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FINAL REMEDIAL INVESTIGATION AND FEASIBILITY STUDY AND TECHNICAL MEMORANDUM 2 HYDROGEOLOGIC ASSESSMENT NAS WHITING FIELD FL

5/1/1992

ABB ENVIRONMENTAL

FINAL

REMEDIAL INVESTIGATION AND FEASIBILITY STUDY

TECHNICAL MEMORANDUM NO. 2 HYDROGEOLOGIC ASSESSMENT

NAVAL AIR STATION WHITING FIELD MILTON, FLORIDA

MAY 1992

SOUTHERN DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
CHARLESTON, SOUTH CAROLINA
29411-0068

FINAL
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PRIOR NOTIFICATION OF THE COMMANDING OFFICER
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MILTON, FLORIDA

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REMEDIAL INVESTIGATION AND FEASIBILITY STUDY

PHASE I

NAVAL AIR STATION, WHITING FIELD
MILTON, FLORIDA

Technical Memorandum No. 2
Hydrogeologic Assessment

UIC: N60508

Contract No. N62467-88-C-0382

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GLOSSARY

ABB-ES	ABB Environmental Services, Inc.
AVGAS	aviation gasoline
bls	below land surface
CERCLA	Comprehensive Environmental Response, Compensation, and Liability Act
CFR	Code of Federal Regulations
cm/sec	centimeter per second
FDER	Florida Department of Environmental Regulation
ft/day	feet per day
ft^2/day	square feet per day
ft/ft	feet per foot
GAC	granulated activated carbon
HRS	Hazard Ranking System
IAS	Initial Assessment Study
IR	Installation Restoration
K	hydraulic conductivity
μg/kg	micrograms per kilogram
mg/ℓ	milligrams per liter
msl	mean sea level
NAS	Naval Air Station
NCP	National Contingency Plan
NEESA	Naval Energy and Environmental Support Activity
NGVD	National Geodetic Vertical Datum of 1929
NPL	National Priorities List
PA	Preliminary Assessment
PCBs	polychlorinated biphenyls
PCPT	piezocone penetrometer
PVC	polyvinyl chloride
RI/FS	Remedial Investigation and Feasibility Study
SARA	Superfund Amendments and Reauthorization Act
SI	Site Inspection
SOUTHNAVFACENGCOM	Southern Division, Naval Facilities Engineering Command
TCL	Target Compound List
TRAWING FIVE	Training Air Wing Five

GLOSSARY (Continued)

USEPA	U.S. Environmental Protection Agency
UST	underground storage tanks
VOC	volatile organic compound

1.0 INTRODUCTION

ABB Environmental Services, Inc. (ABB-ES), under contract to the Department of Navy, is submitting Technical Memorandum No. 2 for the Phase I Remedial Investigation and Feasibility Study (RI/FS) for Naval Air Station (NAS) Whiting Field located in Milton, Florida, to the Department of Navy, Southern Division, Naval Facilities Engineering Command (SOUTHNAVFACENGCOM). The RI/FS is being conducted under contract number N62467-88-C-0382.

Technical Memorandum No. 2, Hydrogeologic Assessment, is the second in a series of six technical memoranda that summarizes the results and transmits data gathered during the Phase I RI. The Phase I RI field program was carried out during the period December 1990 to May 1991. These technical memoranda form the supporting basis for scoping a Phase II RI Sampling and Analysis Plan for NAS Whiting Field.

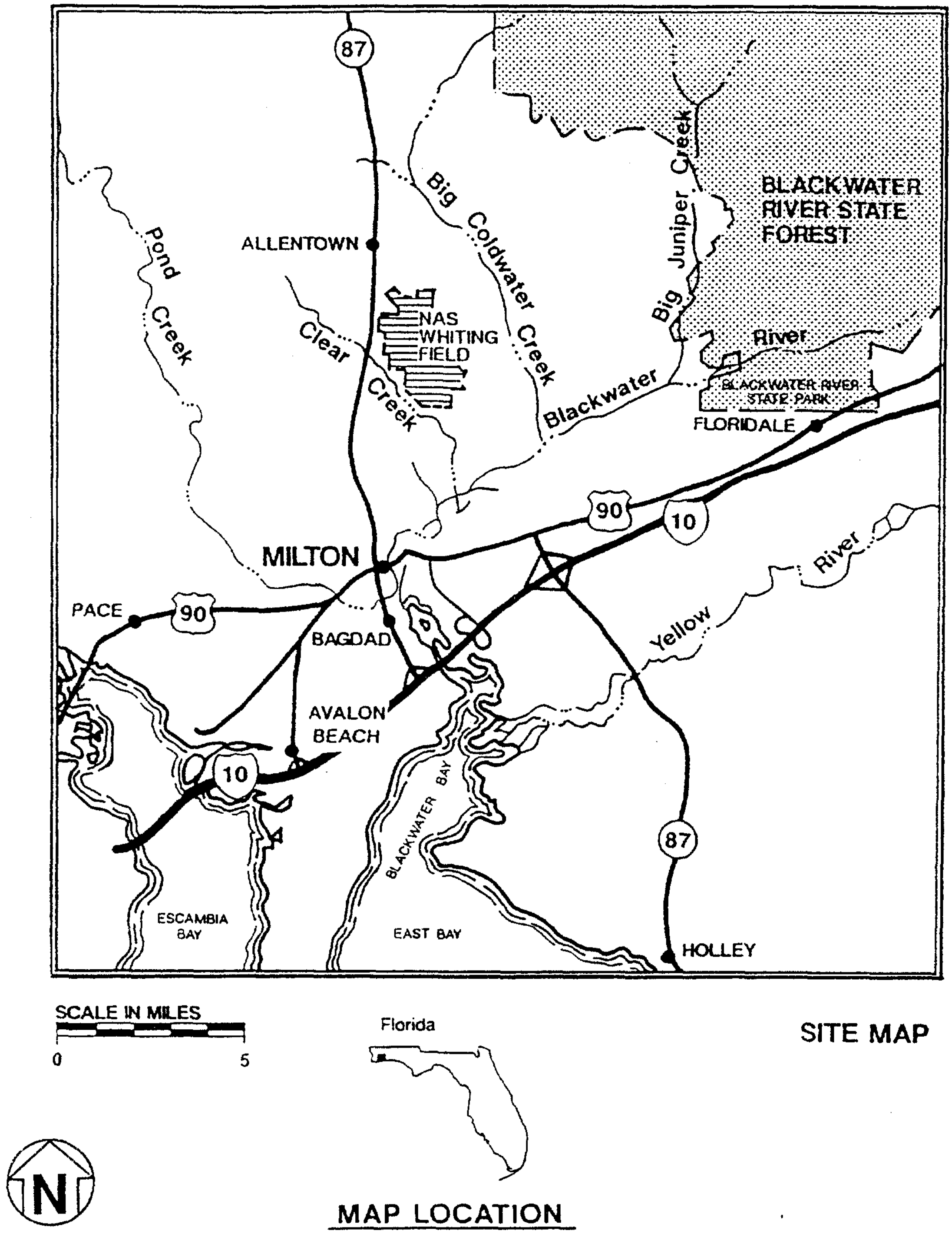
NAS Whiting Field is located in Florida's northwest coastal area approximately 7 miles north of Milton and 20 miles northeast of Pensacola (Figure 1-1). NAS Whiting Field presently consists of two air fields separated by an industrial area and covers approximately 2,560 acres in Santa Rosa County. Figure 1-2 presents the installation layout.

NAS Whiting Field, home of Training Air Wing Five (TRAWING FIVE), was constructed in the early 1940's. It was commissioned as the Naval Auxiliary Air Station Whiting Field in July 1943 and has served as a naval aviation training facility ever since. The field's mission has been to train student naval aviators in basic instruments, formation and tactic phases of fixed-wing, and propeller-driven aircraft, and in the basic and advanced portions of helicopter training.

NAS Whiting Field lies within the Western Highlands physiographic division of Santa Rosa County in the Coastal Plain Province. The Western Highlands are characterized by a well drained, southward sloping, plateau with numerous streams. Land surrounding NAS Whiting Field primarily consists of agricultural land to the northwest, residential and forested areas to the south and southwest, and forested land around the remaining boundaries. This land use distribution is shown in Figure 1-3.

Located on an upland area, elevations at Whiting Field range from 150 to 190 feet above sea level. The facility is bounded by low-lying receiving waters; Clear Creek to the west and south and Big Coldwater Creek to the east. These two streams are tributaries of the Blackwater River, which discharges to the estuarine waters of the East Bay of the Escambia Bay coastal system.

1.1 PURPOSE AND BACKGROUND. The purpose of the NAS Whiting Field RI/FS is to identify a range of remedial alternatives to address any identified risks to public health and the environment posed by toxic or hazardous chemicals present as a result of past waste disposal practices or spills. To achieve this objective, the RI must collect data sufficient to assess the nature and distribution of chemicals associated with each site. The data collected in the RI will be used in the FS to screen, evaluate, and select remedial alternatives to provide permanent, feasible solutions to environmental contamination problems at NAS Whiting Field.



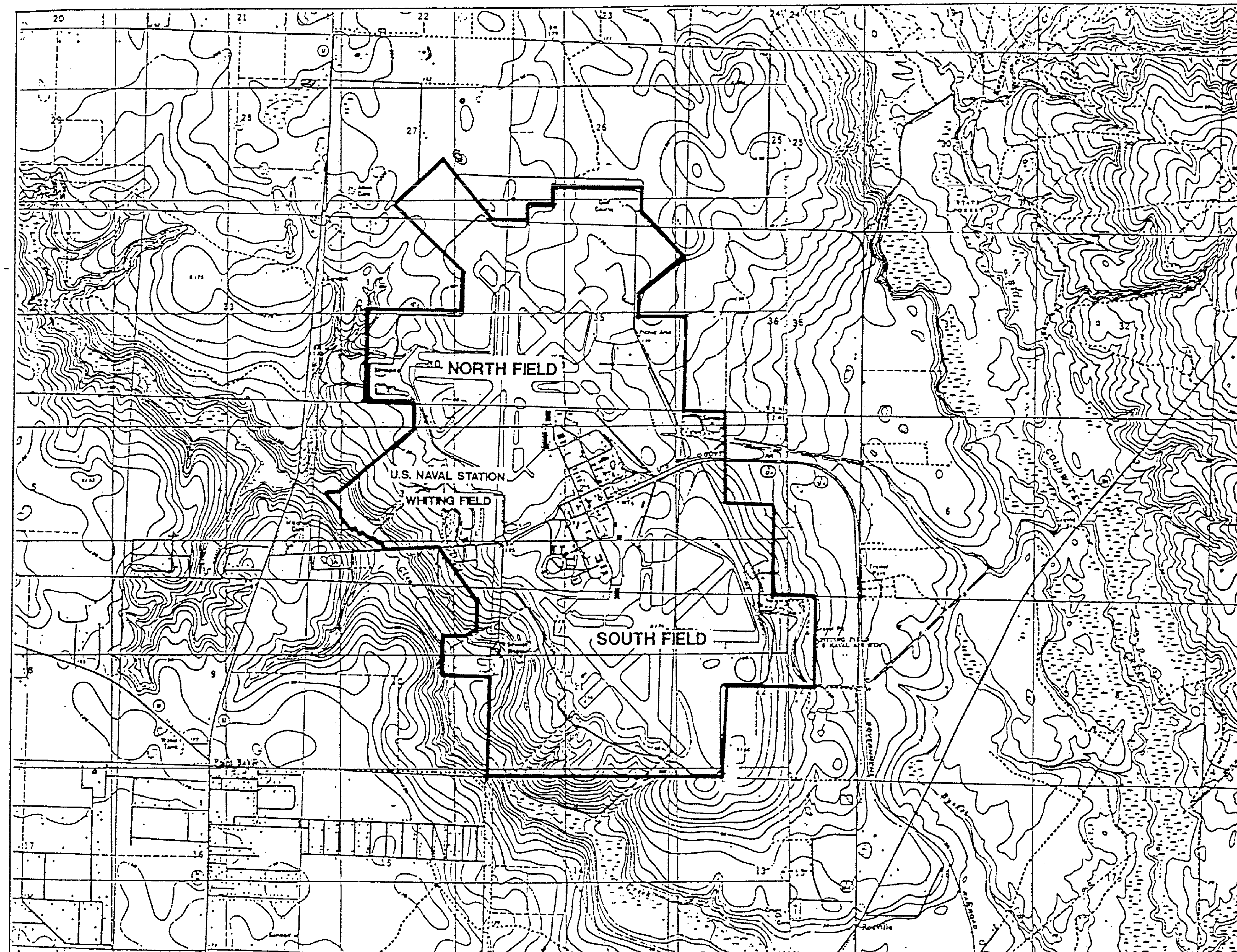
SOURCE: ABB ENVIRONMENTAL SERVICES, INC., 1991

FIGURE 1-1

FACILITY LOCATION MAP

RI/FS PROGRAM

NAS WHITING FIELD
MILTON, FLORIDA



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SOURCE:
USGS QUADRANGLE MILTON NORTH, FLORIDA
PHOTOREVISED 1987
AND USGS QUADRANGLE HAROLD, FLORIDA 1973.

FIGURE 1-2

NAS WHITING FIELD

RI/FS PROGRAM

NAS WHITING FIELD
MILTON, FLORIDA

00225I01Z

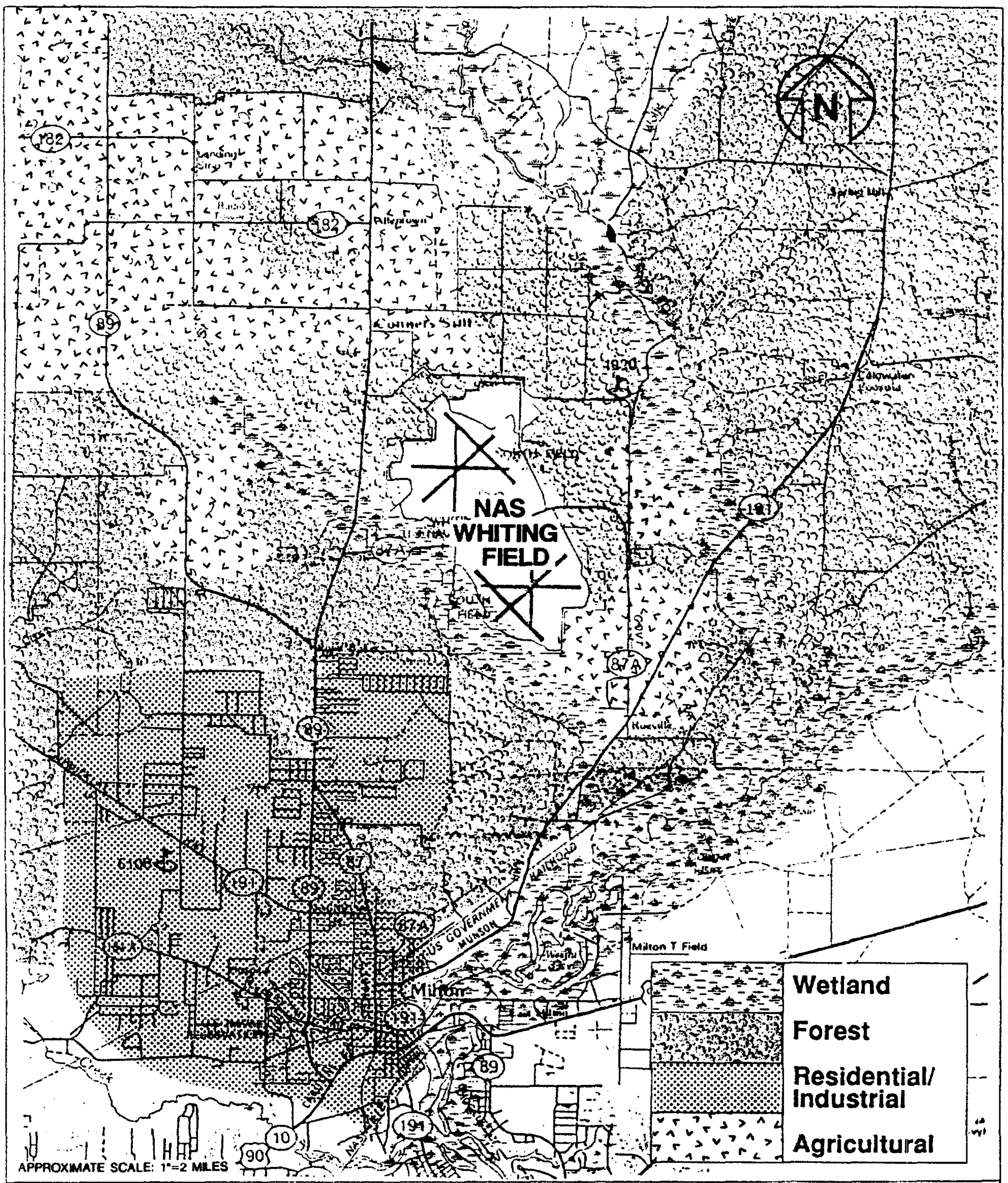


FIGURE 1-3

Land Use Distribution in the Vicinity of NAS Whiting Field

FLORIDA ATLAS & GAZETTEER, DeLORME MAPING COMPANY, 1987

RI/FS PROGRAM

NAS WHITING FIELD
MILTON, FLORIDA

The Navy IR program was designed to identify and abate or control contaminant migration resulting from past operations at Naval installations. The IR program is the Navy response authority under Section 120 of the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) of 1980 as amended by the Superfund Amendments and Reauthorization Act (SARA) of 1986 and Executive Order 12580. CERCLA requires that Federal facilities comply with the act, both procedurally and substantively. SOUTHNAVFACENGCOM is the agency responsible for the Navy IR program in the Southeastern United States. Therefore, SOUTHNAVFACENGCOM has the responsibility to process NAS Whiting Field through Preliminary Assessment (PA), Site Inspection (SI), priority listing, RI/FS, and remedial response selection in compliance with the guidelines of the National Oil and Hazardous Substances Pollution Contingency Plan (NCP) [40 Code of Federal Regulations (CFR) 300].

Section 105(a)(8)(A) of SARA required the U.S. Environmental Protection Agency (USEPA) to develop criteria in order to set priorities for remedial action based on relative risk to public health and the environment. To meet this requirement, USEPA has established the Hazard Ranking System (HRS) as Appendix A to the NCP. The HRS is a scoring system designed to assess relative threat due to documented or potential releases at a site. First promulgated in 1982, the HRS was amended in December 1990, effective March 14, 1991 (55 Federal Register No. 241:51532-51667), to comply with requirements of Section 105(c)(1) of SARA to increase the accuracy of the assessment of relative risk. The newly promulgated HRS II has been substantially revised and is designed to prioritize sites after the SI phase of the CERCLA process. The SI or extended SI is used to present the required data to expeditiously perform an HRS II ranking. At NAS Whiting Field, the SI was conducted as a Contamination Study, Verification Phase.

The RI/FS conducted at NAS Whiting Field is a component of the Navy IR program. The preliminary HRS score for NAS Whiting Field indicates that it may qualify for the National Priorities List (NPL). As such, the RI/FS for NAS Whiting Field follows the requirements of the NCP, as amended by SARA, and guidance for conducting Remedial Investigations and Feasibility Studies under CERCLA (USEPA, October 1988).

