Dewater cofferdam	1	lump sum	$12,000	$12,000		Geomatrix estimate
1.2e	Excavate cofferdam	1	lump sum	$30,000	$30,000		Geomatrix estimate
1.2f	Install cofferdam divider, backfill cofferdam	1	lump sum	$50,000	$50,000		Geomatrix estimate
1.2g	Flood cofferdam, remove sheetpiles, bracing, divider	1	lump sum	$20,000	$20,000		Geomatrix estimate
1.2h	Install cap, bollards, final grading	1	lump sum	$12,000	$12,000		Geomatrix estimate
		Est. Cont. Costs - PTW Extension Installation			**$264,000**		
		Estimated Contractor Costs			**$274,500**		
1.3	**Contingency, Supervision and Design Costs (CSD)**						
1.3a	Scope contingency (20% est. contractor costs)	1	lump sum	$54,900	$54,900		REM IV
1.3b	Construction management (10% est. contractor costs incl. cont.)	1	lump sum	$32,940	$32,940		REM IV
1.3c	Engineering design costs (10% est. contractor costs incl. cont.)	1	lump sum	$32,940	$32,940		Geomatrix estimate
		Estimated CSD Costs			**$120,780**		
		ESTIMATE TOTAL COSTS			**$395,280**		

Note:
Estimate does not include the cost of disposal of soil and water or health and safety issues related to the presence of Sr-90 at the site.

GEOMATRIX

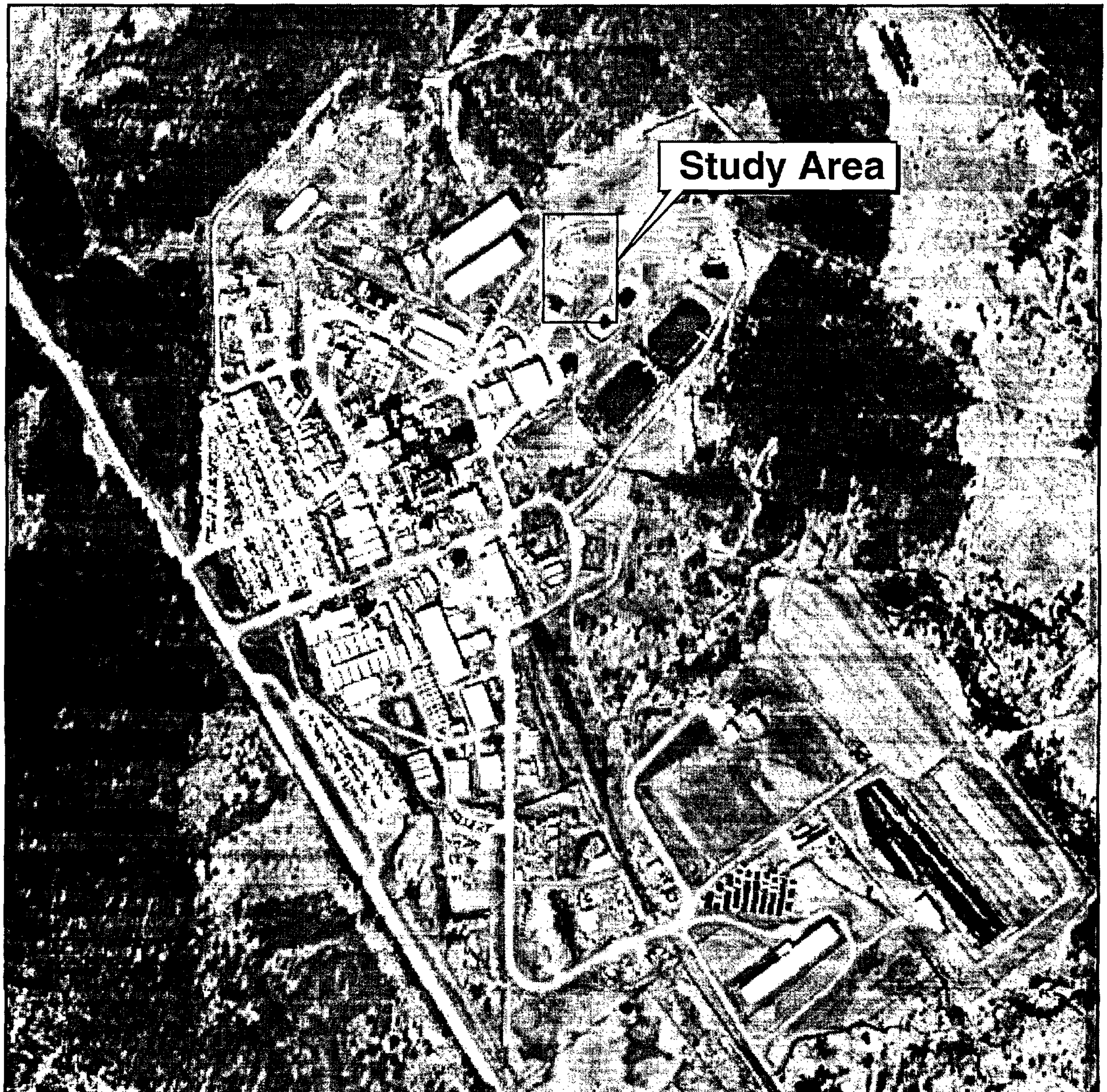
TABLE 3

ESTIMATED COST
CONCEPTUAL ALTERNATIVE NO. 4
West Valley Nuclear Services, LLC
West Valley, New York

