USACE CONTRACT NO. DACW33-91-D-0005
DELIVERY ORDER NO. 003
EBASCO SERVICES INCORPORATED

FINAL CRITERIA SUMMARY REPORT

PERFORM PREDESIGN INVESTIGATION
AND DEVELOP DESIGN CRITERIA AT
DAVIS LIQUID WASTE SUPERFUND SITE
SMITHFIELD, RI

VOLUME I

JUNE 1994

June
1994

LEACH

EBASCO

Investigation and Design of Miscellaneous Civil Works Hazardous Waste Projects, Various Locations in New England

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1.0 INTRODUCTION

Under Delivery Order No. 3 of the U.S. Army Corps of Engineers Contract No. DACW33-91-D-005, Ebasco Services Incorporated (Ebasco) performed a Predesign Investigation and developed design criteria for a groundwater extraction and treatment system at the ~~Davis Liquid Waste Superfund site located in Smithfield, Rhode Island~~. The Predesign Investigation included evaluation of the chemical, physical, and subsurface data contained in various site documents, as well as derived from field investigation and modeling activities. These data were utilized to establish design criteria required for a groundwater extraction and treatment system at the site. Specifically, this report presents the results of the field investigations, groundwater modeling, delineation of the 100-year floodplain, wetlands delineation, topographic survey, electric power and telephone service plan, and an engineering evaluation of discharge options.

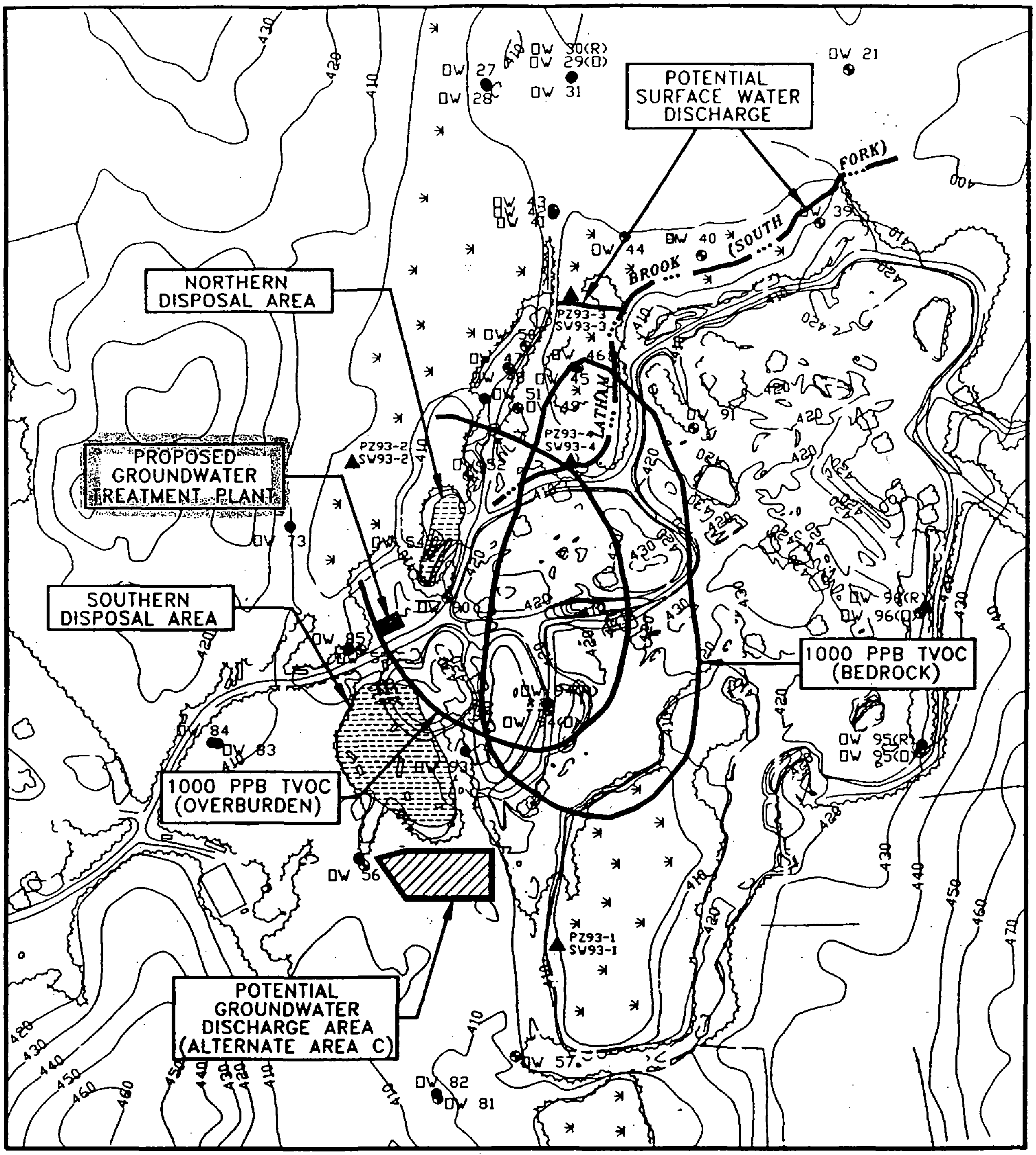
1.1 Project Description and Background Information

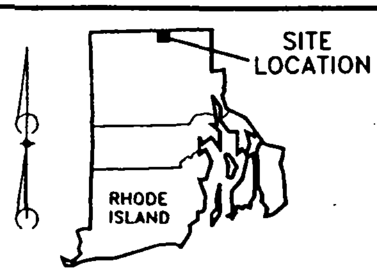
The Davis Liquid Waste Superfund Site (the Site) is located on the property of William and Eleanor Davis, in Smithfield, Rhode Island and served as a disposal location for various liquid and solid wastes. Liquid wastes were brought to the site in tank trucks and drums, and were dumped in several unlined lagoons and seepage pits. Periodically, the semi-solid materials from the lagoon were excavated and dumped in several locations on the site and covered with available soil. Two specific areas, the Southern Disposal Area (SDA) and Northern Disposal Area (NDA) (Figure 1-1) were identified in a Remedial Investigation (CDM, 1986) as source areas containing contaminated soils.

The Remedial Investigation/Feasibility Study (RI/FS) was conducted from 1984-1986 by Camp Dresser McKee (CDM, 1986 and 1987). In September 1987, the USEPA Regional Administrator signed a Record of Decision (ROD) for the site. The selected remedy includes onsite Source Control and Management of Migration components. The Source Control component includes excavation and onsite treatment of soils, while the Management of Migration component includes restoration of overburden and bedrock aquifers contaminated with volatile organic compounds using onsite treatment. In addition, the Management of Migration component of the remedy includes construction of an alternate public water supply off-site. From 1990 to 1992, Woodward-Clyde Consultants (WCC) performed pre-design activities to provide additional information for the Source Control excavations and Management of Migration components, including additional borings and a pump test for an extraction well system.

A groundwater extraction and treatment system ~~(GWTS)~~ is planned for the site. ~~The system ultimately is intended to accomplish active restoration of the overburden and bedrock aquifers contaminated with volatile organic compounds (VOCs) using on-site treatment involving air stripping and carbon adsorption.~~ However, the initial GWTS will only partially meet the objectives identified in the ROD since the initial remedy extracts groundwater from the area contaminated with greater than 1000 parts per billion (ppb). Full implementation of the ROD objectives, that is, areas which are above the ROD mandated levels for individual contaminants will be pursued after evaluating the operation and effectiveness of the interim system.

The Groundwater Treatment System will include a series of overburden and shallow bedrock wells for the collection of groundwater emanating from the source area and from the areas immediately downgradient of the source areas where Total Volatile Organic Compounds (TVOC) concentrations exceed 1,000 parts per billion (ppb) as defined in the Draft Pre-Design Engineering Report II (WCC, 1992b). This area is termed the initial groundwater extraction area. Discharge of treated water will be to either the ground or surface water; this report evaluates each of these discharge alternatives.





LEGEND:

- ● BEDROCK WELL
- ⊕ OVERBURDEN WELL
- ▲ PIEZOMETER

0 100 200 300 400

SCALE IN FEET

EBASCO SERVICES INCORPORATED

U.S. ARMY CORPS OF ENGINEERS
DAVIS LIQUID WASTE SITE
SMITHFIELD, RHODE ISLAND

SITE MAP

CONTRACT NO. DACW33-91-D-0005 | FIGURE 1-1

OWO:03011

1.2 Groundwater Treatment System Predesign Objectives

In order to proceed with the design of the initial GWTS, Ebasco was tasked to establish Design Criteria for the interim Management of Migration remedy. Specific design criteria developed and included in this report are:

- Extraction well configuration
- Estimated pumping rates
- Influent contaminant concentrations
- Combined influent flow rate
- Site plan for the treatment system
- Combined treated water discharge flow rate

Ebasco was also tasked to perform an engineering evaluation to determine the best method for discharging treated water from the GWTS. Based upon the results of the engineering evaluation, other design criteria include:

- Method and location of discharge
- Required chemical concentration of the treated water effluent
- Pre-treatment requirements
- A cost comparison for the discharge options

1.3 Organization of the Report

In establishing design criteria for the interim remedy, Ebasco performed field investigations to generate site specific data for evaluation as well as to support a groundwater computer model. The computer model was utilized as an aid to develop design flow rates, optimize extraction well placements, and optimize capture of contaminated groundwater from the initial extraction area. This Criteria Summary Report presents the results of that modeling effort and includes an evaluation of hydraulic conductivity and permeability, and the modeling results for discharging the treated effluent flow back to ground at the Alternate Area C location (Figure 1-1). Also presented are the estimated combined discharge flowrate, the estimated influent concentrations, and a 100-year flood plain evaluation for determining both the adequacy of a GWTS building location and the hydraulic impact on the 100-year floodplain from discharging treated effluent flow to a surface water. A preliminary Groundwater Treatment System location and influent pipe run layout are also presented. Following development of the criteria, an evaluation of the potential discharge locations is discussed, and a cost comparison presented.

The document is organized such that this section, Section 1.0, summarizes the project background, predesign objectives and report organization. Section 2.0 describes the field work and data collection undertaken to support modeling efforts, estimates of influent concentration, siting of the GWTS, and evaluation of discharge options. Section 3.0 summarizes the groundwater modeling effort; and Section 4.0 discusses the development of discharge flow rates. Section 5.0 presents the expected influent concentrations developed for the extraction well network and flow from other site sources. Section 6.0 summarizes the 100-year flood plain evaluation performed for GWTS siting and discharge evaluation purposes, Section 7.0 presents the proposed GWTS site location and preliminary influent pipe runs, and Section 8.0 presents the Engineering Evaluation of Discharge Options, including development of the discharge options, treatment system design considerations, GWTS siting, and preliminary effluent piping runs. Section 9.0 provides the Cost Analysis between the retained discharge alternatives, including: capital and operations and maintenance costs. Finally, Section 10.0 provides the comparison of alternatives, and recommendation of discharge location. The appendices contain a more detailed presentation of data and backup calculations to support key information presented in the main body of the report, including data tables, permeability test results, groundwater modeling and 100-year flood plain evaluation reports, the combined discharge flowrate calculation package, the influent mass balance calculations, the electrical power and telephone service plan, the wetlands delineation report, thermal impact calculations, the off-site culvert evaluation report, a conceptual treatment system development, and cost analysis calculations.

2.0 FIELD WORK AND DATA COLLECTION

A field program and home office data compilation effort were performed to support groundwater modeling, evaluation of groundwater discharge feasibility at Alternate Area C, 100-year flood plain evaluation for treatment system siting, calculation of expected influent concentrations, and the engineering evaluation of discharge options. These field and home office efforts are discussed individually in the following subsections: 2.1 Hydrologic Data Collection, 2.2 Alternate Area C Permeability, 2.3 Compilation of Analytical Data, 2.4 Site Survey, 2.5 Electric Power and Telephone Service Plan, 2.6 Selection of Surface Water Discharge Locations, 2.7 Wetlands, 2.8 Environmental Baseline Assessment, and 2.9 Evaluation of Culverts.

2.1 Hydrologic Data Collection

2.1.1 Piezometers

The Scope of Work required installation of four piezometers in site wetlands to measure the vertical gradients between groundwater and surface water elevations in support of groundwater modeling activities. The piezometers (PZ93-1 through PZ93-4) were installed at the locations depicted as triangles on Figure 1-1. The piezometers were constructed from 1.5-inch O.D. stainless steel piping with three-foot long wire-wrapped screens (.010-inch slot size) and threaded caps. The piezometers were driven by hand into the substratum at depths ranging from 3.6 to 7.4 feet. A summary of piezometer installation depths and screened intervals is presented in Table 2-1. All piezometers were leveled and surveyed to the state plane grid. Water levels were measured in the piezometers and surrounding wetlands on 23 September 1993 and are tabulated with the other groundwater elevations in Section 2.1.3.

Table 2-1

Piezometer Installation Summary

Piezometer	Depth (ft - bgs)	Elevation Top of Riser (ft - msl)	Screened Interval Elevation (ft - msl)
PZ93-1	7.4	407.62	398.08 - 401.08
PZ93-2	4.9	408.01	401.80 - 404.80
PZ93-3	5.9	405.89	398.11 - 401.11
PZ93-4	3.6	406.31	399.88 - 402.88

Notes: Ft - bgs = feet below ground surface
Ft - msl = feet above mean sea level
All piezometers were installed on 21 September 1993

2.1.2 Data Loggers

EPA installed two data loggers in overburden wells OW48 and OW94(O) to record the transient response of the water table to seasonal fluctuations and rain events. Water levels were automatically recorded at one hour intervals from 3 August until 23 October 1993 and downloads of raw data were transmitted to Ebasco on a monthly basis. Tabulated data for each well are compiled in Appendix A. The plot of the data (Figure 2-1) show a relatively parallel response between the wells. Water level data suggest a saw-toothed diurnal fluctuation which may be indicative of thermal stresses from the warming and cooling of the metal cased wells or else reflect 24-hour potential evapotranspiration cycles. The highest water levels occur at or near 1000 hours in OW48 and 1400 hours OW94(O) with the lowest levels seen at night time. The data logger responses to precipitation are discussed in Section 2.1.4.

2.1.3 Water Level Measurements

The Scope of Work required collection of groundwater elevations from select monitoring wells on site. Groundwater elevations were collected using an oil/water interface probe on 23 September 1993. This round of water elevations provided an additional piezometer data set for verification of the calibrated groundwater model. A total of 36 monitoring wells, 4 piezometers and 4 surface water elevations were measured. The surface water measuring points were collocated with the piezometers. Three wells listed in the Scope of Work, OW39, OW40 and OW47 were not located during an August site visit or in September during the field data collection effort. Subsequent site visits have located OW40 and remnants of OW39 (broken riser pipe at ground surface). OW47 is presumed to have been destroyed. Several nonstudy area wells - OW28, OW29, OW30, OW31, OW83 and OW84 were also measured and used to support modeling efforts. All wells measured in this program and their indicated water elevations are compiled in Table 2-2 and contoured in Figures 2-2 and 2-3 for overburden and bedrock elevations, respectively. Only one well, OW57, showed indications of separate phase liquids. A very thin floating product thickness of approximately 0.02 feet was measured. This well is south of the known disposal activity sites and in an area currently being logged for wood. Gasoline containers and spent chain saw oil canisters were observed on the ground near the well during water level measurements.

2.1.4 Precipitation Records

The Scope of Work required the collection of digital records of hourly and daily precipitation, pan evaporation and temperature from NOAA stations closest to the site. Daily precipitation and temperature records for the period from June 1975 to October 1993 were obtained for the National Weather Service stations at Woonsocket, North Foster, Providence and Kingston, Rhode Island. Pan evaporation data were available only for Kingston and on a seasonal basis (April - October). The most recent data (March 1993 to October 1993) were only available in paper format. These data were collected to support groundwater model recharge parameter estimates. Comparison of monthly averages over the reporting period 1975-1991 shows a similarity in response for all four stations (Figure 2-4).

Groundwater levels appear to respond quickly to precipitation as shown in Figure 2-1 which illustrates data logger response to the precipitation measured at the two closest stations, North Foster and Woonsocket. Overall, groundwater response appears to be linear for both recharge (rising water levels in response to precipitation) and discharge (falling water levels after precipitation ceases). There appears to be a better correlation for groundwater levels to the regional weather station data trend for seasonal rainfall than to individual storm events. There are several events (7 August 1993 and 29 August 1993) where data logger response is either masked or precipitation was different onsite.

Further discussion of the use of the meteorological data within the model can be found in Section 2.2.4 of the Groundwater Modeling Report in Appendix D.

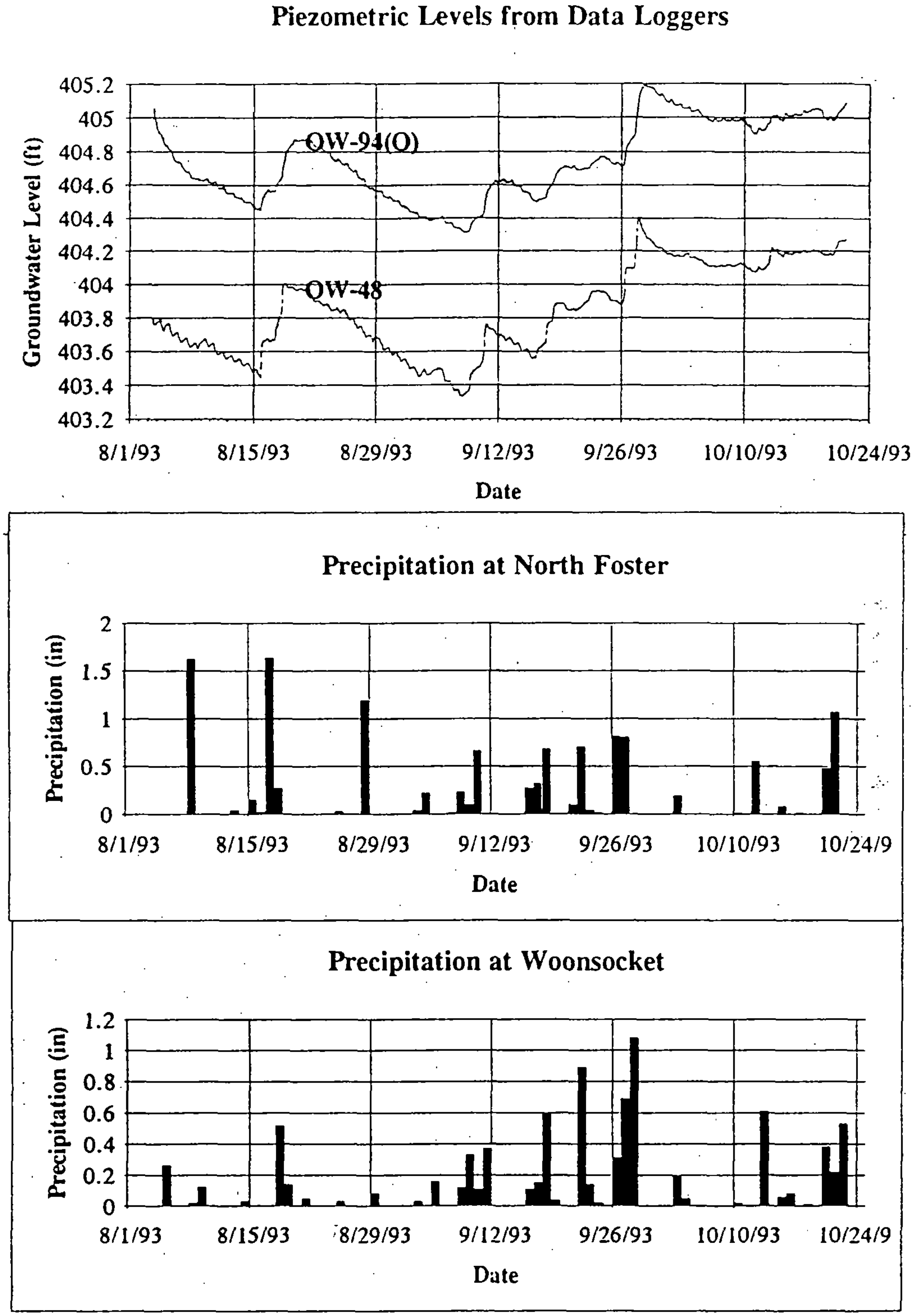


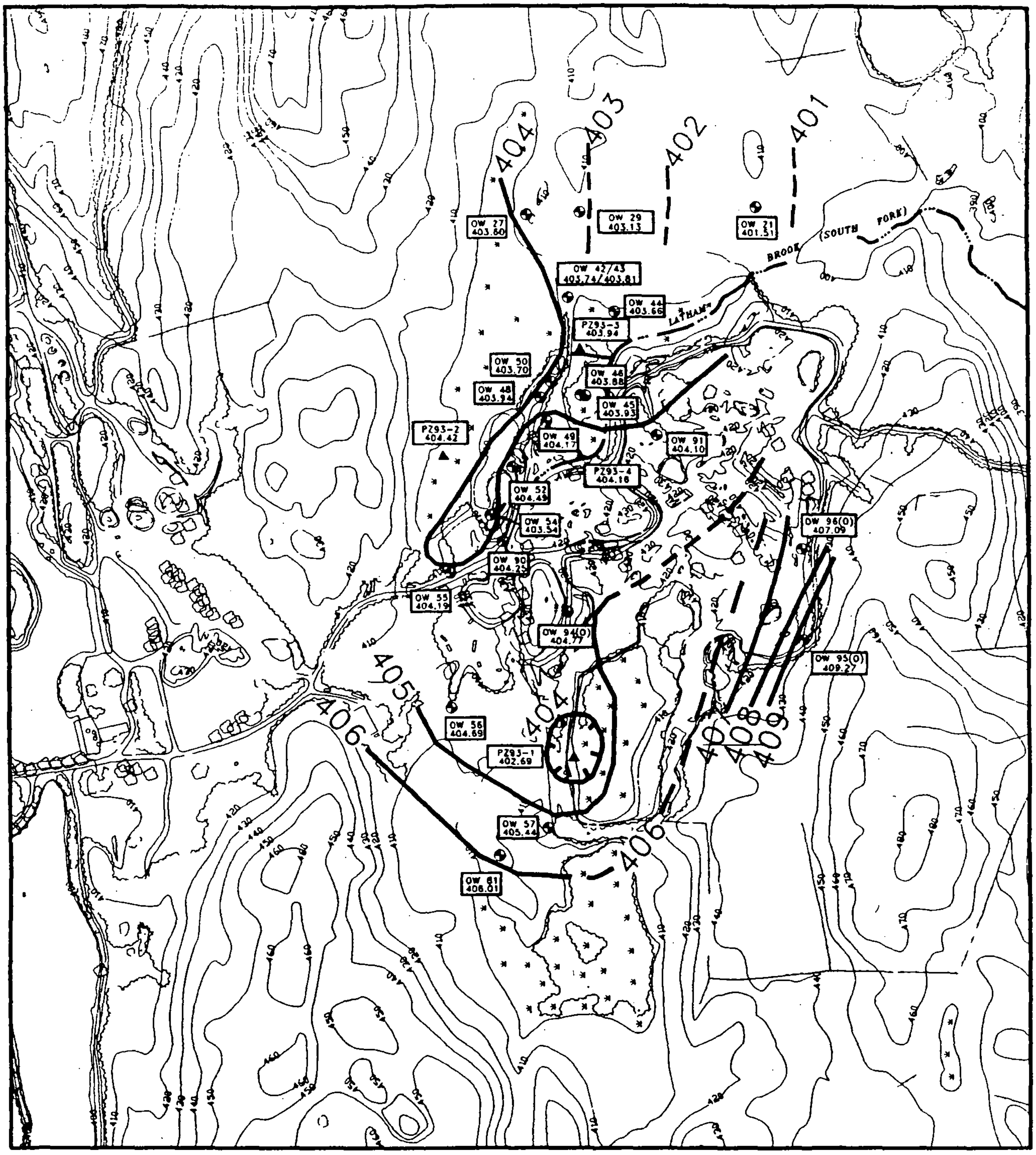
Figure 2-1 Data Logger Data and Meteorologic Conditions in August-October 1993

Table 2-2
Water Level Measurements

Well #	Depth to GW (ft)	Depth of Well (ft)	Standing Water (ft)	Ground Elev. (ft msl)	Meas. Pt. Elev. (ft msl)	Water Elev. (ft msl)	OVA (ppm)	O2 (%)	LEL (%)	Notes
OW-21	6.20	14.29	8.09	405.21	407.71	401.51	0	20.8	0	
OW-27	6.15	9.35	3.20	408.00	409.75	403.60	0	20.8	0	
OW-28	5.68	26.61	20.93	407.89	408.94	403.26	0	20.9	0	
OW-29	6.51	12.67	6.16	409.06	409.64	403.13	0	20.8	0	MP is 0.58 ft to grd
OW-30	6.38	8.56	2.18	409.15	409.74	403.36	0	20.8	0	MP is 0.59 ft to grd
OW-31	6.21	16.71	10.50	409.42	410.04	403.83	0	20.8	0	MP is 0.62 ft to grd
OW-41	4.53	69.27	64.74	406.74	408.16	403.63	0	20.9	0	
OW-42	4.43	7.83	3.40	406.50	408.17	403.74	0	20.9	0	
OW-43	4.34	24.98	20.64	406.70	408.15	403.81	0	20.9	0	MP is 1.42 ft to grd
OW-44	4.14	12.53	8.39	404.15	407.80	403.66	0	20.9	0	MP is 4.65 ft to grd
OW-45	3.48	22.33	18.85	404.50	407.41	403.93	0	20.9	0	
OW-46	3.40	31.80	28.40	404.40	407.28	403.88	0	20.9	0	
OW-48	4.91	Data logger block		407.65	408.85	403.94	0	20.9	0	
OW-49	6.60	17.83	11.23	408.04	410.77	404.17	0	20.9	0	
OW-50	5.36	23.13	17.77	406.28	409.06	403.70	7	20.9	0	MP is TOC
OW-51	7.55	16.39	8.84	406.11	409.36	401.81	2	20.9	0	
OW-52	11.24	33.10	21.86	412.88	415.73	404.49	3.5	20.9	0	MP is TOC
OW-54	3.96	5.23	1.27	405.10	407.50	403.54	2	20.9	0	
OW-55	9.45	31.74	22.29	411.39	413.64	404.19	0	20.9	0	
OW-56	15.02	22.61	7.59	417.15	419.71	404.69	0	21.0	0	MP is TOC
OW-57	6.14	12.40	6.26	410.68	411.58	405.44	0	20.9	0	Product @ 6.12 ft
OW-73	8.34	14.59	6.25	410.21	412.51	404.17	0	21.0	0	MP is 2.58 ft to grd
OW-81	7.37	21.73	14.36	411.14	413.38	406.01	0	21.0	0	
OW-82	6.43	> 100	> 93.57	410.70	412.20	405.77	0	21.0	0	MP is TOC
OW-83	11.11	14.18	3.07	414.10	417.55	406.44	0	21.0	0	
OW-84	11.50	91.11	79.61	414.00	416.27	404.77	0	21.0	0	
OW-85	8.23	90.95	82.72	410.22	412.62	404.39	0	20.9	0	
OW-90	11.18	33.49	22.31		415.43	404.25	0	21.0	0	MP is TOC
OW-91	6.93	16.21	9.28	408.53	411.03	404.10	0	21.0	0	
OW-93	Dry >15.58	15.58	15.58	419.15	421.45	< 405.87	0	21.0	0	
OW-94O	12.98	Data logger block		415.50	417.75	404.77	0	21.0	0	
OW-94R	12.21	55.06	42.85	416.20	416.91	404.70	0	21.0	0	
OW-95O	18.81	23.23	4.42	429.30	428.08	409.27	0	20.9	0	
OW-95R	20.21	49.11	28.90	428.30	428.02	407.81	0	20.8	0	
OW-96O	4.96	13.45	8.49	413.10	412.05	407.09	0	20.8	0	
OW-96R	3.80	37.08	33.28	412.10	411.93	408.13	12	20.8	0	
PZ 93-1	4.93	9.54	4.61	405.02	407.62	402.69	0	20.9	0	
PZ 93-2	3.59	6.21	2.62	404.91	408.01	404.42	0	20.9	0	
PZ 93-3	1.95	7.78	5.83	403.69	405.89	403.94	0	20.9	0	
PZ 93-4	2.15	6.43	4.28	401.81	406.31	404.16	0	20.9	0	
SW 93-1				405.02	407.62	407.62				2.58 ft to surface water
SW 93-2				404.91	408.01	< 404.91				Dry
SW 93-3				403.69	405.89	405.89				2.10 ft to surface water
SW 93-4				401.81	406.31	406.31				2.48 ft to surface water

Notes:
TOC = Top of casing — OW = Monitoring Well
TOR = Top of Riser — PZ = Piezometer
MP = Measuring Point — SW = Surface Water measuring point
GRD = Ground

Water levels were measured on 23 September 1993, all levels measured to top of riser unless noted otherwise.
Standing water is height of water column within well or piezometer.
EPA data loggers prevented sounding of wells OW-48 and OW-94(O), only water level was measured.
Ground elevations from WCC, 1992a and CDM, 1986. Top of piezometer elevations from M. Nyberg for this study.
Ground elevations at piezometers and surface water points are estimated from surveyed top of riser and stickup.
SW 93-X indicates surface water measuring point. SW93-2 is located in wetland that was dry on 23 September 93.
Surface water measuring point SW93-X is colocated with corresponding piezometer PZ93-X.