Prior to the implementation of the Phase I RI/FS Program, a PA and two sampling and analysis programs had been conducted at NAS Whiting Field. The PA, conducted as an Initial Assessment Study (IAS), was performed by Envirodyne Engineers in 1984 and published in 1985 (Envirodyne Engineers, 1985). Based on historical data, aerial photographs, field inspections, and personnel interviews, 16 disposal or spill sites of potential contamination and/or contaminant migration were initially identified at NAS Whiting Field by the IAS team. These are sites where waste disposal or accidents have occurred in the past.

The May 1985 IAS concluded that 15 of the 16 sites warranted further investigation, under the Navy's IR Program, to assess potential long-term impacts. Only Site 2, the Northwest Open Disposal Area, was judged to not warrant further consideration. A Confirmation Study, including sampling and monitoring of the sites, was recommended to confirm or deny the existence of the suspected contamination and to quantify the extent of any problems that may exist. The results of the Confirmation-Verification Study would then be used to evaluate the necessity of conducting mitigating actions or cleanup operations.

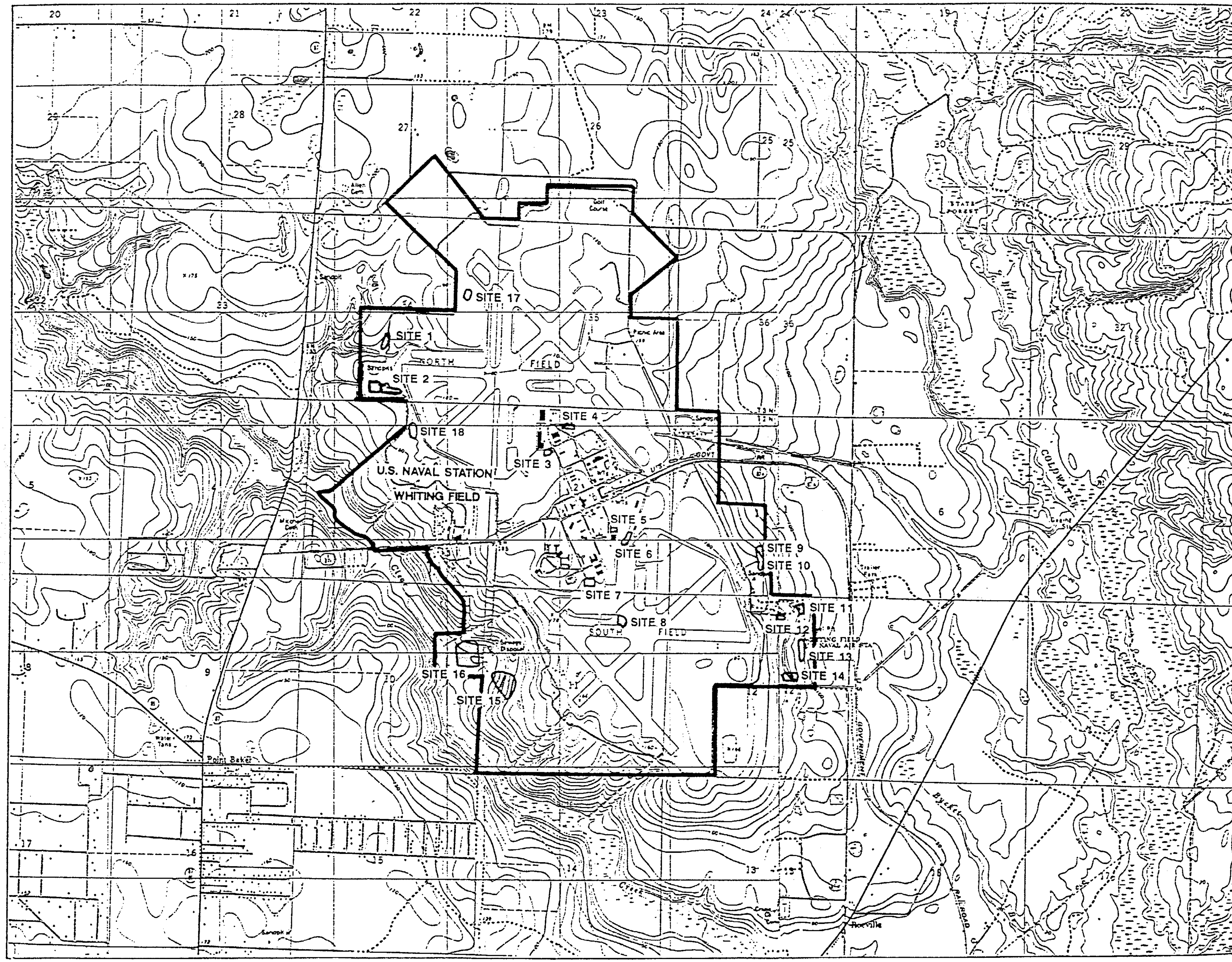
In November 1985, Geraghty & Miller, Inc., prepared for the Navy a plan of action entitled *Naval Assessment and Control of Installation Pollutants, Verification Study, NAS Whiting Field* (Geraghty & Miller, 1985b), which was subsequently submitted to the Florida Department of Environmental Regulation (FDER). This plan contained details of the proposed scope of work for the Verification Study. During discussion with FDER in December 1985, two additional sites (17 and 18) were added to the Verification Study. Both were active sites at that time where waste oils and fuels were burned in firefighting training exercises.

In addition, during 1985 one of the sites (Site 5, Battery Acid Seepage Pit) was investigated under Consent Order with the FDER. Data from this investigation has been compiled in a report entitled *Detection and Monitoring Program, Battery Shop Site, NAS Whiting Field, Florida* (Geraghty & Miller, November 1985a).

The location of the 18 sites are shown in Figure 1-4. Each of the sites was evaluated with regard to contamination characteristics, migration pathways, and pollutant receptors. Table 1-1 summarizes the information collected on these sites.

Work conducted during the course of the Verification Study began with the collection and assimilation of existing data and literature pertinent to the project and included the findings from the IAS. The field work was performed in May and June of 1986. Sixteen monitor wells were installed at locations around the facility. One surface water, 16 groundwater, and 46 soil samples were then collected for chemical analyses.

Historical records indicate that throughout the years of operation, NAS Whiting Field has generated a variety of wastes related to pilot training, the operation and maintenance of aircraft along with ground support equipment, and the station's facility maintenance activities. Prior to the establishment of hazardous waste management programs and programs to recycle waste oil, most of the hazardous wastes were reportedly disposed of onsite. Waste materials were disposed either in dumpsters that were emptied into onsite disposal areas or they went into waste oil bowsers, which probably were used for firefighting training. Envirodyne Engineers (1985) estimated that thousands of gallons of wastes including waste paints, paint thinners, solvents, waste oils, waste gasoline, hydraulic fluids, aviation gasoline (AVGAS), tank bottom sludges, polychlorinated biphenyls (PCBs) transformer fluids, and paint stripping wastewater were potentially dumped into onsite disposal areas. These disposal areas consisted of natural or man-made depressions located within the confines of the air station. In addition to the waste materials routinely disposed of onsite in the disposal areas, additional materials were reportedly released onsite as the result of accidents or equipment failure.



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SOURCE:
USGS QUADRANGLE MILTON NORTH, FLORIDA
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FIGURE 1-4

Location of Sites at
NAS Whiting Field

RI/FS PROGRAM

NAS WHITING FIELD
MILTON, FLORIDA

00225I02Z

Table 1-1 (Continued)
Summary of Potential Disposal Sites

Technical Memorandum No. 2
NAS Whiting Field
Milton, Florida

Site No.	Site Name and Type	Location	Period of Operation	Types of Material Disposed	Comments
1	Northwest Disposal Area (landfill)	North Field, west side	1943-1965	Refuse, waste paints, thinners, solvents, waste oils, and hydraulic fluids.	Secondary disposal area during this period; site covers 5 acres.
2	Northwest Open Disposal Area (landfill)	North Field, west side	1976-1984	Construction and demolition debris, tires, and furniture.	Former borrow pit location, commonly referred to as the "Wood Dump."
3	Underground Waste Solvent Storage Area (tank)	North Field, south of Building 2941	1980-1984	Waste solvents, paint stripping residue, and 120-gallon spill.	Wastes generated by paint stripping operations.
4	North AVGAS Tank Sludge Disposal Area	North Field, north of Tow Lane	1943-1968	Tank bottom sludge containing tetraethyl lead.	Sludge disposal in shallow holes near tanks.
5	Battery Acid Seepage Pit (contaminated soil)	South Field, near Building 1478	1964-1984	Waste electrolyte solution containing heavy metals and waste battery acid.	Pits located 110 feet from potable supply well (W-S2).
6	South Transformer Oil Disposal Area (contaminated soil)	South Field, Building 1478	1940's-1960's	PCB-contaminated dielectric fluid.	Disposal in "0-2" drainage ditch.
7	South AVGAS Tank Sludge Disposal Area (landfill and tanks)	South Field, west of Building 1406	1943-1968	Tank bottom sludge containing tetraethyl lead.	Sludge disposed in shallow holes near tanks.
8	AVGAS Fuel Spill Area (contaminated soil)	South Field, south of Building 1406	Summer 1972	AVGAS containing tetraethyl lead.	Fuel spill of about 25,000 gallons on an area of about 2 acres.
9	Waste Fuel Disposal Pit (landfill)	South Field, east side	1950's-1960's	Waste AVGAS containing tetraethyl lead.	Fuel disposed in former borrow pit.
10	Southeast Open Disposal Area (A) (landfill)	South Field, southeast area	1965-1973	Construction and demolition debris, waste solvents, paint, oils, hydraulic fluid, PCBs, pesticides, and herbicides.	Secondary disposal area during this period; site covers about 4 acres.

See notes at end of table.

Table 1-1 (Continued)
Summary of Potential Disposal Sites

Technical Memorandum No. 2
NAS Whiting Field
Milton, Florida

Site No.	Site Name and Type	Location	Period of Operation	Types of Material Disposed	Comments
11	Southeast Open Disposal Area (B) (landfill)	South Field, southeast area	1943-1970	Construction and demolition debris, waste solvents, paint, oils, hydraulic fluid, and PCBs.	Secondary disposal area during this period; site covers about 3 acres.
12	Tetraethyl Lead Disposal Area (waste pile)	South Field, southeast area	May 1, 1968	Tank bottom sludge and fuel filters contaminated with tetraethyl lead.	Disposal area posted with warning; site consists of two earth covered mounds; 25 foot by 25 foot area.
13	Sanitary Landfill (landfill)	South Field, southeast area	1979-1984	Refuse, waste solvents, paint, hydraulic fluids, and asbestos.	Primary sanitary landfill, potentially received hazardous wastes the first year of operation.
14	Short-Term Sanitary Landfill (landfill)	South Field, southeast area	1978-1979	Refuse, waste solvents, oils, paint, and hydraulic fluids.	Primary sanitary landfill for brief period; relocated due to drainage problems.
15	Southwest Landfill (landfill)	South Field, southwest area	1965-1979	Refuse, waste paints, oils, solvents, thinners, asbestos, and hydraulic fluid.	Primary landfill for this time period; covers about 15 acres.
16	Open Disposal and Burning Area (landfill)	South Field, southwest area	1943-1965	Refuse, waste paints, oils, solvents, thinners, PCBs, and hydraulic fluid.	Primary disposal area for this time period; covers about 10 acres.
17	Crash Crew Training Area (contaminated soil)	North Field, west side	1951-Present	JP-4.	Waste fuels and some solvents ignited, then extinguished.
18	Crash Crew Training Area (contaminated soil)	North Field, west side	1951-Present	JP-4.	Waste fuels and some solvents ignited, then extinguished.

Notes: AVGAS = aviation gasoline.
PCB = polychlorinated biphenyls.

The results of the Verification Study reported to SOUTHNAVFACENGCOM by Geraghty & Miller (*Verification Study: Assessment of Potential Ground-Water Pollution at Naval Air Station Whiting Field*, December 1986) provided an incomplete assessment of the physical as well as the chemical conditions currently existing at NAS Whiting Field. Groundwater contamination was detected at some sites and not at others. The study concluded that many of the monitoring wells were not located downgradient of the intended study site and that additional work was needed to characterize the hydrogeologic conditions and the chemical contamination conditions that exist at NAS Whiting Field. The Verification Study is the former IR program counterpart to the SI.

Of the 18 sites identified to date, 13 are scheduled for further study under the Navy's IR program. Due to the fact that it only received construction and demolition debris, Site 2, the Northwest Open Disposal Area, was judged to warrant no further consideration early in the IR program. Site 5, the Battery Acid Seepage Pit, was extensively studied in 1985 (Geraghty & Miller, 1985a) in response to an FDER Consent Order (84-0253). Results indicated no significant contamination resulting from past activities at the Battery Acid Shop and the Consent Order was recommended to be rescinded on April 15, 1987. However, the presence of benzene in the existing monitoring wells surrounding the seepage pit warrants further consideration. As such, the investigation of benzene contamination around Site 5 is coupled with the field and laboratory investigation proposed for production well W-S2. Sites 4, 7, and 8 are slated for investigation and remediation, if necessary, under the Navy's Underground Storage Tank (UST) program and, therefore, are not incorporated in the Navy's IR program. Table 1-2 presents a summary of past and projected investigative programs for the 18 sites within the RI/FS and UST programs.

The Jordan Phase I RI Workplan (June 1990) provides a summary of the regional and installation-specific environmental setting, current and historical industrial operations, and summary of the verification study and the Site 5, Battery Shop data which will not be repeated in the technical memorandum. As appropriate, data from these sources will be incorporated into the assessment.

<u>1.2 OBJECTIVES OF HYDROGEOLOGICAL INVESTIGATION</u>. The objectives of the RI Phase I hydrogeological investigation were to:

- characterize the regional groundwater flow system,
- characterize the groundwater flow system at the following six sites or site groupings (Sites 1/17/18, Site 3, Sites 4/5/7/8, Sites 9/10, Sites 11/14, and Sites 15/16),
- estimate the aquifer characteristics (e.g., hydraulic conductivities, storativity, and transmissivity), and
- gain additional hydrogeologic data (e.g., horizontal gradients and seepage velocities).

Several tasks, including installation of monitoring wells and piezometers, water level measurements, slug tests, pumping test, piezocone penetrometer (PCPT) soundings, and *in-situ* groundwater sampling, were conducted during the RI Phase I field program to define the hydrogeologic regime at NAS Whiting Field.

Table 1-2
Summary of Site Investigations

Technical Memorandum No. 3
NAS Whiting Field
Milton, Florida

Site Number	Site Name	Previous Studies: IAS	Previous Studies: Verification Study	Previous Studies: Consent Order	Ongoing RI/FS	Navy's UST Program
1	Northwest Disposal Area	*	*		*	
2	Northwest Open Disposal Area	*				
3	Underground Waste Solvent Storage Area	*	*		*	
4	North AVGAS Tank Sludge Disposal Area	*	*			*
5	Battery Acid Seepage Pit	*		*		
6	South Transformer Oil Disposal Area	*	*		*	
7	South AVGAS Tank Sludge Disposal Area	*	*			*
8	AVGAS Fuel Spill Area	*	*			*
9	Waste Fuel Disposal Pit	*	*		*	
10	Southeast Open Disposal Area (A)	*	*		*	
11	Southeast Open Disposal Area (B)	*	*		*	
12	Tetraethyl Lead Disposal Area	*	*		*	
13	Sanitary Landfill	*	*		*	
14	Short-Term Sanitary Landfill	*	*		*	
15	Southwest Landfill	*	*		*	
16	Open Disposal and Burning Area	*	*		*	
17	Crash Crew Training Area		*		*	
18	Crash Crew Training Area		*		*	

Notes: IAS = Initial Assessment Study.
RI/FS = Remedial Investigation/Feasibility Study.
UST = underground storage tank.
AVGAS = aviation gasoline.

Information derived from the above tasks will be used to provide sufficient data to propose a no further action (for groundwater) remedial alternative or provide information to optimize explorations to further delineate the nature and extent of groundwater contamination.

The methods and the results of the physical hydrogeologic investigation are detailed in the following sections. Methods and results pertaining to groundwater quality are presented in Technical Memorandum No. 5.

2.0 FIELD PROGRAM SUMMARY

2.1 WATER LEVELS. Two synoptic rounds of water level measurements were collected during Phase I of the RI. The groundwater measurements were collected using a Solinst™ electronic water level indicator. The first round of water level measurements was collected from 16 of the 20 existing monitoring wells. Water level measurements could not be collected from four existing monitoring wells at Site 5, because the wells were equipped with groundwater sampling devices that restricted access to the groundwater in the wells.

The second round of water level measurements was collected at the completion of the RI Phase I field program. Water levels were measured in 16 existing monitoring wells, 1 monitoring well from Site 5 (the groundwater sampling device was removed for pumping test monitoring), and the 8 newly installed monitoring wells and piezometers.

2.2 SLUG TESTS. Single-hole, *in-situ*, permeability tests (slug tests) were performed on 15 existing or new monitoring wells. Data collected from the slug tests were used to calculate the hydraulic conductivity in the sand-and-gravel aquifer. Results of the slug test analysis are presented in Section 3.2.3.

Three rising head and three falling head slug tests were performed in each monitoring well. The slug tests were conducted in accordance with USEPA Method 9100. Generally, the test consists of the introduction and withdrawal of a slug of water or a weight, and the measurement of the change in water level, or fluid pressure, in the well over time.

The slug tests conducted at NAS Whiting Field used a weighted slug (a 5-foot long, 2-inch diameter, polyvinyl chloride (PVC) tube filled with sand) to displace a volume of water in the monitoring well. The rising head slug test consisted of placing the slug below the groundwater level in the monitoring well, allowing the groundwater to return to static conditions, quickly removing the slug from the groundwater, and measuring the increase in water level over time with a pressure transducer and a Hermit™ 1000 C data logger. The falling head slug test is just the opposite of a rising head slug test. The slug is introduced into groundwater in the monitoring well and the resultant drop of the water level is measured.

Slug tests were not conducted in monitoring wells WHF-3-E, WHF-3-W, and WHF-7-1 because contamination was present nor in four of the Site 5 monitoring wells because groundwater sampling devices were present in the wells.

2.3 PUMPING TEST. The pumping test was conducted at the south production well to calculate hydraulic properties of the sand-and-gravel aquifer. This test began on March 14, 1991.

Groundwater levels in the deep observation well (WHF-5-OW-1), the two piezometers (WHF-5-P2-1 and WHF-5-P2-2), and one monitoring well (GMW-3) at the Battery Shop Site 5 were monitored by pressure transducers and two Hermit™ data loggers. Water levels in the production well were collected manually. Pumping rates of the south production well were monitored at the water plant. Barometric pressure readings were collected on one of the data logger channels and were used to correct the water level data reading.

Water levels in the shallow observation well were not monitored due to an obstruction in the bottom of the well. The three remaining monitoring wells at the Battery Shop were also not monitored due to the presence of dedicated sampling devices in the wells.

The pumping portion of the test was terminated on March 20, 1991, after a senior geohydrologist had reviewed the accumulated data and determined the data were sufficient to characterize the aquifers' hydraulic properties. Once the pump in the production well was turned off, the pumping test recovery segment began. The same wells and piezometers monitored with pressure transducers and data loggers during the pumping segment were monitored during the recovery segment.