Conceptual Alternative 4: Install New PTW

							SOURCE
1	**INVESTIGATION AND DESIGN COSTS**						
1.1a	Soil and groundwater investigation	1	lump sum	$	100,000	$100,000	Geomatrix estimate
1.1b	Engineering design	1	lump sum	$	90,000	$90,000	Geomatrix estimate
1.1c	Design and installation of monitoring network	1	lump sum	$	50,000	$50,000	Geomatrix estimate
		Estimated Contractor Costs -Inv. and Design				**$240,000**	
2	**CONSTRUCTION COSTS**						
2.1	**Contractor and Equipment Costs - Site Preparation**						
2.1a	Surveying	1	lump sum		$2,500	$2,500	Geomatrix estimate
2.1b	Site Preparation	1	lump sum		$8,000	$8,000	Geomatrix estimate
		Estimated Contractor Costs - Site Prep.				**$10,500**	
2.2	**Contractor and Equipment Costs - PTW Construction**						
2.2a	Mobilization/demobilization	1	lump sum		$20,000	$20,000	Geomatrix estimate
2.2b	Purchase sheet piles	1	lump sum		$0	$0	Sheets already on site
2.2c	Install sheetpile cofferdam	1	lump sum		$120,000	$120,000	Geomatrix estimate
2.2d	Dewater cofferdam	1	lump sum		$12,000	$12,000	Geomatrix estimate
2.2e	Excavate cofferdam	1	lump sum		$30,000	$30,000	Geomatrix estimate
2.2f	Install cofferdam divider, backfill cofferdam	1	lump sum		$50,000	$50,000	Geomatrix estimate
2.2g	Flood cofferdam, remove sheetpiles, bracing, divider	1	lump sum		$20,000	$20,000	Geomatrix estimate
2.2h	Install cap, bollards, final grading	1	lump sum		$12,000	$12,000	Geomatrix estimate
		Est. Cont. Costs - PTW Construction				**$264,000**	
		Estimated Contractor Costs				**$274,500**	
2.3	**Contingency, Supervision and Design Costs (CSD)**						
2.3a	scope contingency (20% est. contractor costs)	1	lump sum		$102,900	$102,900	REM IV
2.3b	construction management (10% est. contractor costs incl. cont.)	1	lump sum		$61,740	$61,740	REM IV
2.3c	engineering design costs (10% est. contractor costs incl. cont.)	1	lump sum		$37,740	$37,740	Geomatrix estimate
		Estimated CSD Costs				**$202,380**	
		ESTIMATE TOTAL COSTS				**$716,880**	

Note:
Estimate does not include the cost of disposal of soil and water or health and safety issues related to the presence of Sr-90 at the site.



Source: www.terraserver.microsoft.com

DRAFT

GEOMATRIX

SITE LOCATION MAP
West Valley Nuclear Services
West Valley, New York

Project No.
7061

Figure
1.1

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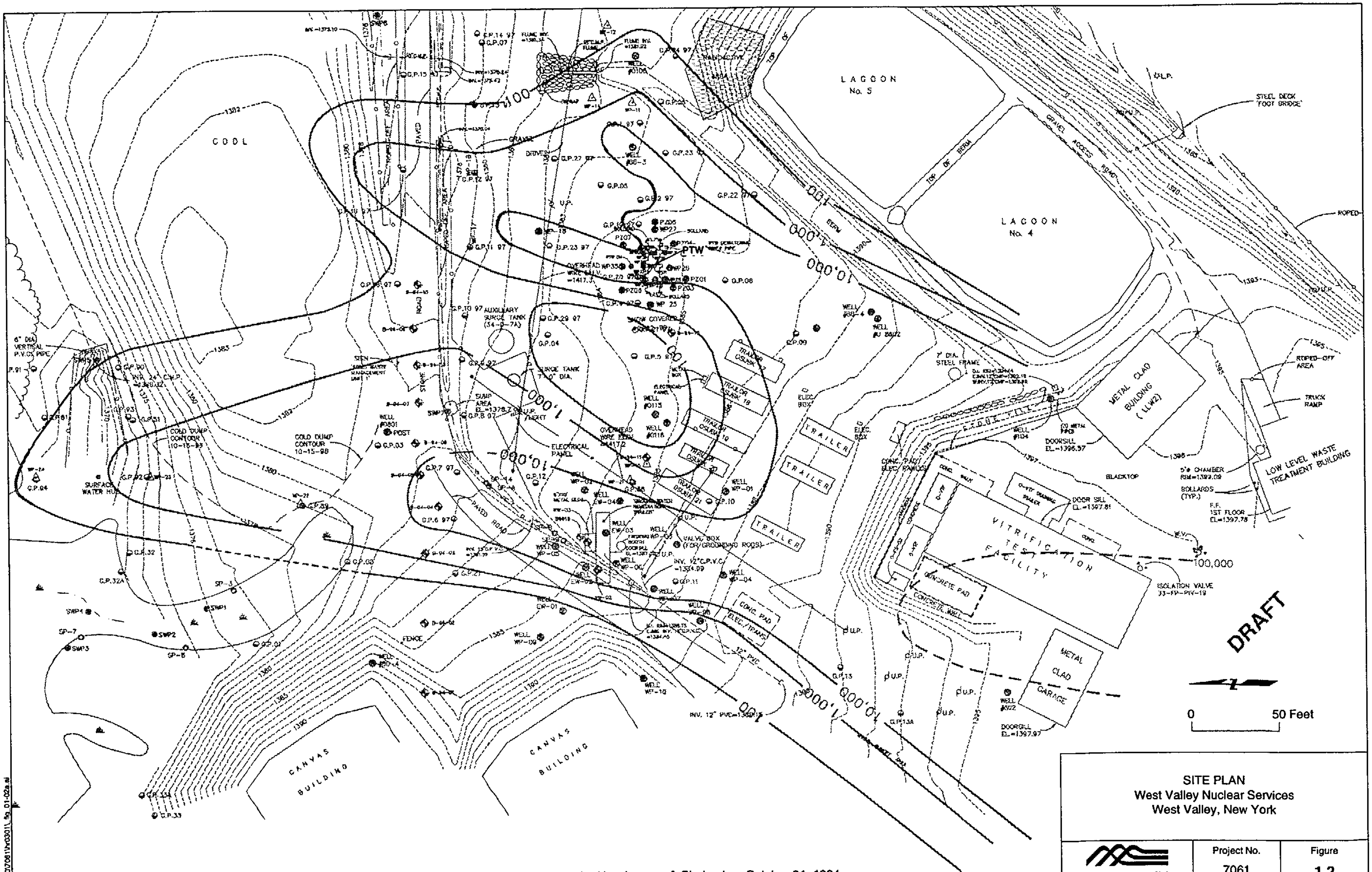