LEGEND:

⊕ OVERBURDEN WELL

▲ PIEZOMETER

0 100 200 300 400

SCALE IN FEET

NOTES:

1. WATER ELEVATIONS IN FEET MSL.
2. WATER LEVELS MEASURED ON 23 SEPTEMBER 1993.
3. SURFACE WATER MEASURING POINTS ARE COLOCATED AT PIEZOMETERS (PZ).

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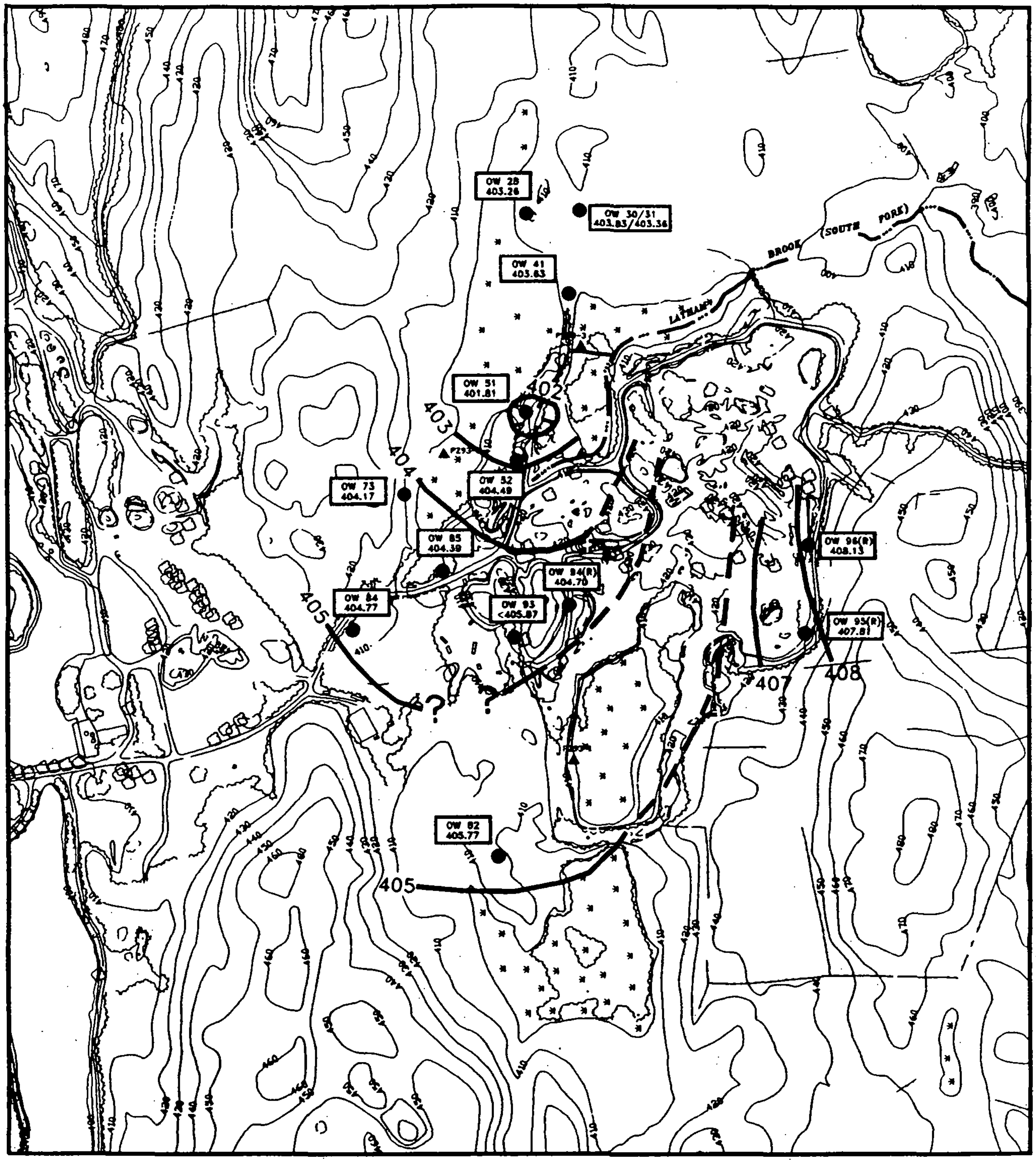
DAVIS LIQUID WASTE SITE
SMITHFIELD, RHODE ISLAND

GROUNDWATER CONTOURS SEPTEMBER 1993 – OVERBURDEN –

CONTRACT NO. DACW33-91-D-0005 | FIGURE 2-2

DWG:03015



LEGEND:

- ● BEDROCK WELL
- ▲ PIEZOMETER

0 100 200 300 400

SCALE IN FEET

NOTES:

1. WATER ELEVATIONS IN FEET MSL.
2. WATER LEVELS MEASURED ON 23 SEPTEMBER 1993.
3. SURFACE WATER MEASURING POINTS ARE COLOCATED AT PIEZOMETERS (PZ).

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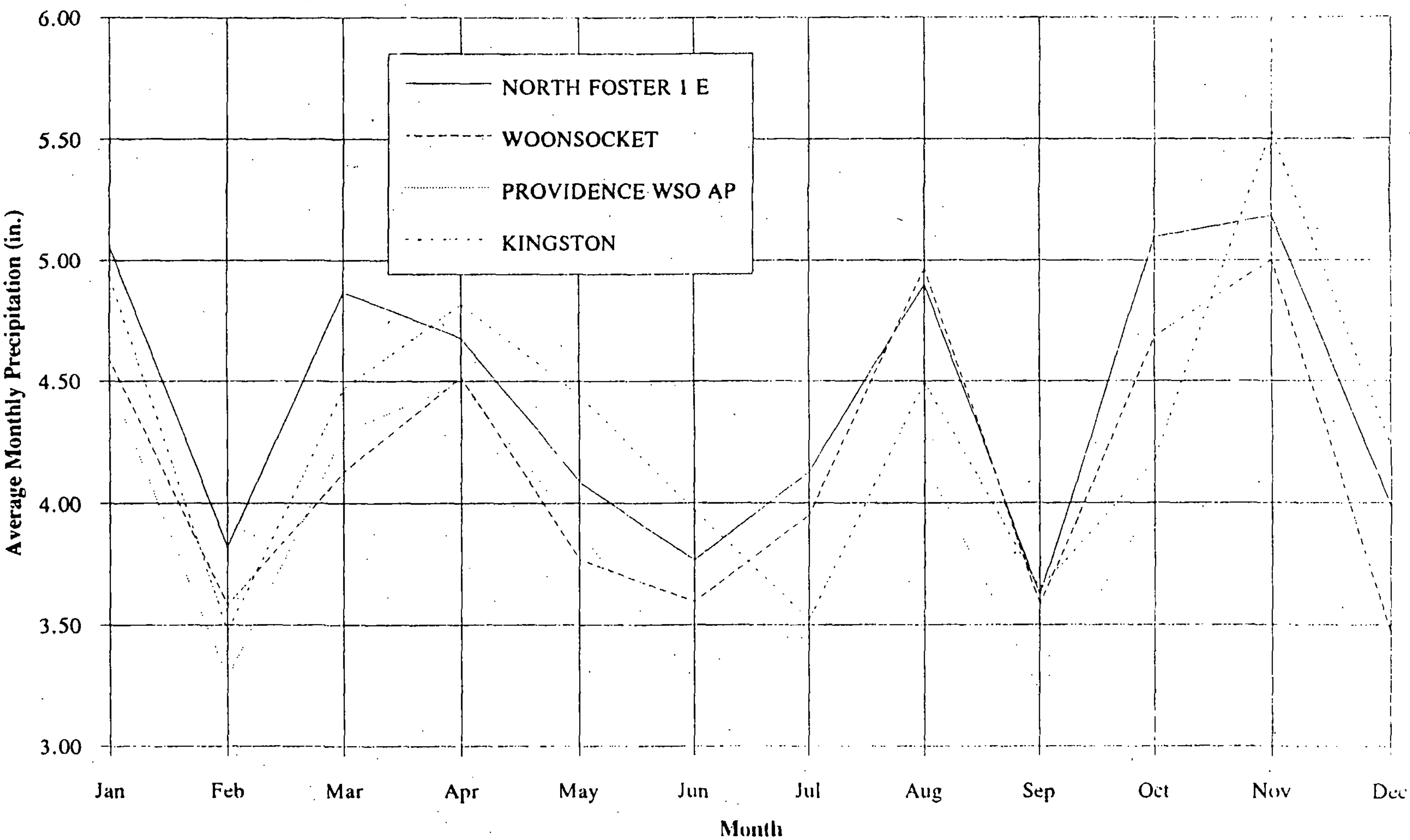
DAVIS LIQUID WASTE SITE
SMITHFIELD, RHODE ISLAND

GROUNDWATER CONTOURS SEPTEMBER, 1993 – BEDROCK –

CONTRACT NO. DACW33-91-D-0005 | FIGURE 2-3

DWG:D5014

Figure 2-4 Average Monthly Precipitation (1975-1991)



2.2 Alternate Area C Permeability

The Scope of Work required the evaluation of an area in the southern portion of the site, designated Alternate Area C in an earlier report (WCC, 1992a), as a potential location for discharge of treated effluent to groundwater. This area is considered to be the best area for groundwater discharge based on its proximity to the proposed Groundwater Treatment System and greater depths to groundwater relative to other parts of the site. In order to evaluate site suitability, a permeability test as specified in the Scope of Work was conducted on the morning of 24 September 1993 in well OW56 at Alternate Area C (see Figure 1-1). The data derived from this test were used to establish location specific hydraulic conductivity parameters for the groundwater model that would be used in the evaluation of the suitability of this area to serve as a potential groundwater discharge site.

OW56 is a 1.25-inch (I.D.) steel well and had been redeveloped two days earlier on 22 September 1993; specific conductance had stabilized immediately and relative turbidity was stable after three hours of development. A total of 55 gallons of water were pumped during the redevelopment period.

Although the Scope of Work specified both rising and falling head tests using a slug for water displacement, the site conditions precluded the use of the slugs brought for the test. The small diameter of the well limited the width of the slug that could be used and exposed screen area over the unsaturated thickness required a significant slug length to generate a compensating rise in the water table. With concurrence from the on-site USACE-NED geotechnical engineer, methodology was switched to a falling head test by injection of water to the well.

A funnel was modified to create a wider orifice and secured to a two foot section of 1.25-inch I.D. steel pipe and coupling. The coupling was fitted over the top of the well riser pipe and the assembly extended above the protective casing of the well. A pressure transducer was lowered through the funnel assembly to approximately 0.5 feet above the bottom of the well and the cable secured with duct tape to the well casing. The cable was attached to a Hermit SE 1000C data logger and data recorded at gradually increasing time intervals with more frequent measurements in the early part of the test.

Four tests were conducted. Test 1 was conducted using 3 gallons of distilled water and water level rose to only 0.197 feet above the static level. A larger volume of water was needed and the on-site USACE-NED representative (geotechnical engineer) approved the use of purge water from the well as a falling head test source based on telephone direction from EPA and Rhode Island Department of Environmental Management (RIDEM) to dispose of the purge water back downhole. The three subsequent tests were conducted using 10 gallons of purge water for each test.

Data from the four tests were downloaded and the results plotted as time versus the logarithm of the water level. Early time data showing the water table fluctuations associated with the water being poured downhole were filtered and the remaining data exhibiting a continuous decline in water level were used in permeability computations based on the Hvorslev equation (1951) as detailed in Dawson and Istok (1991). An average hydraulic conductivity of 10.55 ft/day was calculated using the geometric mean of the results.

A detailed discussion of the permeability testing, plotting of data, calculation of parameters and tabulated pressure transducer data can be found in the Permeability Testing Report in Appendix B.

2.3 Compilation of Analytical Data

The Scope of Work required tabulation of previous groundwater and surface water data to delineate potential expansion areas for the groundwater extraction system, support future discharge permitting activities, and to estimate expected chemical concentrations for the Groundwater Treatment System. Analytical data from the CDM RI (1987) and WCC Pre-Design Report (1992b) were compiled and tabulated for selected groundwater locations in the general area of the 1,000 ppb TVOC overburden and bedrock isocons based on the criteria discussed below in Sections 2.3.1.1 and 2.3.1.2. Analytical data were also tabulated for all surface water locations. This effort included a comprehensive compilation of all available data from these locations for volatile organics, semivolatile organics, pesticide/PCBs, and metals. These data are presented in Section 5 through 8 of Appendix C for groundwater, and Section 9 through 12 of Appendix C for surface water. In addition, the maximum contaminant concentrations historically noted in any wells in the extraction or expansion areas have been summarized in Sections 1 through 4 of Appendix C. These data (maximum concentrations) are presented such that any well for which only one round of data was available is identified with the date of that data, and wells for which more than one round of data are available are labeled "MAX".

2.3.1 Groundwater

2.3.1.1 Initial Extraction Area

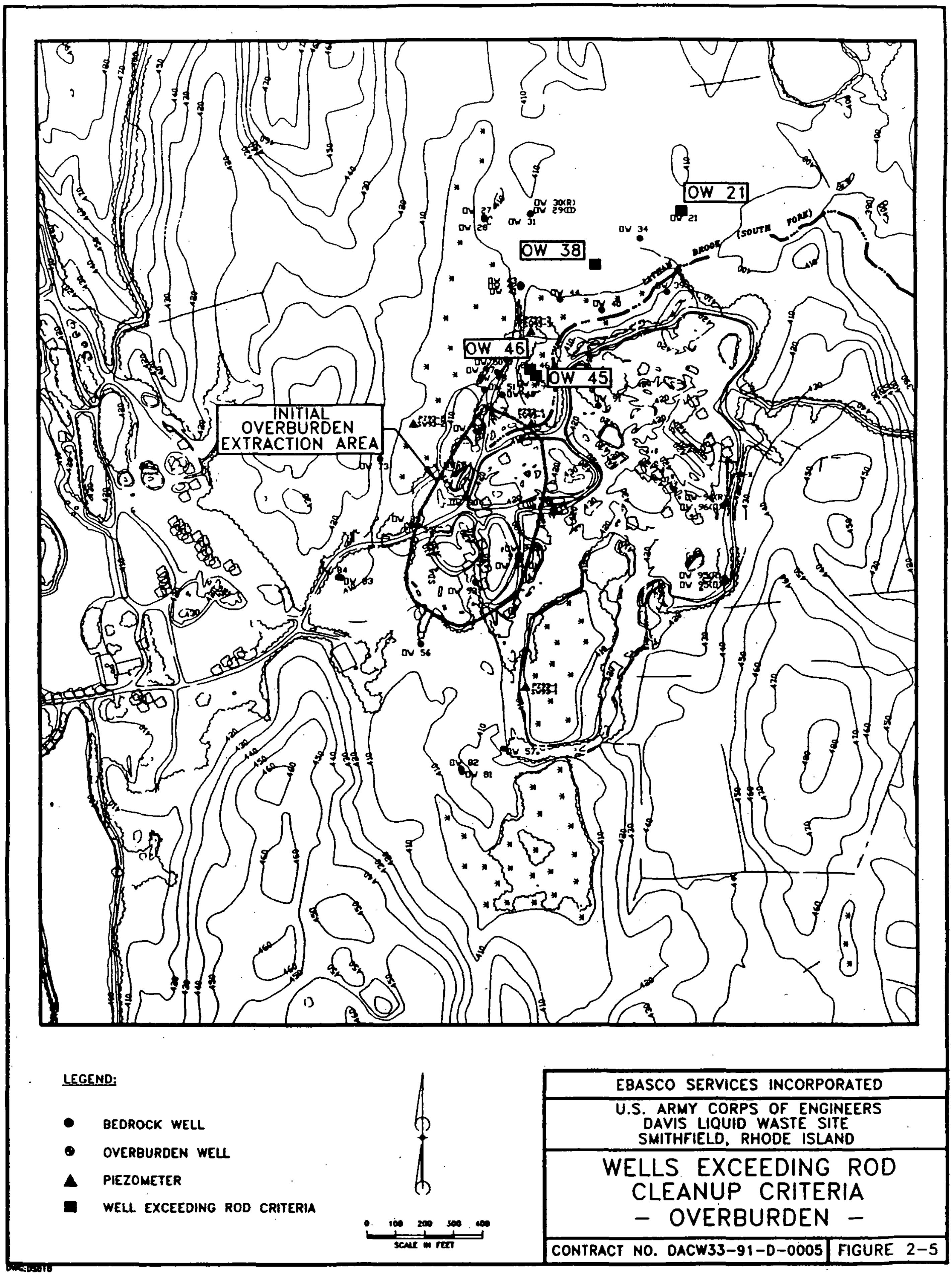
Data were compiled for those wells in the initial extraction area to support estimation of the expected influent concentrations from the extraction wells. The CDM data were derived from three sampling events, November 1984, December 1984 and August 1985. For the CDM RI validated data tabulations, blanks were interpreted as non detects unless otherwise indicated. This is a less conservative practice than assuming that there are no data available. The WCC data were from a September 1991 sampling event. For some wells, WCC selected only certain metals species for analysis. Tabulated data are grouped by analytical fractions for overburden and bedrock wells in Appendix C. It should be noted that not every well was sampled during any one event.

2.3.1.2 Potential Future Expansion Areas

The Scope of Work requires estimation of the combined discharge flowrate. One component of this flow is groundwater extracted from potential future expansion areas. The Record of Decision mandates groundwater cleanup levels of 5 ug/l (ppb) for benzene, trichloroethylene (TCE) and tetrachloroethylene (PCE). For purposes of defining potential groundwater extraction expansion areas, groundwater data sitewide were examined and tabulated for those wells outside of the 1,000 ppb TVOC overburden and bedrock isocons showing a historic exceedence greater than 5 ug/l for either benzene, TCE or PCE. Four (4) overburden wells (OW21, OW38, OW45, and OW46) and seven bedrock wells (OW07, OW33, OW34, OW36, OW41, OW51 and PT-02R) exceeded these criteria and were tabulated (see Appendix C). It should be noted that OW51 is screened in both the overburden and the bedrock. These overburden and bedrock wells are shown on Figures 2-5 and 2-6, respectively. Based on a subsequent redefinition of the initial extraction area by USACE-NED (as shown in Figures 2-5 and 2-6 and discussed in Section 3.5.1.), OW55 is now included in the extraction area and is not considered part of the potential future expansion area.

2.3.2 Surface Water

Surface water data across the site were compiled and tabulated to support potential surface water discharge permit activities. These data are tabulated in Appendix C.



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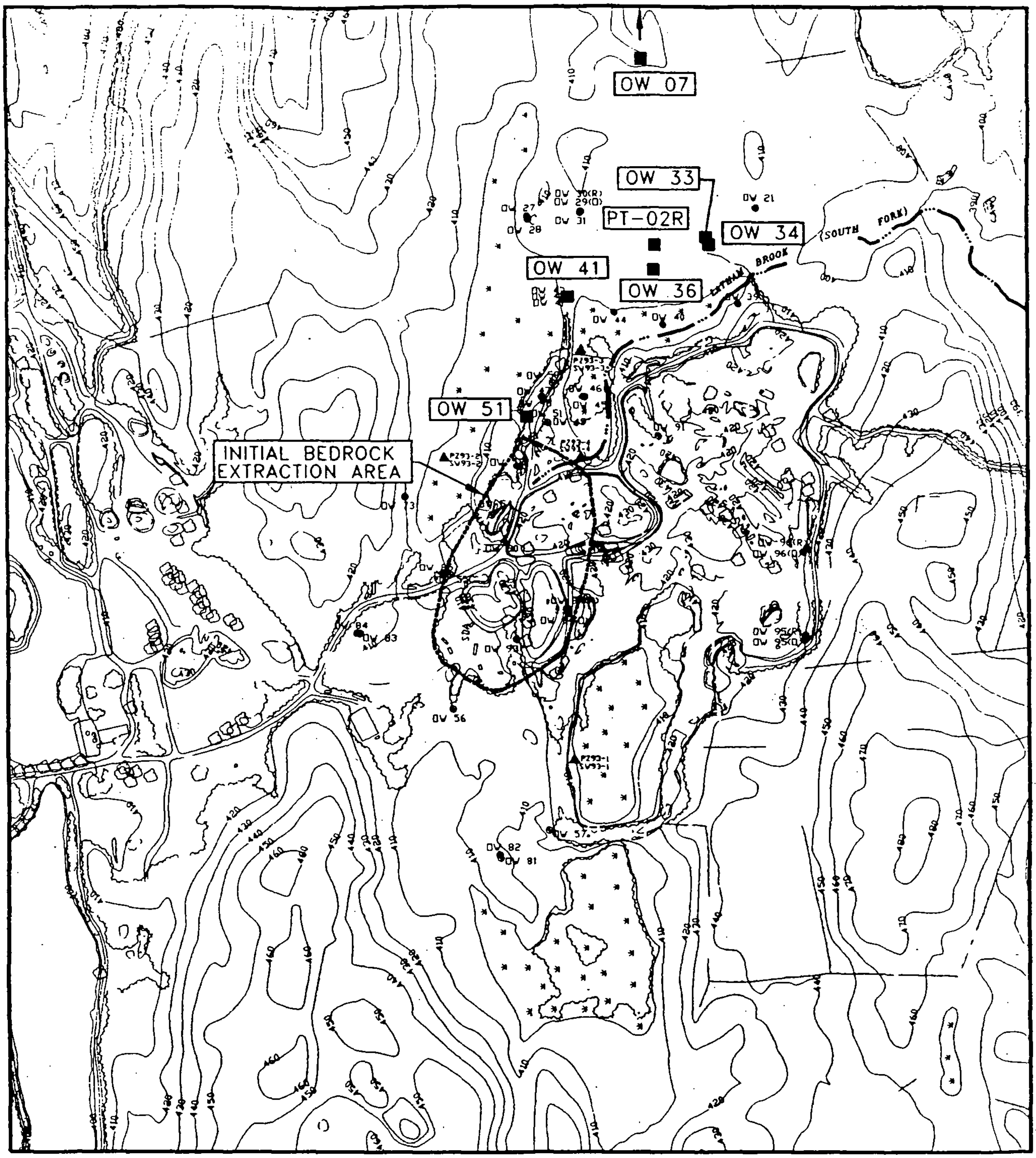
DAVIS LIQUID WASTE SITE
SMITHFIELD, RHODE ISLAND

WELLS EXCEEDING ROD CLEANUP CRITERIA – OVERBURDEN –

CONTRACT NO. DACW33-91-D-0005 | FIGURE 2-5



LEGEND:

- ● BEDROCK WELL
- ⊕ OVERBURDEN WELL
- ▲ PIEZOMETER
- ■ WELL EXCEEDING ROD CRITERIA

0 100 200 300 400
SCALE IN FEET

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SMITHFIELD, RHODE ISLAND

WELLS EXCEEDING ROD CLEANUP CRITERIA – BEDROCK –

CONTRACT NO. DACW33-91-D-0005 | FIGURE 2-6

DWG:DAVSITE9

2.4 Site Survey

The first phase of ongoing site survey work was conducted by Ebasco's subcontractor, Marc Nyberg Associates, as part of the field work performed during fall 1993. The survey consisted of the following activities.

- Piezometers - Four piezometers, installed by Ebasco, were levelled and surveyed on September 23, 1993. The piezometers were designated PZ93-1 through PZ93-4 and are shown as triangles on Figure 1-1. The piezometers provided water level measurements to support the groundwater modeling tasks (See Section 2.1.1).

- Transects for HEC-2 Model - Eight transects from the confluence of the south and north forks of Latham Brook to the proposed location of the Groundwater Treatment System were surveyed on October 1, 1993 and used in the HEC-2 model for the 100 Year Flood Plain Evaluation (Section 6.0). The end points of each transect were properly staked and flagged in the field. Transect locations and profiles are presented in Section 6.0.