Three groundwater samples from the south production well were collected ahead of the granular activated carbon (GAC) filter during the pumping test. One sample was collected on the first day of the pumping test, one on the second day, and one on the final day. The samples were shipped to Savannah Laboratories and Environmental Services, Inc., Tallahassee, Florida, for analysis of target compound list (TCL) volatile organic compounds (VOCs).

3.0 RESULTS AND INTERPRETATION

3.1 REGIONAL HYDROGEOLOGY. There are three major groundwater aquifers within the region. The first is a shallow aquifer, which is both artesian and non-artesian (the sand-and-gravel aquifer), and two other deep artesian aquifers (the Upper Floridan aquifer and the Lower Floridan aquifer). Virtually all groundwater withdrawn in Escambia and Santa Rosa Counties comes from the surficial sand-and-gravel aquifer. Descriptions of the aquifers and accompanying stratigraphic units (Geraghty & Miller, 1985) are presented in the NAS Whiting Field Workplan (Jordan, 1990) and summarized below. A generalized hydrogeologic section for Santa Rosa County is shown in Figure 3-1.

- **Sand-and-Gravel Aquifer**. Sediments, extending to a depth of about 350 feet, comprise the sand-and-gravel aquifer, which is subdivided into two units. The water table or upper part of the sand-and-gravel aquifer does not constitute a source for large water supplies; however, its primary importance is to recharge the lower more productive zone of the aquifer. According to an aquifer test in the Milton area, the clayey sand, locally confining, unit separating the upper and lower aquifer zones is very leaky. Most large capacity wells in the area, such as the NAS Whiting Field supply wells, are screened into the lower part of this aquifer from about 180 to 330 feet below land surface (bls).

  The sand-and-gravel aquifer includes the upper Miocene coarse clastics, the Citronelle Formation, and marine terrace deposits. These three units have similar hydraulic properties and sometimes are indistinguishable. The aquifer consists of poorly sorted, fine- to coarse-grained sands with gravel and lenses of clay, which may be as much as 60-feet thick. In some areas, the formation also contains wood fragments of all sizes, including whole tree trunks, occurring mostly in layers that may be as much as 25-feet thick.

  The formation contains lensatic zones within the sand that are cemented by iron-oxide minerals. The lenses, known locally as hardpans, have lower permeabilities and, along with the clay lenses, are responsible for the occurrence of perched water tables and semi-artesian conditions in the aquifer.

  The water from the sand-and-gravel aquifer is considered to be of excellent quality. Total dissolved solids and total hardness are generally less than 50 milligrams per liter (mg/ℓ). However, because of high levels of dissolved carbon dioxide, the water is acidic with an ambient pH as low as 5.0 and locally it may contain high concentrations of iron.

- **Floridan Aquifer System**. Underlying the sediments of the sand-and-gravel aquifer is the thick (±300 feet), relatively impermeable Pensacola clay, below which are thick layers of limestone and shale to a depth of nearly 2,000 feet.

  The limestone layers constitute the regionally extensive Floridan aquifer system, which, in this area, is divided into an upper and lower part separated by the Bucatunna Clay member of the Byram Formation. The Upper

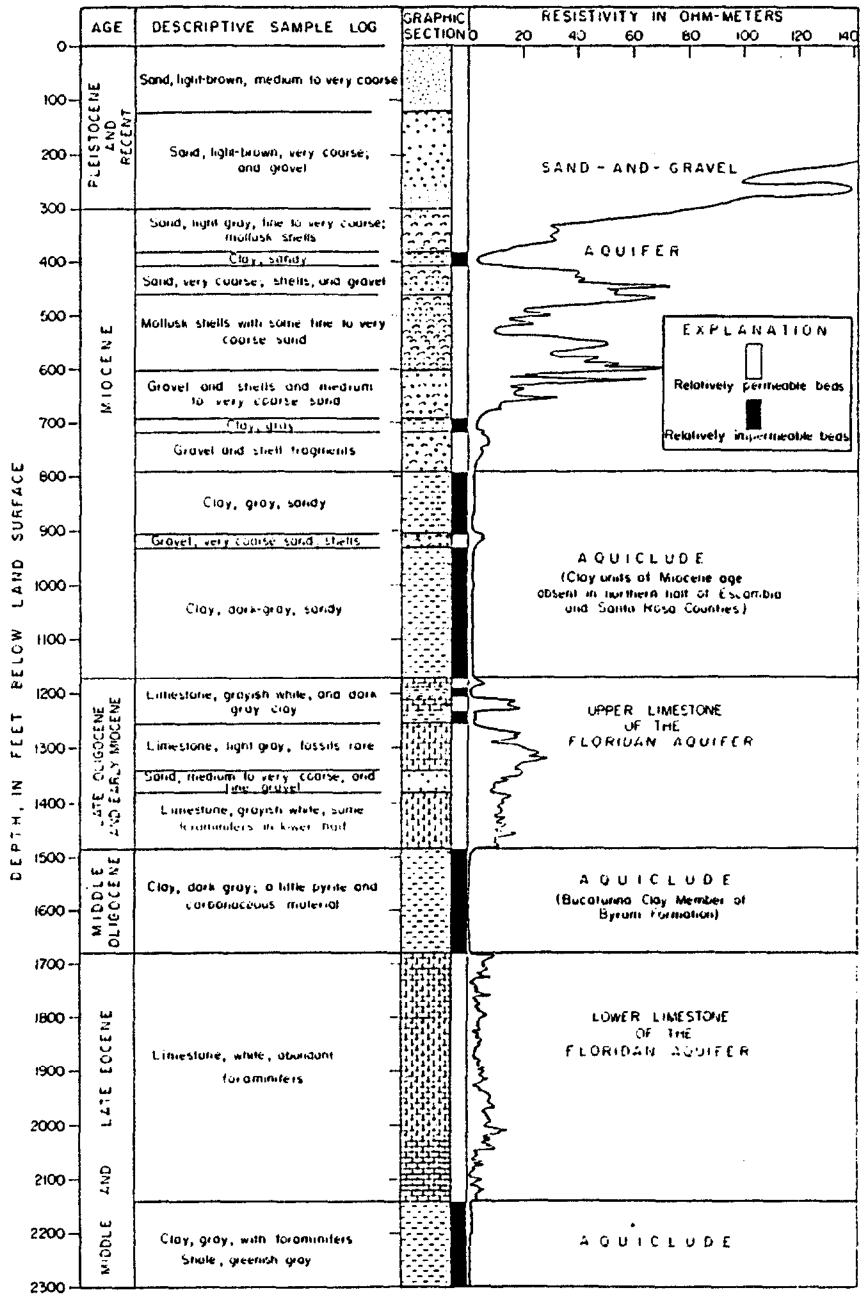


FIGURE 3-1

Generalized Hydrogeological Section of Santa Rosa County

RI/FS PROGRAM

NAS WHITING FIELD
MILTON, FLORIDA

Floridan aquifer is an important source of water in areas east of Santa Rosa County; however, toward the west, it is increasingly mineralized and is generally not used as a water supply. The Lower Floridan aquifer is highly mineralized in the NAS Whiting Field area and is, in fact, designated for use as a waste disposal injection zone. The Floridan aquifer receives little or no recharge from the sand-and-gravel aquifer because of the Pensacola clay confining unit. The potentiometric surface of the Floridan aquifer system in the NAS Whiting Field area is about 50 to 55 feet above mean sea level (msl) and the direction of groundwater flow is southeast.

3.2 INSTALLATION-SPECIFIC HYDROGEOLOGY. The installation-specific aquifer system is the same as the regional aquifer system described in Section 3.1. As previously discussed, the three aquifer systems are the sand-and-gravel, the Upper Floridan, and the Lower Floridan.

Data from the boring logs and PCPT soundings indicate the sand-and-gravel aquifer below NAS Whiting Field is primarily composed of poorly graded, fine- to coarse-grained quartz sand and gravel with interbedded silt and clay layers that may be as much as 30-feet thick. Prior to the RI Phase I field program, a continuous clay layer, approximately 20- to 30-feet thick, was believed to be a continuous confining unit throughout the sand-and-gravel aquifer at depths ranging from 50 to 180 feet bls. Based on results of the Phase I RI field program, this clay unit appears to be discontinuous under the installation creating only locally confining conditions. Geologic logs and cross sections are described in Technical Memorandum No. 1, Geologic Assessment.

The potentiometric surface of the sand-and-gravel aquifer at NAS Whiting Field ranges from 10 to 128 feet bls (40 to 80 feet above the National Geodetic Vertical Datum [NGVD] of 1979). Depth to groundwater measurements and groundwater elevations collected in November 1990 and July 1991 are summarized in Table 3-1.

3.2.1 Groundwater Flow Directions Figure 3-2 presents an interpreted groundwater contour map developed from depth to groundwater measurements collected during the RI Phase I field program. Groundwater below the western part of NAS Whiting Field appears to flow in a southwesterly direction toward Clear Creek at a relatively steep gradient. Groundwater beneath the eastern half of NAS Whiting Field flows in a southeasterly direction and bends towards Big Coldwater Creek at a more moderate gradient. In the center of NAS Whiting Field, groundwater migrates in a southeasterly direction at a relatively flat gradient.

In order to evaluate groundwater flow at the identified sites, sites in close proximity to each other were grouped together to create a larger groundwater database. The site groupings are as follows.

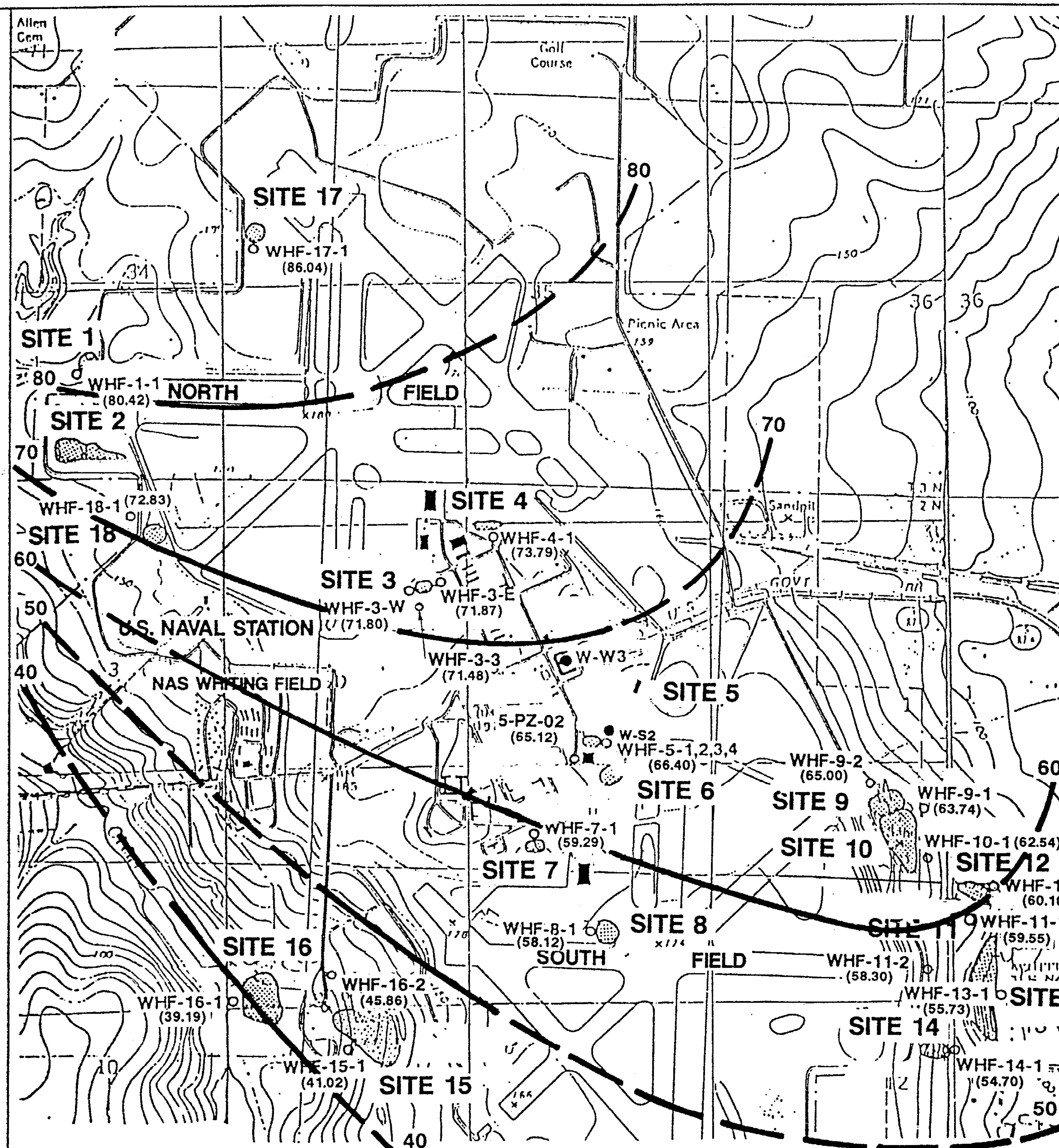
Sites 1, 2, 17, and 18 (the northwestern area)
Site 3 (the north field area)
Sites 4, 5, 6, 7, and 8 (the industrial and production well area)
Sites 9 and 10 (the eastern area)
Sites 11, 12, 13, and 14 (the eastern area)
Sites 15 and 16 (the southeastern area)

Table 3-1
Groundwater Elevations

Technical Memorandum No. 2
NAS Whiting Field
Milton, Florida

Well Designation	Depth to Water, Feet Below TOC		TOC Elevation (feet NGVD)	Groundwater Elevation (feet NGVD)	
	11/12/90	7/23/91		11/12/90	7/23/91
WHF-1-1	62.69	62.18	142.60	79.91	80.42
WHF-3-1	103.50	103.10	174.90	71.40	71.80
WHF-3-2	104.03	103.55	175.42	71.39	71.87
WHF-3-3	--	106.78	178.26	--	71.48
WHF-4-1	99.34	98.70	172.49	73.15	73.79
WHF-5-1	--	117.82	184.22	--	66.40
WHF-7-1	128.56	128.48	187.77	59.21	59.29
WHF-8-1	115.69	114.80	172.92	57.23	58.12
WHF-9-1	86.51	92.86	146.60	60.09	63.74
WHF-9-2	--	96.19	161.19	--	65.00
WHF-10-1	86.84	84.23	146.77	59.93	62.54
WHF-11-1	60.18	57.15	116.70	56.52	59.55
WHF-11-2	--	89.86	148.17	--	58.30
WHF-12-1	79.10	76.39	136.49	57.39	60.10
WHF-13-1	49.88	46.96	102.69	52.81	55.73
WHF-14-1	87.69	85.03	139.73	52.04	54.70
WHF-15-1	26.00	25.19	66.21	40.21	41.02
WHF-16-1	11.09	10.70	49.89	38.80	39.19
WHF-16-2	--	36.22	82.08	--	45.86
WHF-17-1	108.10	108.62	194.66	86.56	86.04
WHF-18-1	91.61	90.66	163.49	71.87	72.83
WHF-5-OW-01	--	121.36	185.80	--	64.44
WHF-5-OW-02	--	115.64	186.02	--	70.38
WHF-5-PZ-01	--	121.87	186.01	--	64.14
WHF-5-PZ-02	--	120.78	185.90	--	65.12

Notes: TOC = top of casing.
feet NGVD = feet above National Geodetic Vertical Datum of 1929.



N

LEGEND

- MONITORING WELL (ELEVATIONS IN FEET NGVD)
- (XX.XX) WATER LEVEL
- GROUNDWATER SUPPLY WELL
- 70 POTENTIOMETRIC SURFACE CONTOUR

APPROXIMATE SCALE

0 2000 4000 8000

FIGURE 3-2
GROUNDWATER CONTOUR MAP
JULY 1991

RI/FS PROGRAM

NAS WHITING FIELD
MILTON, FLORIDA

00225I03Z

Based on groundwater elevations calculated from groundwater measurements collected in July 1991, groundwater flow directions at each of the above site groupings could be determined.

Groundwater contour maps for the six site groupings are presented in Figures 3-3 through 3-8. Groundwater flow direction at each of the site groupings is summarized as follows.

Site Grouping	Groundwater Flow Direction
Sites 1, 2, 17, and 18	South-southwest
Site 3	South
Sites 4, 5, 6, 7, and 8	South
Sites 9 and 10	Southeast
Sites 11, 12, 13, and 14	Southeast
Sites 15 and 16	Southwest

The specific site grouping groundwater flow directions correlate well with the overall installation groundwater flow pattern. No localized groundwater flow anomalies appear to be present.

3.2.2 Horizontal Gradients Horizontal gradients of the sand-and-gravel aquifer potentiometric surface were calculated from groundwater level measurements collected in July 1991 from the existing and newly installed monitoring wells.

The steepest horizontal gradient estimated at the installation, 0.0075 feet per foot (ft/ft), was from Site 1 to Clear Creek. The flattest gradient, 0.0018 ft/ft, was from Site 17 to Site 7.

The estimated horizontal gradient across the site groupings ranged from 0.0016 ft/ft (Sites 4, 5, 6, 7, and 8) to 0.0076 ft/ft (Sites 15 and 16). Horizontal gradients across the site groupings are summarized as follows.

Site Grouping	Horizontal Gradient (ft/ft)
Sites 1, 2, 17, and 18	0.0029
Site 3	0.0021
Sites 4, 5, 6, 7, and 8	0.0016
Sites 9 and 10	0.0023
Sites 11, 12, 13, and 14	0.0034
Sites 15 and 16	0.0076

3.2.3 Hydraulic Conductivity and Seepage Velocity

Hydraulic Conductivity. Data collected from the single-hole permeability tests (slug tests) were evaluated using the Aqtesolv™ groundwater software package to estimate hydraulic conductivity of the sand-and-gravel aquifer. The data was analyzed within the Aqtesolv™ program using a method developed by Bouwer and Rice (1976) for calculating the hydraulic conductivity of an aquifer from partially penetrating wells in an unconfined aquifer.