Source: Topographic survey by Nussbaumer & Clarke, Inc, October 31, 1994.

SITE PLAN
West Valley Nuclear Services
West Valley, New York

GEOMATRIX

Project No.	Figure
7061	1.2

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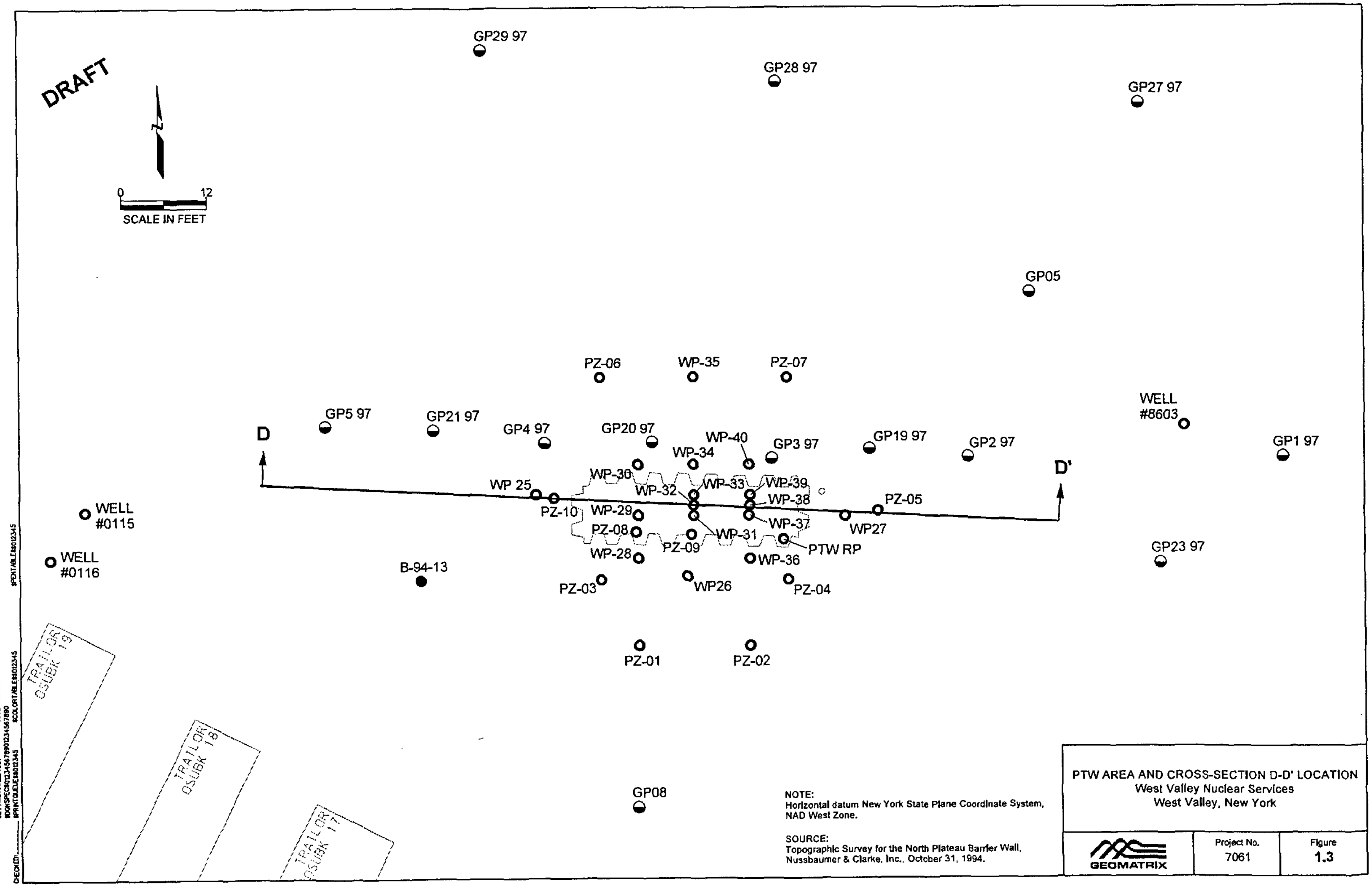