- Flagged-in 100 Year Flood Plain - Following identification of the 100-year Flood Plain elevations using the HEC-2 model, Ebasco provided the surveying subcontractor with the predicted elevation for the 100-Year Flood Plain on each side of Latham Brook at each transect. The surveying contractor then surveyed each of these elevations on December 28, 1993 and flagged the locations accordingly on each side of Latham Brook. The delineated 100-year flood plain is presented in Section 6.0.

- Wetland Edges - The wetland edges, as flagged in the field by Ebasco were surveyed between September 24 and October 7, 1993 and delineated on a site plan. This plan was submitted to the RIDEM wetland division for wetland edge verification. On December 28, 1993 representatives of the RIDEM visited the site to verify the wetland edge locations identified by Ebasco. RIDEM requested several minor modifications to wetland "A". A wetland edge map which includes the changes requested by RIDEM during the December 28, 1993 site visit is included in Appendix I. However, Ebasco has not yet received formal comments on the wetland edge verification package submitted to the RIDEM.

The survey data obtained from the activities described above were incorporated into the existing AutoCAD site plan which was translated by Ebasco from the Microstation 10/91 site plan provided by the USACE-NED as part of this Delivery Order. This AutoCAD site plan was then used in preparation of the various figures included in this Criteria Summary Report. A site plan containing all information surveyed to date as part of this Delivery Order is also included in Appendix I. Following approval of the proposed locations, the ground topography will be supplemented by performing a one-foot topographic survey.

2.5 Electric Power and Telephone Service Plan

Site remediation activities planned at the Davis site include a Groundwater Treatment System and a temporary on-site thermal treatment facility (TTF). These remediation facilities and the temporary construction facilities necessary for their installation will require electrical power and telephone service. However, there are no utility services directly available on-site, and electrical power and telephone service in the site vicinity is limited. therefore, both of these utilities will have to be brought on-site.

The purpose of this task is to determine and quantify electric power and telephone requirements for the GWTS and for potential future needs, and develop a conceptual plan for delivering sufficient electrical power and telephone service utilities to the site in order to support the planned remediation facilities and other site activities. This task involved a determination of utility requirements, an assessment of available utilities in the site vicinity and an evaluation of available options to deliver these utilities to the site. Several options were identified and screened. Two viable options were subsequently evaluated. The evaluation criteria included procurement and installation costs, construction scheduling and lead time requirements, and local utility prerequisites. Appendix H presents a complete discussion of the evaluation.

Based on this evaluation, the following option is recommended for delivering utilities to the site:

- Extend existing 12.47 kv, 3 phase electrical power lines from the Forge Road and Log Road intersection to the site via cart path. Extend existing telephone lines from Log Road to the site via the cart path.

The preliminary cost estimate for installation of utility service is estimated at $514,000. This cost includes an allowance for obtaining easement rights along the cart path on private land owners' properties located between the site and Log Road, private contractor installation of overhead utility service along the cart path and public utility companies' off-site installation of 3 phase power along Log road for an approximate distance of 1.8 miles.

The lead time required to meet utility requirements for remedial action mobilization is approximately 14 months inclusive of both on-site and off-site activities. Based on the evaluation conducted, the overall length of the utility service installation schedule is dependent on the public utility companies' off-site installation activities which are estimated to take up to 14 months. On-site installation, which can be performed concurrently, is estimated to take a lesser amount of time.

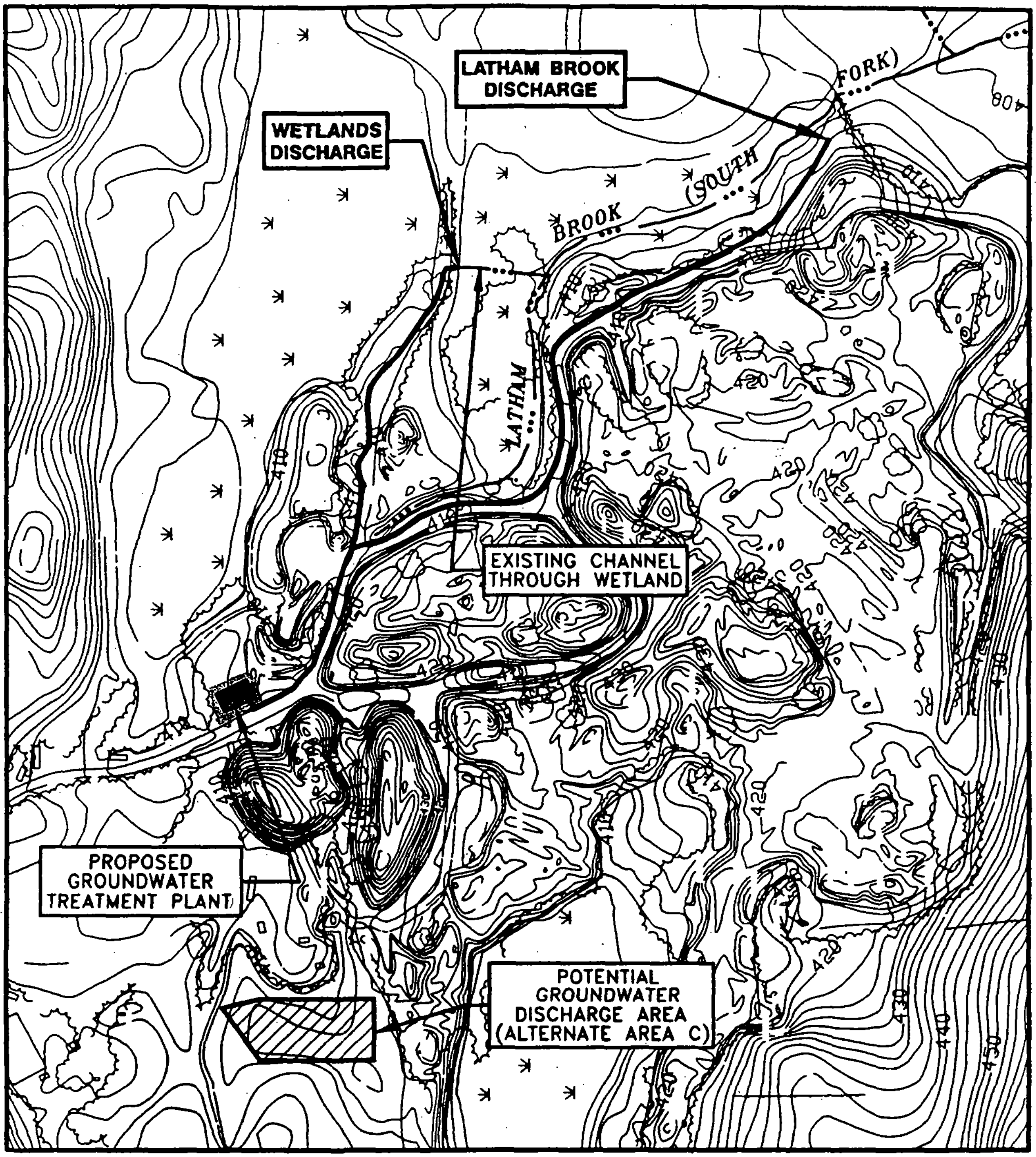
To implement on-site activities, it is estimated that easement procurement will take 6 months. Planning and installation of on-site utility services along the cart path is estimated at 7 months. On-site installation activities could not commence until easement rights have been procured. Therefore, a total of 13 months will be required for installation of on-site services. Easement procurement and on-site service installation activities, as stated above, can be conducted during off-site installation activities. However, assurances that private landowners' are to provide easements will be necessary prior to commencement of off-site installation.

Local prerequisites include local electrical permits, tree clearing permits/approvals and wetland permitting. Time required for local prerequisite compliance is within the estimated 14 month lead time.

Implementation of this option is dependent on whether easements are obtainable. Therefore, land owner contact should be initiated as soon as possible in support of project objectives. If easements cannot be procured, Option 1, as presented in Appendix H, is alternatively recommended.

2.6 Selection of Surface Water Discharge Locations

As part of the field activities, a site walkdown was performed on November 17, 1993 to evaluate site conditions and select potential discharge locations for both the wetlands and brook areas. Based on the walkdown, two specific locations were selected to form the basis for the engineering evaluation presented in this report, as well as for the cost analysis. These discharge locations are identified on Figure 2-7.



KEY:

PROPOSED PIPING RUNS

0 100 200 300 400

SCALE IN FEET

EBASCO SERVICES INCORPORATED	
U.S. ARMY CORPS OF ENGINEERS DAVIS LIQUID WASTE SITE SMITHFIELD, RHODE ISLAND	
POTENTIAL DISCHARGE LOCATIONS	
CONTRACT NO. DACW33-91-D-0005	FIGURE 2-7

DWG:DS020

Prior to the site walkdown, two proposed discharge locations were identified on the site plan. The criteria developed to assess these two potential locations for discharging to surface water included:

- The stream channel at the discharge point should be well defined with sufficient slope to facilitate mixing and flow of water downstream. The channel should be of sufficient size to receive a maximum discharge flow of 45 gpm.

- Discharge piping runs should follow the site roads as much as possible, to avoid disruption of the site tire storage and avoid contaminated areas;

- The discharge location(s) should be at a lower elevation than the GWTS, to potentially allow gravity discharge;

- A minimum cover of 4'-6" over the top of the piping is desirable to maintain frost protection;

- Construction of the piping in the wetlands or wetlands buffer zone should be avoided, if possible. Impacts to the wetlands should also be minimized to the greatest extent practicable;

- The discharge location(s) should have adequate space to allow for construction of an energy dissipation structure, if necessary, and be situated so as to minimize impact on surrounding wetlands during construction;

- Distance from the GWTS to the discharge location should be minimized, to minimize the length and resulting cost of the required piping runs; and

- The discharge location should not require significant velocity and/or erosion controls.

Based on the assessment conducted during the walkdown the locations selected for the surface water discharges as shown on Figure 2-7 are appropriate, and meet the criteria described above.

The selected location for wetlands discharge is near the outlet of an existing culvert under the cart path on site. This location has the advantage of discharging into a portion of the wetlands which is characterized by a discrete flow channel, due to the existing culvert discharge. This channel acts as a stream and during seasonal high flow periods could potentially provide additional flow for effluent mixing and dilution. Space is also available to allow for construction of an energy dissipation apron as part of the outfall structure. Construction of this apron would occur near the cart path, minimizing wetland impact. The piping route leading to this location is relatively flat, but it appears that discharge can occur under gravity flow. Also, freeze protection measures will be required for a portion of the pipe length, since a 4'-6" depth of cover can not be maintained near the outfall structure.

The selected location for discharge to Latham Brook was also confirmed as a suitable location for discharge. This location had the advantage of following the natural terrain such that the piping would exit to an outlet structure through the side of a small hill. This allows the required depth of cover to be maintained throughout the entire length of the discharge piping, without significantly altering the natural terrain. Discharge could be accomplished under gravity flow. Additionally, this location also contains adequate space for an energy dissipation apron between the hillside and the stream bank. The stream channel at the point of discharge is discrete with well defined banks and slope to receive discharges. The channel is of adequate size to receive the additional flow.

Based on the above, the two discharge locations selected in this section will be carried forward and used in the Engineering Evaluation.

2.7 Wetlands

2.7.1 Wetlands Delineation

Wetlands on the Davis site were delineated in the following areas:

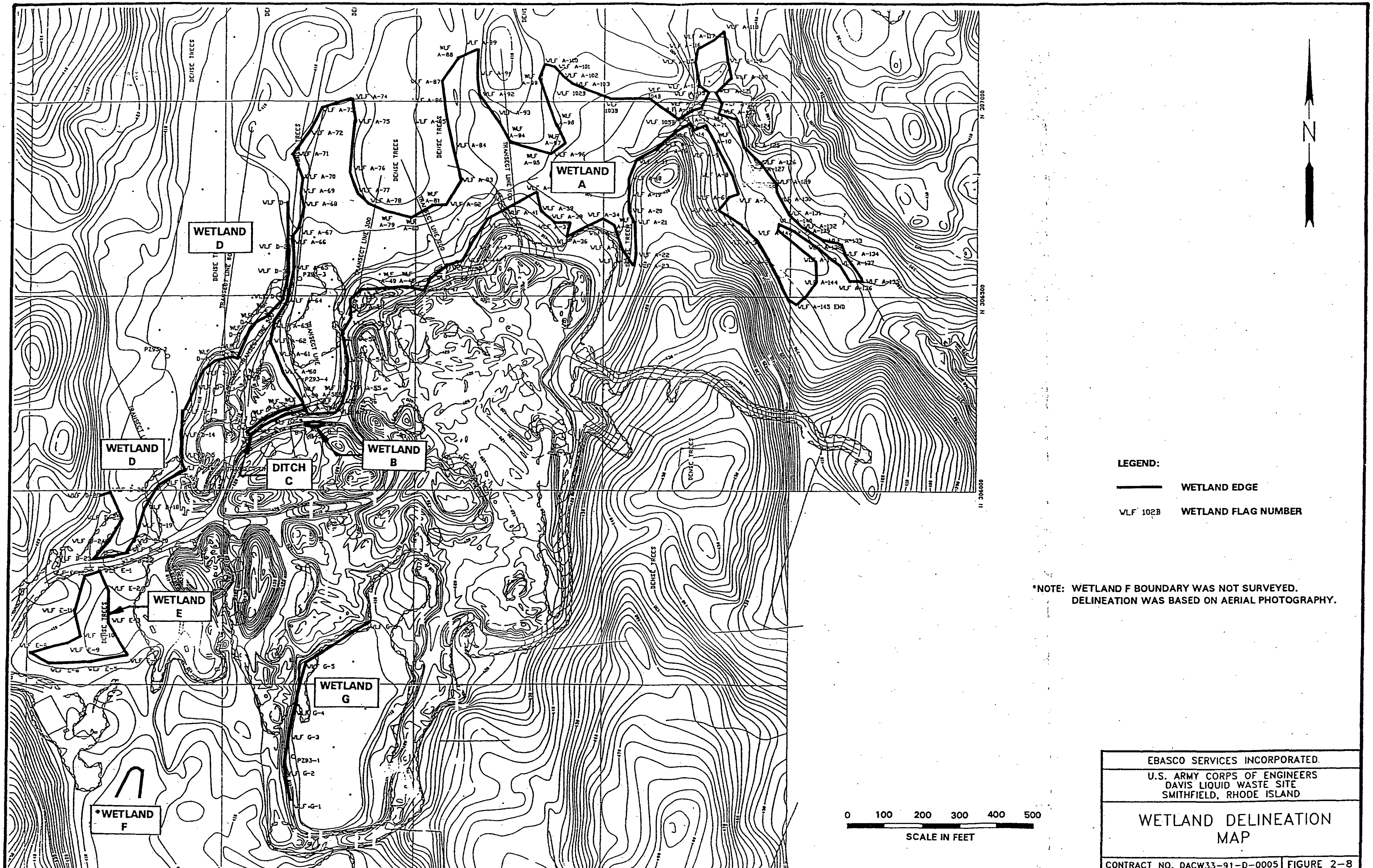
- within 200 feet of the proposed Groundwater Treatment System location;
- areas potentially affected by the proposed groundwater withdrawal; and
- areas potentially impacted by the discharge of the treated groundwater to the groundwater or surface water (Latham Brook).

The wetlands were initially identified using a color aerial photo of the Davis site. The wetland edges were then field delineated and flagged using wetland criteria endorsed by the Rhode Island Department of Environmental Management (RIDEM). RIDEM regulates the delineated wetlands, and also regulates an area of land within 50 feet of a freshwater wetland (known as the "Perimeter Wetland", RIDEM Wetland Rule 5.09). These criteria include the presence of greater than 50% hydrophytic vegetation, hydric soils, and hydrologic indicators. Wetlands were classified according to Tiner (1989), and Golet and Larson (1974).

The edges of six wetland areas, designated wetlands A, B, D, E, F and G, were delineated and flagged. The drainage ditch from the Northern Disposal Area to wetland A was flagged and designated as wetland C. This ditch is contaminated since it collects drainage from the Northern Disposal Area, and is planned to be remediated during the Source Control Remedial Action. Therefore, it has not been included in the evaluation. The landowner, Mr. Davis, later removed the flagging from Wetland F and denied permission to reflag this wetland. The USACE-NED instructed Ebasco to show the general edge of wetland F on the base map, and not to attempt to reflag the edges. The flagged edges of wetlands A, B, D, E, G, and ditch C were then surveyed by Marc N. Nyberg Associates, Inc. (Nyberg), and their location added to the site base map. RIDEM later field checked the wetland delineation. Based on RIDEM verification, some slight modifications to the edges of Wetland A were made and resurveyed by Nyberg. The surveyed edges of the five wetlands and the ditch, and the general boundary of wetland F are shown on Figure 2-8, and the G-size figure is included in Appendix I.

Three plant community types comprise the six wetland areas - forest swamp, shrub swamp and emergent wetland (marsh and wet meadow). Wetlands A, B, D, E and F are primarily forest swamp wetlands. A, D, and E also have some shrub swamp and emergent wetland, while Wetlands B and F also have some shrub swamp. Wetlands A and D are comprised of approximately 70 percent forest swamp and 30 percent shrub swamp and emergent wetland. Wetland B has 60 percent forest swamp and 40 percent shrub swamp, while Wetland E has 60 percent forest swamp and 40 percent shrub swamp and emergent wetland. Wetland F is 80 percent forest swamp with 20 percent shrub swamp and some open water. Wetland G is comprised of a mixture of shrub swamp and emergent wetland.

The forest swamps are principally made up of an overstory tree layer of red maple. Wetland B, a very small wetland, has an overstory of black willow. In these forest swamps, highbush blueberry, sweet pepperbush, swamp azalea, spice bush, winterberry and maleberry are the dominant shrub species. Common greenbriar and summer grape are the dominant vine species, and cinnamon fern, jewelweed and sphagnum moss are some of the dominant herbaceous species.



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SMITHFIELD, RHODE ISLAND

WETLAND DELINEATION MAP

CONTRACT NO. DACW33-91-D-0005 FIGURE 2-8

The shrub swamps are made up of the same shrub and vine species as the forest swamps as well as steeplebush, northern arrowwood, leatherleaf, swamp sweetbells and smooth alder. Some small red maple trees are also present. The dominant herbaceous species in the shrub swamps and emergent wetlands include those found in the forest swamps as well as sensitive fern, marsh fern, tussock and other sedges, beggar-ticks, rattlesnake grass, swamp candles, grass-leaved goldenrod, false nettle, and lesser bur-reed.

The complete Wetland Delineation Report and a listing of the wetland plant species found in the wetlands on the Davis site are provided in Appendix I.

2.7.2 Endangered and Threatened Species Survey

A survey for state and federally listed endangered and threatened plant and wildlife species was performed in the Davis Liquid site wetlands. The Rhode Island Natural Heritage Program (RINHP) was contacted prior to the survey to obtain the list of federal and state endangered and threatened plant and animal species in Rhode Island (see Appendix I), and to determine if there were known or potential occurrences of any of the listed endangered and threatened species in the specified wetlands.

RINHP reported no documented or potential occurrences of federally or state listed species on the Davis Liquid site and vicinity (Enser, 1993). An endangered and threatened species survey was performed in the six wetland areas on October 1 and 4, 1993. Additional observations were made while performing the wetland delineation and other activities on October 5 and 7. No endangered or threatened plant or animal species were observed.

2.7.3 Wetlands Impact Assessment of Groundwater Withdrawal

An assessment of the potential impact of the groundwater withdrawals from the proposed Groundwater Treatment System on the functions and values of the wetlands on the Davis site was made. Some of the more important functions and values of freshwater wetlands in Rhode Island include (Tiner, 1989):

- fish and wildlife habitat
- rare, endangered and threatened species habitat
- aquatic productivity
- flood control
- water quality maintenance
- groundwater recharge
- recreation

The six wetland areas on the Davis site provide several of these functions and values, including wildlife habitat, aquatic productivity, flood control, groundwater recharge, and water quality maintenance. These wetlands currently provide habitat for several species of mammals, birds, amphibians and reptiles. Some wildlife species observed included frogs, salamanders, deer and several bird species. No fish or rare, endangered or threatened species of plants or animals are known to occur in the six wetlands or in the portion of Latham Brook located within the site. Since this is private property, no public hunting or other public recreational use of the wetlands occurs, so the wetlands do not provide significant recreational value.

The proposed groundwater extraction system currently consists of eight pairs of extraction wells located as shown on Figure 7-2. Based on the groundwater modeling extraction well configuration simulations (see Appendix D), it was determined that the eight well pairs pumping at a total flow (withdrawal) rate of 22.5 gpm could result in drawdown of the wetland water table during the summer season. Based on the simulations, seasonal adjustments to the pumping rate could be required to avoid dewatering of wetlands, with the summer pumping rate limited to 17 gpm. The estimated maximum drawdown is predicted to be less than six inches (see Figures 3-8 and 4-14 in Appendix D to compare predicted drawdowns). Under this well configuration and with the ability to limit the pumping rate to minimize wetland drawdown, the groundwater withdrawal would have minimal impact on the six wetlands or on their aforementioned functions and values. Water levels in the wetlands are expected to be monitored during system operation to ensure compliance with acceptance criteria yet to be developed with RIDEM.

The drawdown under the most likely (6 inches) and worst (12 inches) case scenarios would not significantly impact the wetland plant communities and their functional values. Due primarily to their smaller size, wetlands B, E and G would be most vulnerable to drawdown impact. Wetland B is presently merely a very small remnant wetland area serving no real functional value. Both wetlands E and G are usually inundated by surface water in the wet seasons (winter - spring). The likely and worst case drawdowns would create drier conditions in these wetlands, but hydric soil conditions should remain to maintain these areas as wetlands. Some shift in plant composition may occur where wetland species preferring drier conditions would become more dominant. However, many of the plant species presently in these wetlands have a wide water level tolerance range and will survive drier conditions. Wetlands E and G would continue to function as wetlands and provide the same functional values prior to drawdown.

Due to their large size, wetlands D and A will show an even lesser effect to drawdown. Both wetlands receive surface water recharge from wetland E during the flood periods. During drawdowns, less flood waters would discharge from wetland E to these wetlands; however, these wetlands receive substantial surface water runoff from the surrounding uplands, as well as groundwater recharge. The impact of the drawdown on the wetlands under either scenario would be minimal. Again, the slightly lowered water levels in these wetlands should not result in a change of these areas to upland communities. They will remain wetlands. Little to no change in plant species composition would result with the slightly drier conditions. The wetland plant species presently occurring in these wetlands are able to tolerate drier wetland situations. No reduction in their functional values is expected to result under either drawdown scenario.

2.8 Environmental Baseline Assessment

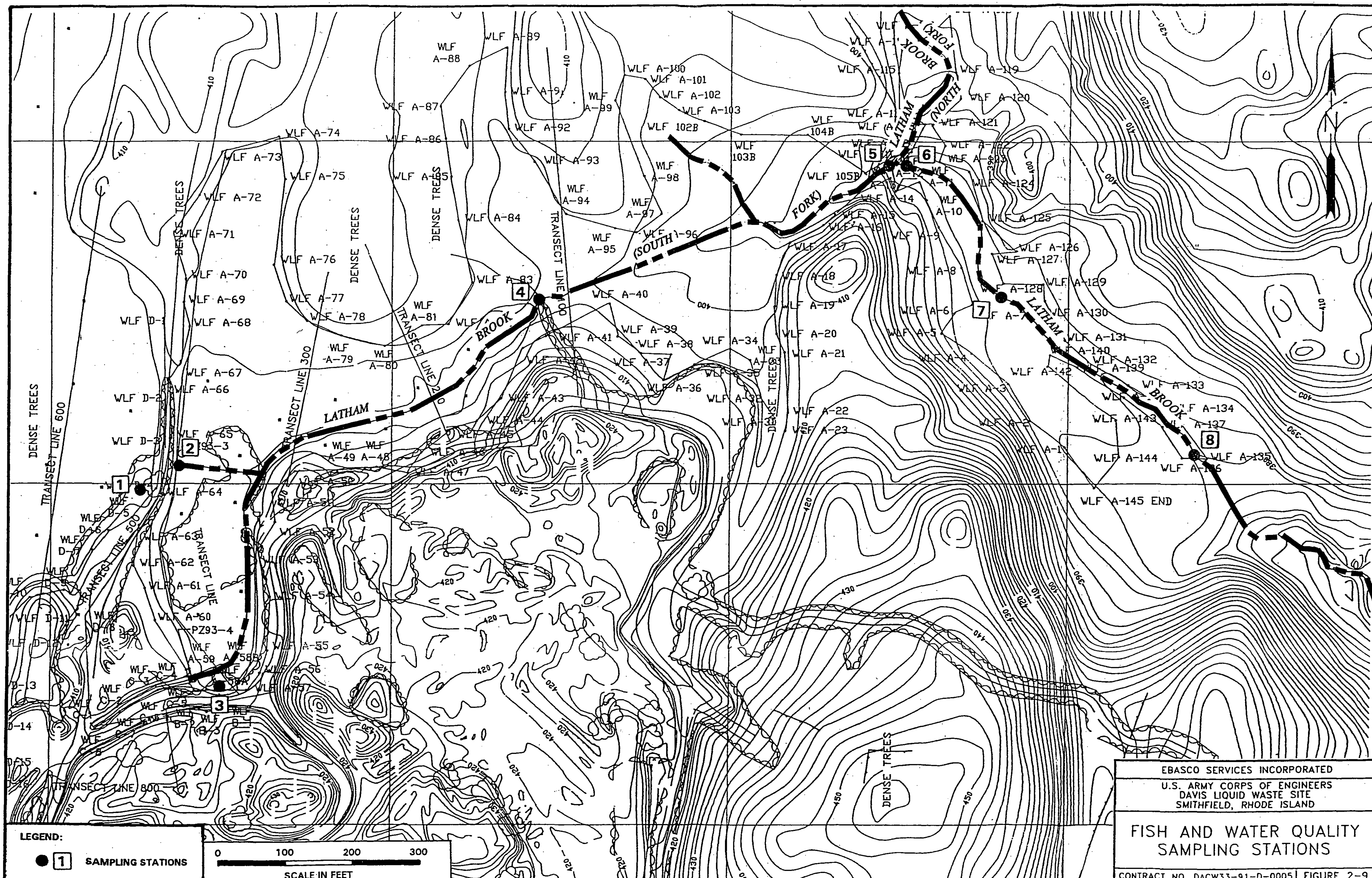
2.8.1 Latham Brook Fish Community Study

2.8.1.1 Fish Community Study Area

A survey was conducted to qualitatively characterize the fish community in Latham Brook downstream of the potential surface water discharge locations for the treated groundwater. Given the potential surface water discharge locations, as presented in Section 2.6, are the South Fork of Latham Brook near monitoring well OW-39 and the headwater wetlands upstream of the South Fork, the fish survey area was defined to include the South Fork and the main channel of Latham Brook to approximately 620 feet downstream of the confluence of the South and North forks and any ponded areas in the headwater wetlands where fish could exist. These locations are shown in Figure 2-9.