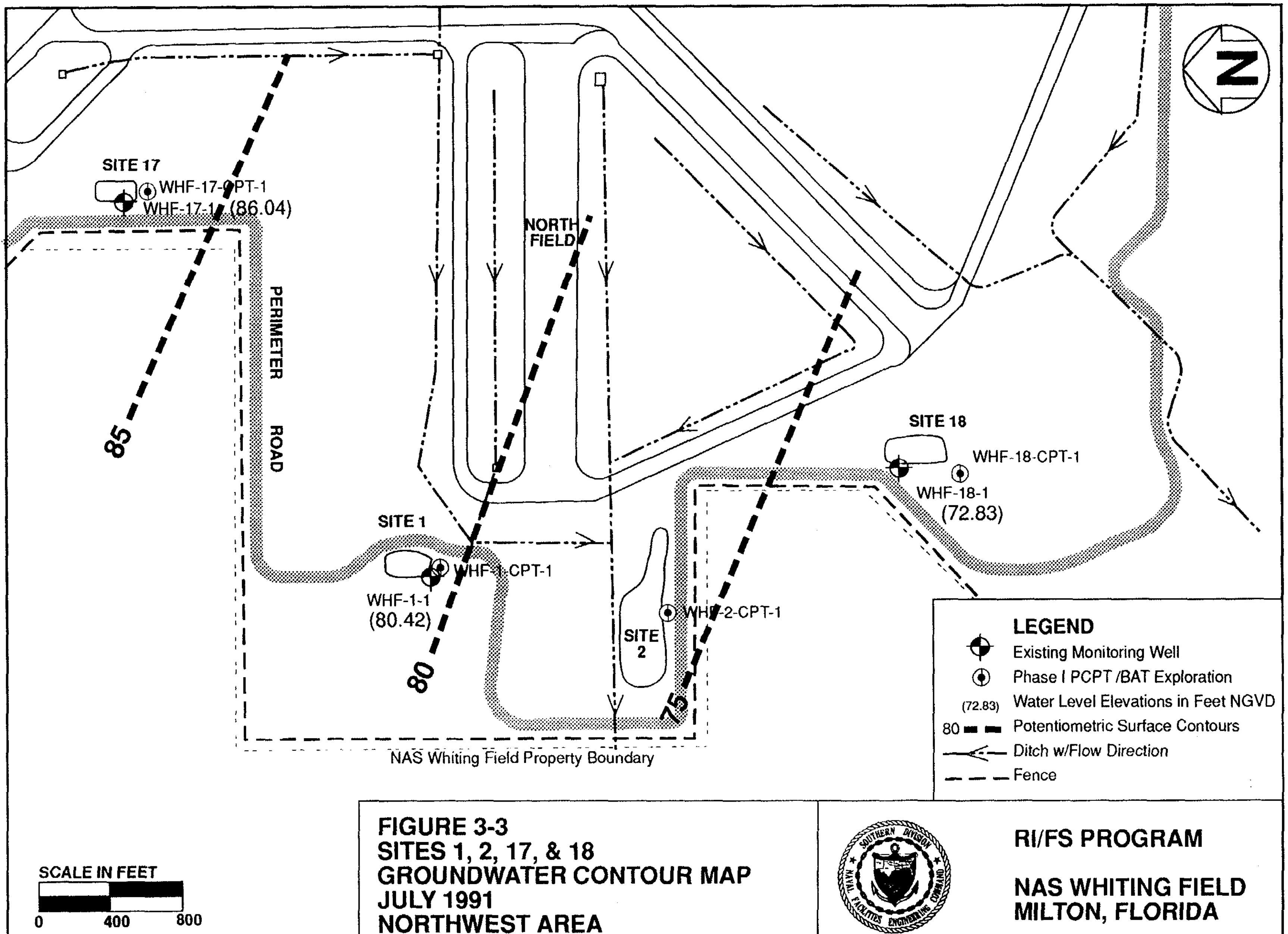


FIGURE 3-3
SITES 1, 2, 17, & 18
GROUNDWATER CONTOUR MAP
JULY 1991
NORTHWEST AREA

RI/FS PROGRAM

NAS WHITING FIELD
MILTON, FLORIDA

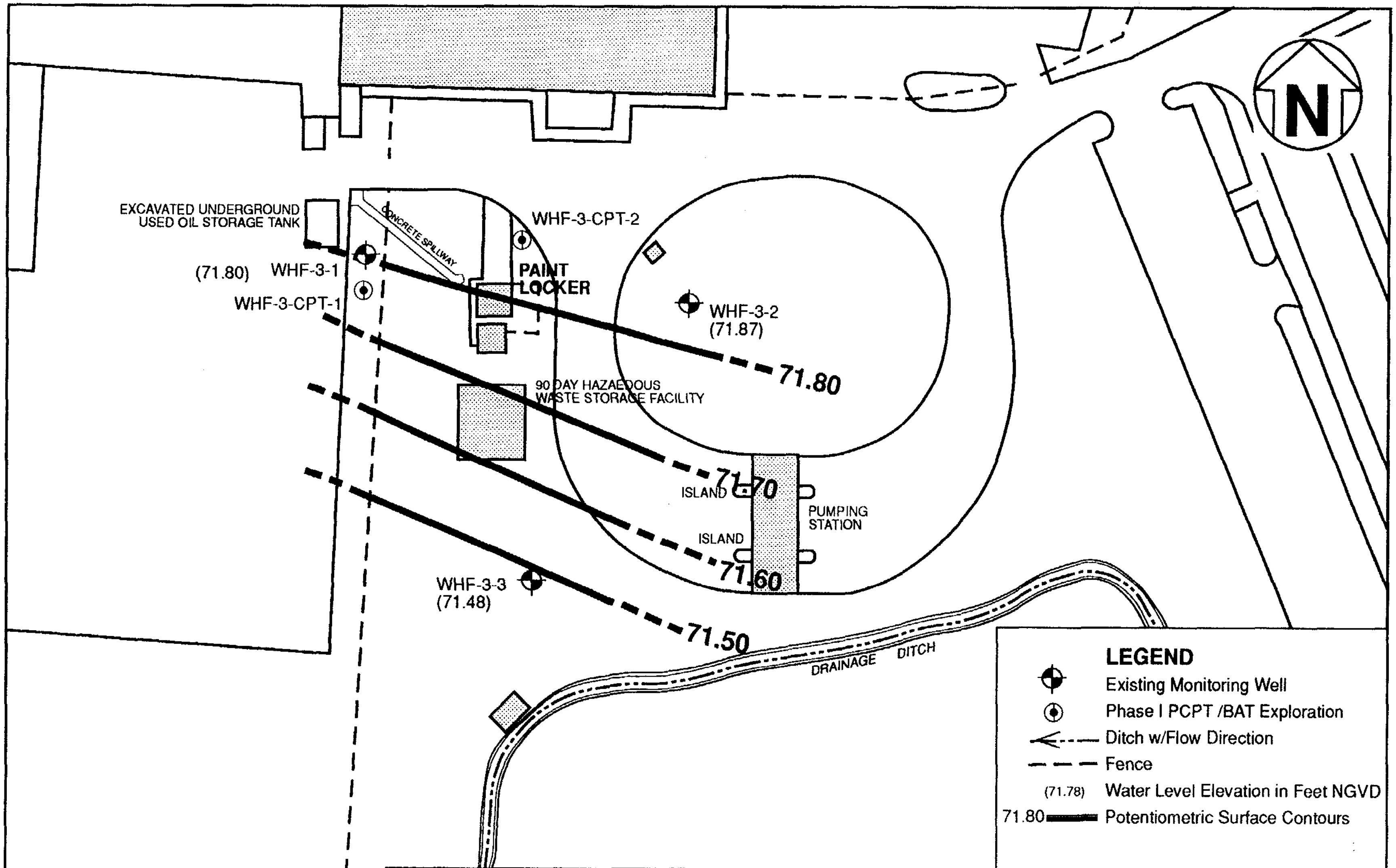


FIGURE 3-4
SITE 3
GROUNDWATER CONTOUR MAP
JULY 1991

RI/FS PROGRAM

NAS WHITING FIELD
MILTON, FLORIDA

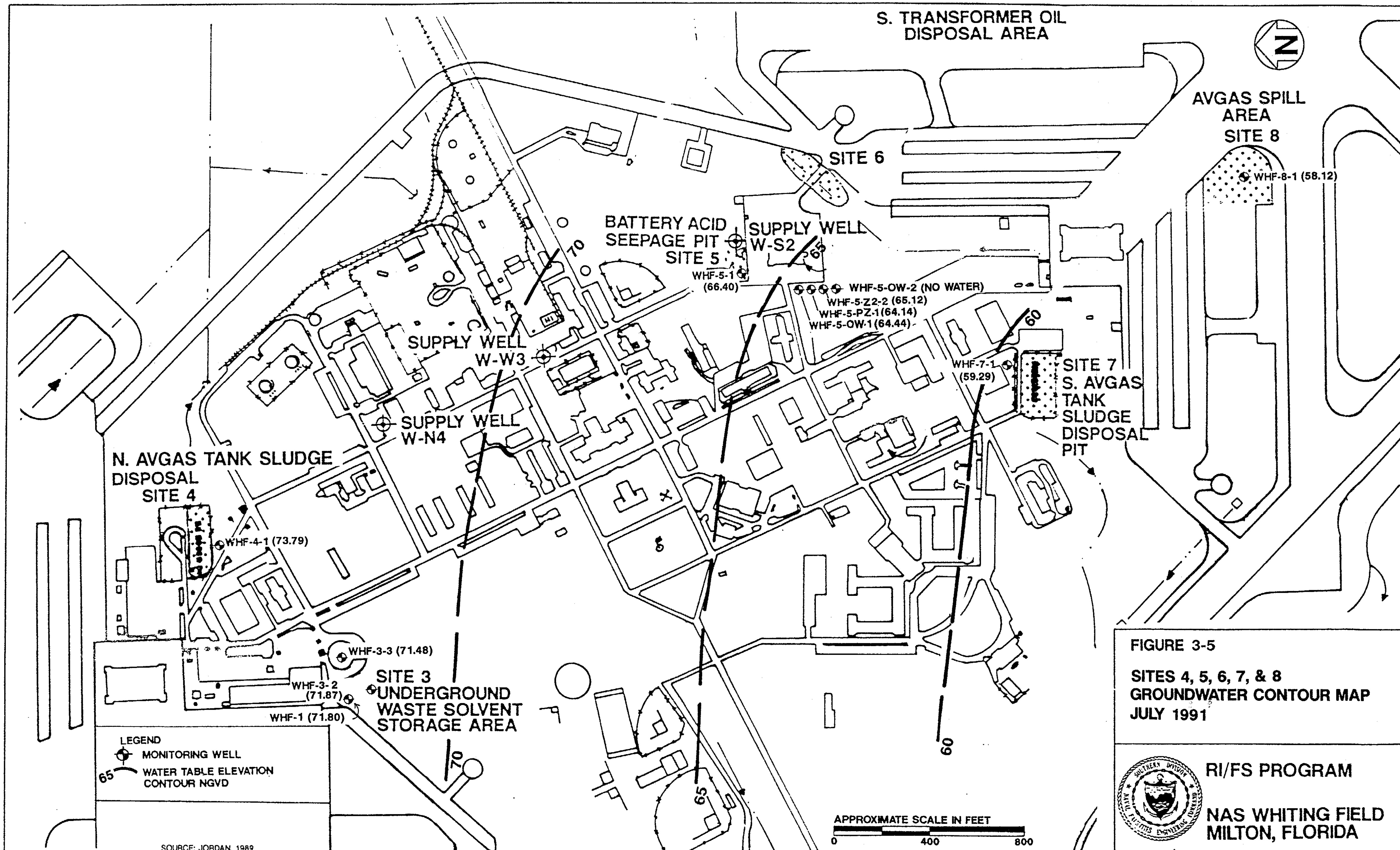


FIGURE 3-5

SITES 4, 5, 6, 7, & 8 GROUNDWATER CONTOUR MAP JULY 1991

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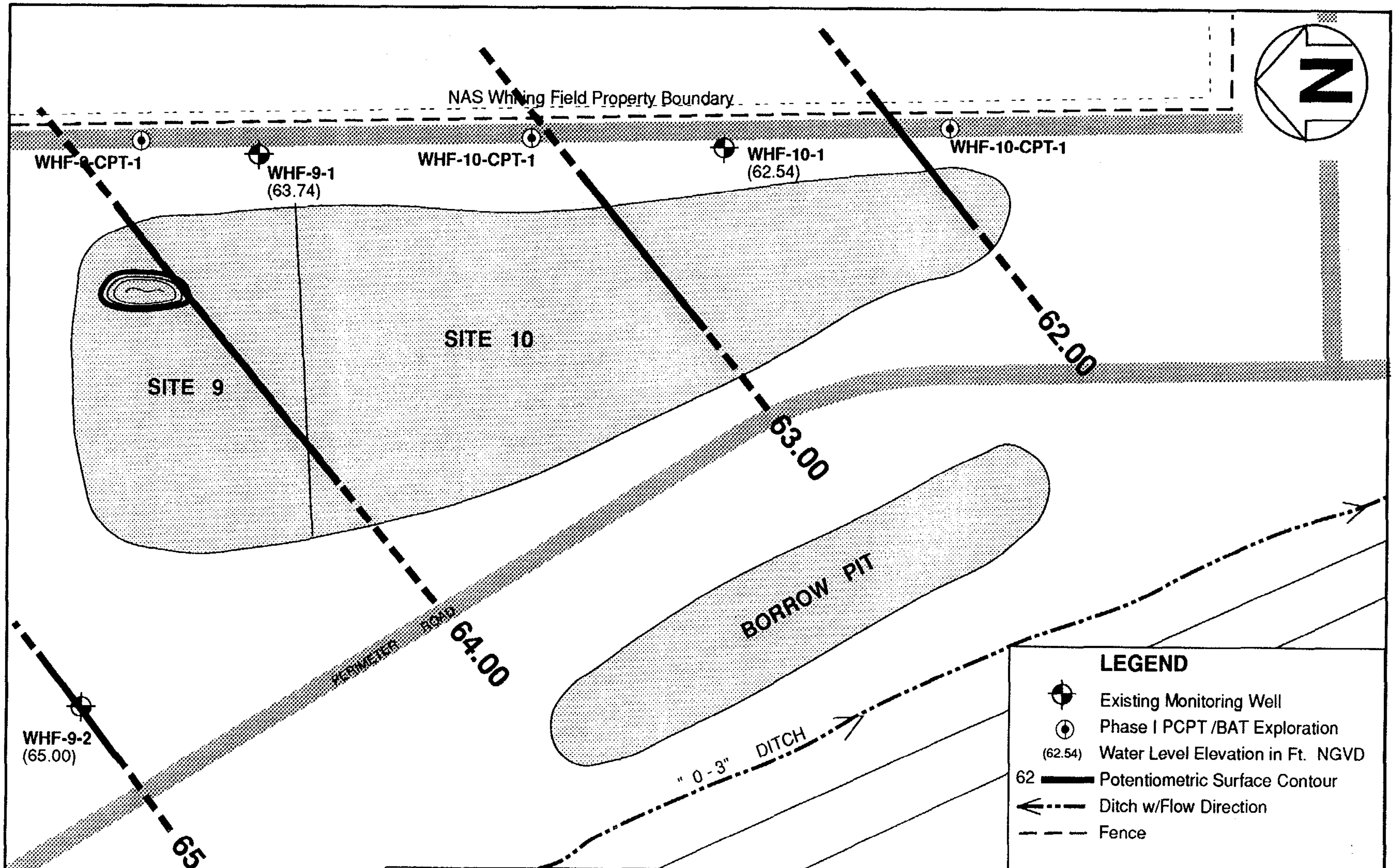
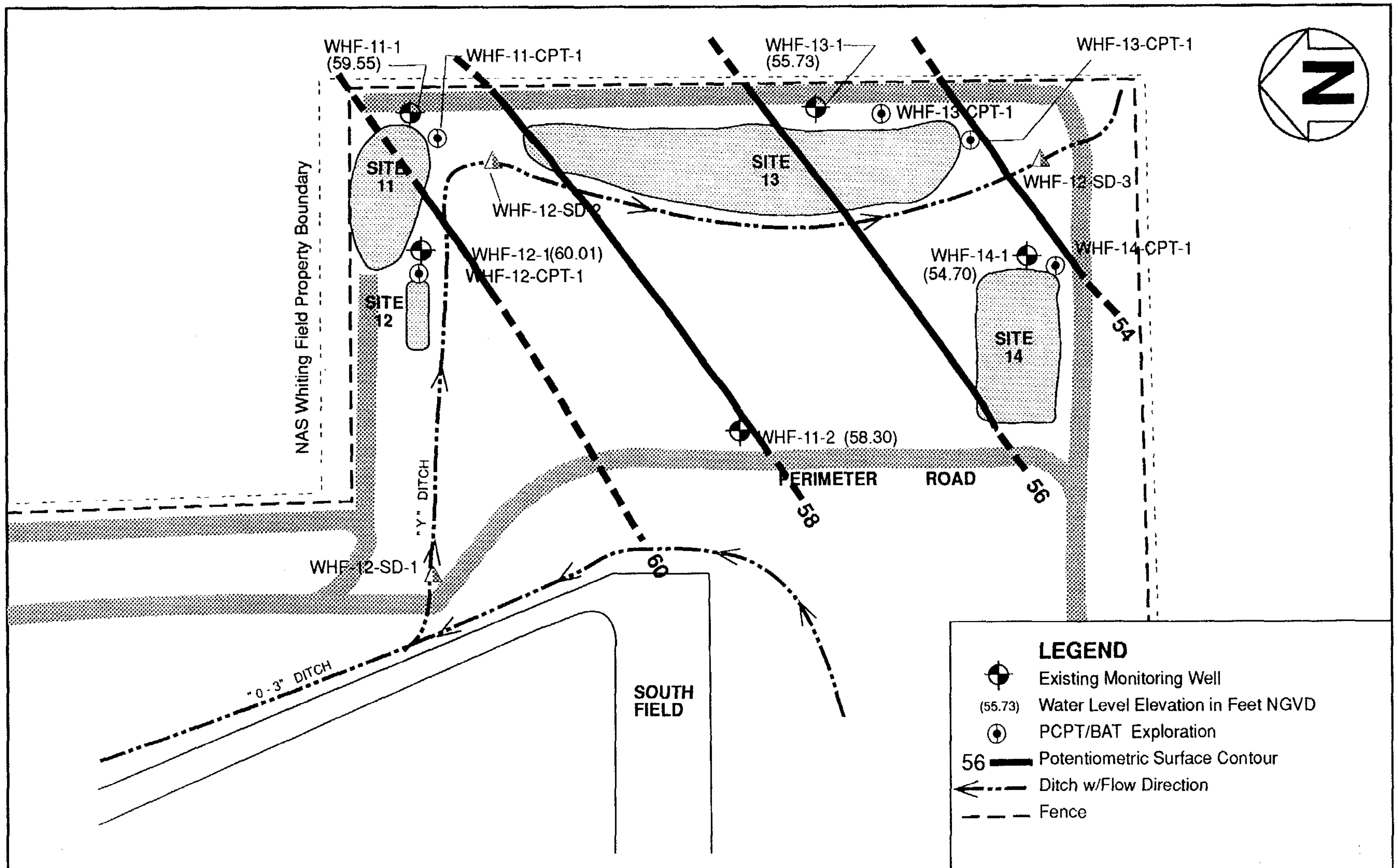