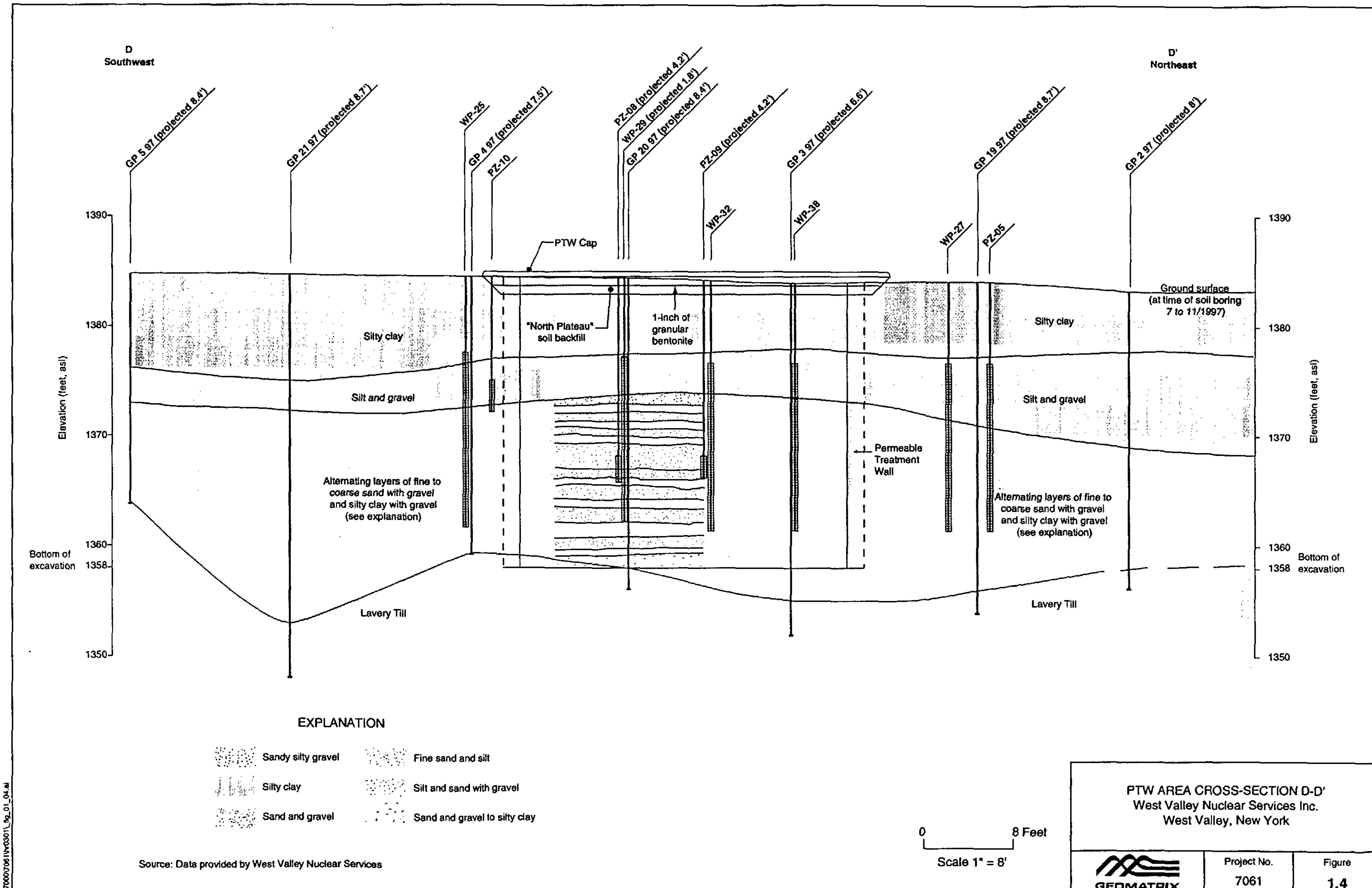


NOTE:
Horizontal datum New York State Plane Coordinate System, NAD West Zone.

SOURCE:
Topographic Survey for the North Plateau Barrier Wall, Nussbaumer & Clarke, Inc., October 31, 1994.