2.8.1.2 Fish Community Study Methods

Representative habitats within the designated reaches of the brook and headwater wetlands were sampled for fish using a backpack electroshocker (Smith Root Type XI High Energy Pulsator), on September 23,



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FISH AND WATER QUALITY SAMPLING STATIONS

CONTRACT NO. DACW33-91-D-0005 | FIGURE 2-9

1993 to determine fish species and relative abundance. Additional data (DO and air temperature) were collected on October 1, 1993. A total of eight sampling locations, designated as stations 1-8, were surveyed. Two stations were in the headwater wetlands on either side of the wet path (stations 1 and 2), three in the South Fork (stations 3-5), and three in the main channel of Latham Brook (stations 6-8) (see Figure 2-9). At each station, water quality parameters including the water temperature, dissolved oxygen (DO), specific conductivity and pH were measured. Instrumentation used included a YSI Model 51B DO Meter, YSI 33 SCT meter and a Beta Technology Model HYDAC conductivity/temperature and pH unit. The width, depth and bed substrate characteristics of the waterbody were noted at each station.

2.8.1.3 Fish Community Study Results

During the electrofishing surveys, no fish were caught or observed at the eight stations. Other aquatic species were caught or observed at six stations, including the green frog (Rana clamitans), unidentified young salamanders and crawfish. The waterbody parameter measurements and characteristics, and animals caught at each station are provided in Table 2-3.

The electrofishing survey results indicated that no fish were present in this particular reach of Latham Brook (South Fork and main channel) and the headwater wetlands. Measurements at all stations indicated water quality generally good for fish life, except at stations 2 and 3 where the DO was low (2.2 mg/l and 2.4 mg/l) in the two very small pockets of standing water. The absence of fish at these stations (Stations 2 and 3) is due to the lack of favorable habitat conditions (both water quality and available water depth). The absence of fish in the other headwater wetland (station 1) area may be due to other reasons. The ponded area in this wetland (D) (see Wetland Delineation Section) is probably permanent, but the measured acidity (4.43 standard pH units) probably makes the water unsuitable to fish. Station 3, the other ponded area in wetland A, has a fluctuating water level and low DO levels which prohibit fish life. The absence of fish along the other reaches of the brook (stations 4-8) is probably due to the intermittent nature of this section of the brook. During the summer, parts of the South Fork and main channel of Latham Brook are known to become dry, preventing a fish community from becoming established.

2.9 Evaluation of Culverts

This section presents a summary of the evaluation of the off-site culverts on Latham Brook downstream of the site. A copy of the full report is included as Appendix J. The evaluation includes a physical description of the culverts, estimate of flow capacity, assessment of culvert integrity and photographic documentation. Additional HEC-2 runs were also performed as part of this evaluation to determine the impact of various flood flows on the first downstream culvert. The culverts evaluated were designated as follows moving downstream from the site on Latham Brook (see Figure 2-10):

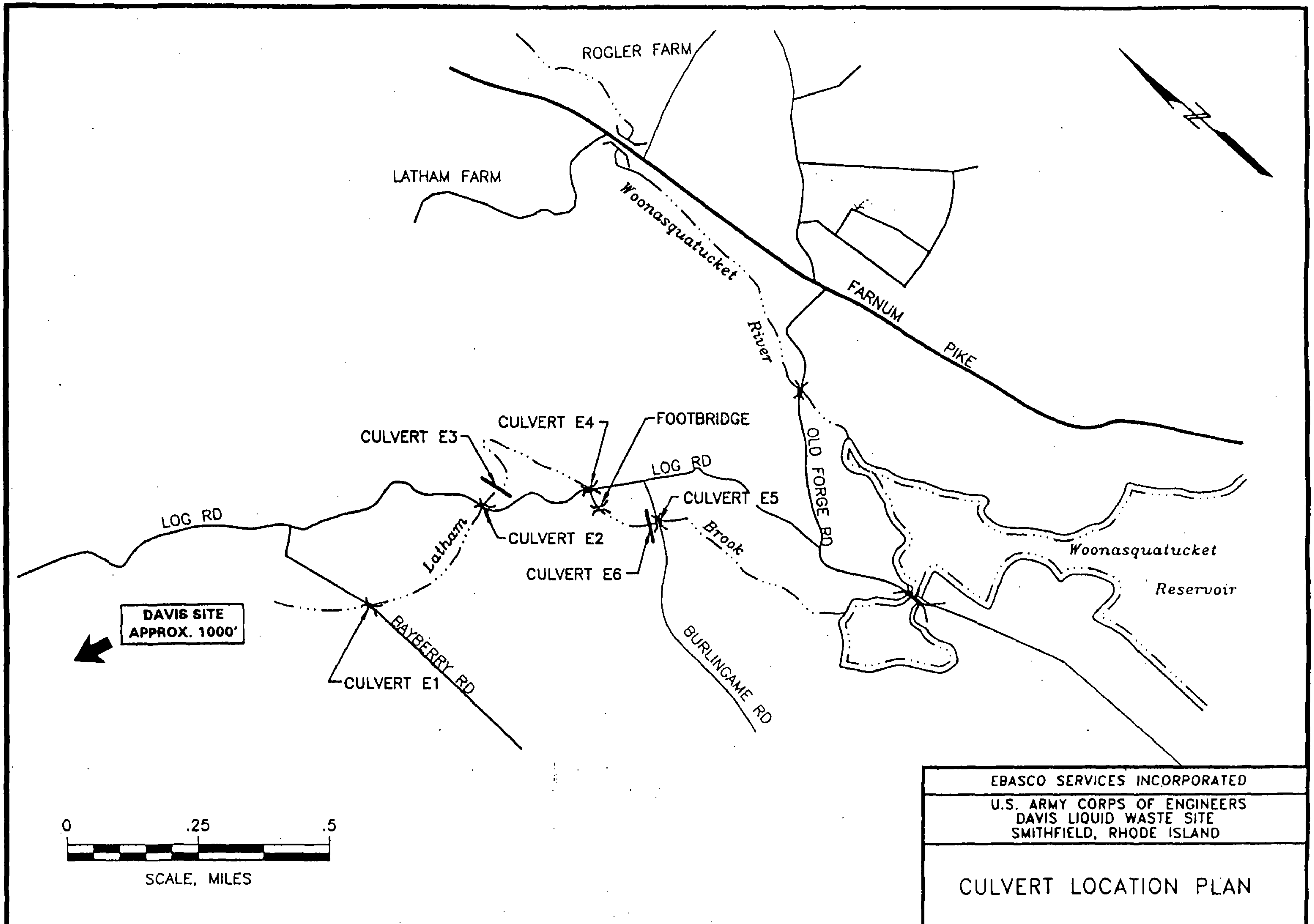
- E1, where Bayberry Road crosses Latham Brook
- E2, where Log Road crosses Latham Brook (upstream location)
- E3, where Latham Brook passes through a small fieldstone dam just downstream of E2
- E4, where Log Road crosses Latham Brook (downstream location)
- A recently constructed footbridge downstream of E4
- E5, where Burlingame Road crosses Latham Brook
- E6, small partially completed dam upstream of E5.

Table 2-3
Electrofishing Survey Data Taken at the Davis Liquid Waste Superfund Site
September 23 and October 1, 1993*

Station **	Sampling Location	Temperature (°C)		DO (mg/l)	Specific Conductivity (mS/cm)	pH (s.u.)	Waterbody Characteristics	Animals Caught
		Water	Air					
1	Wetland D Ponded Area	8 11.5	10	12.6	42 118	4.43	Width - 12-30 ft. Depth - 3-4 ft. Substrate - organic silt muck	Green frog (subadults and tadpoles)
2	Wetland A-Two Very Small Ponded Areas west and east at PZ-93-3	8 12.5,13.6	14	2.2	172 128, 139	5.98	Width - 2-3 ft. Depth - 5 in. Substrate - organic silty loam	No animals caught
3	South Fork Latham Brook at PZ-93-4	10 13.5	14	2.4	172 110.5	6.27	Width - 4-15 ft. Depth - 4-12 in. Substrate - organic silt	Green frog (subadults and tadpoles)
4	South Fork Latham Brook-610 ft. upstream from confluence with North Fork	10 12.5	11	10.2	125 97.5	6.02	Width 3-4 ft. Depth 4-6 ft. Substrate - silty sand	Green frog (subadults and tadpoles) Crawfish
5	South Fork Latham Brook-5 ft. upstream of confluence with North Fork	8 11.2	13.5	10.6	108 91.3	6.36	Width - 10 ft. Depth - 2-4 in. Substrate - silty sand	No animals caught
6	Latham Brook-5 ft. downstream of confluence of South and North Forks	8 11.6	13.5	10.6	108 92.7	6.36	Width - 10 ft. Depth - 2-4 in. Substrate - silty sand	No animals caught
7	Latham Brook-260 ft. downstream of confluence of South and North Forks	8 11.6	12	11.4	108 94.1	6.31	Width - 10-15 ft. Depth - 4-6 in. Substrate - boulders, silty sand	Green frog (subadults and tadpoles) Crawfish
8	Latham Brook 620 ft. downstream of confluence of South and North Forks	8 11.5	12	11.0	108 90.5	5.98	Width - 15 ft. Depth - 6 in. Substrate - boulders, silty, sandy muck	Green frog (subadults and tadpoles) Salamander (young)

* DO and air temperature measured on September 23, pH measured on October 1, all other measurements reported were taken on September 23 (first line) and October 1 (second line).

** Sampling Station locations are shown on Figure 2-9



Source: GEI Consultants, Inc.

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SMITHFIELD, RHODE ISLAND

CULVERT LOCATION PLAN

CONTRACT NO. DACW33-91-D-0005 | FIGURE 2-10

Revised 05/27/94

2.9.1 Assessment of Culverts' Physical Condition and Flow Capacity

A site visit was conducted on October 8, 1993 to evaluate the six downstream culverts and a recently constructed footbridge.The flow through each of the culverts was estimated at the time of the evaluation, as well as the adequacy of the culvert in passing the flow.

During the site visit the integrity of the culverts was assessed. Complete photographic documentation is presented in Appendix J. The oldest culverts appear to be E2 and E3, approximately 100 years old. Culverts E1, E4, and E6 appear to be over 20 years old and culvert E5 appears to be less than two years old. All of the culverts appear to be in generally good condition. The estimated flow at the time of the site visit ranged from a low of 25 gpm (.06 cfs) at E5 to a high of 103 gpm (.23 cfs) at E4. A discharge rating curve was computed for each culvert for flows ranging up to 660 cfs, the 100-year flood flow calculated by WCC using HEC-1 and HEC-2 modeling (WCC PDER II, 1993). The method used for computing these flows was the U.S. Department of Transportation Federal Highway Administration HY8 Culvert Analysis Program. Table 2-4 indicates the estimated flow for each culvert during the site visit. The estimated flow capacity of each culvert is also presented. For each culvert, the flow capacity is considered equal to the flow which overtops the road above each culvert. The water elevation at which overtopping of the road occurs (i.e., the actual road elevation) is also included in Table 2-4.

Table 2-4

Estimated Flow and Flow Capacity of Culverts

Culvert Designation	Estimated Flow at Site Visit (cfs)	Flow Capacity of Culvert - Estimated Flow which Causes Overtopping of Road (cfs)	Water Elevation at which Overtopping of Road Occurs (ft. NGVD)
E1	.07	55.1	356.01
E2	.15	85.3	311.82
E3	.19	206.6	310.73
E4	.23	116.2	278.85
Footbridge	N/A	284.2	276.00
E5	.06	162.4	264.50
E6 (left culvert)	.08	91.3	265.97

Note: N/A indicates not evaluated.

Based on a comparison of the observations made during the site visit, the estimated flowrate at the time of the site visit and the estimated flow capacity of each culvert, the culverts are considered adequate in passing the flow at the time of the site visit.

2.9.2 Impact of Flood Flows on First Downstream Culvert (E1)

HEC-2 model runs were performed for the 2, 10, 25 and 100-year flood events to determine the potential impact of these flood flows on the first downstream culvert. The basis for the HEC modeling and the complete results are included in Appendix G. The predicted flowrates and surface water elevations at the first downstream culvert (E1) are presented in Table 2-5.

In Table 2-4, the flow capacity of Culvert E1 is estimated at 55.1 (cfs) with an elevation at which overtopping of the road above the culvert occurs of 356.01 (ft. NGVD). A comparison of results

Table 2-5

Flowrates and Surface Water Elevations at Culvert E1

Flood Event	2-Year	10-Year	25-Year	100-Year
Predicted Flow Q (cfs)	88	223	332	481
Predicted HEC2 Surface Water Elevation at Culvert E1 (ft. NGVD)	356.33	356.83	357.04	357.28

presented in Table 2-4 and 2-5 indicates that Culvert E1 will be overtopped by the predicted 2-year flood flow of 88 cfs and, in fact, by all of the modeled flood events. Under the 2 year scenario, the road is overtopped by approximately 4 inches and by approximately 15 inches during the 100-year flood event.

The 2-year flood scenario was then run to include the expected 45 gpm (.10 cfs) discharge from the on-site treatment system. Table 4-2 in Appendix G contains the HEC-2 modeling results and indicates that there was no change in surface water elevation for the 2-year flood event with the additional treatment system discharge included. The treatment system discharge of 0.10 cfs is very small (3 orders of magnitude) compared to the storm runoff imposed on the first downstream culvert E1 created by the 2-year, 10-year, 25-year, and 100-year flood events. Therefore, the additional flow attributed to the on-site treatment system discharge is considered to not have a significant impact on the first downstream culvert (E1), as the culvert already appears to be incapable of handling recurring flood events. Ebasco recommends contacting local residents to obtain historical accounts to further evaluate the HEC modeling results.

3.0 GROUNDWATER MODELING

The Scope of Work requires the establishment of design criteria for an extraction system that effectively contains the groundwater emanating from the main source areas and best captures the groundwater from the areas of the 1,000 ppb TVOC overburden and bedrock isocons as depicted by Woodward-Clyde Consultants (WCC, 1992). To aid in the design of the groundwater extraction system, the USGS three-dimensional groundwater flow model, MODFLOW (McDonald and Harbaugh, 1988) was used to develop estimated pumping flow rates, optimize well placements, and optimize groundwater capture in the initial extraction area. Aquifer parameters were derived from existing site data and the September 1993 field efforts described in Section 2.0. The model was also used to evaluate the suitability of Alternate Area C for groundwater discharge of treated effluent. This section summarizes the groundwater modeling effort. A more detailed discussion of the groundwater model is found in the Groundwater Modeling Report in Appendix D.

3.1 Numerical Model Construction

A model area of 1950 feet in an east to west direction by 2500 feet north to south direction was specified, and a grid was developed with a resolution of 25 feet in the area containing the 1,000 ppb TVOC isocons. The resolution in the remaining modeled area was 50 feet. Transition cells with 35 feet and 40 feet spacings were specified between the two areas of resolution for model stability considerations. The model grid is shown in Figure 3-1. The lateral boundaries were selected so that the grid extended to topographic highs to the east and west where no-flow conditions could be specified. The grid was extended to the south to ensure that groundwater discharge to Alternate Area C would not influence specified heads along that boundary, and to the north to ensure that extraction well flows would not influence specified heads along that boundary. The model contains a total of 52 columns and 68 rows.

The site stratigraphy consists of 20-30 feet of overburden materials lying above bedrock (CDM, 1986). The upper bedrock is assumed to be somewhat fractured based on the boring logs available, and the competent bedrock below is assumed to be sufficiently impervious to warrant a no-flow boundary at the bottom of the model. In many of the lower-lying areas, water "collects" or "accumulates" in ponds on the surface and creates wetlands. The lowland areas of the model are relatively flat, with the exception of the northeast corner where the South Fork of Latham Brook steeply exits the system.

To approximate the significant effect of the wetlands and the subsurface strata on the overall hydrogeologic system, a four-layer model was developed (Figure 3-2). Layer 1 represents the wetlands, and is modeled as a zone of high horizontal hydraulic conductivity and low vertical conductivity, in the wetland areas. Layers 2 and 3 represent the upper and lower parts of the overburden at the site. Two layers were specified in the overburden: (1) to provide a smoother geometric transition from the thin overlying wetland layer, and (2) to provide resolution in the overburden for estimates of groundwater heads and particle trajectories. Layer 4 represents the upper portion of the underlying fractured bedrock, that is assumed to be in hydraulic communication with the overburden.

A more detailed discussion of the site conceptual model and initial model construction is found in Appendix D.

3.2 Model Parameters

Initial model parameters were based on available data and are summarized on Table 3-1. In many cases, site specific data germane to the modeled area were unavailable, and either text book values or regional values were extrapolated to the model grids. A telephone conference call (October 27, 1993) with

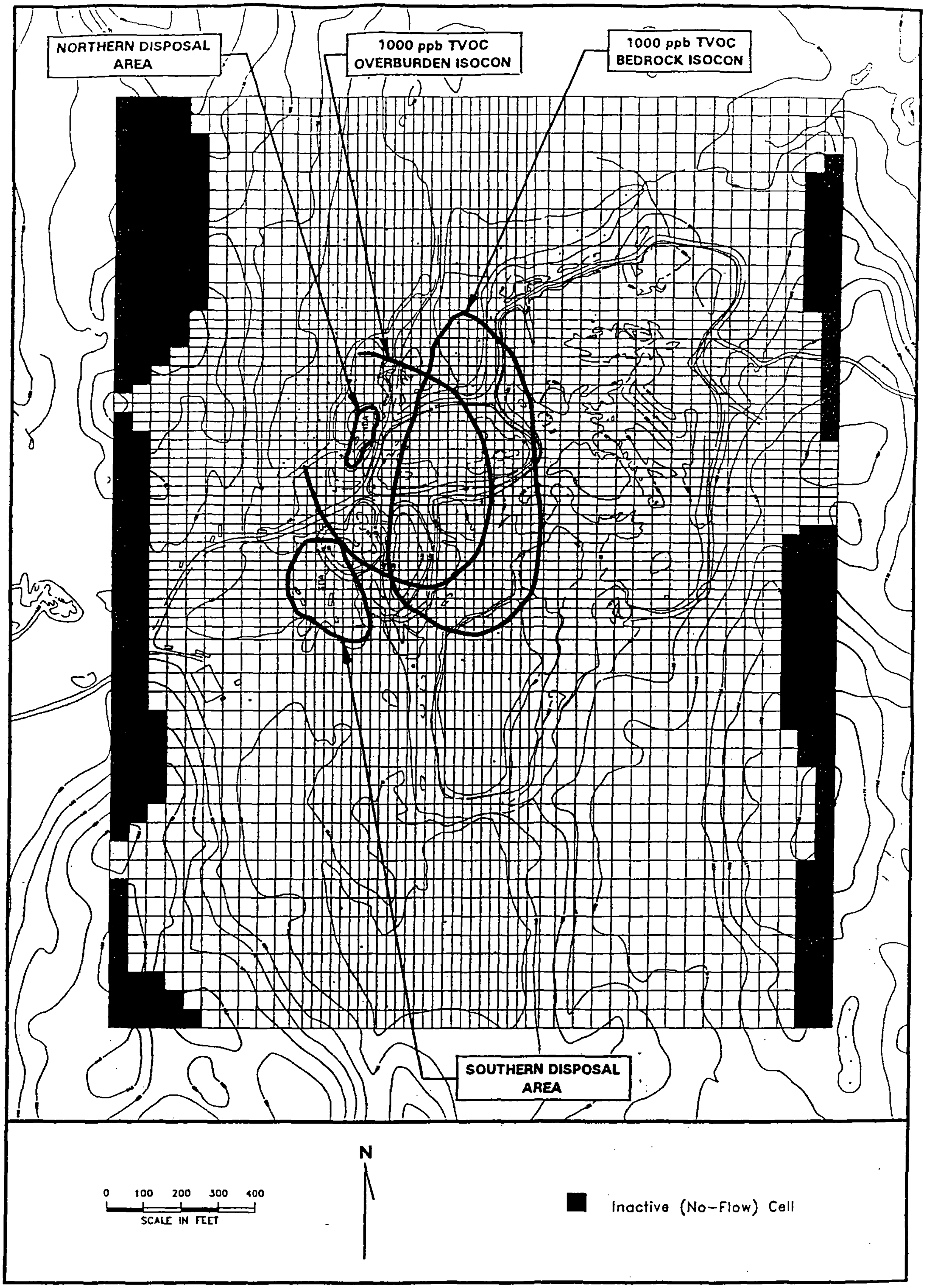


Figure 3-1 Computational Grid for Groundwater Model

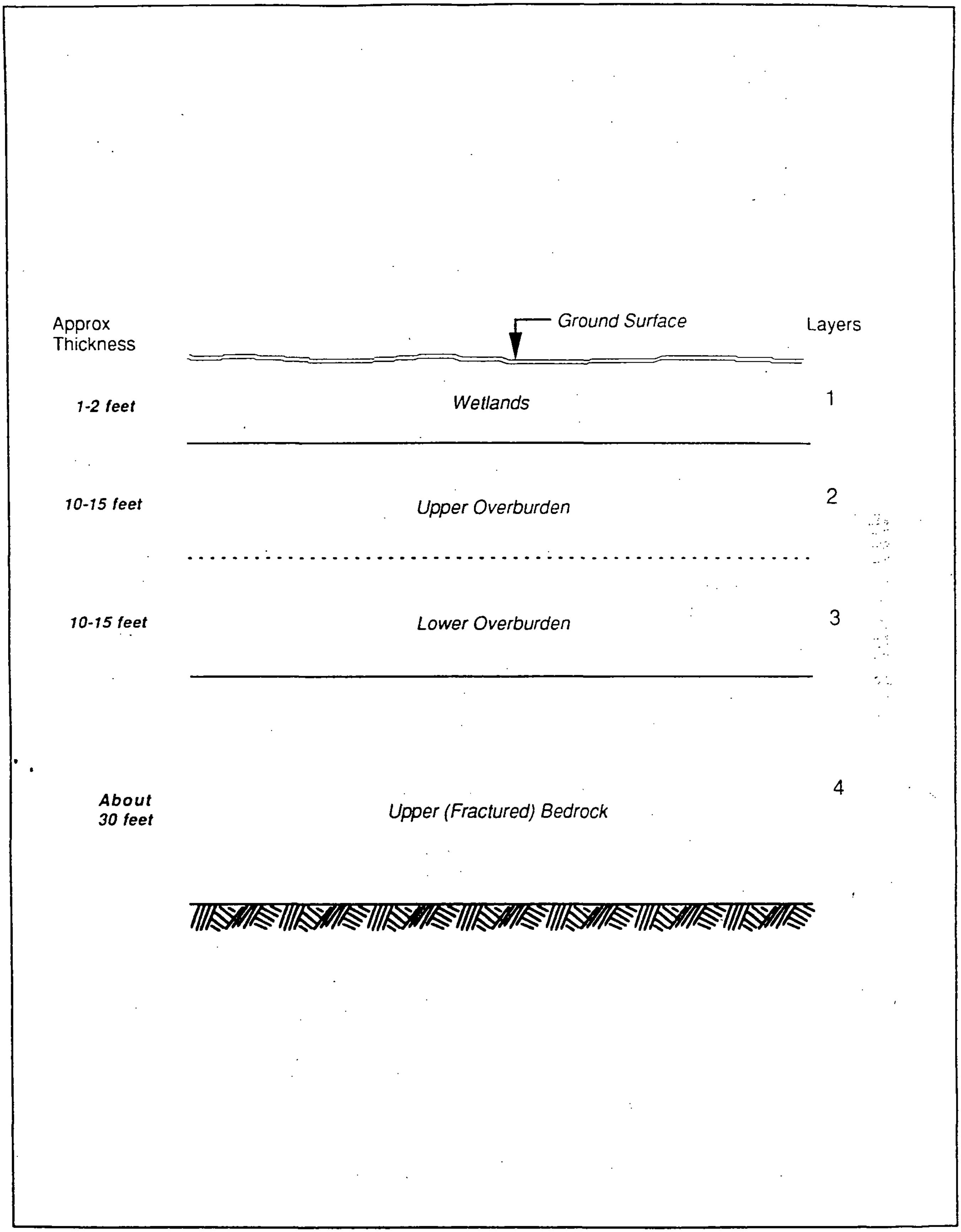


Figure 3-2 Model Layers

Table 3-1

Summary of Initial Groundwater Model Parameters

Parameter or Condition	Values
Model grid resolution	25 ft in extraction areas; 50 ft elsewhere
Model layer thicknesses	1-2 ft in wetlands 10-15 ft in upper overburden 10-15 ft in lower overburden 30 ft in upper bedrock
Layer top elevations	Layer 1 is surveyed ground surface Layer 2 = Layer 1 - 2 ft Layer 3 = 0.5*(Layer 1 + Layer 4) Layer 4 = interpolated top of rock
Bottom of model	Elevation 350 ft
Horizontal hydraulic conductivities	10,000 ft/day in wetlands Variable in upper overburden (1-100 ft/day) Variable in lower overburden (1-100 ft/day) Variable in upper bedrock (.001-1 ft/day)
Vertical conductivities	10:1 anisotropy everywhere
Porosity	0.35 in overburden (Freeze and Cherry, 1979) 0.05 in bedrock (Freeze and Cherry, 1979)
Storage coefficients	0.005 in overburden (mean of site values) Model also uses primary storage coefficient of 0.35 for sand and silt deposits based on drainable porosity. 0.0075 in bedrock (mean of site values)
Boundary conditions	Fixed heads along S and N No flow along E and W (except small part of W boundary)
Fixed heads	Interpolated from 12/91 data
Initial heads	Interpolated from 12/91 data
Precipitation	2.97 in/month (0.008 ft/day) (Average of Woonsocket and North Foster weather station data)
Potential evapotranspiration (PET)	1.21 in/month (0.003 ft/day) (Blaney-Criddle for 41°N and temperatures at North Foster. Adjusted for limited pan evaporation at Kingston Station)
Extinction Depth	5 feet

members of the modeling team (Ebasco, USGS and USACE-NED) established concurrence with the parameters presented in Table 3-1.

3.3 Model Calibration

Using the conceptual model described above as a starting point, three initial simulations were performed to refine the hydraulic conductivity parameter distribution. Modifications were made to incorporate as much of the existing information as possible.