FIGURE 3-6
SITES 9 & 10
GROUNDWATER CONTOUR MAP
JULY 1991

RI/FS PROGRAM

NAS WHITING FIELD
MILTON, FLORIDA



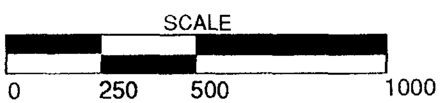


FIGURE 3-7
SITES 11, 12, 13, & 14
GROUNDWATER CONTOUR MAP
JULY 1991

RI/FS PROGRAM

NAS WHITING FIELD
MILTON, FLORIDA

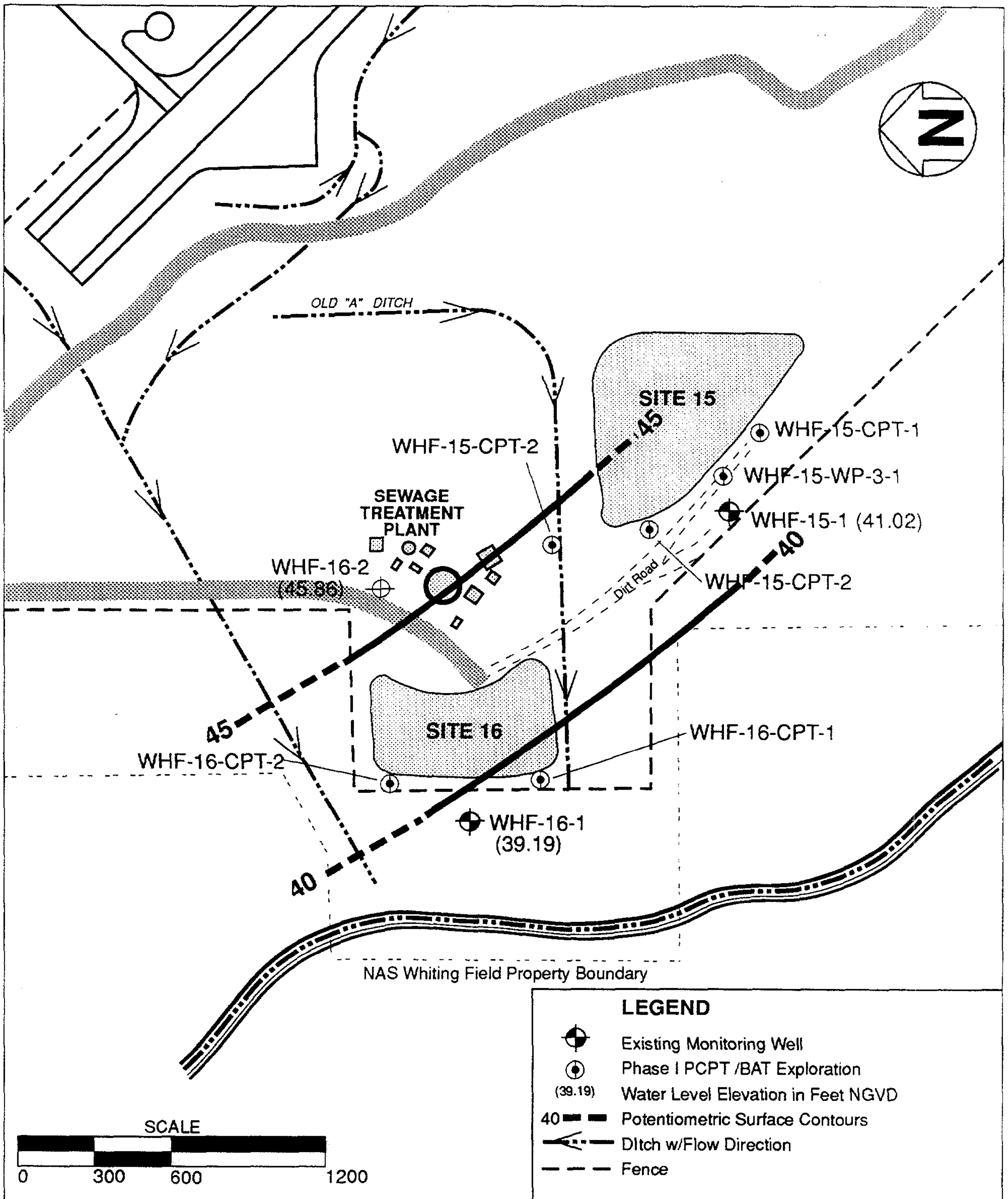


FIGURE 3-8
SITES 15 & 16
GROUNDWATER CONTOUR MAP
JULY 1991

RI/FS PROGRAM

NAS WHITING FIELD
MILTON, FLORIDA

Table 3-2
Comparison of Average Falling and Rising Head Slug Test

Technical Memorandum No. 2
NAS Whiting Field
Milton, Florida

Well	Falling Head K (cm/sec)	Rising Head K (cm/sec)
WHF-1-1	1.03×10^{-3}	1.06×10^{-3}
WHF-3-3	1.29×10^{-3}	5.67×10^{-3}
WHF-5-1	5.13×10^{-4}	2.88×10^{-4}
WHF-8-1	2.71×10^{-3}	2.62×10^{-3}
WHF-9-2	5.48×10^{-3}	4.96×10^{-3}
WHF-10-1	7.00×10^{-3}	9.31×10^{-3}
WHF-11-1	1.78×10^{-2}	2.01×10^{-2}
WHF-12-1	2.35×10^{-2}	1.81×10^{-2}
WHF-13-1	1.40×10^{-2}	1.07×10^{-2}
WHF-14-1	5.34×10^{-2}	1.67×10^{-2}
WHF-15-1	2.98×10^{-2}	1.62×10^{-2}
WHF-16-1	9.72×10^{-3}	5.89×10^{-3}
WHF-16-2	2.40×10^{-3}	2.40×10^{-3}
WHF-17-1	9.46×10^{-3}	9.78×10^{-3}
WHF-18-1	6.80×10^{-3}	4.67×10^{-3}

Notes: Average is the geometric mean.
K = conductivity.
cm/sec = centimeter per second.

Three rising head and three falling head slug tests were conducted in each monitoring well. Comparison of the average calculated rising and falling head hydraulic conductivity results for each monitoring well tested are presented in Table 3-2.

The geometric mean of the calculated hydraulic conductivities for the three rising and falling head slug tests conducted in each monitoring well is summarized in Table 3-3. Geometric mean values ranged from 5.34×10^{-2} to 2.88×10^{-4} centimeters per second (cm/sec). This considerable variability (2 orders of magnitude) in hydraulic conductivity within the same aquifer is a reflection of the wide range of grain sizes (clay to gravel) and interbedding characteristic of the sand-and-gravel aquifer.

Table 3-3
Geometric Mean Hydraulic Conductivity by Site Group

Technical Memorandum No. 2
NAS Whiting Field
Milton, Florida

Site Grouping	Hydraulic Conductivity (cm/sec)
1, 2, 17, and 18	3.61×10^{-3} (8.96 ft/day)
3	8.17×10^{-3} (23.16 ft/day)
4, 5, 6, 7, and 8	3.87×10^{-3} (9.55 ft/day)
9 and 10	6.53×10^{-3} (18.51 ft/day)
11, 12, 13, and 14	1.91×10^{-2} (54.14 ft/day)
15 and 16	6.80×10^{-3} (19.27 ft/day)

Notes: cm/sec = centimeters per second.
ft/day = feet per day.

Hydraulic conductivities across the site groupings were developed by taking the geometric mean of the calculated hydraulic conductivities associated with all monitoring wells that were slug tested in the grouping.

Hydraulic conductivities for the following monitoring wells were not calculated due to the various listed conditions.

WHF-3-E	Groundwater contamination, not tested
WHF-3-W	Groundwater contamination, not tested
WHF-7-1	Groundwater contamination, not tested
WHF-11-2	Inconsistent data, not analyzable
WHF-9-1	Inconsistent data, not analyzable
WHF-5-OW-2	Insufficient amount of water in well to test

Seepage Velocity. The average linear pore water velocity or seepage velocity across the site groupings can be calculated by using the following modified version of Darcy's law (accounting for a porous medium):

$$V=\frac{K_i}{n} \tag{1}$$

where

V = seepage velocity in feet per day (ft/day),
K = hydraulic conductivity in ft/day,
i = hydraulic gradient in ft/ft, and
n = effective porosity.

As mentioned in the previous sections, the hydraulic conductivity and hydraulic gradient have been calculated for each site grouping. The effective porosity for silty sands to well sorted sands ranges from 0.18 to 0.27 (Fetter, 1980). An average value for effective porosity of 0.23 was selected for the seepage velocity calculations.

Using the above equation, seepage velocities for each site grouping are calculated as follows.

Sites 1, 2, 17, and 18

$$V=\frac{(8.96\ ft/day)\,(0.0029\ ft/ft)}{0.23}$$

$$V = 0.11\ \text{ft/day}$$

Site 3

$$V=\frac{(23.16\ ft/day)\,(0.0021\ ft/ft)}{0.23}$$

$$V = 0.21\ \text{ft/day}$$

Sites 4, 5, 6, 7, and 8

$$V=\frac{(9.55\ ft/day)\,(0.0016\ ft/ft)}{0.23}$$

$$V = 0.07\ \text{ft/day}$$

Sites 9 and 10

$$V=\frac{(18.51\ ft/day)\,(0.0023\ ft/ft)}{0.23}$$

V = 0.19 ft/day

Sites 11, 12, 13, and 14

$$V=\frac{(54.14\ ft/day)\,(0.0034\ ft/ft)}{0.23}$$

V = 0.80 ft/day

Sites 15 and 16

$$V=\frac{(19.27\ ft/day)\,(0.0076\ ft/ft)}{0.23}$$

0.64 ft/day

These seepage velocities represent the pore velocity at which groundwater is moving horizontally throughout the upper part of the sand-and-gravel aquifer. These velocities may not be representative of the contaminant transport velocity due to the interaction with other physical and chemical variables.

<u>3.2.4 Aquifer Characteristics</u> The pumping test conducted from March 14 to 20, 1991, enabled the aquifer characteristics of the sand-and-gravel aquifer to be estimated.

The data collected during the pumping test was evaluated using the Hantush (1955) leaky aquifer and Boulton (1955) delayed-drainage methods to provide estimates of transmissivity, hydraulic conductivity, and storativity for the sand-and-gravel aquifer. These data and a discussion of the pumping test results are detailed in the pumping test report in Appendix A.

Calculated transmissivity values ranged from 10,000 to 20,000 square feet per day (ft^2/day). This range is not large considering the typical natural variation in aquifer composition. The corresponding range in lateral hydraulic conductivity of the aquifer is approximately 100 to 150 ft/day, which is the range expected for sand with dispersed clay or thin clay lenses under pumping stress.

The late-time Boulton storativities (0.045 and 0.08) are also reasonable for unconfined conditions in sand aquifers containing clay.

To summarize the pumping test analysis, the aquifer above and in the production zone contains localized thin lenses of clay type material that are not really contiguous. These layers function to delay vertical water level response but do not function as true aquitards. Because of this, vertical migration from the water table to the production zone can occur readily. The influence of pumping of the west well and the aquifer behavior does indicate that the system is horizontally stratified; therefore, responses to pumping in a given depth interval may be transmitted rapidly. Detailed quantitative analysis of the system was complex due to the long-term antecedent heavy rainfall, the inability to feasibly control pumping rates closely, and the presence of other pumping wells in the vicinity that could not feasibly be shutdown. Overall, the system appears to behave as an unconfined system.

As expected, hydraulic conductivities derived from slug test analysis (approximately 9 to 54 ft/day) were lower than the hydraulic conductivity range (100 to 150 ft/day) estimated from the pumping test evaluation. The range of hydraulic conductivities determined from the pumping test analysis is probably more representative of the sand-and-gravel aquifer in the production zone than the range of hydraulic conductivities calculated from the slug test data from the shallower zones. Aquifer test data will likely provide more reliable estimates of the overall conductivity in the production zone than slug test data in the same system for the following reasons:

- the length of the pumping test was several days compared to a few minutes for each slug test, thereby creating a large database;
- the volume of water displaced during a slug test is small and the results will be influenced by the movement of water through the filter pack; and
- changes in water levels of several monitoring wells (often screened at various depths) are measured simultaneously during a pumping test rather than one monitoring well (screened at one specific interval) during a slug test.

However, due to the economies of scale and the large distance between monitoring wells at the perimeter sites, pumping tests cannot be conducted at all sites.

3.3 OVERALL HYDROGEOLOGIC INTERPRETATION. The groundwater system at NAS Whiting Field is composed of three aquifers: the sand-and-gravel aquifer, the Upper Floridan aquifer, and the Lower Floridan aquifer.

The groundwater flow direction of the sand-and-gravel aquifer at NAS Whiting Field is in a south-southwesterly direction towards Clear Creek in the western half of the installation and to the southeast towards Big Coldwater Creek in the eastern half.

The gradient of the sand-and-gravel aquifer potentiometric surface ranges from approximately 0.0016 to 0.0075 ft/ft. Hydraulic conductivities calculated from slug test and pumping test data ranged from 9 to 150 ft/day. Seepage velocities across the six site groupings ranged from 0.11 to 1.38 ft/day.

Based on the pumping test analysis, the transmissivity of the sand-and-gravel aquifer ranges from 10,000 to 20,000 ft^2/day.

Due to the depth of the Floridan aquifer production zones and the potential of cross contamination, no exploration or aquifer characterization was conducted for these deeper systems during the RI field program.

4.0 HYDROGEOLOGIC CONCLUSIONS

Based on the hydrogeologic data collected during Phase I of the RI at NAS Whiting Field and regional literature the following conclusions can be assumed.

Aquifer System. The regional and installation aquifer systems are the same. The aquifer system is composed of the sand-and-gravel aquifer, the Upper Floridan aquifer, and the Lower Floridan aquifer. Soils of the sand-and-gravel aquifer generally consist of very fine- to coarse-grained sand and gravel with interbedded clay and silty clay lenses (0.5 to 30 feet thick).

Both Floridan aquifers are primarily composed of thick layers (up to 400 feet thick) of limestone with some interbedded sand and dolomite.

Groundwater Flow Directions. The groundwater flow direction of the sand-and-gravel aquifer at NAS Whiting Field appears to be to the south-southwest (toward Clear Creek) in the western half of installation and to the southeast (toward Big Coldwater Creek) in the eastern half.

The groundwater flow direction at the six site groupings generally follows the overall installation groundwater flow pattern. Groundwater flow directions at the site groupings are as follows.

Site Grouping	Groundwater Flow Direction
Sites 1, 2, 17, and 18	South-southwest
Site 3	South
Sites 4, 5, 6, 7, and 8	South
Sites 9 and 10	Southeast
Sites 11, 12, 13, and 14	Southeast
Sites 15 and 16	southwest

Horizontal Gradients. Horizontal gradients of the sand-and-gravel aquifer ranged from 0.0016 ft/ft to 0.0076 ft/ft. Calculated horizontal gradients across the six site grouping are as follows.

Site Grouping	Horizontal Gradient (ft/ft)
Sites 1, 2, 17, and 18	0.0029
Site 3	0.0021
Sites 4, 5, 6, 7, and 8	0.0016
Sites 9 and 10	0.0023
Sites 11, 12, 13, and 14	0.0034
Sites 15 and 16	0.0080

Hydraulic Conductivity. Hydraulic conductivity of the sand-and-gravel aquifer calculated from single-hole permeability test data ranged from 5.34×10^{-2} to 2.88×10^{-4} cm/sec. The geometric mean of hydraulic conductivities for the site groupings ranged from 1.91×10^{-2} to 3.61×10^{-3} cm/sec. The variability in hydraulic conductivity in the sand-and-gravel aquifer is a result of the wide range of grain sizes and degrees of soil exhibited in the soils of the aquifer.

Seepage Velocity. Calculated seepage velocities across the six site groups ranged from 0.07 to 0.64 ft/day.

Aquifer characteristics. The sand-and-gravel aquifer characteristics calculated from the pumping test are as follows.

Transmissivity = 10,000 to 20,000 ft^2/day
Hydraulic conductivity = 100 to 150 ft/day
Storativity = 0.045 and 0.08

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APPENDIX A

AQUIFER TEST REPORT

1.0 INTRODUCTION

This report presents the results of the pumping test conducted at NAS Whiting Field in March 1991. Pumping test rationale, setup and description are detailed in Section 2.3 of this technical memorandum.

2.0 GENERAL SITE HYDROGEOLOGY

At the production well W-S2 test site, the sand aquifer system extends from the base of the unsaturated zone at approximately 120 feet below land surface (bls) to top of the shale at 270 feet bls. Drilling logs for the pumped well and the observation well cluster indicate a 2-foot thick plastic clay at 146 feet bls and at 10-foot thick clayey layer spanning the interface between saturated and unsaturated soils. At W-S2, the thicker zone appears to contain substantial sand and was found to be absent in borings from other locations on base, indicating it is not a continuous barrier to vertical groundwater movement.

Clay beds approximately 0.2 to 0.3 foot thick were also logged at the WHF-5-OW-1 (OW-1) well cluster at depths of 135 and 150 feet bls. These occurrences are typical of the fluvial depositional sequence, and individual lenses are thought to be limited in areal extent in comparison to the area of drawdown influence.

3.0 IMPLEMENTATION OF AQUIFER PUMPING TEST

3.1 WATER LEVEL MONITORING FOR BACKGROUND TRENDS. Determination of antecedent "long-term" trends in water level fluctuation at the test site is important so that adjustments to the drawdown and recovery data can be made to remove extraneous effects caused by factors other than pumping of W-S2. Factors of concern for this test were recent pumping of the west production well W-W3, heavy rainfall several weeks prior to the test period, and barometrically induced water level changes.

In-situ Hermit™ electronic digital recorders were used to record water levels in OW-1, piezometers WHF-5-PZ-1 (PZ-1) and WHF-5-PZ-2 (PZ-2), and monitoring well GMW-3 (on the west side of the battery shop) from 09:57 March 14 to 10:21 March 27. During the first 1000 minutes of the pumping and recovery periods, the pre-programmed log cycle provided frequent readings for data storage. After 1000 minutes, readings were collected at 30 minute intervals. Prior to the start of W-S2 pumping, data were recorded at 5 minute intervals for 95 minutes (Figure 1).

The interval of background water level record prior to pumping was relatively short, and therefore the recovery record was extended to seven days, one day longer than the pumping period.