PTW AREA AND CROSS-SECTION D-D' LOCATION
West Valley Nuclear Services
West Valley, New York

Project No.	Figure
7061	1.3



PTW AREA CROSS-SECTION D-D'
West Valley Nuclear Services Inc.
West Valley, New York

Figure 1.4

CONCEPTUAL DESIGN
PENDING RESULTS OF GROUNDWATER MODELING
TASK PERFORMED BY UNIVERSITY AT BUFFALO

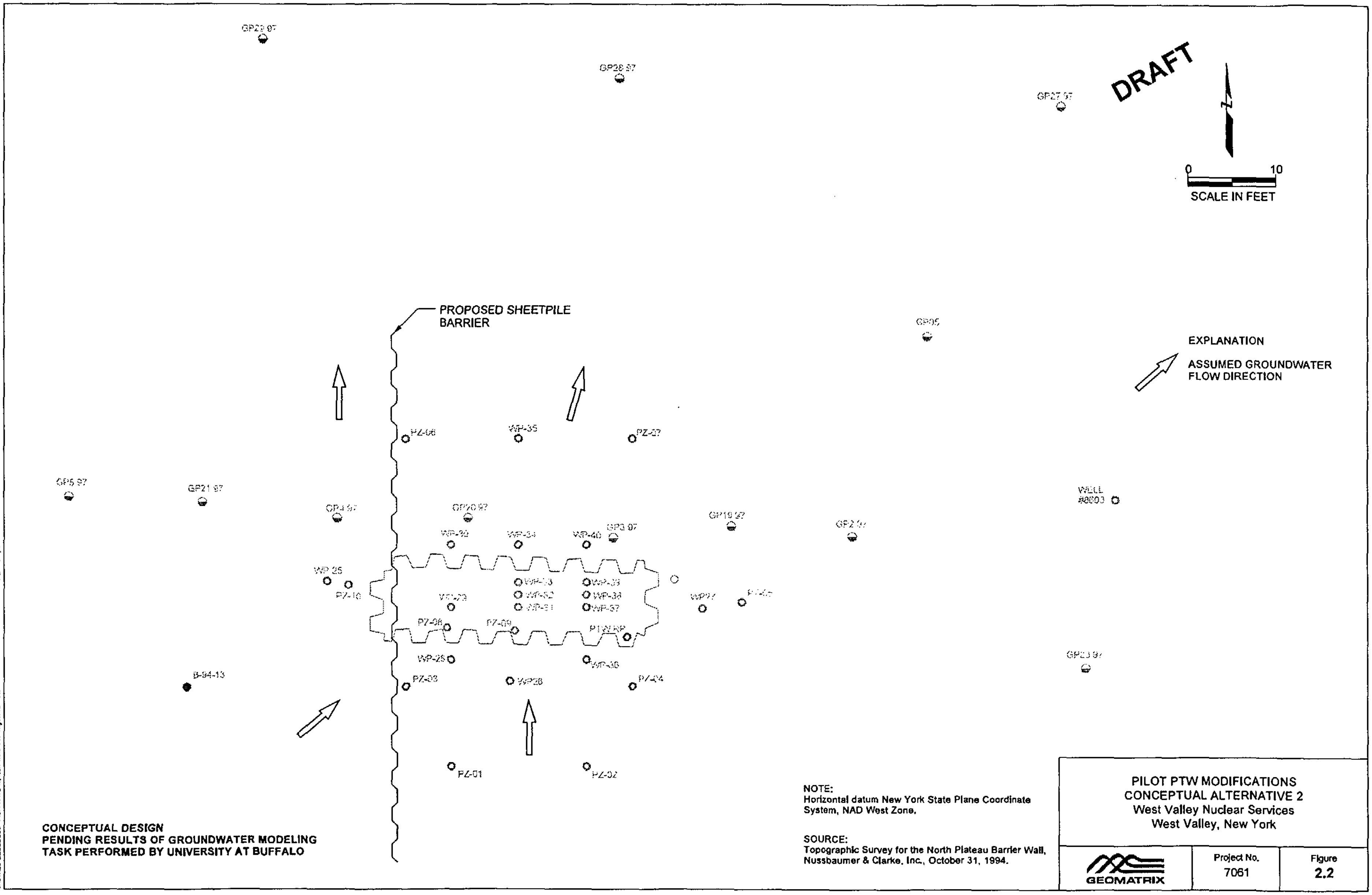
NOTE:
Horizontal datum New York State Plane Coordinate System, NAD West Zone.

SOURCE:
Topographic Survey for the North Plateau Barrier Wall, Nussbaumer & Clarke, Inc., October 31, 1994.

PILOT PTW MODIFICATIONS
CONCEPTUAL ALTERNATIVE 1
West Valley Nuclear Services
West Valley, New York