The major adjustments during the calibration were made to the hydraulic conductivity and precipitation parameters. A zone of higher conductivity was specified in the overburden connecting the observed higher values in the northern and southern parts of the model, consistent with the view of overburden as valley fill material bounded by the hills to the east and west. Hydraulic conductivities were also lowered in the overburden layers covering the hills in the east and west portions of the model, consistent with the view of overburden as a thin cover draped on the surrounding hills.

The precipitation was lowered from 0.008 ft/day to 0.004 ft/day in the hillside areas and to 0 ft/day in the lower, wet areas. This was done in part to decrease the volume of water the model had to discharge. In addition, the hillsides required water to maintain their heads, whereas the lower (wetland) areas appear to be discharge zones, where water subsequently runs off.

Using the PCG2 solver, the model was calibrated with the following tolerances:

Head Change - 70 iterations (outer iteration) = 0.01 feet
Residual Criterion - 5 iterations (inner iteration) = 800 ft^3/day
Total Mass Balance Error = < 1%.

Table 3-2 gives the final calibrated model parameters and other properties. The calibrated model was presented to the modeling team at a meeting on November 22, 1993 and concurrence was reached to proceed with the simulations using the calibrated model. Subsequent groundwater model runs are made using these parameters.

A linear uncertainty analysis was performed on the calibrated model by individually changing a number of input values and running the model. The results indicate that the model is comparatively sensitive to changes in the overburden hydraulic conductivity on the uplands, the recharge, and the magnitude of the evapotranspiration, and least sensitive to changes in the bedrock hydraulic conductivities and the extinction depth for evapotranspiration. Therefore, the "more sensitive" calibrated values can change very little without causing a relatively large change in the result.

The most sensitive hydraulic conductivity in the lowland areas is the overburden material in the northern part of the site. Increasing the hydraulic conductivity in the central part of the overburden across the lowlands in the vicinity of OW-56, was somewhat less sensitive. In this simulation, the heads in nearby wells dropped by only 0.1 feet.

A more detailed discussion can be found in the Groundwater Model Report in Appendix D.

3.4 Model Verification

The model was verified using groundwater heads for the verification period 23 September 1993. Heads were lower (especially at the higher elevation wells OW-95 and OW-96) than during the calibration period

Table 3-2

Summary of Final Calibrated Groundwater Model Parameters

Parameter or Condition	Values
Model grid resolution	25 ft in extraction areas; 50 ft elsewhere
Model layer thicknesses	1.5 ft in wetlands 2-15 ft in upper overburden 2-15 ft in lower overburden 5-30 ft in upper bedrock
Layer top elevations	Layer 1 is surveyed ground surface Layer 2 = Layer 1 - 2 ft Layer 3 = 0.5*(Layer 1 + Layer 4) Layer 4 = interpolated top of rock
Bottom of model	Contoured bottom of 'fractured' bedrock
Horizontal hydraulic conductivities (see Figures 3-1 to 3-3)	1,000 ft/day in wetlands Variable in upper overburden (1-100 ft/day) Variable in lower overburden (1-100 ft/day) Variable in upper bedrock (.001-1 ft/day)
Vertical conductivities	10:1 horizontal to vertical anisotropy, except in tighter materials < 1 ft/day (Freeze and Cherry, 1979)
Porosity	0.35 in overburden (Freeze and Cherry, 1979) 0.05 in bedrock (Freeze and Cherry, 1979)
Storage coefficients	0.005 in overburden (mean of site values) Model also uses primary storage coefficient of 0.35 for sand and silt deposits based on drainable porosity. 0.0075 in bedrock (mean of site values)
Boundary conditions	Fixed heads along S and N and river No flow along E and W
Fixed and initial heads	Interpolated from 12/91 data
Precipitation	0.004 ft/day in uplands 0.0 ft/day in lowlands
Potential evapotranspiration (PET)	1.21 in/month (0.003 ft/day) (Blaney-Criddle for 41°N and temperatures at North Foster. Adjusted for limited pan evaporation at Kingston Station)
Extinction depth	5 feet

of December 1991. However, the rainfall for the preceding month was higher than for the calibration period. Therefore, it was hypothesized that the head response in the hillside areas (primarily the tighter bedrock) must be governed by a longer integration period and a scaling factor was established for precipitation input.

Using the scaled precipitation values, the model responds appropriately in the lowland areas, and most heads are reduced by about one foot. The simulated results agreed closely (within 0.15 feet) with wetland piezometer observations except at PZ93-1 where the observed head appears to be anomalously low. However, the model did not reproduce the dramatic 4-7 feet decline in heads between December 1991 and September 1993 at wells OW-95 (O and R) and OW-96 (O and R). Heads were lowered by 2 to 4 feet at these wells. Given the paucity of hydraulic data for the uplands area, and the fact that these wells are located far enough away from the extraction area that they should not influence drawdown configurations, the model was considered verified. The calibrated model was then utilized for predictive simulations. These simulations are described briefly below and in more detail in the Groundwater Modeling Report in Appendix D.

3.5 Model Simulations

3.5.1 MODPATH Simulation

MODPATH was used to assess the reasonableness of WCC's 1000 ppb TVOC isocons being derived exclusively from the NDA and SDA, by observing the coincidence of particle flow paths with the isocons. Based on the final model calibration and using the resultant heads from the MODFLOW calibration simulation, particles were defined in the SDA and NDA and MODPATH simulations were run. The two simulations included particles released in (1) the overburden, and (2) the bedrock (Figures 3-3 and 3-4). Particles tended to rise into the upper layers and corroborated the upward vertical hydraulic gradients observed in previous site data.

Based on the particle track simulations, performed by Ebasco, the initial extraction areas for overburden and bedrock wells were revised by USACE (via facsimile on 6 December 1993) to the new limits as shown in Figures 3-5 and 3-6, respectively. These areas represent the limits of groundwater capture to be modeled for the extraction well system configuration. For purposes of this discussion and subsequent groundwater modeling efforts, this area will still be referred to as the initial extraction area.

3.5.2 Extraction Well Configuration Simulations

Within the extraction area, potential extraction well locations at the site are limited to the roads and paths. Wells are not to be located within the tires or in the NDA or SDA so as not to interfere with site cleanup operations or Source Control activities. WCC (1992b) proposed using a number of wells, each one pumping at 10 gpm and with individual capture zones of 260 feet in diameter. An initial well configuration for this model was established (Figure 3-7), using WCC's 130 foot radius as a starting point for estimated well separation. Hydrogeological conditions that could affect drawdowns at the well locations are accounted for by the variable layer thicknesses and parameters specified at each node.

To simulate this initial well configuration, an "annual average" stress condition was created by averaging the precipitation and evapotranspiration stresses from the calibration and verification periods as winter and summer respectively. These values are listed in Table 3-3.

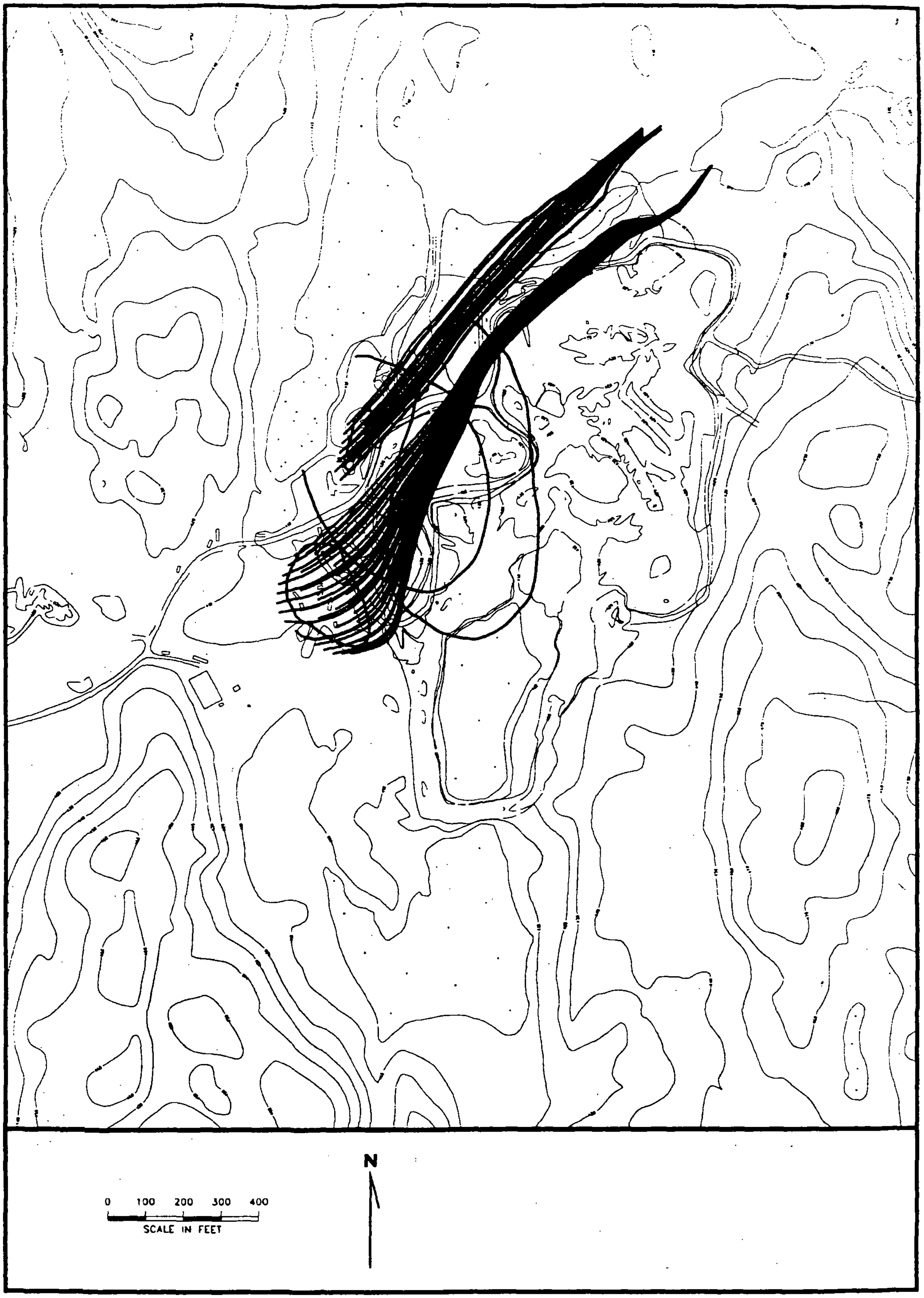


Figure 3-3 Particle Tracks in Overburden for Calibrated Groundwater Model

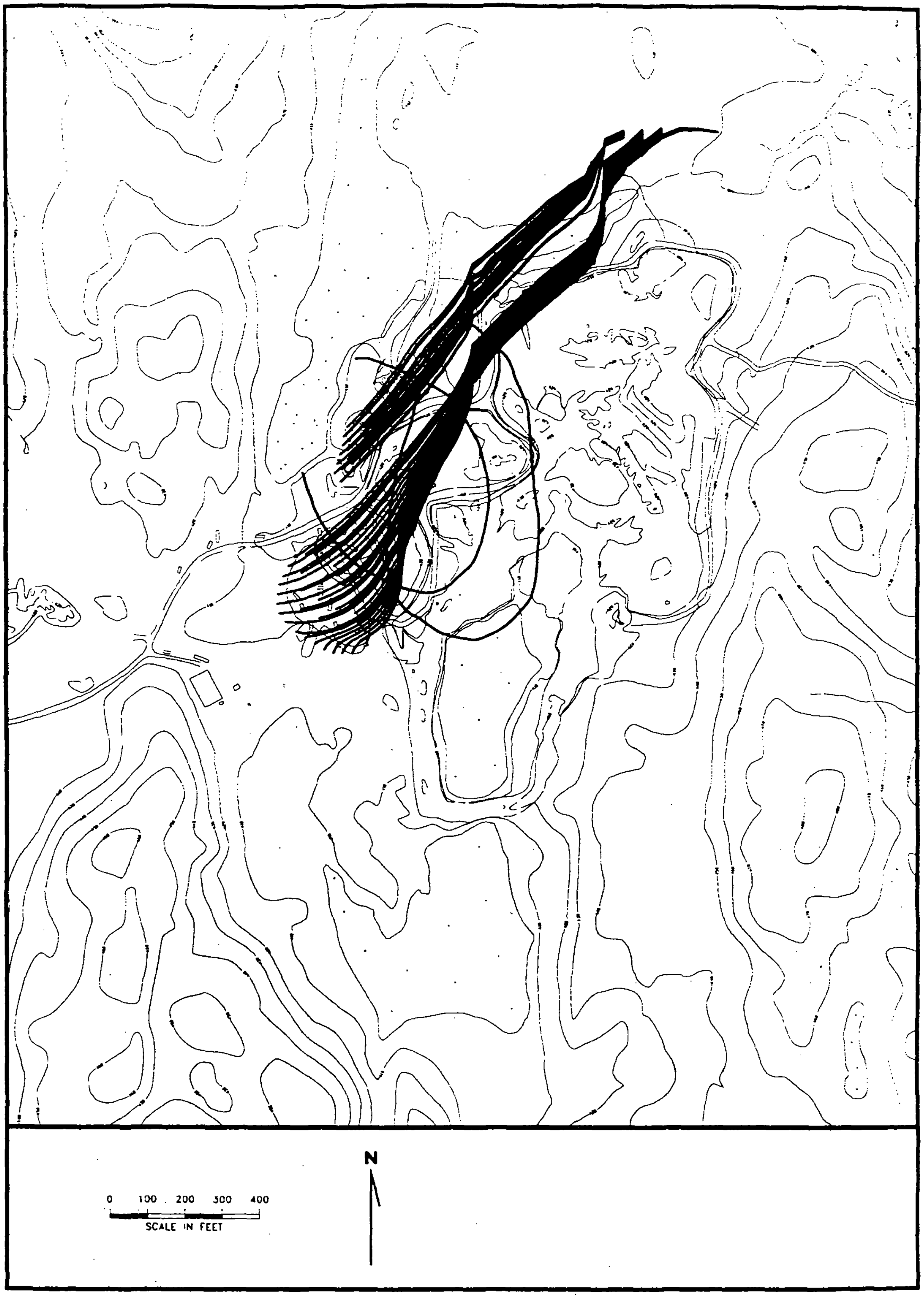
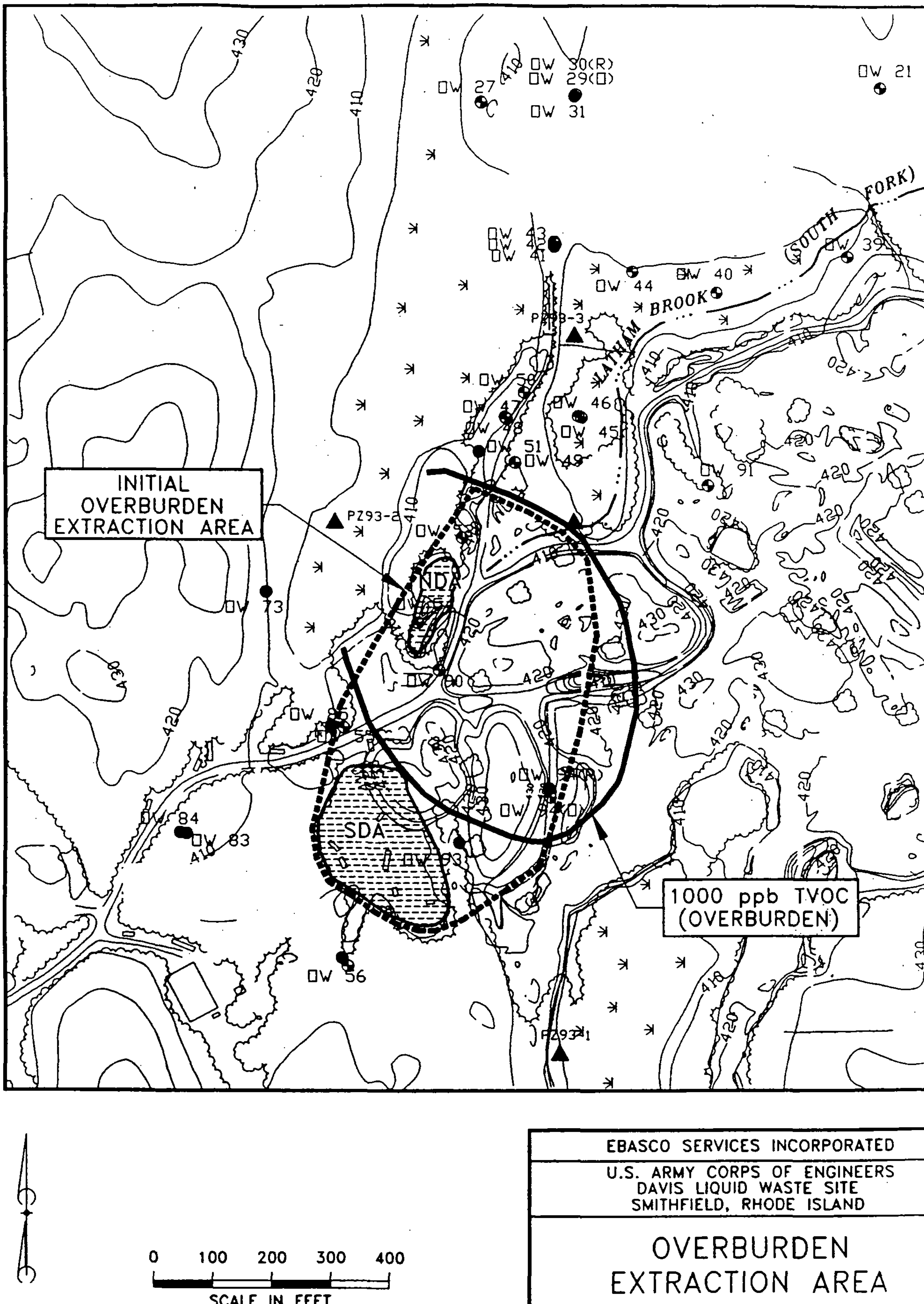


Figure 3-4 Particle Tracks in Bedrock for Calibrated Groundwater Model



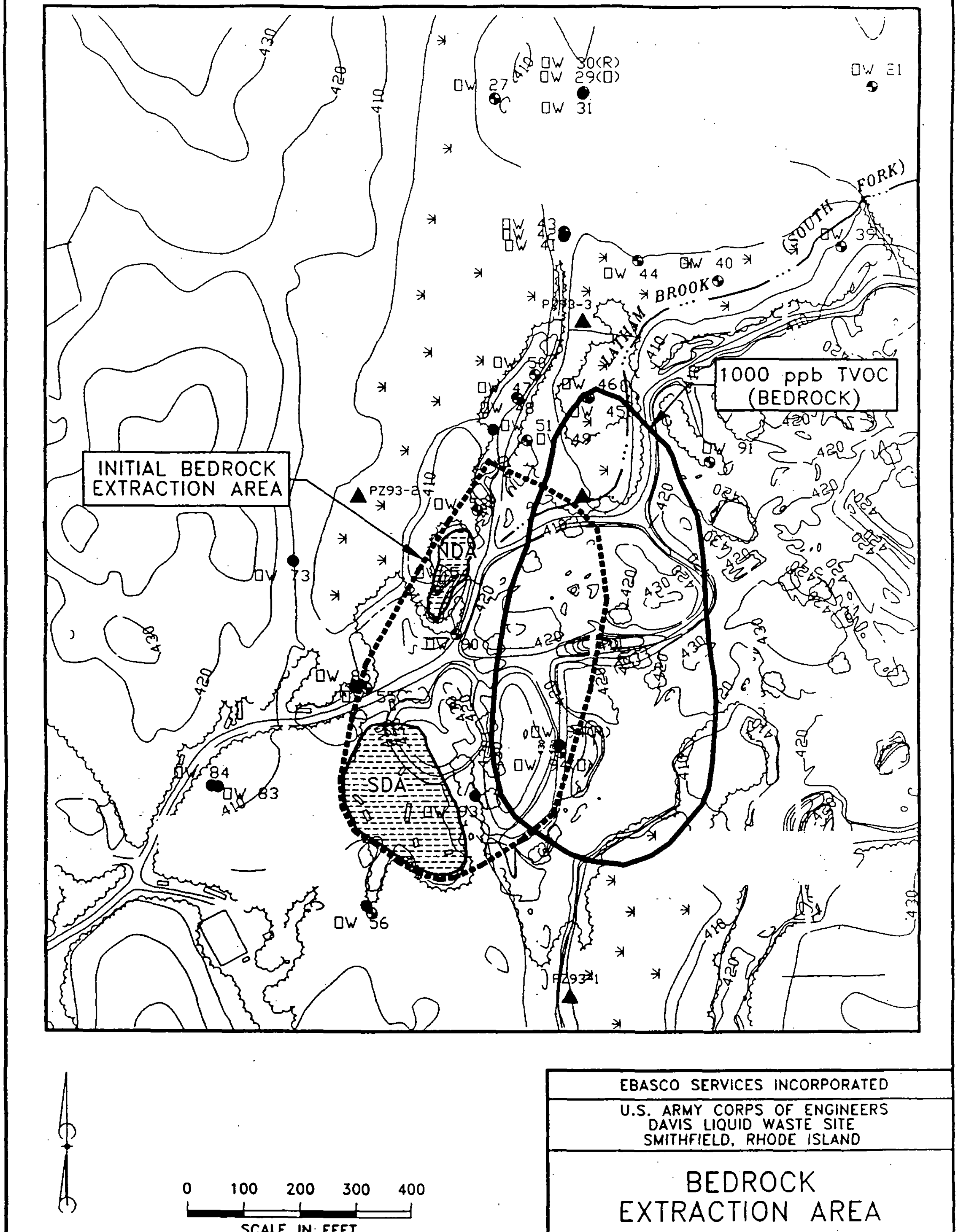
EBASCO SERVICES INCORPORATED

U.S. ARMY CORPS OF ENGINEERS
DAVIS LIQUID WASTE SITE
SMITHFIELD, RHODE ISLAND

OVERBURDEN EXTRACTION AREA

CONTRACT NO. DACW33-91-D-0005 | FIGURE 3-5

DWG:DS012



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BEDROCK EXTRACTION AREA

CONTRACT NO. DACW33-91-D-0005 | FIGURE 3-6

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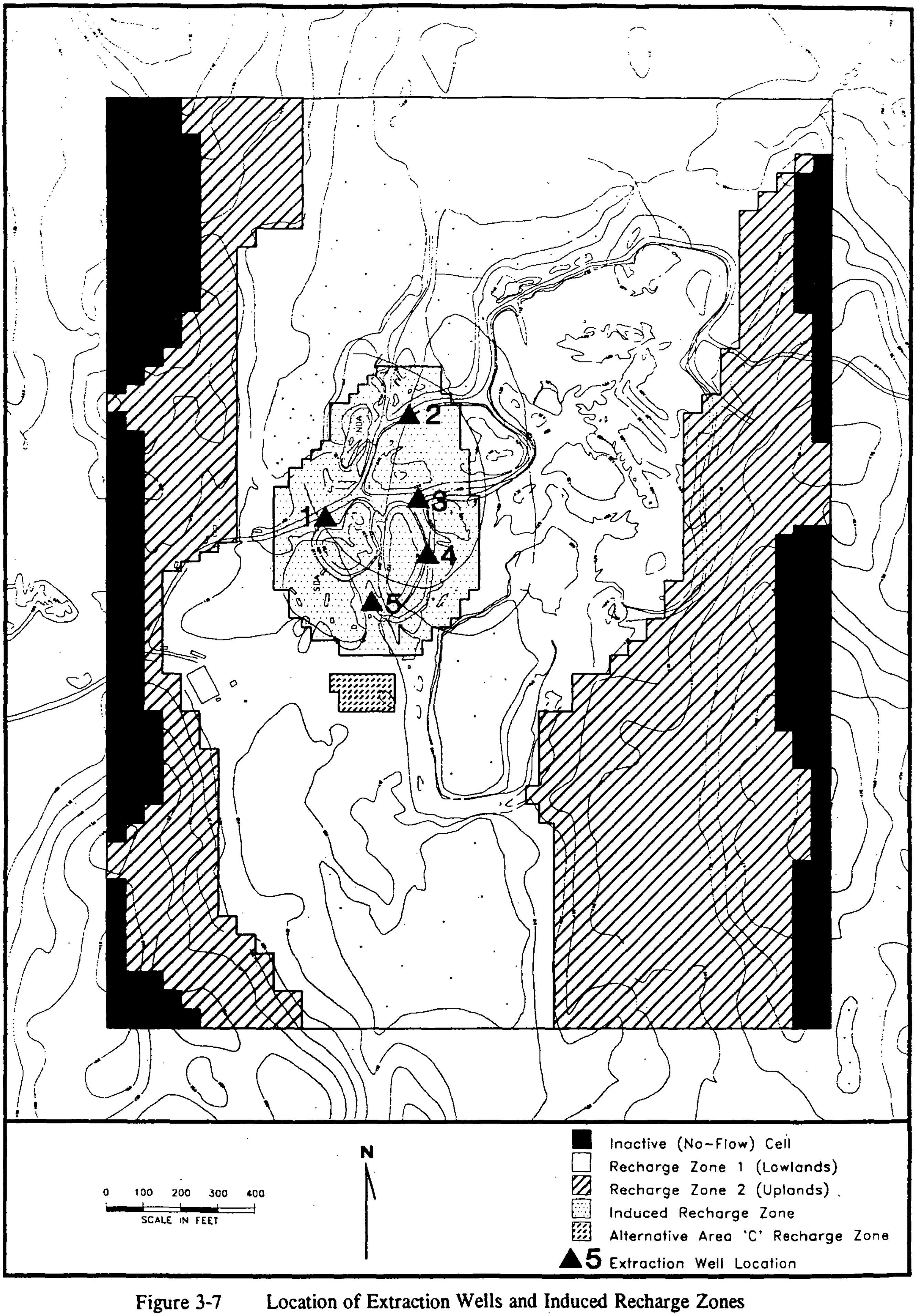

Figure 3-7 Location of Extraction Wells and Induced Recharge Zones

Table 3-3

Summary of Stresses Used for Various Sensitivity Simulations

Stress Parameter	Winter Conditions	Summer Conditions	Annual Average Conditions
Evapotranspiration	0.003 ft/d	0.006 ft/d	0.0045 ft/d
Upland Precipitation	0.004 ft/d	0.0028 ft/d	0.0035 ft/d
Lowland Precipitation	0 ft/d	0 ft/d	0 ft/d

The model had been calibrated by reducing net precipitation in the lowlands to 0.0 ft/day. However, it is reasonable to assume that during extraction, the drawdown around the wells will induce recharge from the surface, thereby reducing discharge of groundwater to surface waterbodies. Therefore, a recharge zone was established in the model that encompassed the estimated capture zone (Figure 3-7). This zone was assigned a maximum possible recharge equal to the average annual rainfall of 45 inches (0.01 ft/day). This is a conservative assumption to establish the upper limits of the expected pumping range. A final mass balance incorporating this recharge was calculated in the Groundwater Modeling Report in Appendix D, and shown to have a less than one percent discrepancy.