3.2 SIX-DAY CONSTANT-RATE PUMPING TEST. The rate of discharge of W-S2 was determined from visual readings taken from an existing in-line totalizing meter in the supply line to the treatment plant. These readings were used to determine interval discharge rates, which are every 5 minutes for the first 275 minutes of

CORRECTED WATER LEVELS IN OBSERVATION WELLS
PRIOR TO START OF MARCH 14, 1991 PUMPING TEST
WHITING FIELD NAS

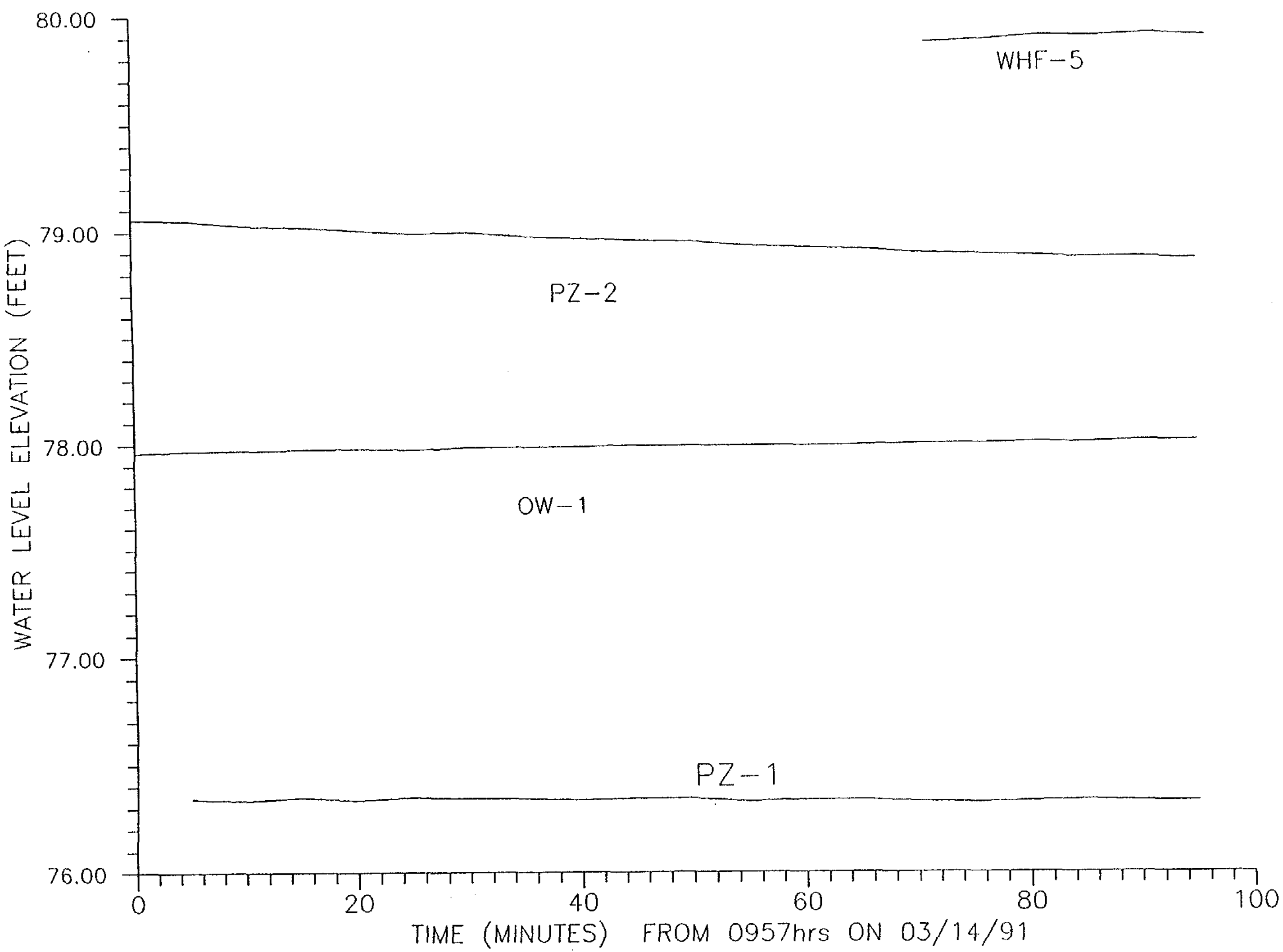


Figure 1

the test (Table 1). During this early test period, the pumping rate fluctuated from 382 gpm to an average of 430 gpm average for the initial 50 minutes. Except for an average of 388 gpm for nearly 2 hours during the second day of pumping, the average rate gradually declined to an ending rate of 368 gpm (Figure 2).

The average pumping rate for the 6 day test was 395 gpm. Based on averages for periods shown on Figure 2, the early pumping rate was approximately 7 percent higher than the test average, and the final rate was approximately 7 percent low. The existing plumbing system did not permit any adjustment of the pumping rate.

3.3 SEVEN-DAY RECOVERY TEST. Once sufficient pumping data had been collected to characterize the aquifer's hydraulic properties the W-S2 production well was shut down and the recovery of the aquifer to static conditions was monitored and recorded. No production wells, except the west production well, at Whiting Field were pumped during the recovery portion of the test on the morning of March 14. Data loggers recorded water levels from OW-1, PZ-1, PZ-2 and GMW-3 over a seven day period.

4.0 ANALYSIS OF THE PUMPING TEST DATA

Analysis of drawdown and recovery data using quantitative methods were performed for three observation wells (OW-1, PZ-2, and GMW-3), and a qualitative assessment was applied to PZ-1. The analyses were complicated by the non-uniform nature of the aquifer system, relatively large fluctuations in pumping rate, and having to approximate antecedent water level trends.

4.1 ADJUSTMENTS TO DRAWDOWN AND RECOVERY DATA. Each recorded water level reading was adjusted to normalize the data for fluctuations in barometric pressure (recorded simultaneously at the test site) and to substract out a general rise in water levels throughout the test period. The basic mechanics of adjusting the data are discussed below.

4.1.1 Barometric Influence The atmospheric pressure measured at ground surface varied 0.24 psi (0.56 feet of water) during the first two days of the pumping period, and lesser amounts thereafter (Figure 3). A direct inverse correlation of barometric fluctuation and water level change is evident in the graph of OW-1, PZ-2, and GMW-5, implying these wells exhibit significant barometric efficiency (Figure 4). A relatively sharp rise of 0.07 psi (0.17 foot of water) in the morning of March 19 produced small identifiable declines in the water levels of observation wells. These changes were used to compute barometric efficiencies of 35, 59, and 53 percent for wells OW-1, PZ-2, and WHF-5, respectively.

The entire arrays of water level data for these wells were adjusted by these percentages relative to the barometric reading at the time pumping started. The resulting water level graphs were noticeably smoother and more amenable to curve-fitting.

4.1.2 General Rising Trend The 95 minutes of data collected prior to the start of the test indicated that water levels in OW-1 and GMW-3 was slowly rising, but levels in PZ-1 and PZ-2 were declining at different rates (see Figure 1). The maximum rate of change was a decline of 0.13 foot/hour in PZ-2. A 0.03 foot/hour

FLOW RATE BASED ON TOTALIZER MEASUREMENTS
AQUIFER PUMPING TEST

MARCH 14, 1991
WHITING FIELD

DATE	CLOCK TIME	ELAPSED TIME (MIN)	DURATION OF MEASUREMENT	ENDING METER READING	BEGINNING METER READING	GALLONS DURING INTERVAL	INTERVAL RATE (GPM)
14-Mar-91	12:30	0	--	--	215312.50	--	--
14-Mar-91	12:35	5	5	215314.25	215312.50	1750	350.0
14-Mar-91	12:40	10	5	215316.35	215314.25	2100	420.0
14-Mar-91	12:45	15	5	215318.20	215316.35	1850	370.0
14-Mar-91	12:50	20	5	215320.10	215318.20	1900	380.0
14-Mar-91	12:55	25	5	215322.05	215320.10	1950	390.0
14-Mar-91	13:00	30	5	215324.10	215322.05	2050	410.0
14-Mar-91	13:05	35	5	215326.15	215324.10	2050	410.0
14-Mar-91	13:10	40	5	215328.30	215326.15	2150	430.0
14-Mar-91	13:14	44	4	215330.05	215328.30	1750	437.5
14-Mar-91	13:20	50	6	215332.80	215330.05	2750	458.3
14-Mar-91	13:25	55	5	215334.75	215332.80	1950	390.0
14-Mar-91	13:30	60	5	215336.90	215334.75	2150	430.0
14-Mar-91	13:35	65	5	215338.95	215336.90	2050	410.0
14-Mar-91	13:41	71	6	215341.65	215338.95	2700	450.0
14-Mar-91	13:46	76	5	215343.75	215341.65	2100	420.0
14-Mar-91	13:50	80	4	215345.45	215343.75	1700	425.0
14-Mar-91	13:55	85	5	215347.50	215345.45	2050	410.0
14-Mar-91	14:00	90	5	215349.55	215347.50	2050	410.0
14-Mar-91	14:05	95	5	215351.85	215349.55	2300	460.0
14-Mar-91	14:10	100	5	215353.90	215351.85	2050	410.0
14-Mar-91	14:15	105	5	215356.00	215353.90	2100	420.0
14-Mar-91	14:20	110	5	215358.05	215356.00	2050	410.0
14-Mar-91	14:25	115	5	215360.15	215358.05	2100	420.0
14-Mar-91	14:30	120	5	215362.35	215360.15	2200	440.0
14-Mar-91	14:35	125	5	215364.35	215362.35	2000	400.0
14-Mar-91	14:40	130	5	215366.55	215364.35	2200	440.0
14-Mar-91	14:45	135	5	215368.75	215366.55	2200	440.0
14-Mar-91	14:50	140	5	215370.95	215368.75	2200	440.0
14-Mar-91	14:55	145	5	215373.00	215370.95	2050	410.0
14-Mar-91	15:00	150	5	215375.05	215373.00	2050	410.0
14-Mar-91	15:05	155	5	215377.05	215375.05	2000	400.0
14-Mar-91	15:10	160	5	215379.10	215377.05	2050	410.0
14-Mar-91	15:15	165	5	215381.30	215379.10	2200	440.0
14-Mar-91	15:27	177	12	215386.70	215381.30	5400	450.0
14-Mar-91	15:30	180	3	215387.80	215381.30	6500	433.3
14-Mar-91	15:35	185	5	215389.90	215387.80	2100	420.0
14-Mar-91	15:40	190	5	215392.00	215389.90	2100	420.0
14-Mar-91	15:45	195	5	215394.05	215392.00	2050	410.0
14-Mar-91	15:50	200	5	215396.10	215394.05	2050	410.0
14-Mar-91	15:55	205	5	215398.20	215396.10	2100	420.0
14-Mar-91	16:00	210	5	215400.55	215398.20	2350	470.0

FLOW RATE BASED ON TOTALIZER MEASUREMENTS (CONTINUED)
AQUIFER PUMPING TEST

MARCH 14, 1991
WHITING FIELD

DATE	CLOCK TIME	ELAPSED TIME (MIN)	DURATION OF MEASUREMENT	ENDING METER READING	BEGINNING METER READING	GALLONS DURING INTERVAL	INTERVAL RATE (GPM)
14-Mar-91	16:05	215	5	215402.60	215400.55	2050	410.0
14-Mar-91	16:10	220	5	215404.80	215402.60	2200	440.0
14-Mar-91	16:15	225	5	215406.70	215404.80	1900	380.0
14-Mar-91	16:20	230	5	215408.89	215406.70	2190	438.0
14-Mar-91	16:25	235	5	215410.95	215408.89	2060	412.0
14-Mar-91	16:30	240	5	215413.08	215410.95	2130	426.0
14-Mar-91	16:35	245	5	215415.20	215413.08	2120	424.0
14-Mar-91	16:40	250	5	215417.28	215415.20	2080	416.0
14-Mar-91	16:45	255	5	215419.38	215417.28	2100	420.0
14-Mar-91	16:50	260	5	215421.45	215419.38	2070	414.0
14-Mar-91	16:55	265	5	215423.59	215421.45	2140	428.0
14-Mar-91	17:00	270	5	215425.90	215423.59	2310	462.0
14-Mar-91	17:05	275	5	215427.78	215425.90	1880	376.0
14-Mar-91	18:21	351	76	215460.00	215427.78	32220	423.9
15-Mar-91	06:24	1074	723	215763.39	215460.00	303390	419.6
15-Mar-91	08:55	1225	151	215826.41	215763.39	63020	417.4
15-Mar-91	13:41	1511	286	215944.11	215826.41	117700	411.5 ave 411.
15-Mar-91	15:37	1627	116	215989.18	215944.11	45070	388.5
16-Mar-91	09:28	2698	1071	216432.00	215989.18	442820	413.5 for 107
16-Mar-91	15:05	3035	337	216569.00	216432.00	137000	406.5
17-Mar-91	08:35	4085	1050	216992.00	216569.00	423000	402.9
18-Mar-91	07:55	5485	1400	217542.00	216992.00	550000	392.9
18-Mar-91	14:59	5909	424	217704.00	217542.00	162000	382.1
19-Mar-91	10:58	7108	1199	218159.00	217704.00	455000	379.5
20-Mar-91	08:42	8412	1304	218638.32	218159.00	479320	367.6

1511
+ 1071
2582

To 2698 min AVE (w/o 388.5) = 416.1 g
" " " (w/ 388.5) = 414.9 g

416.1
− 388.5
27.6 or 6.6% reduced

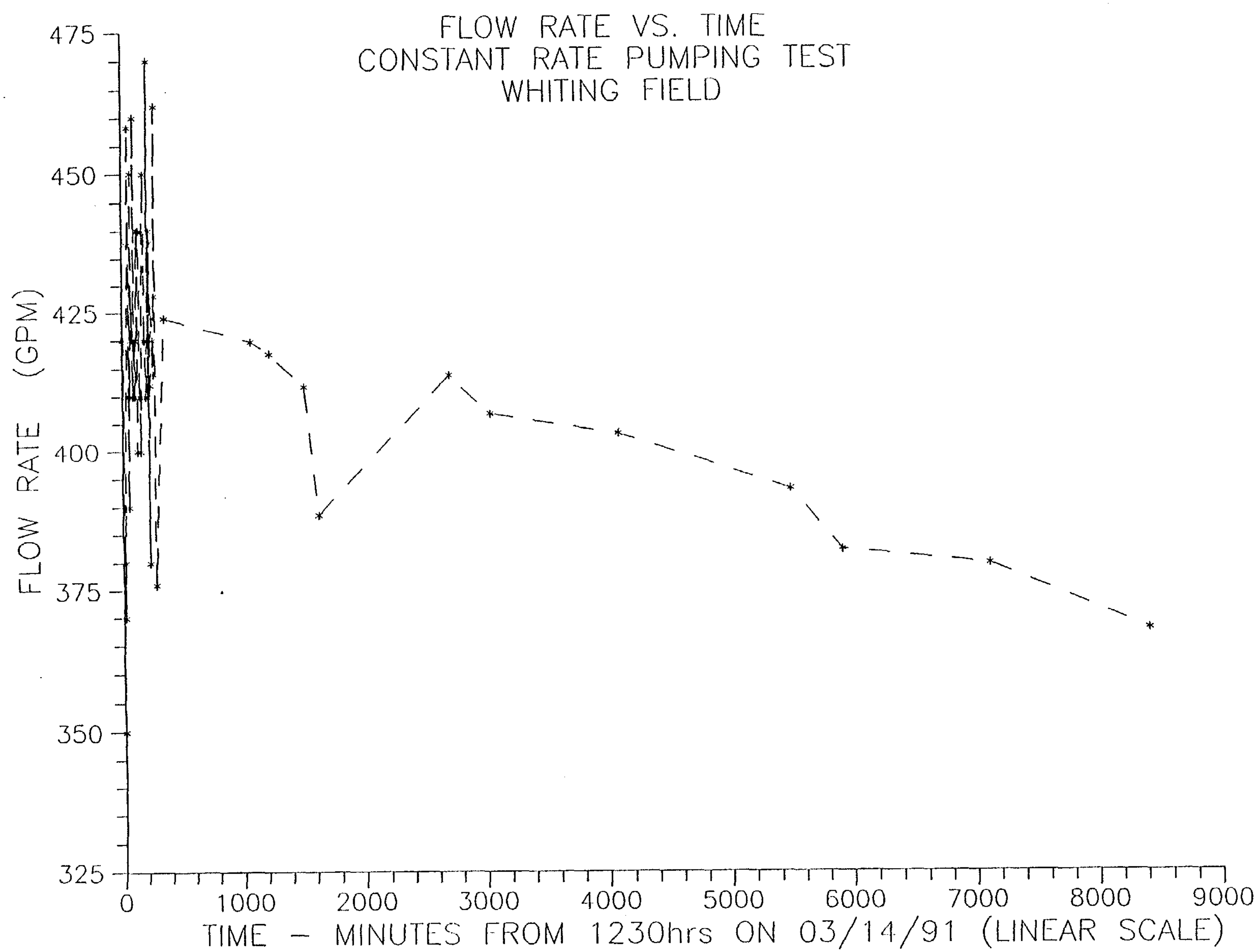

FLOW RATE VS. TIME
CONSTANT RATE PUMPING TEST
WHITING FIELD
FLOW RATE (GPM)
475
450
425
400
375
350
325
0
1000
2000
3000
4000
5000
6000
7000
8000
9000
TIME – MINUTES FROM 1230hrs ON 03/14/91 (LINEAR SCALE)