GEOMATRIX

Project No.	Figure
7061	2.1



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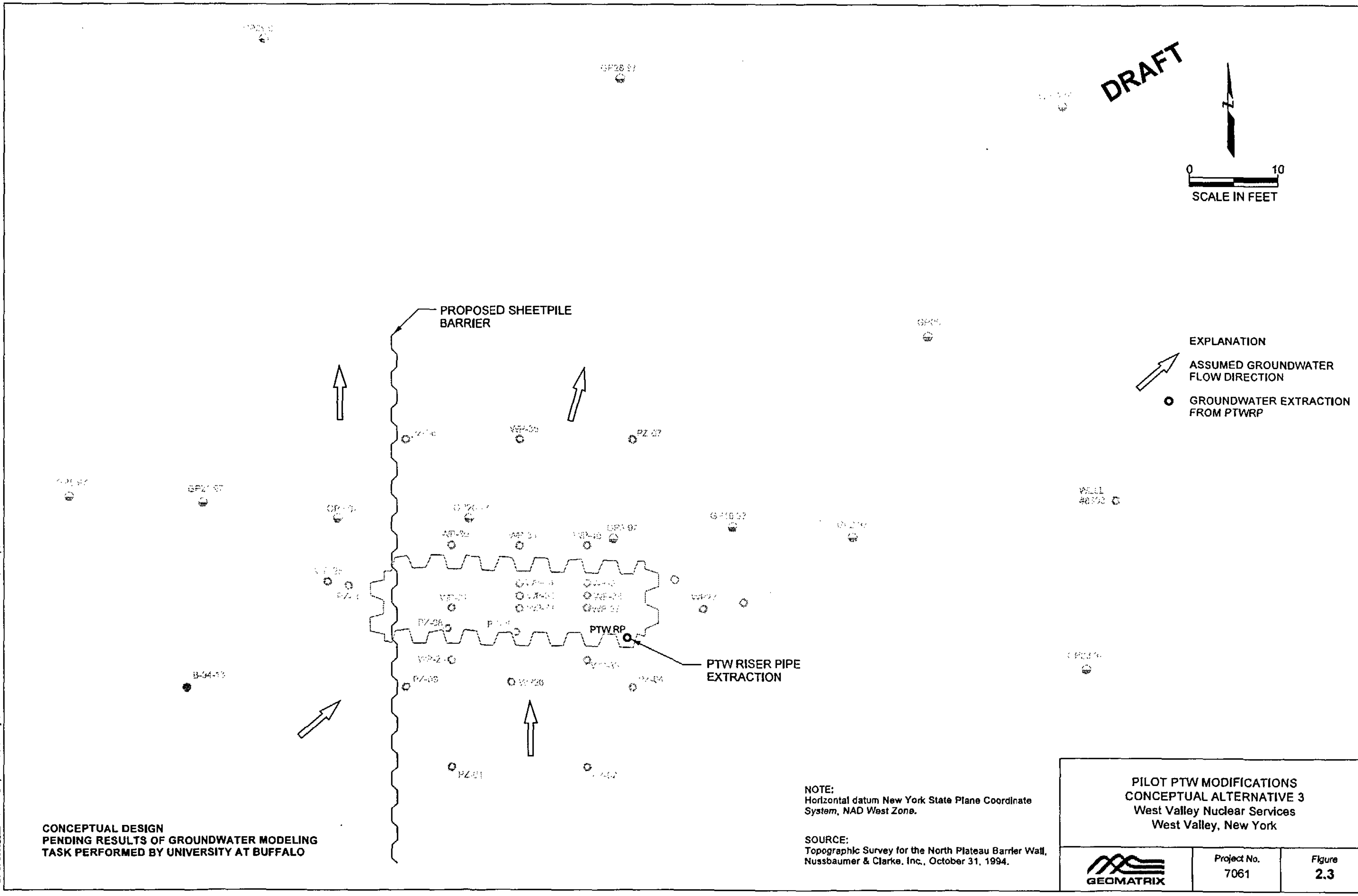
0
10
SCALE IN FEET
PROPOSED SHEETPILE BARRIER
EXPLANATION
ASSUMED GROUNDWATER FLOW DIRECTION
GP28 97
GP5 97
GP21 97
GP4 97
GP20 97
GP3 97
GP19 97
PZ-06
WP-35
PZ-07
WELL #8603
WP-32
WP-34
WP-40
WP-25
PZ-10
WP-29
PZ-08
PZ-09
PTW RP
WP-26
WP-36
PZ-03
WP28
PZ-04
B-94-13
PZ-01
PZ-02
CONCEPTUAL DESIGN
PENDING RESULTS OF GROUNDWATER MODELING
TASK PERFORMED BY UNIVERSITY AT BUFFALO
NOTE:
Horizontal datum New York State Plane Coordinate System, NAD West Zone.
SOURCE:
Topographic Survey for the North Plateau Barrier Wall, Nussbaumer & Clarke, Inc., October 31, 1994.
PILOT PTW MODIFICATIONS
CONCEPTUAL ALTERNATIVE 2
West Valley Nuclear Services
West Valley, New York
GEOMATRIX
Project No.
7061
Figure
2.2