The following sections discuss the various simulations and sensitivity analyses performed to derive a proposed extraction flow rate and pumping well configuration. These simulations are summarized in Table 3-4 and examine variations in well locations, pumping rates, precipitation/evapotranspiration stresses, aquifer parameters and wetland influences.

3.5.2.1 Pumping Overburden Wells Only

The extraction well configuration, shown in Figure 3-7, was initially simulated with each well pumping at 10 gpm. The results indicated drawdowns greater than two feet in much of the northern lowlands.

A second simulation was performed in which each of the wells was pumped at 1,000 ft^3/day (5.2 gpm). Water would be captured in both the overburden and the bedrock by pumping the overburden alone. Although the simulation indicated that hydraulic control would be maintained in the bedrock, the majority of flow would be contributed by the overburden, thus suggesting that bedrock aquifer restoration would be prolonged. Additionally, a greater horizontal to vertical anisotropy ratio than assumed (Table 3-1) would accentuate the division of flow. Likewise, the converse is true - a lower ratio would lessen the effect.

The second simulation results also indicated drawdowns in the western and northern wetlands ranging from approximately 0.25 to 1.5 feet. In the southern wetlands, the results indicated drawdowns from 0.25 feet to just over 2 feet. Overall, the greatest wetland drawdown occur in those areas proximate to pump wells.

Table 3-4
Summary of Groundwater Modeling Simulations

Model Runs	Recharge Stresses				Pumping Stresses					Discharge Stresses				Comments
	Upland (ft/day)	Lowland (ft/day)	Extr. Area (ft/day)	Evp/Trns (ft/day)	Strata	Total (gpm)	Total (ft ^ 3/d)	# of Wells	Well Rate (ft ^ 3/d)	Total (gpm)	Total (ft ^ 3/d)	Area (ft ^ 2)	Load Rate (ft/d)	
Calibration	0.0040	0.0000	0.0000	0.0030		No wells pumping				No discharge				December 1991 data
Validation	0.0028	0.0000	0.0000	0.0060		No wells pumping				No discharge				September 1993 data
Average Annual Overburden Only	0.0030	0.0000	0.0100	0.0045	OB	26.0	5000	5	1000	Discharge to stream				Overburden wells only; drawdown in bedrock
					BR	0.0	0	0	0					
Average Annual Bedrock Only	0.0030	0.0000	0.0100	0.0045	OB	0.0	0	0	0	Discharge to stream				Bedrock wells only; tight drawdown contours at wells; poor mass bal.
					BR	26.0	5000	5	1000					
Average Annual Overburden/Bedrock	0.0030	0.0000	0.0100	0.0045	OB	18.2	3500	5	700	Discharge to stream				Captured all particles from target area edge
					BR	7.8	1500	5	300					
Winter – Sensitivity	0.0040	0.0000	0.0130	0.0030	OB	18.2	3500	5	700	Discharge to stream				Captured most particles from target area edge; lost some on north edge
					BR	7.8	1500	5	300					
Summer – Sensitivity	0.0028	0.0000	0.0080	0.0060	OB	18.2	3500	5	700	Discharge to stream				Captured particles but drawdown in the wetlands
					BR	7.8	1500	5	300					
Summer – Sensitivity (Reduced Pumping)	0.0028	0.0000	0.0080	0.0060	OB	10.4	2000	5	400	Discharge to stream				Less drawdown in wetlands; but target area not captured
					BR	5.2	1000	5	200					
Winter – Sensitivity Expanded Network	0.0040	0.0000	0.0130	0.0030	OB	4.7	900	3	300	Discharge to stream				Captured all particles from target area edge
					OB	10.9	2100	3	700					
					BR	1.6	300	3	100					
					BR	4.7	900	3	300					
Summer – Sensitivity Expanded Network	0.0028	0.0000	0.0080	0.0060	OB	4.7	900	3	300	Discharge to stream				Captured all particles from target area edge; but drawdown in wetlands
					OB	10.9	2100	3	700					
					BR	1.6	300	3	100					
					BR	4.7	900	3	300					
Summer – Sensitivity Expanded Network (Reduced Pumping)	0.0028	0.0000	0.0080	0.0060	OB	4.7	900	3	300	Discharge to stream				Captured all particles from target area edge; minimal drawdown in wetlands
					OB	7.8	1500	3	500					
					BR	1.6	300	3	100					
					BR	3.1	600	3	200					
Average Annual Expanded Network	0.0030	0.0000	0.0100	0.0045	OB	4.7	900	3	300	Discharge to stream				Captured all particles from target area edge
					OB	10.9	2100	3	700					
					BR	1.6	300	3	100					
					BR	4.7	900	3	300					
Winter (K=100 ft/day) Expanded Network	0.0040	0.0000	0.0130	0.0030	OB	4.7	900	3	300	Discharge to stream				Majority of particles not captured at the northern edges
					OB	10.9	2100	3	700					
					BR	1.6	300	3	100					
					BR	4.7	900	3	300					
Wetland – Sensitivity Average Annual (Fix Wetland Nodes)	0.0030	0.0000	0.0100	0.0045	OB	4.7	900	3	300	Discharge to stream				Captured all particles from target area edge; mass balance shows minimal induced recharge
					OB	10.9	2100	3	700					
					BR	1.6	300	3	100					
					BR	4.7	900	3	300					
Transient Run Expanded Network	0.0030	0.0000	0.0100	0.0045	OB	4.7	900	3	300	Discharge to stream				Most significant drawdowns occur in first two months; levels off in four to six months
					OB	10.9	2100	3	700					
					BR	1.6	300	3	100					
					BR	4.7	900	3	300					
Average Annual GW Discharge	0.0030	0.0000	0.0100	0.0045	OB	18.2	3500	5	700	26.2	5037.5	16250	0.31	Generates mound at ground surf. at leaching area (k=1 ft/d)
					BR	7.8	1500	5	300					
Average Annual GW Discharge	0.0030	0.0000	0.0100	0.0045	OB	18.2	3500	5	700	8.4	1625	16250	0.1	Flow is 33% of pumping; breakout at discharge point and along slope
					BR	7.8	1500	5	300					
Transient Run GW Discharge	0.0030	0.0000	0.0100	0.0045	OB	18.2	3500	5	700	8.4	1625	16250	0.1	Breakout in discharge area occurs approximately 45 days after startup
					BR	7.8	1500	5	300					

3.5.2.2 Pumping Bedrock Wells Only

Using the same extraction well configuration, shown in Figure 3-7, the wells were pumped in the bedrock at 1,000 ft^3/day (5.2 gpm) each. The results indicate that pumping effectively from the bedrock requires significantly more wells pumping at lower rates throughout the entire extraction area. This is not practical at the Davis Liquid Waste site due to the limited locations available for well placement. It should also be noted that the model treats the bedrock as a uniform porous media and does not account for site heterogeneities that may limit flow from particular areas being modeled.

3.5.2.3 Combined Overburden and Bedrock Pumping

Based on the above individual simulations of pumping in only one stratum:

1. Five overburden wells, each pumping at 1,000 ft^3/day (5.2 gpm), are sufficient for capture of groundwater within the extraction area.

2. Wells pumping only the overburden are sufficient for capture, but more than likely will miss some of the bedrock contamination based on actual site heterogeneities. The model represents the bedrock as a uniform porous media, whereas in reality, the bedrock probably is comprised of discrete, interconnected fracture zones that can behave as separate flow systems.

3. Wells pumping only the bedrock are not sufficient for complete capture of groundwater within the extraction area.

Extraction from both the overburden and the bedrock was simulated using pairs of wells in the configuration shown in Figure 3-7. For each pair, the overburden well was pumped at 700 ft^3/day (3.7 gpm) and the bedrock well at 300 ft^3/day (1.6 gpm). MODPATH simulations show that particles released in the overburden and bedrock, respectively, around the perimeter of the extraction area are captured by this well configuration under annual average conditions at a total pumping rate of 26 gpm.

Because hydrologic conditions vary at the site throughout a typical year, sensitivity analysis simulations were performed. These simulations were also used to evaluate the range of uncertainty for selected aquifer properties and in the representation of the wetlands in the model. During these sensitivity simulations, it was noted that under "winter" conditions of increased precipitation and lowered evapotranspiration, the present five well pair network pumping at 26 gpm did not predict complete capture of the extraction area. Therefore, the extraction well network was revised to include eight well pairs (Figure 3-8) to provide a more complete capture of the extraction area under varying hydrologic conditions. The first eight well pairs simulation used the same individual well pumping rates (3.7 gpm in overburden, 1.6 gpm in bedrock) as those in the five well pairs configuration. With further simulations, it was noted that flows at individual wells within the eight pairs could be reduced and still maintain hydraulic control of the extraction area. Additionally, under average annual conditions, the model predicts hydraulic control can be maintained using only six of the eight well pairs (Figure 3-9) pumping at a total flow rate of 22.5 gpm (Table 3-5). The sensitivity simulations also suggest that seasonal adjustments to extraction well pumping may be a consideration in order to balance complete capture with wetland drawdowns. Model simulations suggest summer pumping rates (Table 3-5) may be limited to approximately 17 gpm to prevent significant drawdowns (greater than 2 feet) in the wetland areas. However, because actual field conditions could be different than represented in the model, particularly with respect to potentially higher overburden conductivities and fewer interconnected bedrock fractures, a more conservative pumping configuration using eight well pairs is proposed. The two

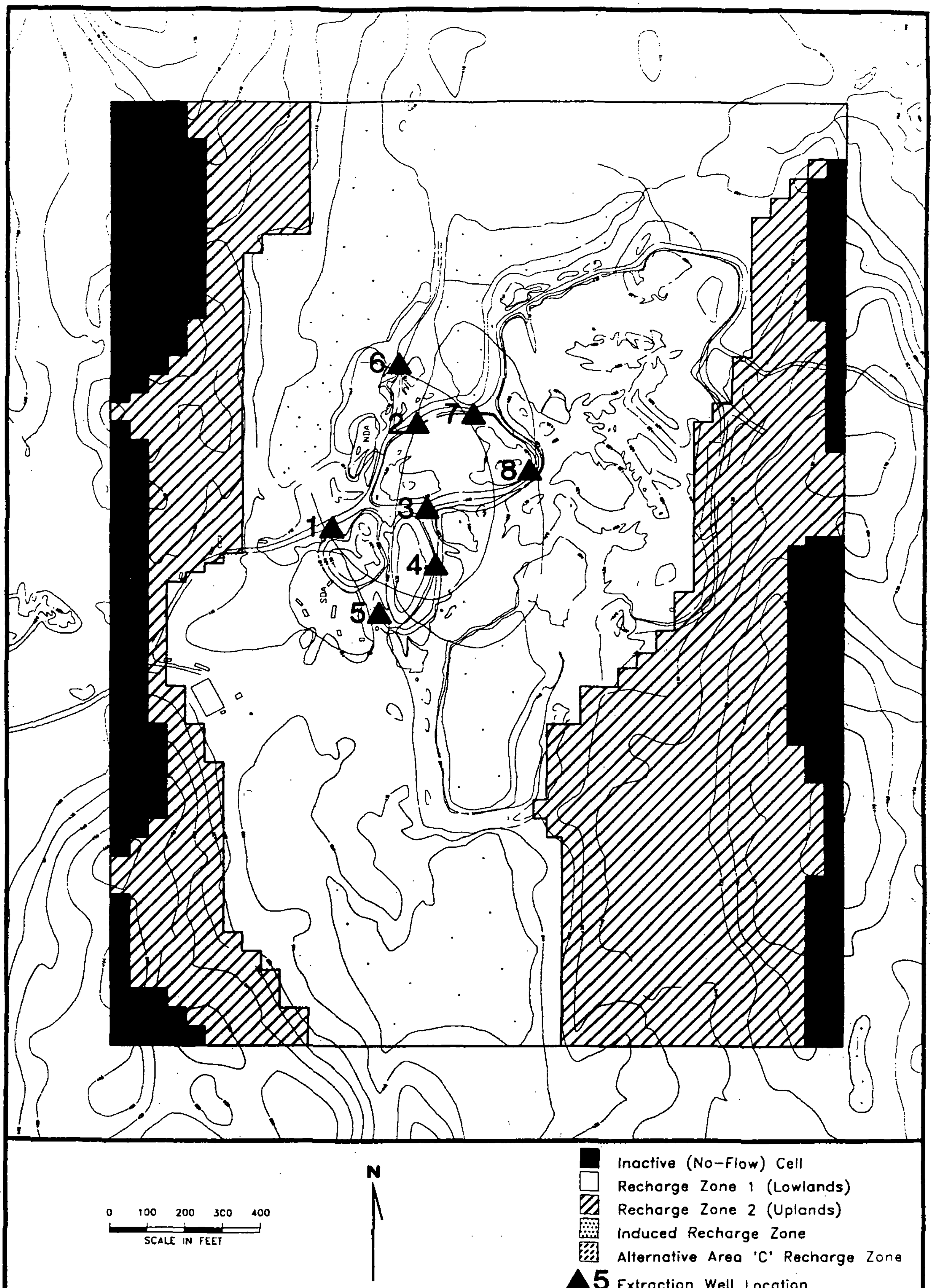

Figure 3-8 Revised Network of Extraction Wells

Table 3–5
Groundwater Model Pumping Rates Based on Model Run – 20 Dec 93

Well #	Avg. Annual (ft^3/d)	Summer (ft^3/d)	Est. Max. (ft^3/d)		Avg. Annual (gpm)	Summer (gpm)	Est. Max. (gpm)
OB–1	0	0	300		0	0	1.6
BR–1	0	0	100		0	0	0.6
OB–2	700	500	700		3.7	2.6	3.7
BR–2	300	200	300		1.6	1.1	1.6
OB–3	700	500	700		3.7	2.6	3.7
BR–3	300	200	300		1.6	1.1	1.6
OB–4	300	300	400		1.6	1.6	2.1
BR–4	100	100	200		0.6	0.6	1.1
OB–5	0	0	300		0	0	1.6
BR–5	0	0	100		0	0	0.6
OB–6	300	300	300		1.6	1.6	1.6
BR–6	100	100	100		0.6	0.6	0.6
OB–7	700	500	700		3.7	2.6	3.7
BR–7	300	200	300		1.6	1.1	1.6
OB–8	300	300	400		1.6	1.6	2.1
BR–8	100	100	200		0.6	0.6	1.1
Totals	4200	3300	5400		22.5	17.7	28.9

Overburden Well #	Avg. Annual (gpm)	Summmer (gpm)	Est. Max. (gpm)		Bedrock Well #	Avg. Annual (gpm)	Summmer (gpm)	Est. Max. (gpm)
OB–1	0	0	1.6		BR–1	0	0	0.6
OB–2	3.7	2.6	3.7		BR–2	1.6	1.1	1.6
OB–3	3.7	2.6	3.7		BR–3	1.6	1.1	1.6
OB–4	1.6	1.6	2.1		BR–4	0.6	0.6	1.1
OB–5	0	0	1.6		BR–5	0	0	0.6
OB–6	1.6	1.6	1.6		BR–6	0.6	0.6	0.6
OB–7	3.7	2.6	3.7		BR–7	1.6	1.1	1.6
OB–8	1.6	1.6	2.1		BR–8	0.6	0.6	1.1
Totals	15.9	12.6	20.1			6.6	5.1	8.8

Note: Pumping rates rounded up to nearest 0.1 gpm

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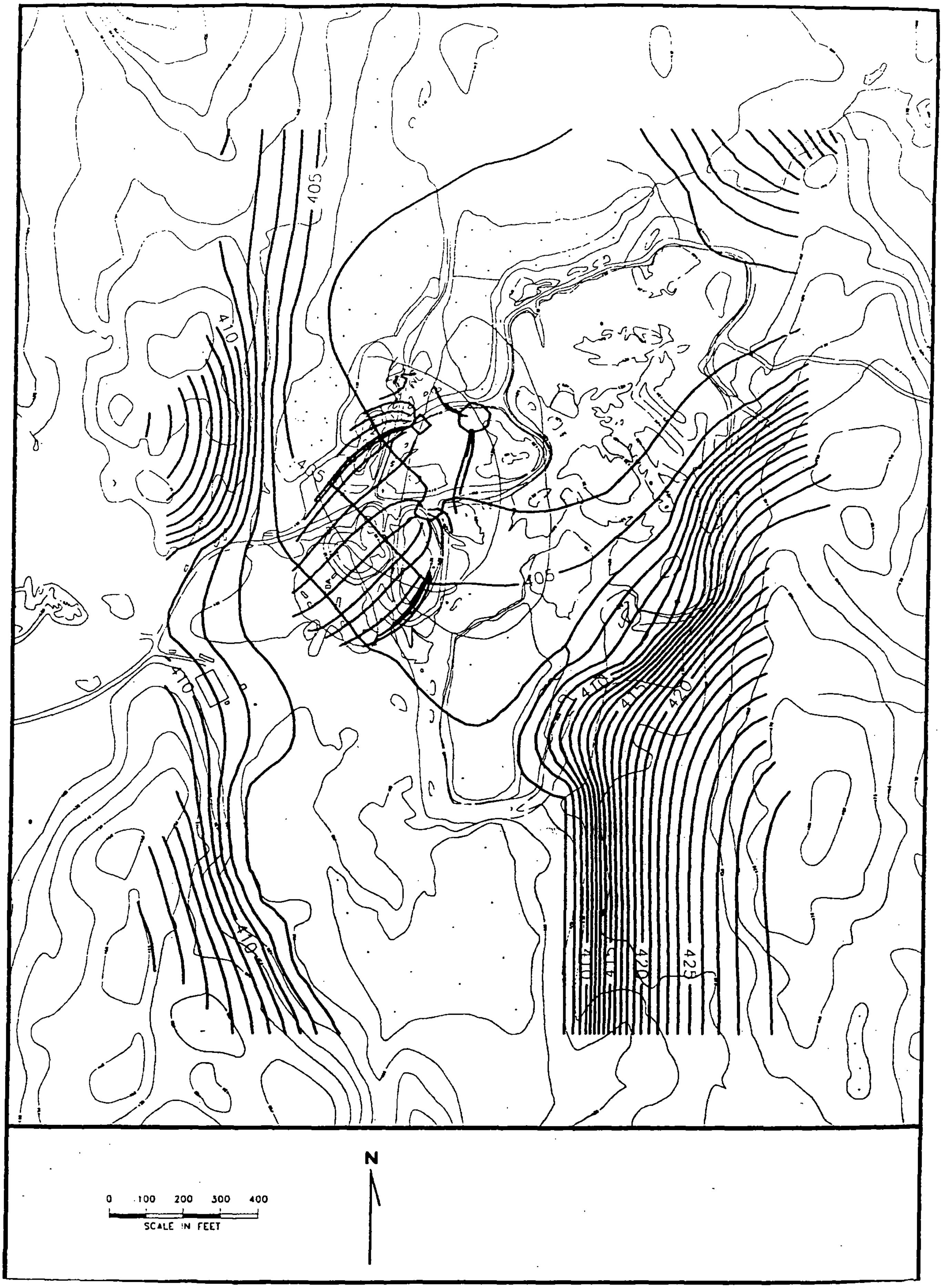


Figure 3-9 Heads and Particle Tracks In Overburden With Revised Extraction System Operating During Annual Average Conditions

additional well pairs provide flexibility to adjust to varying annual and hydrogeologic conditions, which may differ from those parameters assumed in the model.

3.5.3 Transient Analysis of Extraction Well System

The successful simulation conditions of annual average stresses and 22.5 gpm flowrate (only six well pairs pumping, see Table 3-5) were then used in a transient analysis of system performance. The purpose of this simulation was to determine the length of time required to establish the capture zone once the extraction well system was activated. Figure 3-10 shows the results at four model grid nodes within the capture zone and nearby wetlands. The node locations are depicted on Figure 3-11. The majority of drawdown inside the capture zone occurs within the first 120-140 days. After that, heads drop at a much slower rate, and may be influenced by seasonal variations in precipitation and evapotranspiration. In the nearby wetlands, the results indicate a slower drawdown response for the same time period (compared to the pumping well).

3.5.4 Extraction Well Configuration and Estimated Pumping Rate

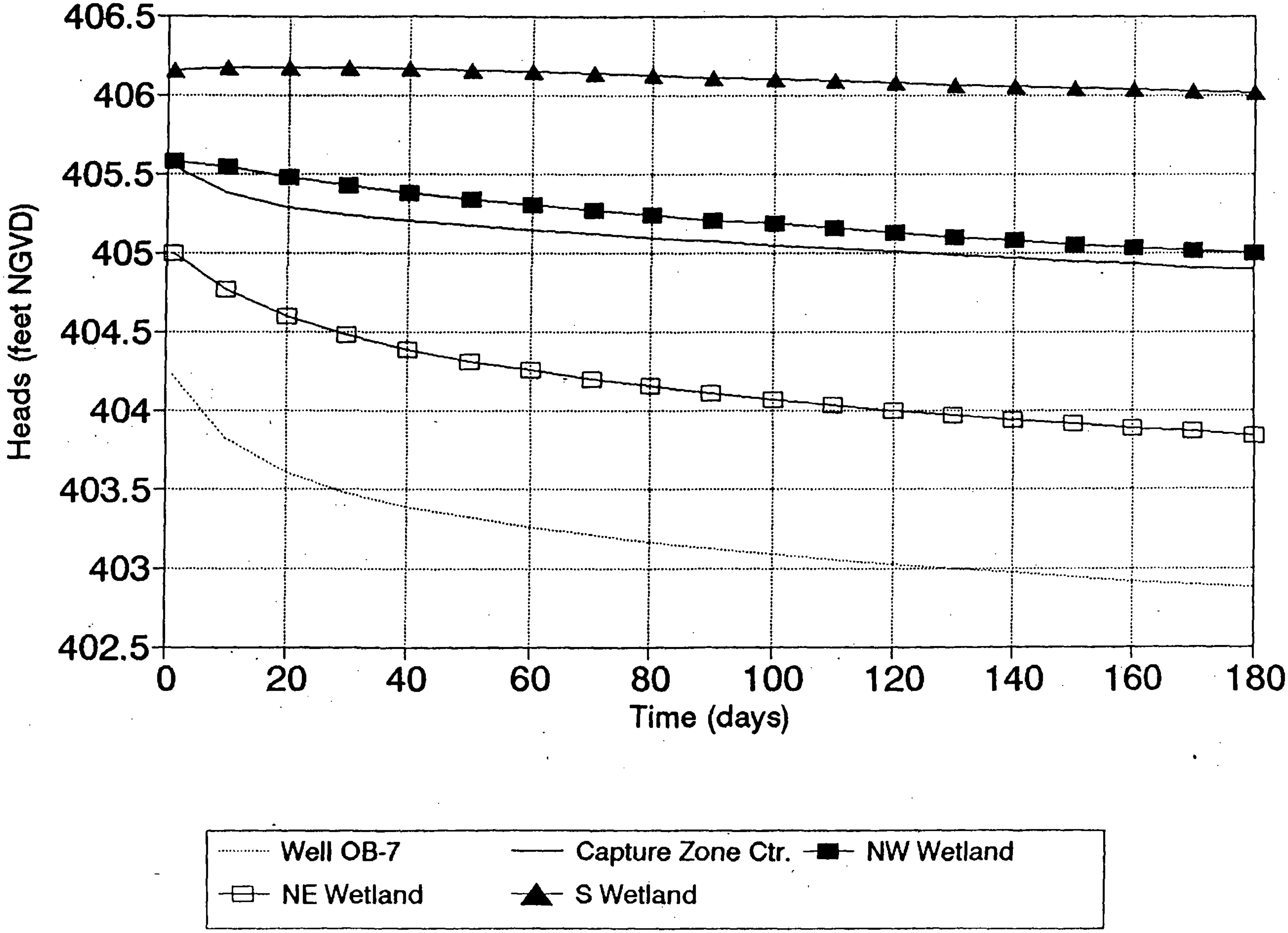
In the above simulations, a successful pumping scenario under annual average conditions was recorded with six of eight well pairs pumping at a total flow rate of 22.5 gpm. However, to account for field conditions and parameters that may be different from those in the model, a higher flow rate may be necessary. Additional bedrock wells may also be needed in the extraction configuration given that the model treats the bedrock as a uniform porous medium - a condition that may not exist at this site. Therefore, to account for potentially higher overburden hydraulic conductivities and reduced bedrock interconnected pathways, it is recommended that eight well pairs, as shown in Figure 3-8, be used as the base system design. An additional 6.4 gpm of extraction flow rate (Table 3-5) is estimated to provide a suitable safety factor to cover the model uncertainties with respect to bedrock capture and increased hydraulic conductivities. This flow rate is based on an extrapolation from other sensitivity analysis simulations; the eight well pairs configuration was not directly modeled. Thus, the estimated pumping rate is 28.9 gpm. It is suggested that initially, the eight well pair extraction configuration be installed and the system pumped at 28.9 gpm flow rate to establish the capture zones. Water levels and well drawdowns can then be measured and adjustments made to the system parameters, uniformly or on an individual well basis, to meet established system performance criteria.

3.5.5 Groundwater Discharge Simulation

All of the above model simulations assumed a surface water discharge. A model simulation was made to evaluate the effect of discharging treated effluent to groundwater in Alternate Area C. The discharge location was represented in the model as a set of recharge nodes over an area of 16,250 $ft.^2$. This grid area was an early approximation of the available space on site for a discharge gallery or leaching field structure. (The actual area available is estimated to be approximately 13,100 $ft.^2$ as shown in Figure 3-12). Such a discharge is expected to create a local groundwater mound. The feasibility of such a discharge will be based on whether the resulting groundwater mound breaks out at the surface of the discharge location, in the adjacent Southern Disposal Area or along the slope between the two locations.