Figure 2

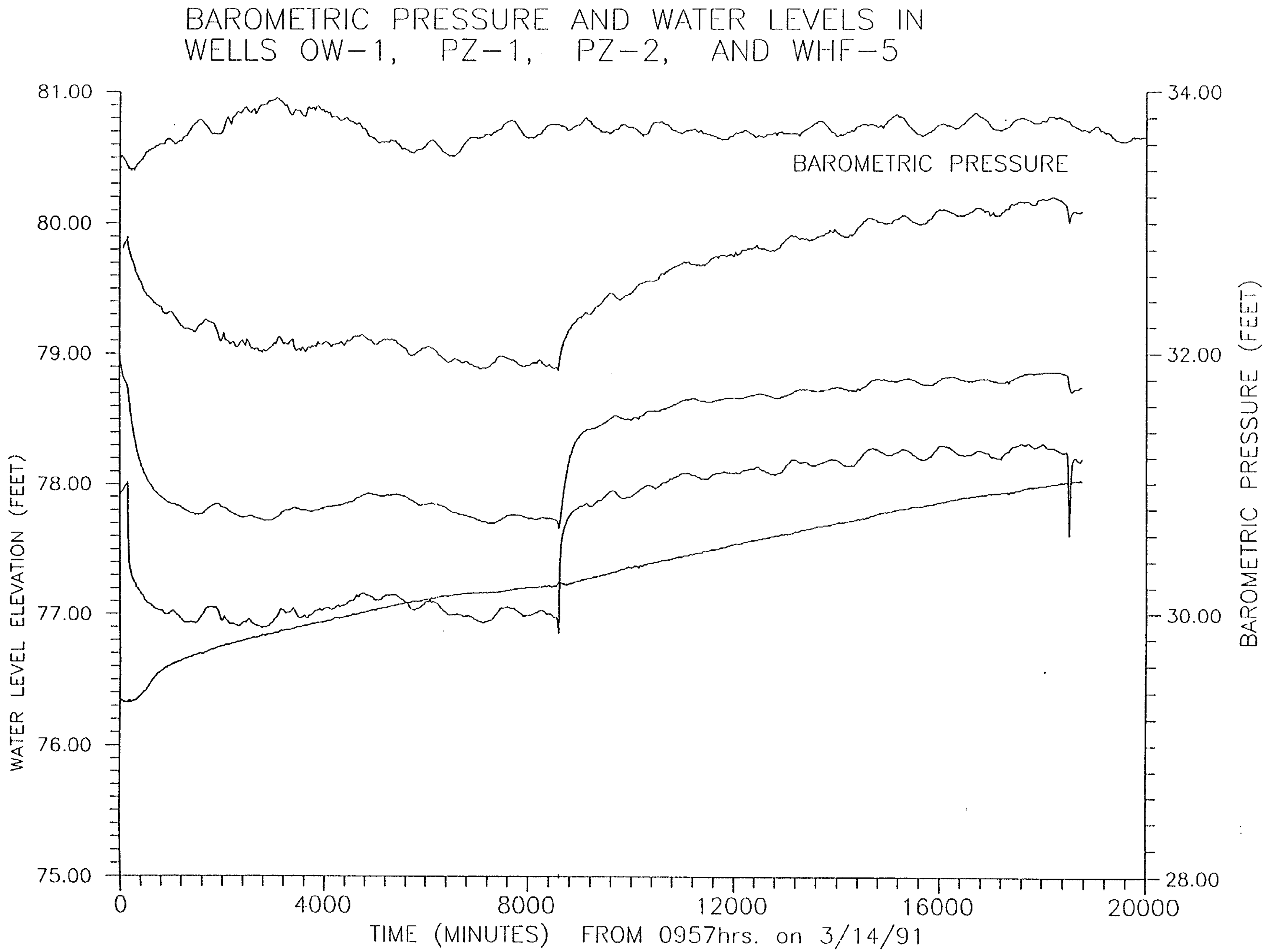
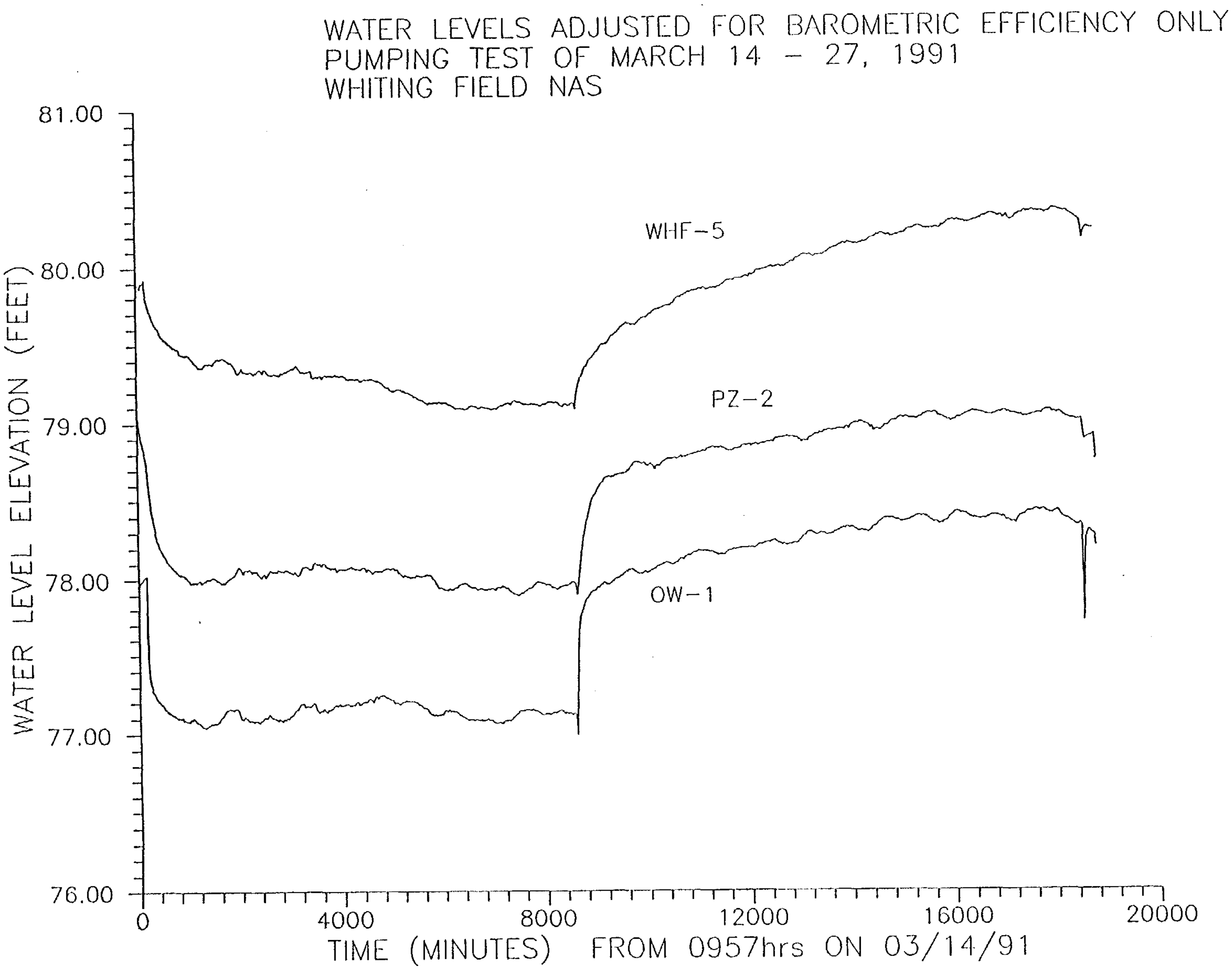

BAROMETRIC PRESSURE AND WATER LEVELS IN
WELLS OW-1, PZ-1, PZ-2, AND WHF-5
BAROMETRIC PRESSURE
WATER LEVEL ELEVATION (FEET)
BAROMETRIC PRESSURE (FEET)
TIME (MINUTES) FROM 0957hrs. on 3/14/91
81.00
80.00
79.00
78.00
77.00
76.00
75.00
34.00
32.00
30.00
28.00
0
4000
8000
12000
16000
20000

Figure 3


WATER LEVELS ADJUSTED FOR BAROMETRIC EFFICIENCY ONLY
PUMPING TEST OF MARCH 14 - 27, 1991
WHITING FIELD NAS
WATER LEVEL ELEVATION (FEET)
81.00
80.00
79.00
78.00
77.00
76.00
WHF-5
PZ-2
OW-1
0
4000
8000
12000
16000
20000
TIME (MINUTES) FROM 0957hrs ON 03/14/91

Fic~re 4

change occurred in OW-1, screened in the aquifer. Both rates are significant to the analysis.

Barometric-adjusted plots showed that fully recovered levels in the observation wells were between one quarter and a half foot higher than levels prior to pumping (Figure 5). During the 13 days, no distinct rise occurred that can be correlated to rain storms. Probable explanations why recovered levels are above the pre-pumping levels are (1) seasonal recharge effects resulted in a general rising trend area-wide, and/or (2) the aquifer was not fully recovered from the termination of general supply pumping at the time the test began. Because neither explanation could be confirmed, a linear adjustment to the drawdown and recovery data was pro-rated for this time interval specific to each well as follows: GMW-3 = -2.61×10^{-5} ft/min; OW-1 = -2.33×10^{-5} ft/min; and PZ-2 = -1.46×10^{-5} ft/min.

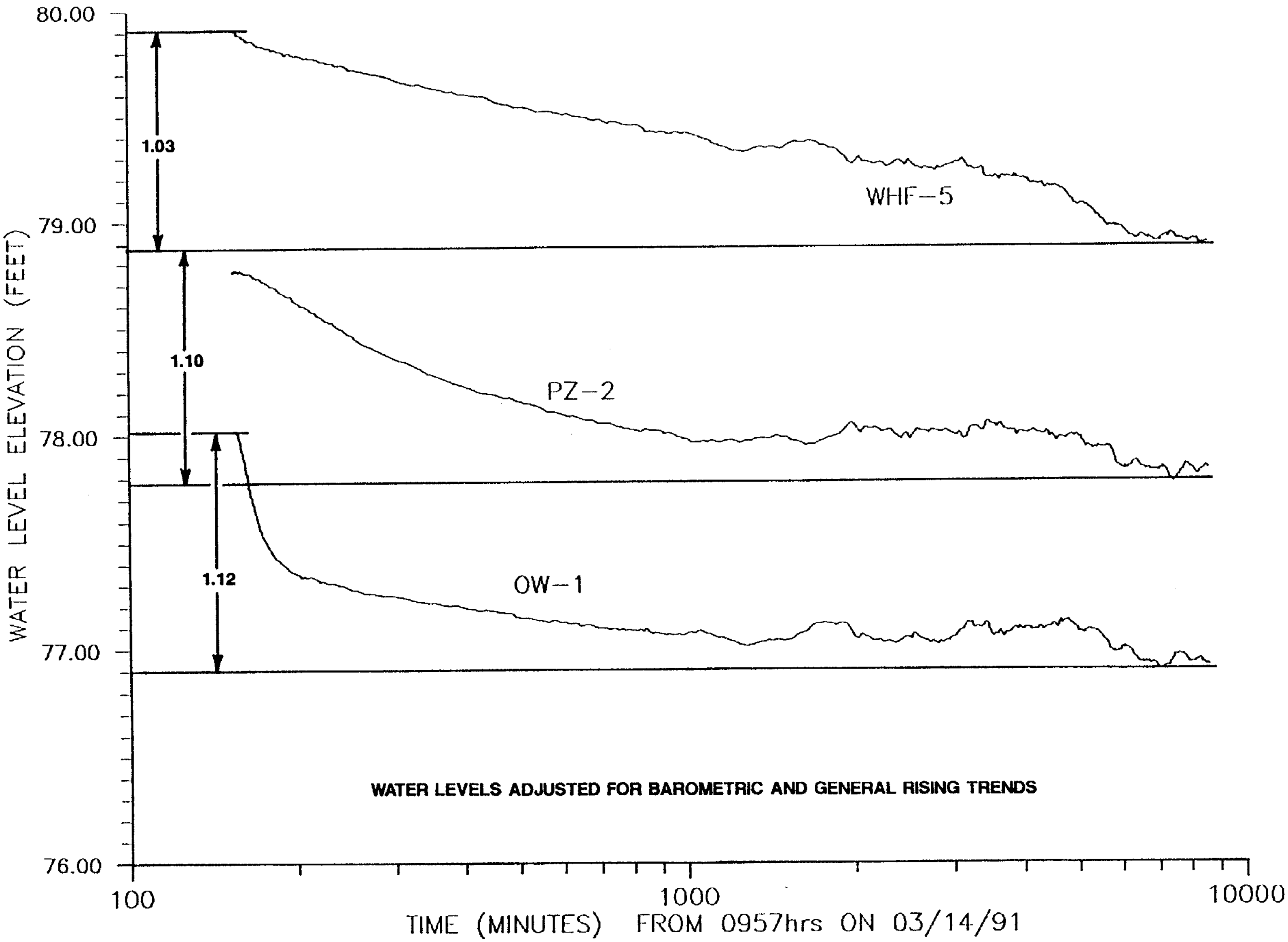
<u>4.2 **DRAWDOWN AND RECOVERY DATA FOR OW-1 AND PZ-2**</u>. The Jacob semi-log straight line method is not applicable for deriving reliable values of aquifer hydraulic parameters: transmissivity (T) and storativity (S). The flatness of the adjusted (barometric and trend) drawdown and recovery curves for both wells during all but the first day of pumping indicates that vertical leakage is occurring.

Assuming confining conditions, the appropriate matching type-curves are those of Hantush (1955) for leakage through semi-confining beds that does not contribute significant water from aquitard storage. Hantush calculations made upon matching the data to type-curve v = 0.3 give a T of 10,400 ft^2/day and a S of 8×10^{-4} for OW-1, and a match to v = 0.2 gave a T of 16,800 ft^2/day and a S of 4.8×10^{-3} for PZ-2 (Figure 6). This analytical method also provides estimates of vertical hydraulic conductivity, or permeability, of 3.8×10^{-2} ft/day at OW-1, and 2.4×10^{-2} ft/day at PZ-2. The variability displayed by these results are within the normal range for fluvial aquifer systems.

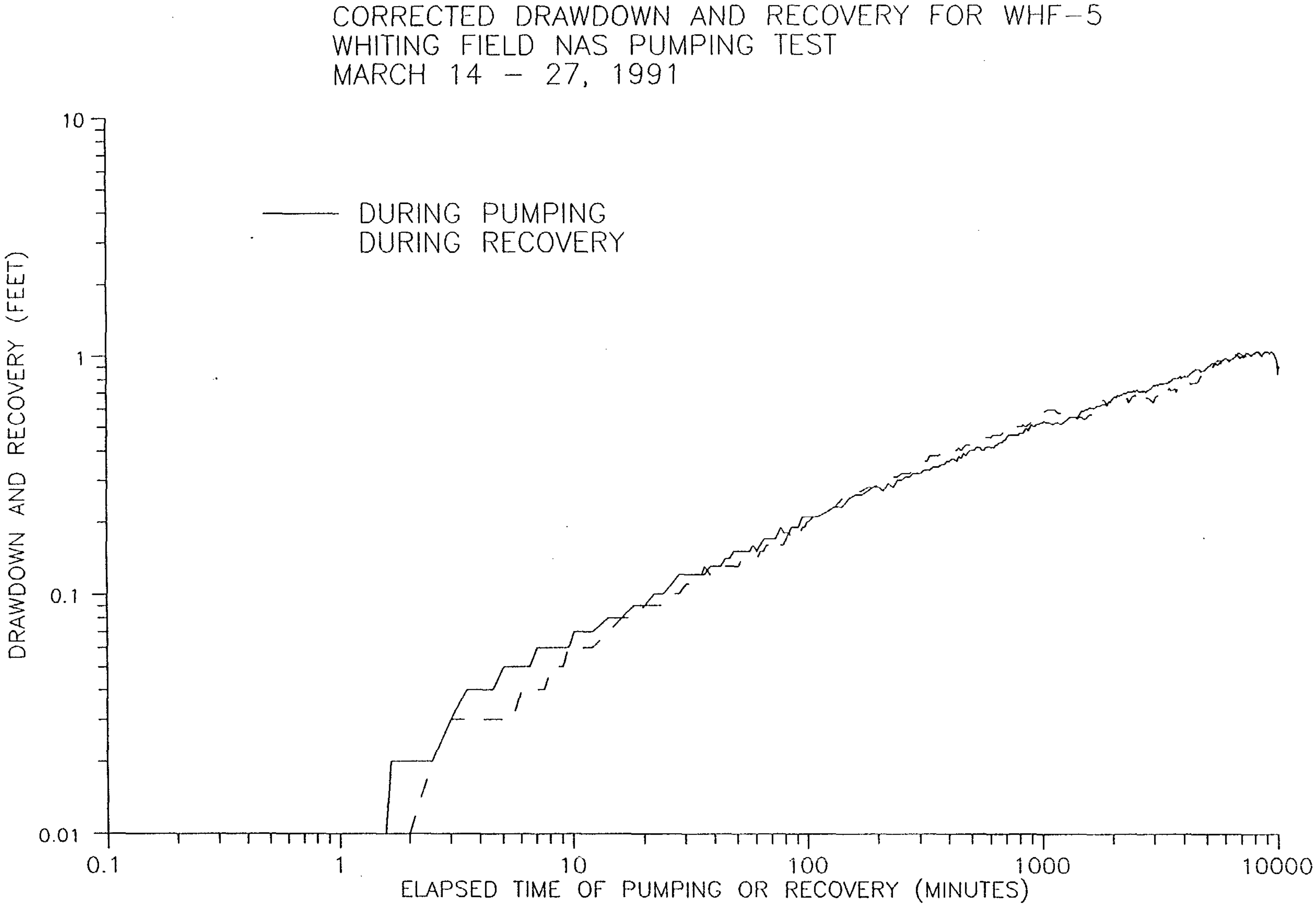
If the test area is largely unconfined, a Boulton delayed-drainage analysis is the most appropriate solution. Recovery plots show good curve matches for both OW-1 and PZ-2 using the Boulton method. In fact, the late-time "B" type-curves, which become pertinent as delayed drainage ceases, match closely with the resumed increase in drawdown evident after 5000 minutes elapsed pumping time. Under the geologic-based interpretation requiring some confinement of the aquifer, the lack of a match between the data and the Hantush curves after 5000 minutes would have to be attributed to improper trend adjustments (i.e., too large during latter half of drawdown, and too small during latter part of recovery). This possibility is discussed in Section 5.3.

Table 2 gives a summary of calculated pertinent hydraulic values for each observation well quantitatively analyzed. The resulting T and S values derived from the Hantush and Boulton assumptions are compared.

<u>4.3 **DRAWDOWN AND RECOVERY DATA FOR GMW-5**</u>. The shape of the drawdown and recovery curves are nearly identical with each other, but are decidedly different than those of OW-1 and PZ-2 (Figure 7). The reason for this appears to be the occurrence of one or two thin clay beds between the pumped aquifer and the sandy stratum tapped by the WHF-3 screen. Because the lateral integrity of the clay beds is questionable, two different analytical approaches were used as possible interpretations of the adjusted data plots.


80.00
79.00
78.00
77.00
76.00
WATER LEVEL ELEVATION (FEET)
1.03
1.10
1.12
WHF-5
PZ-2
OW-1
WATER LEVELS ADJUSTED FOR BAROMETRIC AND GENERAL RISING TRENDS
100
1000
10000
TIME (MINUTES) FROM 0957hrs ON 03/14/91

F re 5



CORRECTED DRAWDOWN AND RECOVERY FOR WHF-5
WHITING FIELD NAS PUMPING TEST
MARCH 14 - 27, 1991
DURING PUMPING
DURING RECOVERY
DRAWDOWN AND RECOVERY (FEET)
10
1
0.1
0.01
0.1
1
10
100
1000
10000
ELAPSED TIME OF PUMPING OR RECOVERY (MINUTES)

Figure 6

TABLE 2: SUMMARY OF CALCULATED HYDRAULIC PARAMETERS

OBS. WELL	RADIAL DISTANCE r (feet)	MAX RAWDOWN (feet)	AQUIFER TRANSMISSIVITY T (feet squared/day)			LATERAL HYDRAULIC AQUIFER CONDUCTIVITY K (feet/day)*	AQUIFER STORATIVITY S (dimensionless)				AQUITARD THICKNESS (feet)	VERTICAL HYDRAULIC CONDUCTIV OF PERTINENT AQUITARD K' (feet/day)		
			HANTUSH (1955)	HANTUSH (1960)	BOULTON (1954)		HANTUSH (1955)	HANTUSH (1960)	BOULTON EARLY TIME	BOULTON LATE TIME		HANTUSH (1955)	HANTUSH (1960)	NEUMAN (1972)
OW-1	445	1.12	10400		14800	105	0.00083		0.00097	0.045	2	0.038		
PZ-2	472	1.10	16800		18900	149	0.0048		0.0061	0.08	2	0.024		
WHF-5	151	1.03		17300		144		0.0011			2			0.00014
				to		to		to						to
				33600		280		0.0026			10		0.0008	0.00029

ANALYTICAL METHODS USED

BOULTON (1954) = unconfined aquifer with delayed drainage from drawdown cone

HANTUSH (1955) = semi-confined aquifer with leakage through aquitard, without water released from storage

HANTUSH (1960) = semi-confined aquifer with leakage through aquitard, and water released from storage

NEUMAN and WITHERSPOON (1972) = semi-confined aquifer, using the ratio of drawdowns between aquifer and aquitard

* Average T for well, divided by an estimated aquifer thickness of 120 feet.
* Boulton and Hantush Ts are averaged for each pertinent well.
  K = T/b
  b estimated at 120 feet