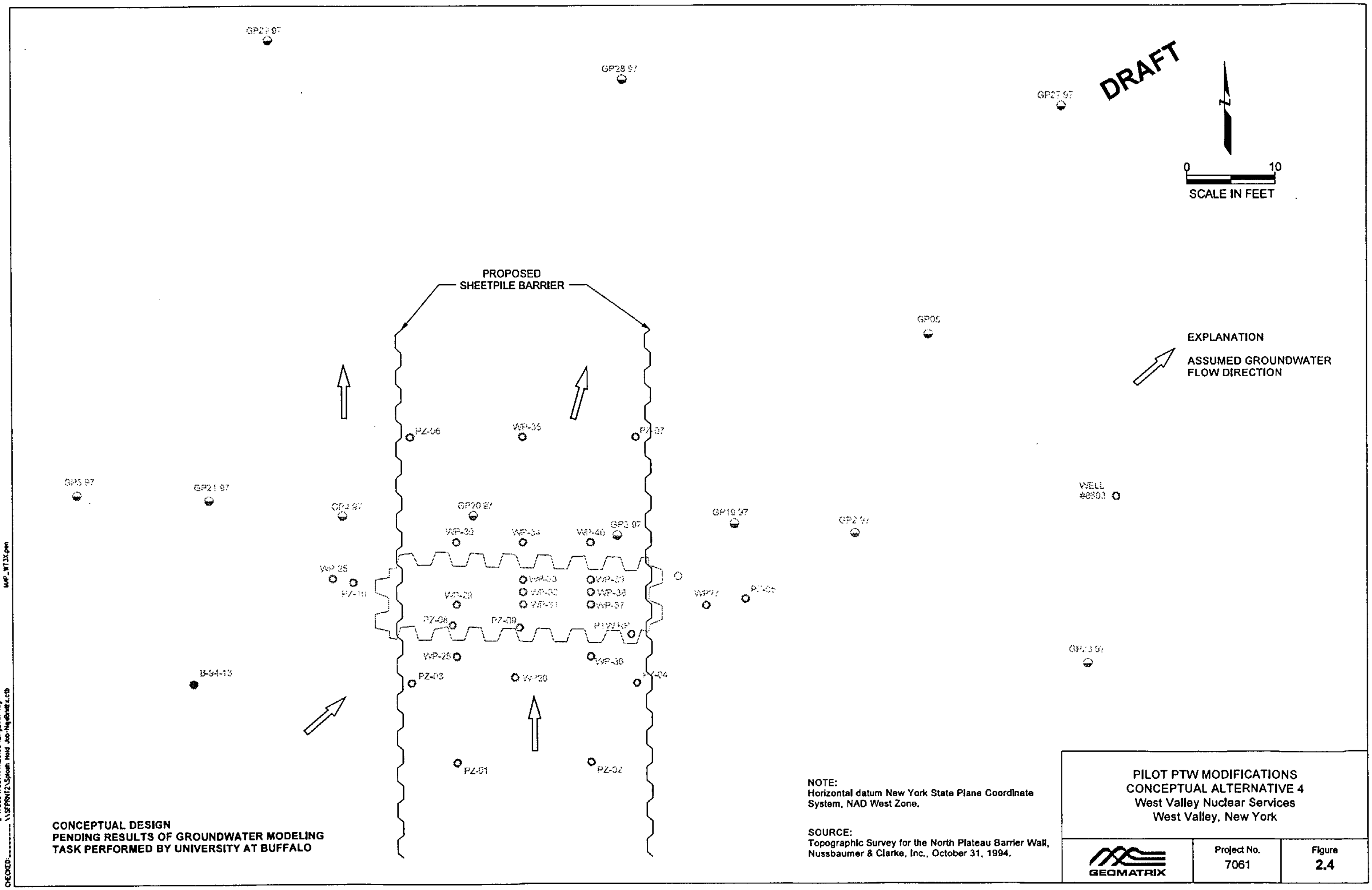
CONCEPTUAL DESIGN
PENDING RESULTS OF GROUNDWATER MODELING
TASK PERFORMED BY UNIVERSITY AT BUFFALO

NOTE:
Horizontal datum New York State Plane Coordinate *System, NAD West Zone.*

SOURCE:
Topographic Survey for the North Plateau Barrier Wall, Nussbaumer & Clarke, Inc., October 31, 1994.

PILOT PTW MODIFICATIONS
CONCEPTUAL ALTERNATIVE 3
West Valley Nuclear Services
West Valley, New York

GEOMATRIX	Project No. 7061	Figure **2.3**



CONCEPTUAL DESIGN
PENDING RESULTS OF GROUNDWATER MODELING
TASK PERFORMED BY UNIVERSITY AT BUFFALO

NOTE:
Horizontal datum New York State Plane Coordinate System, NAD West Zone.

SOURCE:
Topographic Survey for the North Plateau Barrier Wall, Nussbaumer & Clarke, Inc., October 31, 1994.

PILOT PTW MODIFICATIONS
CONCEPTUAL ALTERNATIVE 4
West Valley Nuclear Services
West Valley, New York

GEOMATRIX

Project No.	Figure
7061	2.4

APPENDIX 13

COST ESTIMATE FOR FOCUSED DATA COLLECTION AND ANALYSIS PROGRAM

APPENDIX 13

COST ESTIMATE FOR FOCUSED DATA COLLECTION AND ANALYSIS PROGRAM

This appendix / attachment presents a cost estimate for implementing Components 1-3 of the focused data collection and analysis program presented in section 1.5.1 of the report by Geomatrix Consultants, Inc. titled “Pilot Permeable Treatment Wall Modification Options Report,” dated April 2001.

Component 1: Borehole drilling, well installation, and water level monitoring program

Mobilization / demobilization for drilling rig and 2-person crew, including radiological worker training:

- training $5,000
- mobilization $3,000
- demobilization $3,000

Total for Mobilization / Demobilization $11,000

Complete 4 test borings to a depth of 30-feet each using hollow-stem augers with continuous split-spoon sampling at 2-foot intervals, complete 2 similar test borings to a depth of 22-feet each, and complete 1 similar test boring to a depth of 13-feet (plus 15% contingency).