Based on the established project schedule, the discharge scenario was run using one of the earlier well configurations pumping at 26.2 gpm. A recharge rate of 0.31 ft/day (26.2 gpm divided by 16,250 ft^2) was specified at these nodes. Under steady state conditions, the predicted groundwater mound was significantly higher than the ground surface and thus, did not meet discharge criteria. To approximate

Figure 3-10. Drawdown Time Histories for Revised Extraction Wells (Pumping Rate at 22.5 gpm)



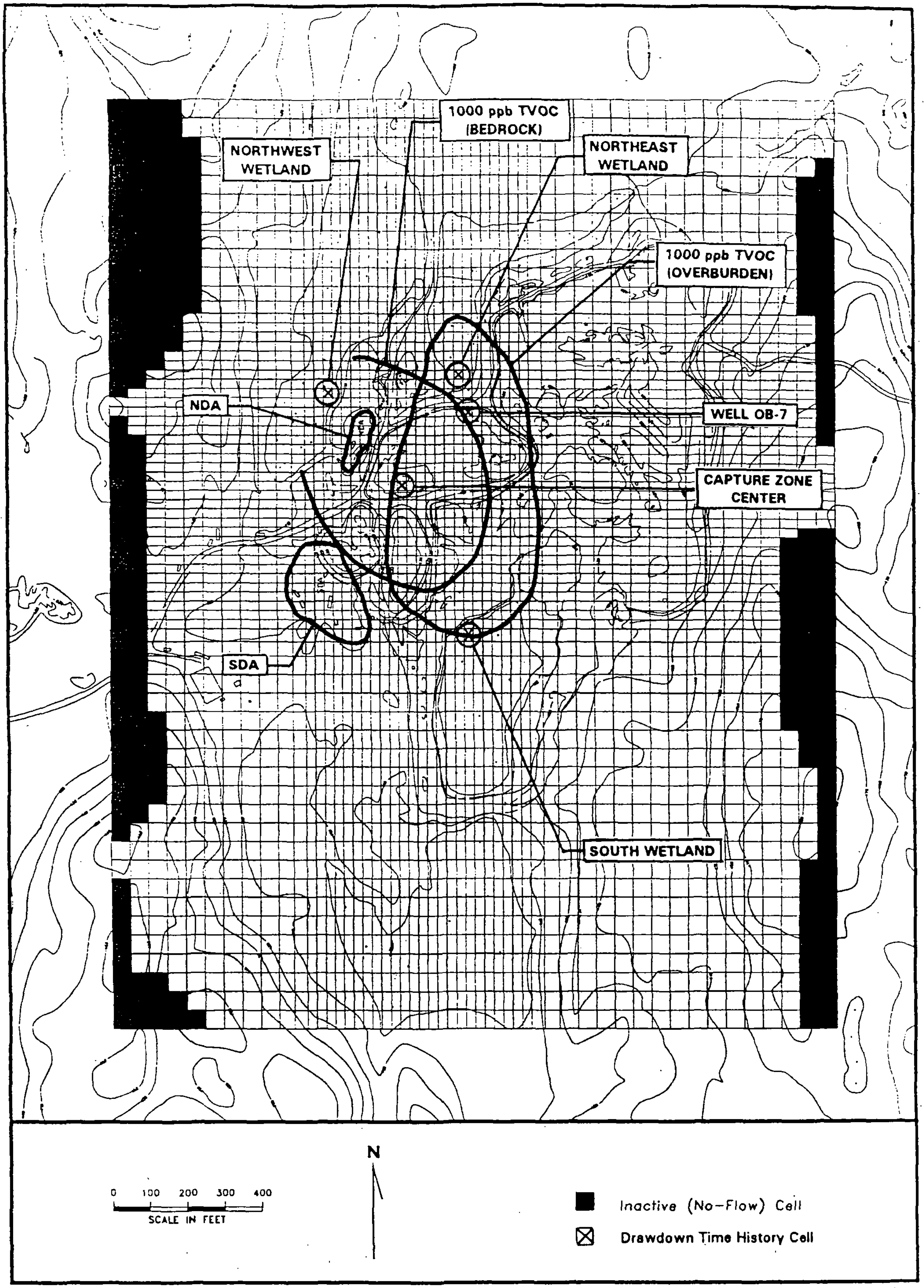
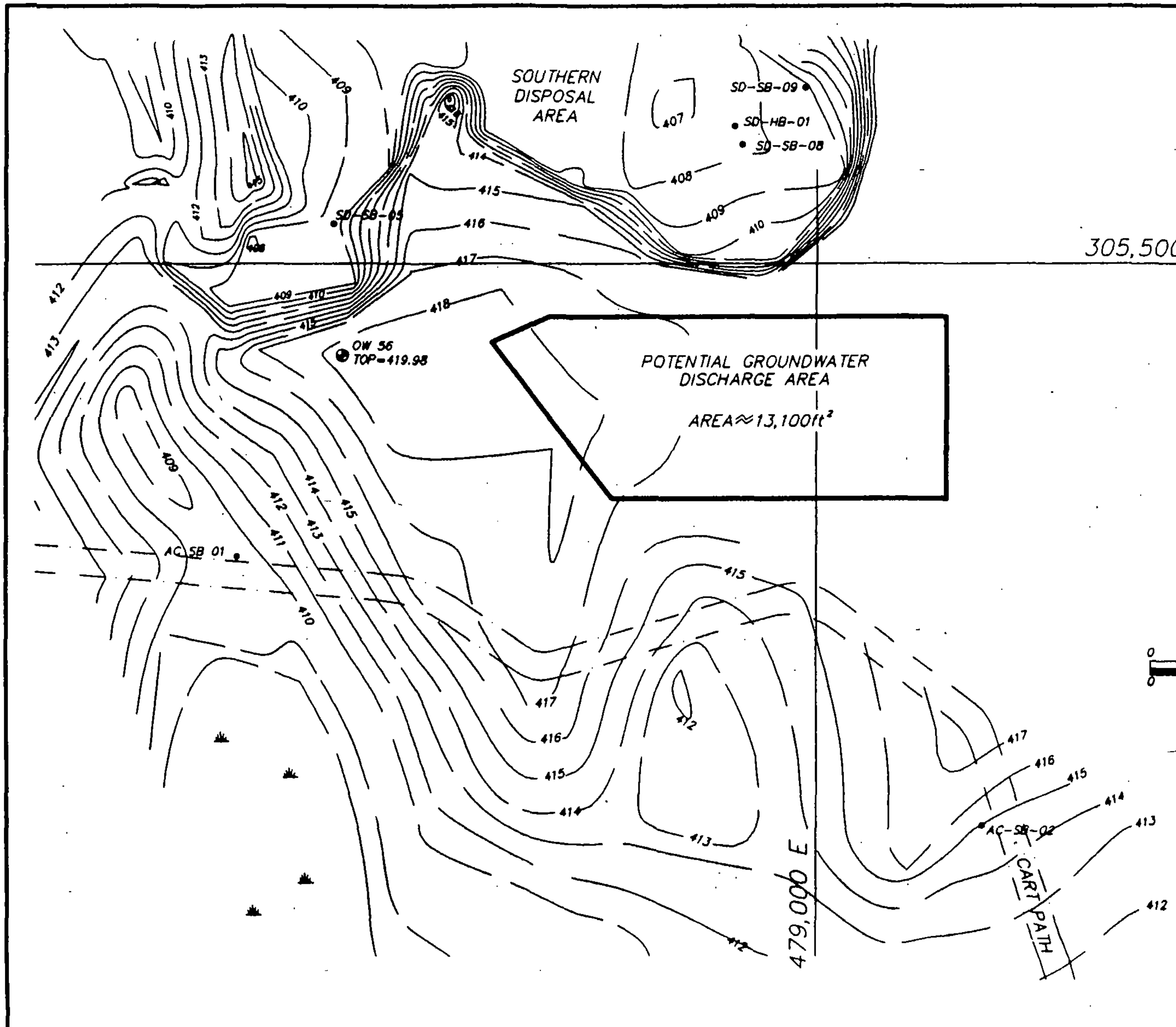


Figure 3-11. Drawdown Time History Locations



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POTENTIAL GROUNDWATER
DISCHARGE AREA
– ALTERNATE AREA C –

CONTRACT NO. DACW33-91-D-0005 | FIGURE 3-12

DWG:DS018

just how much water might be recharged at this location before breaking the ground surface, the model was rerun with a specified recharge in this area of 0.1 ft/day (approximately 8.4 gpm). The results of this loading rate cause ground breakout and fail to meet the discharge criteria.

This second case (8.4 gpm discharge) was also run as a transient simulation to determine how long the mound would take to form. Figure 3-13 shows the time histories of groundwater heads in the upper part of the overburden (1) within the discharge area; (2) at the edge of the discharge area just above the SDA; and (3) in the southern portion of the SDA. With the modeled values of hydraulic conductivity in this area (1 ft/day based on the OW56 permeability value of 0.66 ft/day), it requires a significant gradient and increased hydraulic conductivity to move the discharged effluent (approximately one-third of the extraction well pumpage) away from the discharge area. Not only does the mound intersect the ground surface (elevation 416.5 ft), but there are indications that a seepage face forms on the slope above the SDA (elevation 412 feet). The simulation also shows that the mound takes about 4-6 months to form. Therefore, based on the above simulations, a groundwater discharge to Alternate Area C would have to be less than 8.4 gpm. Because the combined discharge flow rate is currently estimated at 45 gpm (Appendix E), a groundwater discharge at Alternate Area C is not considered feasible.

3.6 Model Summary

In summary, the groundwater model developed for the Davis Liquid Waste site, was calibrated to observations from December 1991, and verified with observations from September 1993. The model has a resolution of 25 feet within the 1000 ppb TVOC isocons, and 50 feet elsewhere.

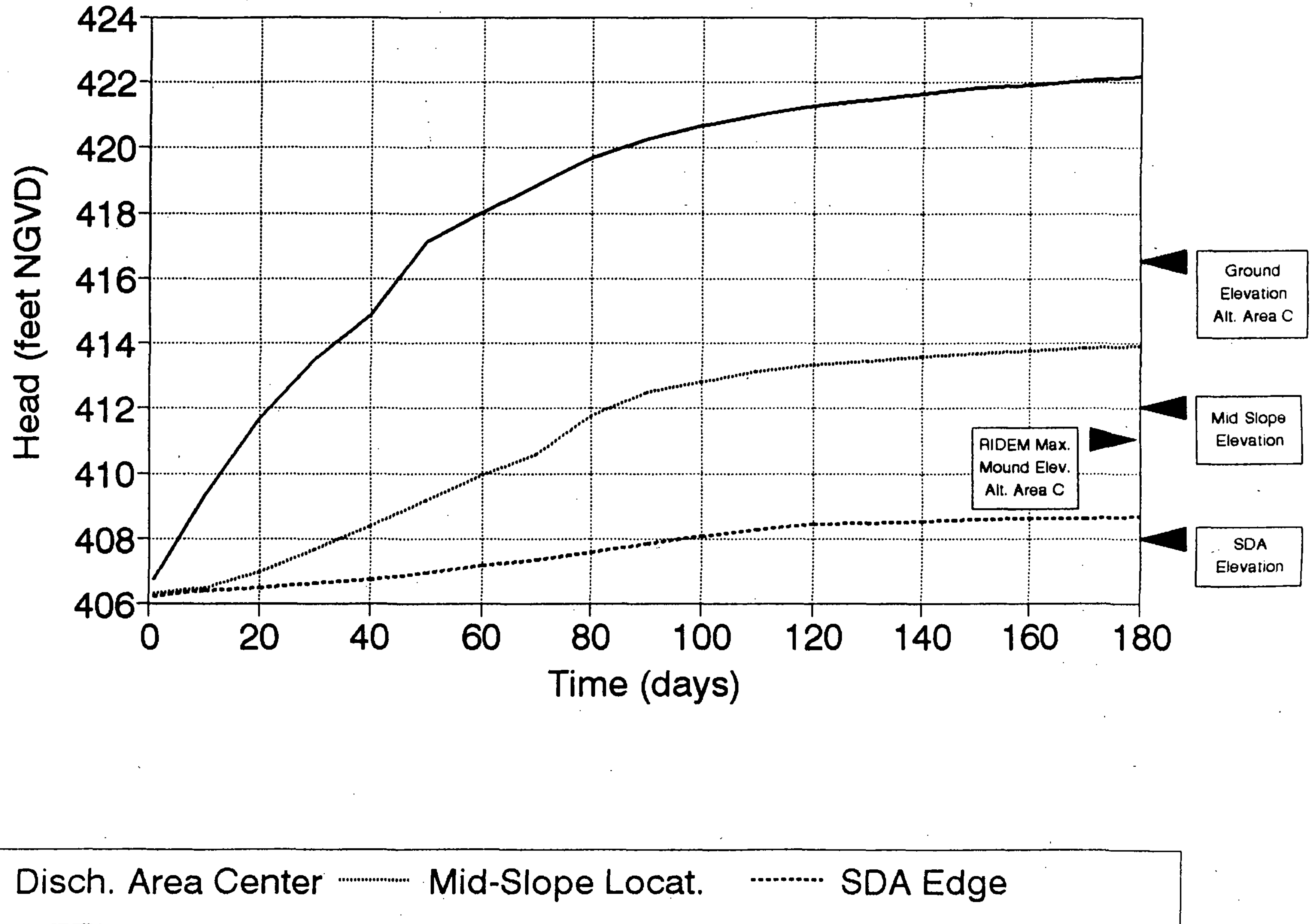
The model was used to estimate the flow rates from an initial extraction well configuration. However, upon further examination during model sensitivity analyses, it was determined that this initial configuration and estimated pumping rate might not be adequate to ensure capture during all periods of the years, and may result in drawdowns in some of the wetlands onsite. Therefore, a revised configuration was proposed and examined. This configuration appears successful in achieving capture while minimizing wetland drawdowns. A transient simulation of the revised extraction well configuration suggests that the majority of capture zone drawdown occurs after 120-140 days.

A number of simulations were also performed to examine what would happen if effluent from the proposed treatment system were discharged to the ground at Alternate Area C. The results indicated that the total extraction well pumpage could not be discharged to this area without causing surface breakout at the discharge area, as well as in the vicinity of the Southern Disposal Area. The area was also shown to be unsuitable at a third of this flowrate.

The groundwater model was used as an aid to develop a recommended extraction well configuration and estimated flowrate. Model assumptions and limitations are discussed fully in Section 7.0 of the Groundwater Modeling Report (Appendix D). The model provided the following benefits:

- Reduced some level of uncertainty associated with the Pre-Design Investigation
- Helped to better redefine the extraction area and reevaluate the potential contaminant contouring artifacts from widely spaced data points on the eastern boundary area of the 1000 ppb TVOC bedrock isocon.
- Reduced the expected pumping rate estimated in the WCC Pre-Design Report (1992).

Figure 3-13. Time History of Discharge Mound Formation (8.4 gpm)



- Permitted an evaluation of the spatial distribution of wells away from the wetlands. This resulted in a "finer tuning" of pumping placements and rates near the wetlands. The model shows that implementation of the WCC Pre-Design configuration might have caused wetland dewatering.

- Provided better control of well placements for capture.

- Provided a flexible approach to capturing groundwater from the target extraction area.

4.0 DEVELOPMENT OF COMBINED DISCHARGE FLOW RATE

An Engineering Evaluation (EE) was performed as part of the Criteria Summary Report to determine the best method for discharging treated water from the Groundwater Treatment System (GWTS). The EE is based upon a combined discharge flowrate that will include flow from the following Davis Site sources:

- Extraction Area wells
- Decontamination water, from on-site remedial activities (including future Source Control)
- Stormwater management
- Dewatering from the source areas during Source Control excavation
- Future expansion area wells

Calculations to quantify flowrates from the above sources have been developed and are included as Appendix E. These flowrates are expected to be updated as necessary and will subsequently serve as the basis for establishing design criteria for the site Groundwater Treatment System (GWTS).

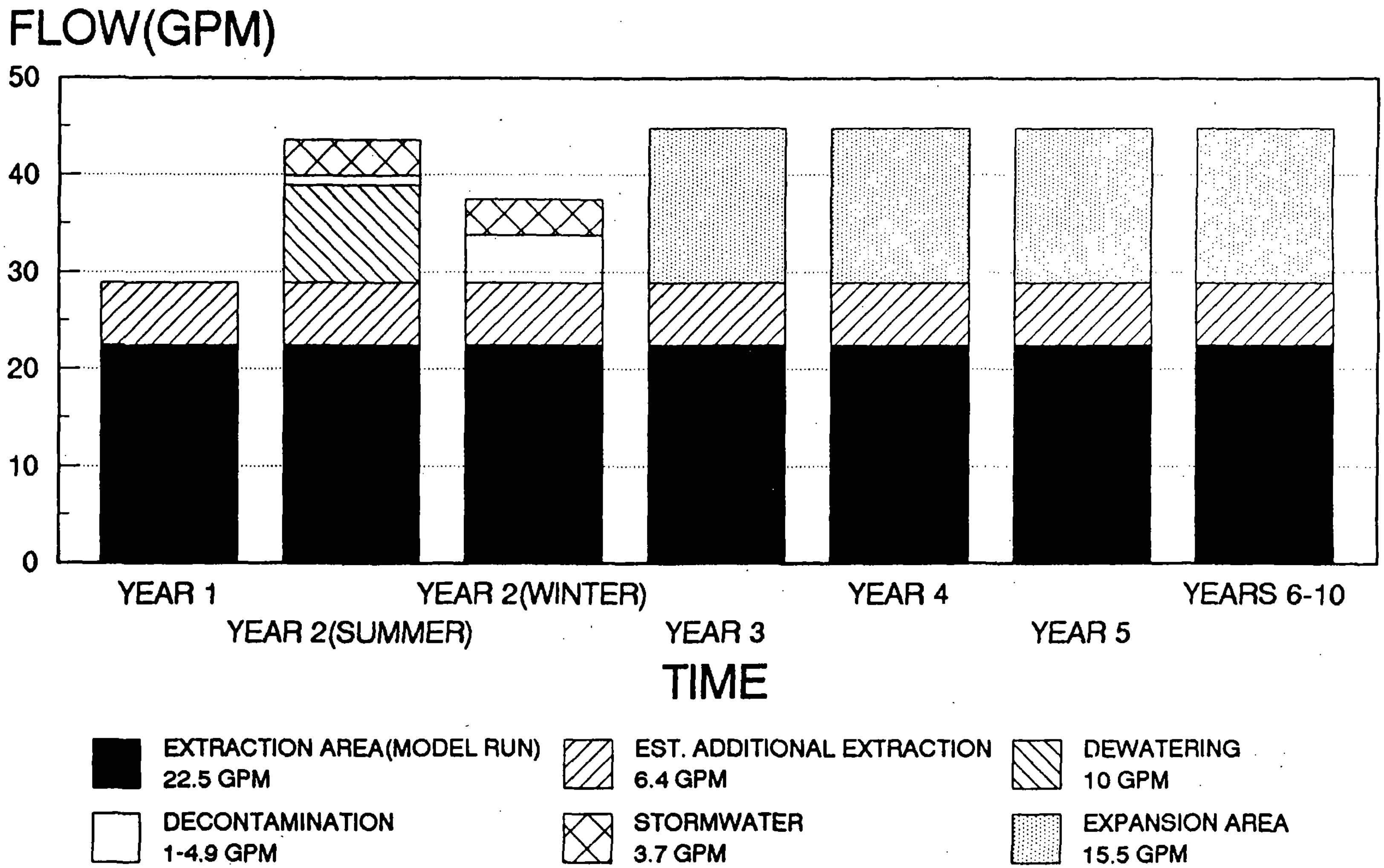
The combined discharge flowrate components presented in Appendix E are as follows:

• Extraction Area Wells (Model Simulation)	22.5	gpm
• Estimated Additional Extraction Area Flow	6.4	gpm
• Dewatering	10	gpm
• Decontamination	1-4.9	gpm
• Stormwater Management	3.7	gpm
• Future Expansion Area	15.5	gpm

As indicated on the flowrate bargraph (Figure 4-1) the estimated maximum combined discharge flowrate is approximately 45 gpm. The makeup of the combined discharge flowrate will vary with the time and phase of remediation.

FIGURE 4-1

DAVIS LIQUID GROUNDWATER PUMP AND TREAT SYSTEM
COMBINED DISCHARGE FLOWRATE

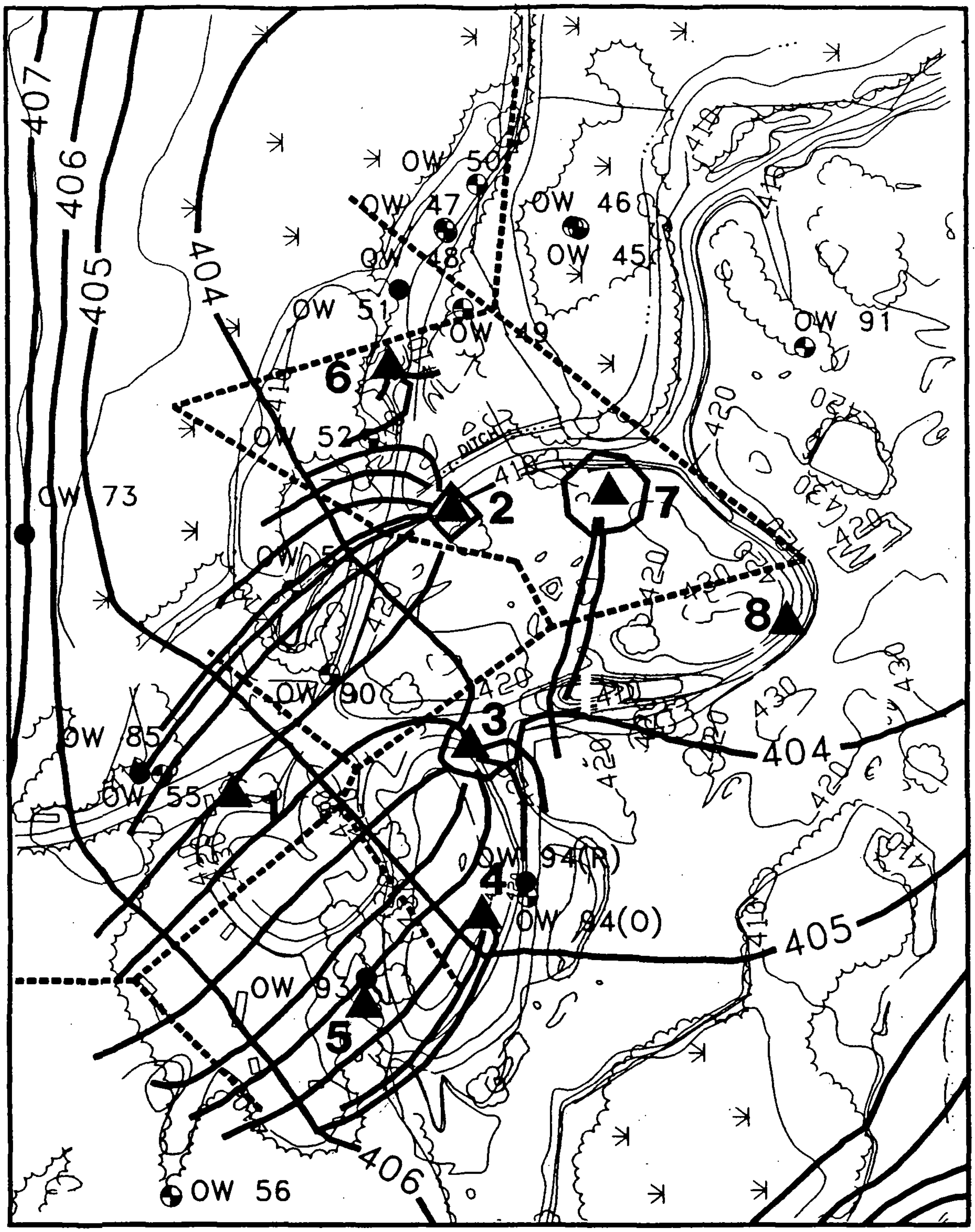


5.0 ESTIMATED INFLUENT CONCENTRATION

Estimated influent concentrations were developed based on the available historical monitoring well data and the probable influent sources to the Groundwater Treatment System (GWTS). The potential sources of groundwater to be treated in the GWTS include the base extraction flow from the area discussed in Section 3.0, and future flows from expansion area wells (Appendix E), should they be remediated. Additionally, the Source Control component of remediation is expected to produce several wastewater streams to be treated in the GWTS. These wastewater streams include groundwater captured during dewatering of the source areas during excavation, water resulting from decontamination of equipment, and potentially contaminated stormwater collected and managed during the Source Control activities. Several flowrate combinations under different site remediation time scenarios were used to estimate influent concentrations to the GWTS. The flowrate combinations and backup calculations are presented in detail in Appendix E.

A mass balance approach was used to calculate influent concentrations with each flowrate time scenario. Influent concentrations from the extraction area wells were estimated using a weighted average based on data from monitoring wells within the capture zones predicted by the groundwater model. The weighted averages were calculated with an apportionment method which utilized Theissen polygons and the extraction well capture flow paths predicted by the groundwater model to distribute the contaminant mass to the extraction wells. More detail on this approach is given in Appendix F. Figure 5-1 shows the polygons and flow paths, while Table 5-1 gives the apportionments. Proposed expansion area well concentrations were based on data from wells historically having exceeded ROD criteria (see Section 2.0 and Appendix F). Exceedences were defined in the ROD as greater than 5 ppb for either benzene, trichloroethene (TCE) or tetrachloroethene (PCE) (Section 2.3.1.2). Source Control related influent flows result from groundwater or surface water which comes into contact with contaminated soils. The data from monitoring well OW54, which is located within the Northern Disposal Area and is in contact with the contaminated soils, was utilized to estimate these concentrations. The maximum historically reported concentration of each contaminant was used in the calculations for conservatism. The conservatism was included to compensate for the limited analytical data available from each monitoring well. Flow rates utilized in the mass balance are described in Section 4.0, Development of Combined Discharge Flow Rates.

The estimated influent concentrations resulting from the series of mass balance calculations are presented in Appendix F. An evaluation of the estimated combinations of influent flow conditions described in Section 4.0 indicated that Year 1 influent concentrations are most appropriate to be used as the GWTS design basis. These concentrations are presented in Tables 5-2, 5-3, and 5-4 for volatile organics, semivolatile organics, and maximum and Woodward Clyde 11/91 total and dissolved metals, respectively. The influent concentrations developed in this scenario (Year 1) include only the base extraction area wells. While this does not reflect the maximum concentration for every contaminant from the various potential sources for each time scenario (Years 2 through 10 as presented in Appendix F), it does provide a reasonable average influent concentration. The design of various components will allow a degree of conservatism which should accommodate the limited contaminant concentrations from the expansion area that are higher than the base extraction area concentrations. Because the duration of the source control activities will be short term, it would be inefficient to use those concentrations for developing the design basis. However, the design of the treatment facility should include the flexibility to add temporary treatment units to the system should they be required for these sources. Such situations could involve the need for bag filters for solids removal, or additional carbon units for polishing during source control activities.