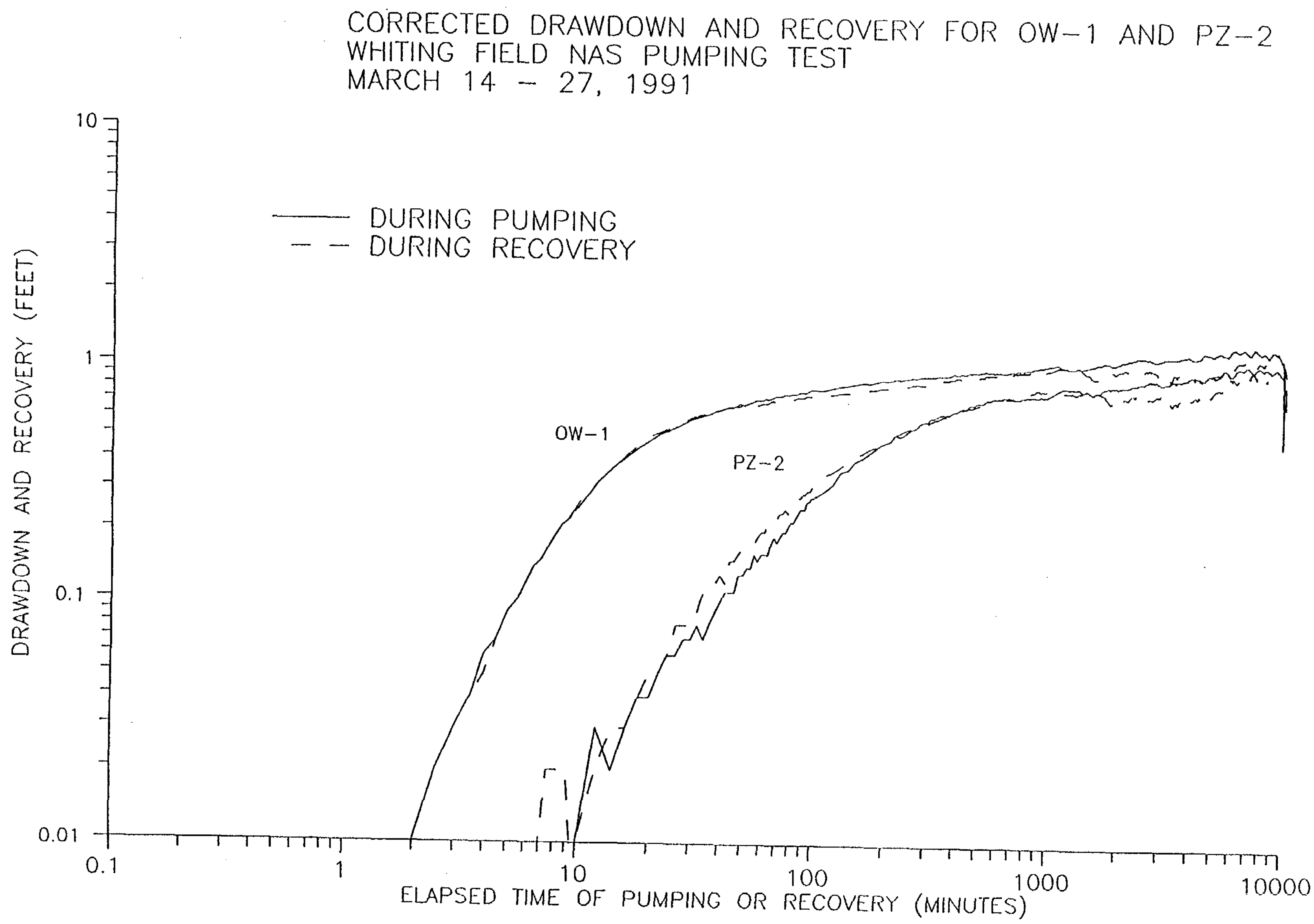

CORRECTED DRAWDOWN AND RECOVERY FOR OW-1 AND PZ-2
WHITING FIELD NAS PUMPING TEST
MARCH 14 - 27, 1991
DURING PUMPING
DURING RECOVERY
OW-1
PZ-2
DRAWDOWN AND RECOVERY (FEET)
10
1
0.1
0.01
0.1
1
10
100
1000
10000
ELAPSED TIME OF PUMPING OR RECOVERY (MINUTES)

Figure 7

The first approach considers that the clays are not continuous within the cone of drawdown, and that GMW-5 is screened within the vertical extent of the pumped aquifer. Furthermore, in this part of the drawdown cone, the aquifer is effectively confined by an aquitard that contributed water to the aquifer from storage in the aquitard. Evidence for this is the 11 feet of clayey soil from 110 to 121 feet noted in the log of W-S2. For this approach to be compatible with the interpretation of the OW-1 and PZ-2 plots, leakage can not occur from aquitard storage at distances from the pumped well as large as the distance to the OW-1 cluster. Using the Hantush (1960) leaky aquifer with storage equations, the best curve match is B = 3, giving a T of 17,300 ft^2/day and a S of $1.1x10^{-3}$.

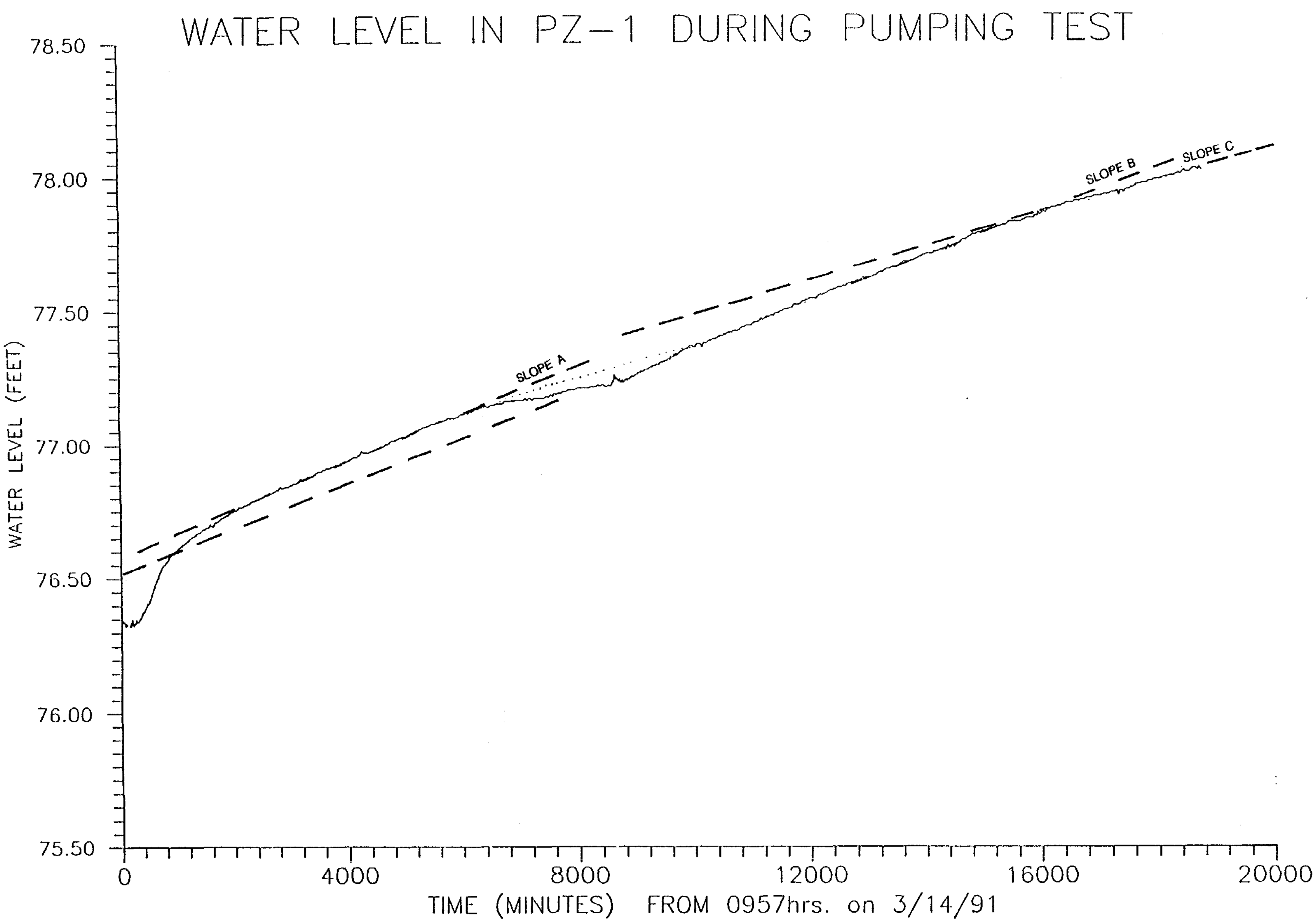
The second approach used to analyze GMW-3 was to acknowledge that the well may be isolated from the pumped aquifer by an effective aquitard, and compute only the aquitard diffusivity (k'/S_S') by the method of Neuman and Witherspoon (1972). This method focuses on quantitative relationships concerning the ratio of aquitard drawdown to aquifer drawdown. In essence, WHF-5 is assumed to represent drawdown in the overlying aquitard, even though it is screened in sandy material. If the well taps a sandy pocket within the clay, the application seems reasonable.

Application of the Neuman-Witherspoon method results in a drawdown ratio of 0.20, leading to a diffusivity of 295 ft^2/day. Then, if a likely range of $5x10^{-6}$ to $1x10^{-5}$ ft^{-1} is assumed for the specific storage of the aquitard, k' is computed to be between $1.43x10^{-4}$ and $2.9x10^{-4}$ ft/day (approximately $5x10^{-8}$ to $1x10^{-7}$ cm/sec). These values are within the realistic range for clayey aquitards.

4.4 WATER LEVEL TREND IN PZ-1. Figure 8 shows a general rise in water level in PZ-1 of approximately 1.5 feet during the test period of 13 days. Barometric adjustments to the water level data could not be determined, and appear negligible. The data plot contains two interruptions in the rising trend, which has three straight-line segments (slopes ranging from $6x10^{-5}$ to $9x10^{-5}$ ft/min). The first interruption is a drawdown and recovery cycle underway at the time the W-S2 pumping test began. The second deviation is a broader, shallower drawdown and recovery cycle that starts approximately 6100 minutes into the pumping test.

Quantitative interpretation of this plot for values of hydraulic parameters is not possible. However, reasonable explanations for the appearance of the plot are offered upon consideration of the drilling log of OW-1 (close to PZ-1). This log suggests that PZ-1 screen is hydraulically isolated from the pumped aquifer to a greater degree than WHF-5 because of a thin second clay bed at a higher elevation. Theoretically, the net effect of two in-between aquitards would be to greatly delay the response time and diminish the drawdown at PZ-1 caused by the migration of pressure reduction from the W-S2 pumping test. It is very likely that the trend interruption beginning at 6100 minutes is a reflection of this process.

The larger water-level deviation at the beginning of the plot is attributed to the operation of the west production well on March 14. The pressure response of PZ-1 to pumping at the West Well is relatively quick, with an apparent drawdown of a quarter foot. In comparison, the multi-day pumping test at W-S2 caused less than 0.1 foot of drawdown.



WATER LEVEL IN PZ-1 DURING PUMPING TEST
78.50
78.00
77.50
77.00
76.50
76.00
75.50
WATER LEVEL (FEET)
0
4000
8000
12000
16000
20000
TIME (MINUTES) FROM 0957hrs. on 3/14/91
SLOPE A
SLOPE B
SLOPE C

Figure 8

4.5 MEASUREMENTS IN OW-2. Five manual measurements were made during the pumping period of W-S2, after an obstruction in the well was removed on March 16. These data suggest that a head decline of approximately a half foot occurred. Because the development of the well is questionable due to apparent vandalism, this change may not represent the true response of the screened stratum to the pumping test, and a quantitative analysis was not attempted.

4.6 LIMITATIONS OF THE PUMPING TEST. Nearly all aquifer pumping tests have aspects which cause the data set to be less than ideal for hydraulic interpretation. Potential causes of problems in quantifying hydraulic parameters accurately for this test are:

- poor spatial lithologic definition within the drawdown cone.
- fluctuation of the pumping rate as much as 7 percent higher to 7 percent lower than the test average.
- the uni-directional location of monitoring wells with respect to the pumped well.
- insufficient antecedent water-level record to establish trends and the degree of interference by other supply wells on base.

Lithologic variation in both the lateral and vertical dimensions is evident from drilling logs, and reinforces the need for differing methods of hydraulic interpretations for individual observation wells. A simple homogeneous "pancake" conceptual model, frequently assumed where supply wells yield large rates of water from unconsolidated soils, does not apply to the pumping test analysis at NAS Whiting Field. The apparent sporadic inter-bedding of clay beds or lenses in the fluvial sequence is interpreted as affecting each observation well response differently.

Although water levels were adjusted for barometric change and interpreted long-term trends, a greater-than-desired amount of fluctuation during drawdown and recovery resulted on the plots.

Because more fluctuation is apparent during drawdown than during recovery, the somewhat random and excessive fluctuation in pumping rate is believed to have contributed "noise" to the plots. The existing plumbing system used to transport pumped water to its discharge point appears complex, and probably caused changes in backpressure on the discharge line. The only obvious correlation between a distinct change in pumping rate and a change in drawdown occurred at approximately 1,500 minutes into the test. A sudden 7 percent drop in pumping rate caused approximately a tenth of a foot rise in water levels. The effect of a large increase in rate during the first 50 minutes of the test is assumed to be insignificant, because recovery curves are nearly identical to the drawdown curves for this period.

All observation wells were located southwest of the pumped well. Within the depth interval of the pumped aquifer, only two observation wells were available. These wells (OW-1 and PZ-2) are practically at the same radius from the pumped wells. The lack of more spatial coverage precludes the use of distance-drawdown methods of analysis. Essentially, aquifer response was obtained at only one location, and the calculated parameters may be directionally biased if the aquifer is anisotropic.

Better background definition of water level trends would have improved confidence in adjustments made to the data plots. The rise of half a foot in aquifer head over the test period is probably not unusual. However, because very little antecedent data were collected, the continuity and linearity of the rise noted between the start of pumping and end of recovery period is not known, and had to be assumed. The 1.5 foot rise in the PZ-1 well, screened above the aquifer, is very significant, even though the period of monitoring does not identify the cause. In-depth analysis of the entire water-level record at each well indicate water levels were not stabilized at the start of the test (contrary to the field interpretation made immediately before starting the test with 2 hours of data). The water-level trend adjustments (previously described) may actually not be linear with time, and pro-rating of these adjustments may result in slightly erroneous drawdown and recovery plots, impacting the accuracy of calculated hydraulic parameters.

5.0 INTERPRETATION OF GROUNDWATER HYDRAULICS

The March 14 to 20, 1991 pumping test is considered adequate to make general characterizations of the groundwater system within the area of observation wells at NAS Whiting Field. The limitations of the test discussed above may have influenced the accuracy of the parameter values to a significant degree. The following interpretation of the general groundwater system characteristics at the site are not acutely dependent on accurate parameter values.

Should remedial design involving groundwater extraction become necessary, another pumping test that attempts to overcome the limitations of this test is recommended.

5.1 AQUIFER HYDRAULIC PARAMETERS. Hantush (1955) leaky aquifer solutions of log-log drawdown and recovery data plots provide the most valid estimates of T, K, S, and k', if the geologic interpretation that the aquifer is partially confined rather than unconfined is correct. In reality, it is not known if the OW-1 drilling log showing confining clayey soils in the top of the saturated zone is representative of the test area. The W-S2 log suggests that the clayey zone is mostly above the natural head elevation, casting some doubt on confinement.

Values of aquifer parameters were calculated by both the Hantush leaky aquifer and Boulton delayed-drainage methods (Table 2). Not unexpectedly, T values are very similar between methods; all values are within the range of 10,000 to 20,000 ft^2/day. This range is not large considering the common natural variation in aquifer composition. The corresponding range in lateral hydraulic conductivity of the aquifer (K) is approximately 100 to 150 feet/day, which seems reasonable for sand with dispersed clay or thin clay lenses.

The Hantush (1960) analysis for the GMW-3 plot is based the possibility that the 2-foot clay at 146-148 feet bls at OW-1 and W-S2 is not a really continuous, and is absent in the WHF-5 area. If so, GMW-3 is actually screened in the aquifer, and not above it. This condition justifies the fitting of Hantush's leaky type-curves (B = 1 to B= 3) to the plot until approximately 5000 minutes of the drawdown period. The sloppier fit of the same type-curves to the recovery plot reduces the confidence level in the Hantush application. But, the resulting T and S are not much different than those resulting from the leakage without aquitard storage analysis of OW-1 and PZ-2. Furthermore, pumping rate variation would be expected to affect GMW-3 plots to a greater degree and cause a sloppy curve fit, because it is one third the distance from the pumping well as compared to OW-1 and PZ-2.

The late-time Boulton storativities (0.045 and 0.08) are also reasonable for unconfined conditions in sandy aquifers containing clay. With the available geologic knowledge, the test area cannot be characterized as to degree of confinement. It is possible that the aquifer may be confined in some areas within the drawdown cone, and not in others areas encountered by outward migrating drawdown.

5.2 INFERRED BOUNDARIES. The distance that drawdown migrated from the pumped well can not be reliable computed because of uncertainties in aquifer storativity, but is probably greater than 1000 feet. The conformance of the drawdown plots of wells OW-1 and PZ-2 to the Boulton type-curves suggests that drawdown did not reach laterally to hydraulic boundaries.

On the other hand, if the steepening of drawdown after 5000 minutes is not due to the end of delayed drainage and trend corrections are correct, a lateral barrier-type boundary may have been encountered. An alternative explanation would be negative boundaries in the vertical dimension, such as depletion of aquitard storage contribution and drawdown in a "source bed" supplying water to leakage. The observed drawdown in PZ-1 after approximately 6000 minutes of pumping supports dewatering of a source bed.

5.3 AQUIFER SYSTEM CHARACTERIZATION. Until many aspects of the hydraulic response of the test were considered in the analysis, the contradiction in direction of water-levels trends prior to the test was puzzling. It became apparent that PZ-1 and PZ-2 have significantly delayed responses to changes in system stress as compared to OW-1 and GMW-3.

The effect of pumping of the West Well on the water level at PZ-1 (discussed in Section 4.4) is greater than the effect of Well W-S2, even though the radial distance is greater. This result indicates that the fluvial aquifer, approximately 120 feet thick, is stratified with respect to hydraulic properties. The West Well apparently is screened in the upper part of the aquifer, while W-S2 is screened in the mid-to-lower section of the aquifer. Because Figure 8 shows that W-S2 also caused drawdown in PZ-1 late in the test period, the entire vertical extent of the fluvial aquifer appears hydraulically connected to a limited degree.

Section 4.6 discussed the 1.5 foot rising trend at PZ-1 during the test. Figure 6. indicates that the brief use of the West Well the morning of the test caused significant but short-term impacts to the trend line. The PZ-1 plot exhibits three segments of trend slope, decreasing slightly with time. Two causes for this are possible: (1) diminishing recharge to the aquifer system from a wet period, and (2) slow rebound in head from recently terminated pumping stress. Heavy rainstorms occurred within a month prior to the test. But also, long-term pumping from the North and West Wells had ceased three days prior to pumping test start-up. Pressure reductions due to months of continuous pumping can be expected to travel relatively long distances in the aquifer as a result of high transmissivity, partial confinement, and large rates of pumping.

In summary, the aquifer system is conceptualized as containing several discrete plastic clay beds in the test area that become sandy in some localities, promoting vertical leakage downward under pumping stress. Where aquitards are sandy, unconfined groundwater conditions may be approached. In these areas, recharge received at the water table will move with only minor delays into the permeable aquifer that supplies water to NAS supply wells.