200 feet at $30/foot = $6,000

Install 4 monitoring wells to a depth of 30-feet each, install 2 wells to a depth of 22-feet each, and install 1 well to a depth of 13-feet. Wells will be constructed of 2-inch ID PVC screen and riser pipe, with a filter sand pack adjacent to the screen, bentonite seal above the sand pack, and cement grout backfill up to ground surface (plus 15% contingency).

200 feet at $30/foot = $6,000

Installation of protective well casings for each well.

7 protective casings at $500/each = $3,500

Decontamination time for drilling rig and crew.

Estimate 20 hours at $150/hour = $3,000

Subtotal for Component 1 = $29,500

Component 2: Focused hydraulic testing program

Purchase or rent equipment for groundwater pumping tests including, but not limited to:

- pumps
- pump controllers
- pipe
- valves
- hoses
- pressure transducers

Estimate $6,000

Subtotal for Component 2 = $6,000

Component 3: Tracer testing program

Purchase or rent equipment for tracer tests including, but not limited to:

- tracer solutions
- tracer dyes
- ion specific electrodes
- colorimeters

Estimate $6,000

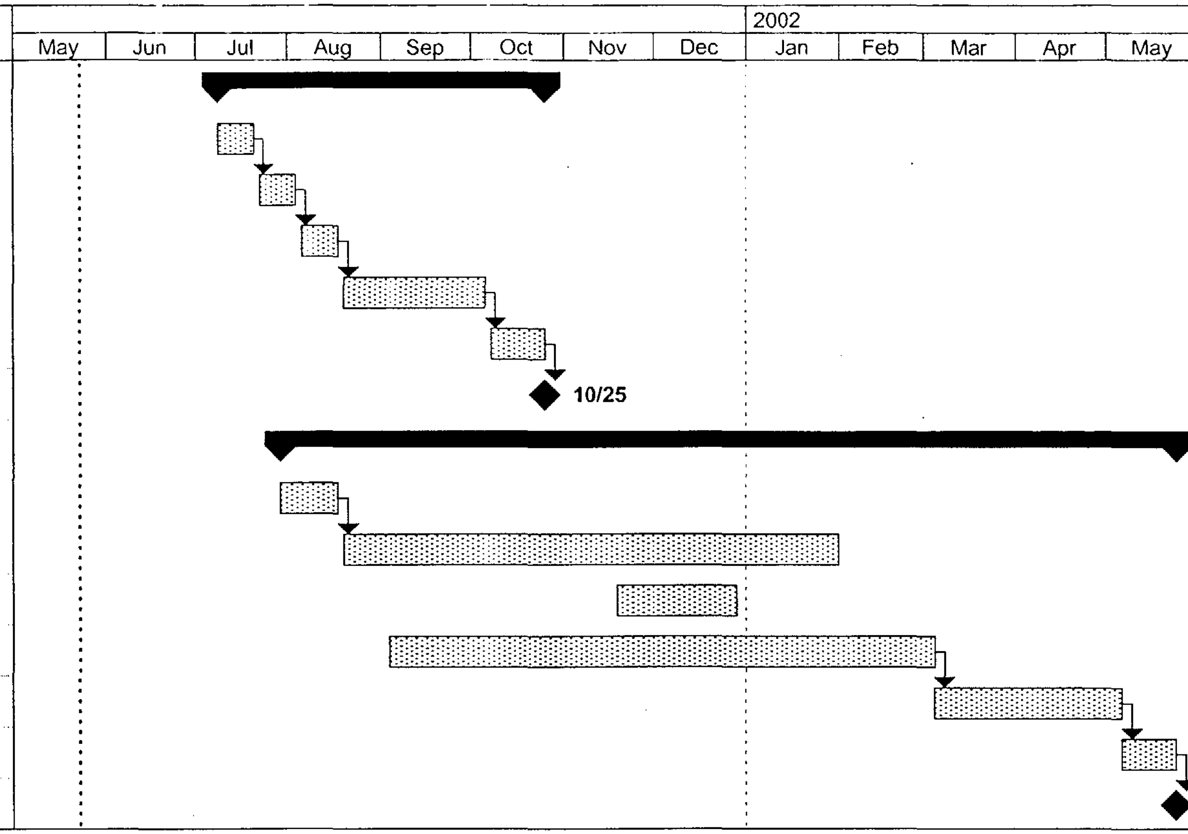
Subtotal for Component 3 = $6,000

Total Estimate for Components 1-3 = $41,500

APPENDIX 14

PATH FORWARD PROJECT SCHEDULE

ID		Task Name
1		**Pilot PTW Path Forward**
2		Field Characterization Program
3		Field Testing
4		Data Assessment
5		3-D Model Development
6		Design Evaluation
7		Decision point on Modification
8		**1st Lobe, Full-Scale Design**
9		Field Characterization Program
10		Field Testing and Data Collection
11		Preliminary Design
12		Data Evaluation
13		Conceptual Design
14		Design Evaluation
15		Decision Point to Proceed



Project: North Plateau Path Forwarc
Date: May 16, 2001
PM: Beth Carpenter

Task	Summary	Rolled Up Progress
Split	Rolled Up Task	External Tasks
Progress	Rolled Up Split	Project Summary
Milestone	Rolled Up Milestone	