- - - - - - - MONITORING WELL ZONE OF INFLUENCE
——— PARTICLE FLOW PATHS
● BEDROCK WELLS
◐ OVERBURDEN WELLS
2 ▲ EXTRACTION WELLS

0 100 200
SCALE IN FEET

N

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EXTRACTION WELL
APPORTIONMENT

CONTRACT NO. DACW33-91-D-0005 | FIGURE 5-1

DWG:D5826

TABLE 5-1

DAVIS LIQUID WASTE - DELIVERY ORDER NO. 3 Mass Balance Calculations - Chemical Mass/Flow Apportionment											
Monitoring Well Apportionment for Average Annual Flow = 22.5 gpm Base Extraction - 6 Pairs of Wells											
Ex. Well	OW50	OW51	OW52	OW54	OW55	OW56	OW85	OW93	OW94(o)	OW94(r)	Background
OB-1											
OB-2			50%	35%	15%						0%
OB-3				20%	20%			20%	40%		0%
OB-4								50%	50%		0%
OB-5											
OB-6		10%	90%								0%
OB-7			70%						20%		10%
OB-8									5%		95%
BR-1											
BR-2			50%	15%			15%				20%
BR-3				20%			20%	20%		40%	0%
BR-4								50%		50%	0%
BR-5											
BR-6			90%								10%
BR-7			50%							20%	30%
BR-8										5%	95%

NOTES:
1) Data from OW96(O) and OW96(R) are used as background well concentration values
2) Extraction wells OB-1, OB-5, BR-1 and BR-5 are reserve wells and considered inactive

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TABLE 5-2

DAVIS LIQUID WASTE - DELIVERY ORDER NO. 3 Estimated Influent Concentrations - Volatile Organics All concentrations in ug/l (ppb)	
Compound	Year 1
Acetone	644
Benzene	55
Bromodichloromethane	0
Bromoform	0
Bromomethane	0
MEK	333
Carbon disulfide	1
Carbon tetrachloride	0
Chlorobenzene	4
Chloroethane	7
Chloroform	376
Chloromethane	0
Dibromochloromethane	0
1,1-Dichloroethane A	142
1,2-Dichloroethane	2
1,1-Dichloroethene E	32
1,2-Dichloroethene (total)	3834
1,2-Dichloropropane	0
cis-1,3-Dichloropropene	0
trans-1,3-Dichloropropene	0
Ethylbenzene	2410
Methylene chloride	132
MIBK	8
Styrene	0
1,1,2,2-Tetrachloroethane	0
Tetrachloroethene	856
Toluene	4462
1,1,1-Trichloroethane	2159
1,1,2-Trichloroethane	2
Trichloroethene	3185
Vinyl Acetate	0
Vinyl Chloride	188
Xylenes (total)	7929
4-Methyl-2-Pentanone	59
2-Chloroethylvinylether	0
Total Concentration (ppb)	26820
Expected Pumping Rate (gpm)	28.9
Expected Total Mass (pounds/day)	9.3

NOTE: This table lists ALL compounds analyzed for;
0 indicates not detected.

TABLE 5-3

DAVIS LIQUID WASTE - DELIVERY ORDER NO. 3 Estimated Influent Concentrations - SemiVolatile Organics All concentrations in ug/l (ppb)	
Compound	Year 1
Phenol	8
bis(2-Chloroethyl) ether	4
2-Chlorophenol	0
1,3-Dichlorobenzene	1
1,4-Dichlorobenzene	14
Benzyl Alcohol	2
1,2-Dichlorobenzene	137
2-Methylphenol	6
Bis-2-chloroisopropyl ether	0
2,2'-Oxybis (1-chloropropane)	0
4-Methylphenol	13
N-Nitroso-di-n-propylamine	0
Hexachloroethane	0
Nitrobenzene	0
Isophorone	7
2-Nitrophenol	4
2,4-Dimethylphenol	10
Benzoic Acid	224
bis(2-Chloroethoxy)methane	0
2,4-Dichlorophenol	0
1,2,4-Trichlorobenzene	24
Naphthalene	32
4-Chloroaniline	1
Hexachlorobutadiene	0
4-Chloro-3-Methylphenol	0
2-Methylnaphthalene	6
Hexachlorocyclopentadiene	0
2,4,6-Trichlorophenol	0
2,4,5-Trichlorophenol	0
2-Chloronaphthalene	0
2-Nitroaniline	0
Dimethylphthalate	3
Acenaphthylene	0
2,6-Dinitrotoluene	0
3-Nitroaniline	0
Acenaphthene	0
2,4-Dinitrophenol	0
4-Nitrophenol	0
Dibenzofuran	0
2,4-Dinitrotoluene	0
Diethylphthalate	0
4-Chlorophenyl-phenylether	0
Fluorene	0
4-Nitroaniline	0
4,6-Dinitro-2-methylphenol	0

TABLE 5-3

DAVIS LIQUID WASTE - DELIVERY ORDER NO. 3 Estimated Influent Concentrations - SemiVolatile Organics All concentrations in ug/l (ppb)	
Compound	Year 1
N-nitrosodiphenylamine(1)	0
4-Bromophenyl-phenylether	0
Hexachlorobenzene	0
Pentachlorophenol	11
Phenanthrene	0
Anthracene	0
Carbazole	0
Di-n-butylphthalate	6
Fluoranthene	0
Pyrene	0
Butylbenzylphthalate	8
3,3'-Dichlorobenzidine	0
Benzo(a)anthracene	0
Chrysene	0
Bis(2-ethylhexyl)phthalate	12
Di-n-octylphthalate	0
Benzo(b)fluoranthene	0
Benzo(k)fluoranthene	0
Benzo(a)pyrene	0
Indeno(1,2,3-cd)pyrene	0
Dibenz(a,h)anthracene	0
Benzo(g,h,i)perylene	0
Total Concentration (ppb)	533
Expected Pumping Rate (gpm)	28.9
Expected Total Mass (pounds/day)	0.19

NOTE: This table lists ALL compounds analyzed for;
0 indicates not detected.

TABLE 5-4

DAVIS LIQUID WASTE - DELIVERY ORDER NO. 3 Estimated Influent Concentrations - Metals All concentrations in ug/l (ppb)			
Compound	Year 1 Mass Balance	Year 1 WCC Total	Year 1 WCC Dissolved
Aluminum	34414	23962	21
Antimony	11	11	0
Arsenic	18	7	7
Barium	801	703	79
Beryllium	3	2	0
Cadmium	11	11	0
Calcium	18701	14305	8212
Chromium	38	27	0
Cobalt	40	23	3
Copper	79	54	0
Iron	62665	61433	8897
Lead	23	16	0
Magnesium	14302	11242	1305
Manganese	4335	2162	1464
Mercury	2	1	0
Nickel	52	39	8
Potassium	23023	16558	1786
Selenium	0	0	0
Silver	0	0	0
Sodium	12585	5463	7630
Thallium	1	0	0
Tin	1	0	0
Vanadium	82	66	0
Zinc	570	497	30
Total Concentration (ppb)	171757	136582	29442
Expected Pumping Rate (gpm)	28.9	28.9	28.9
Expected Total Mass (pounds/day)	59.5	47.3	10.2

NOTE: This table lists ALL compounds analyzed for;
0 indicates not detected.

6.0 100-YEAR FLOOD PLAIN EVALUATION

The Scope of Work required delineation of the 100-year flood plain on site using the estimated maximum surface water elevations associated with the 100-year, 24-hour storm event. These elevations, both for natural flow conditions, and in combination with treatment system discharges, are required by the Scope of Work to determine if the proposed Groundwater Treatment System (GWTS) location is within the boundaries of the 100-year flood plain and, if so, what mitigation measures may be required. Woodward-Clyde Consultants (WCC, 1992) previously applied the basin hydrology model, HEC-1, and the basin hydraulics model, HEC-2, to determine the 100-year flood plain on the North Fork of Latham Brook (Figure 6-1). Ebasco was tasked specifically to use these particular models and WCC's data sets. In this evaluation the WCC model was modified to simulate the South Fork of Latham Brook to support the treatment system siting objective.

This section provides a summary of the application of the HEC-1/HEC-2 models. Complete details and model runs can be found in Appendix G, 100-Year Flood Plain and Surface Water Runoff Evaluation.

6.1 Previous Modeling Work

In 1992, WCC modeled the North Fork of Latham Brook from the Nipsachuck Swamp to below the confluence with the South Fork of Latham Brook (Figure 6-2). WCC used HEC-1 to simulate basin hydrology for the 100-year, 24-hour storm event from two subbasins (North Subwatershed Area and South Subwatershed Area) covering the North and South Forks, respectively. The East Subwatershed Area, also shown in Figure 6-2, was not included in the analysis.

The WCC (1992) HEC-1 model estimated a peak 100-year, 24-hour discharge of 660 cfs at the second downstream culvert from the site. The rainfall depth for this storm event was estimated to be 7.00 inches, with an SCS Type III distribution. Figure 6-3 shows the resulting distribution and cumulative rainfall depth. The North Subwatershed was delineated at the outlet of Nipsachuck Swamp, with an area of 0.54 square miles. The South Subwatershed was delineated at the mouth of the South Branch of Latham Brook, with an area of 0.55 square miles. The SCS curve number and percent impervious area assigned to both subwatersheds was 63 and 5, respectively. The lag time (approximately time to peak) was 0.80 hours for the North Subwatershed, and 1.26 hours for the south. WCC (1992) only described the selection and acceptance of the 100-year, 24-hour rainfall depth and distribution, and the SCS curve number, that had been previously estimated by CDM in the RI Report (CDM, 1986).

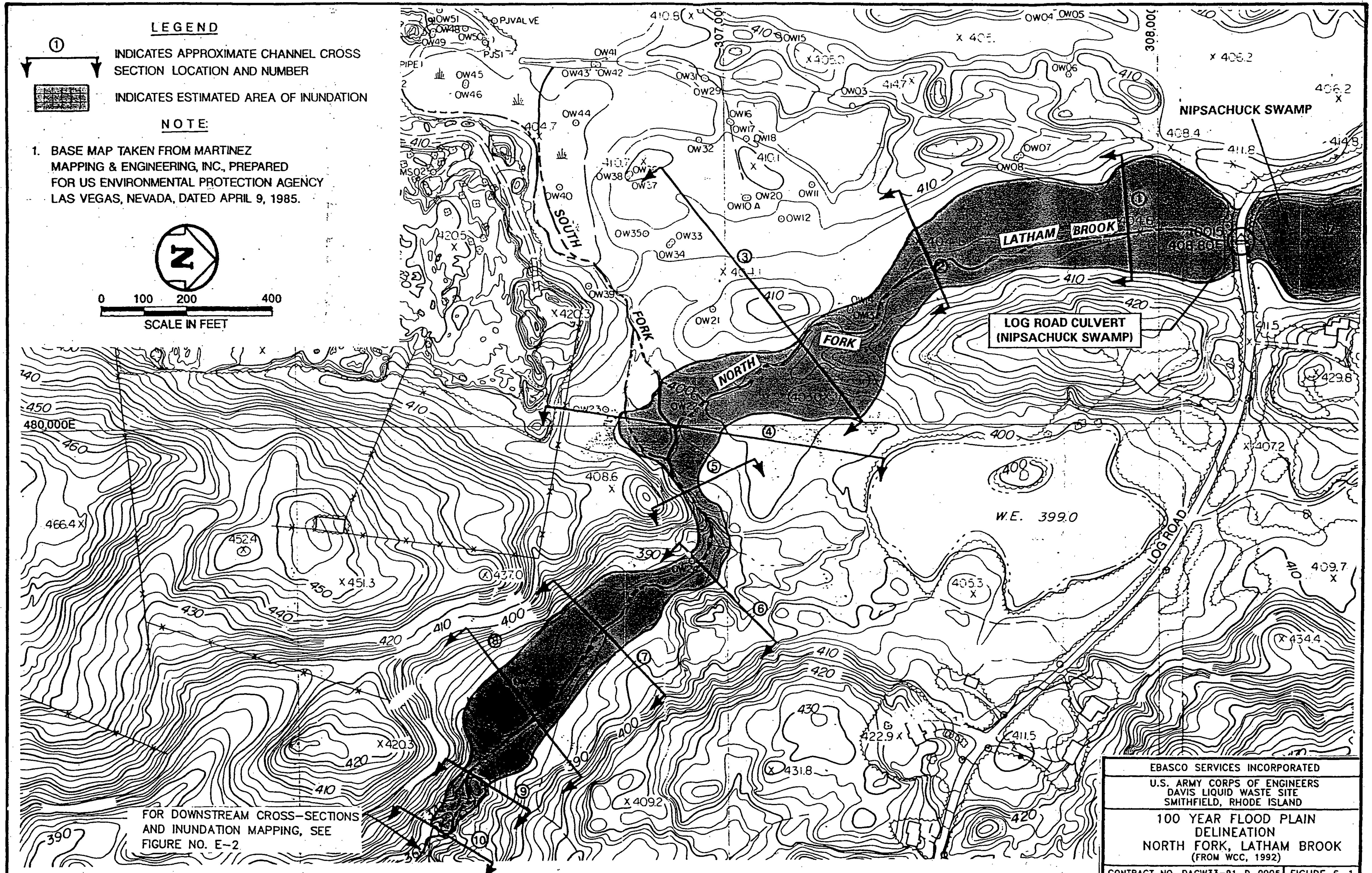
The HEC-1 results (WCC 1992) were:

1. The peak runoff from the North Subwatershed Area is 370 cfs.

2. The peak flow through the outlet culvert from the Nipsachuck Swamp is 4 cfs (the roadway, Log Road, was not overtopped even for the 100-year event).

3. The peak runoff from the South Subwatershed Area is 290 cfs.

The existing WCC model used the unit hydrographs from the North and South Subwatersheds to calculate flood hydrographs that were then combined to determine the peak discharge. WCC (1992) then proceeded with the HEC-2 modeling by assuming a "worst case scenario" in which the Log Road culvert would be washed out, and that the combined flow of 660 cfs from the two subwatershed areas would be routed downstream from the most upstream section. WCC used this constant discharge of 660 cfs throughout all cross-sections of the existing HEC-2 model. The results of this HEC-2 simulation were



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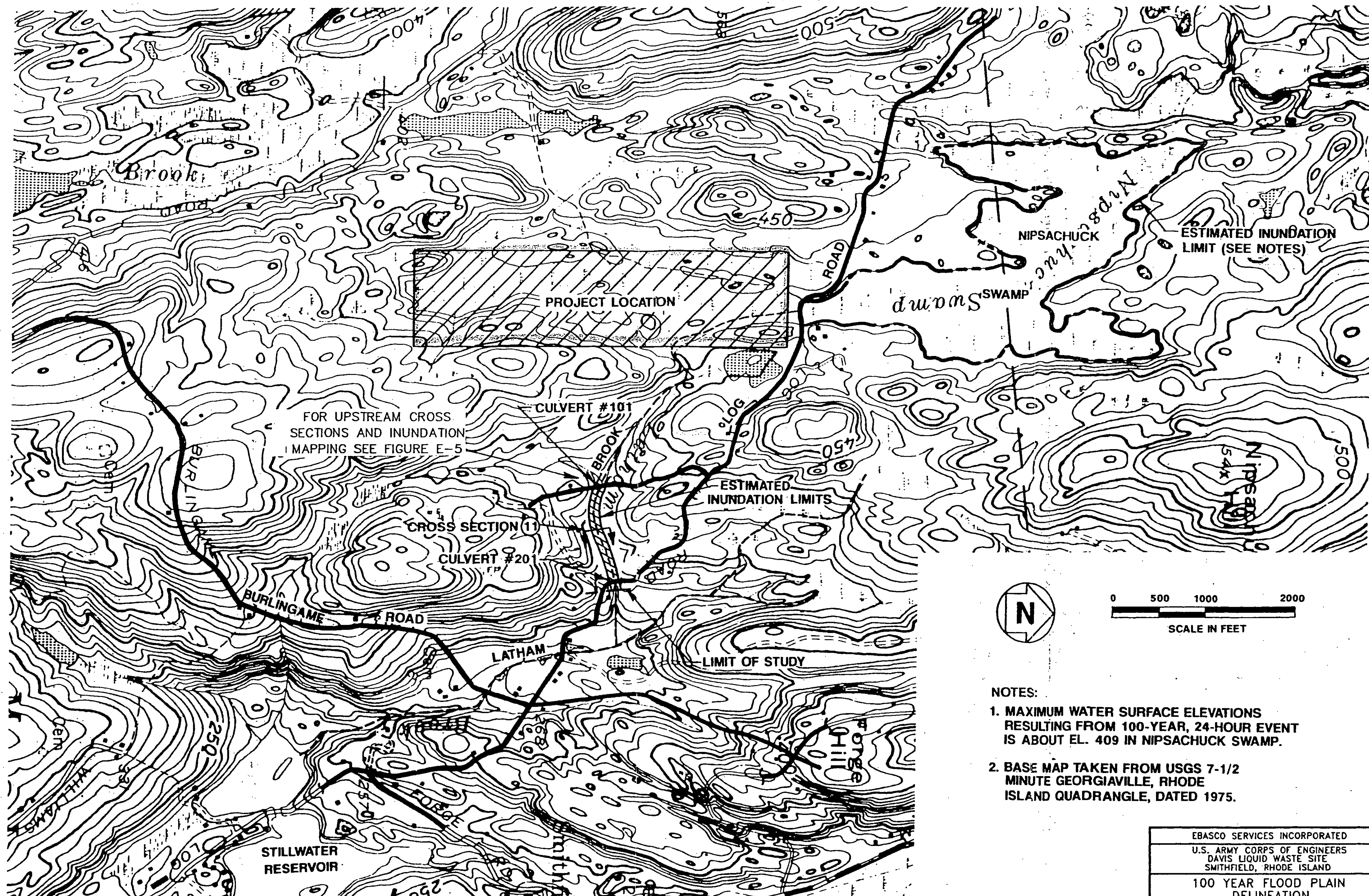
100 YEAR FLOOD PLAIN DELINEATION
NORTH FORK, LATHAM BROOK
(FROM WCC, 1992)

CONTRACT NO. DACW33-91-D-0005 | FIGURE 6-1



NOTES:

1. MAXIMUM WATER SURFACE ELEVATIONS RESULTING FROM 100-YEAR, 24-HOUR EVENT IS ABOUT EL. 409 IN NIPSACHUCK SWAMP.

2. BASE MAP TAKEN FROM USGS 7-1/2 MINUTE GEORGIAVILLE, RHODE ISLAND QUADRANGLE, DATED 1975.

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100 YEAR FLOOD PLAIN DELINEATION NORTH FORK, LATHAM BROOK (FROM WCC, 1992)
CONTRACT NO. DACW33-91-D-0005 FIGURE 6-1A

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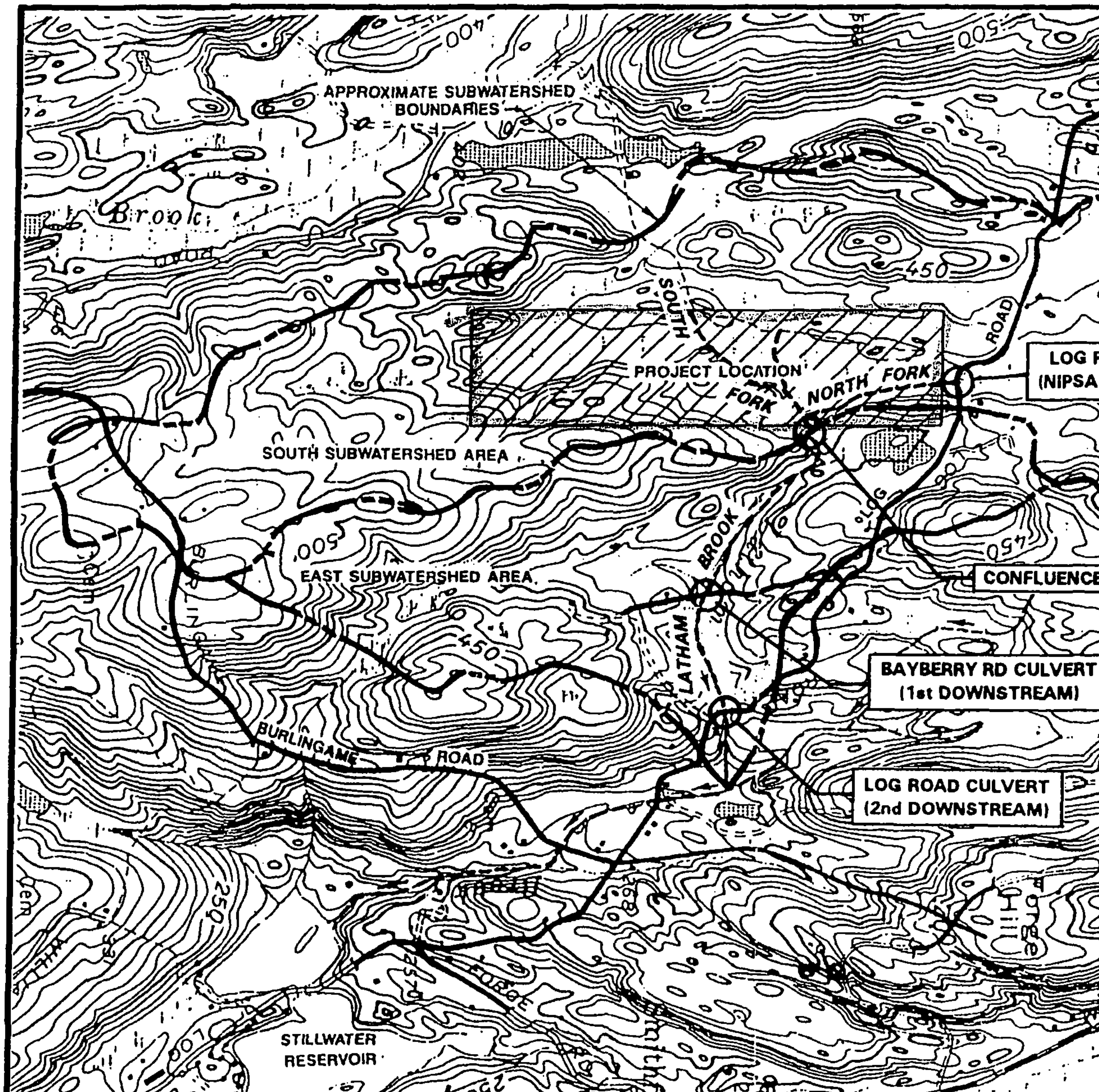

NOTE:
BASE MAP TAKEN FROM USGS 7-1/2 MINUTE GEORGIAVILLE, RHODE ISLAND QUADRANGLE, DATED 1975.

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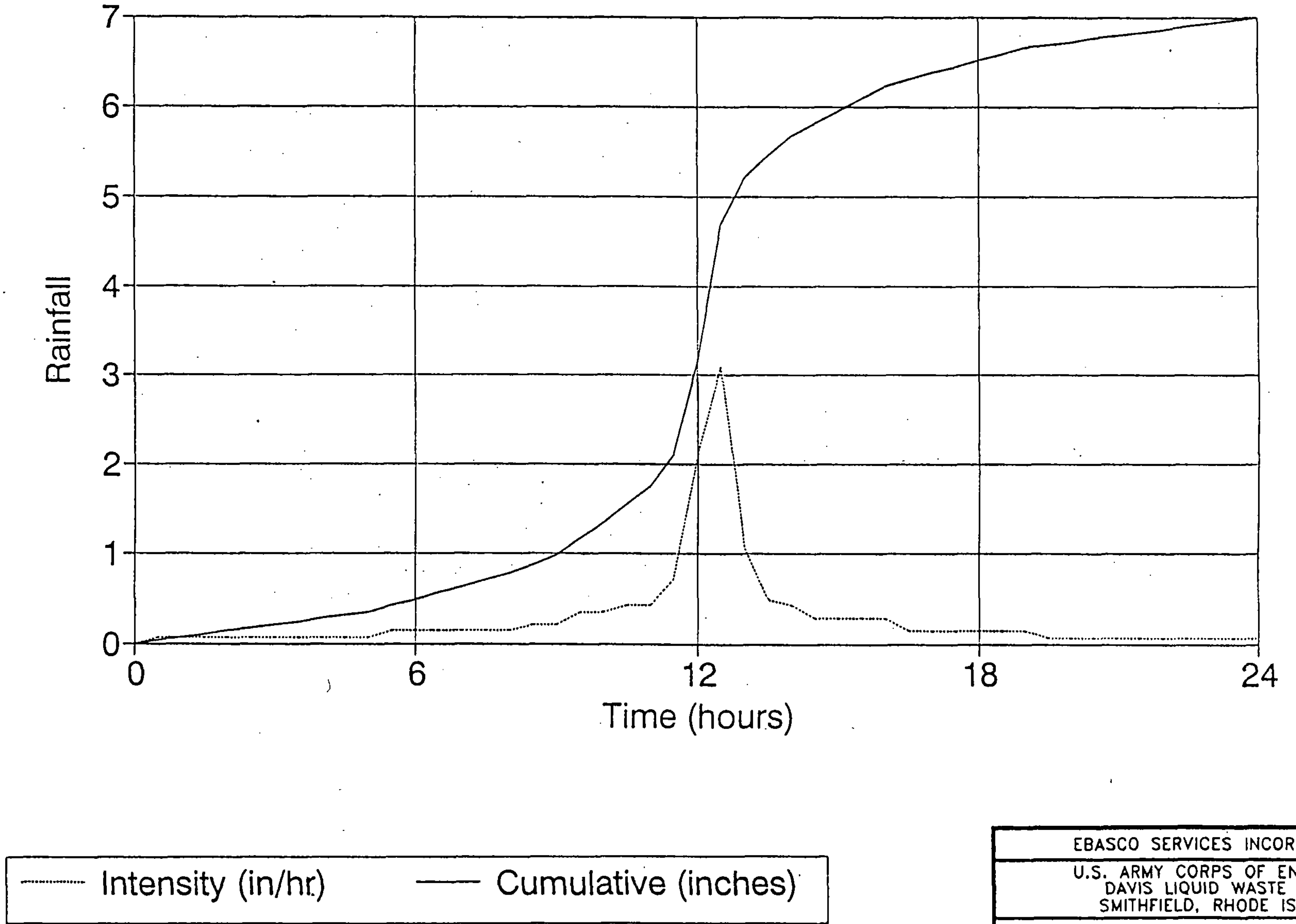
WCC SUBBASIN MAP
(FROM WCC, 1992)

CONTRACT NO. DACW33-91-D-0005	FIGURE 6-2



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100-YEAR, 24-HOUR RAINFALL DISTRIBUTION

CONTRACT NO. DACW33-91-D-0005 | FIGURE 6-3

used by WCC to develop a 100-year flood plain map for the North Fork of Latham Brook, in the vicinity of the site area.

In reviewing the work performed by WCC (1992), Ebasco noted the following observations that might affect the HEC-1/HEC-2 simulations performed in this evaluation:

1. The East Subwatershed Area was not included in the WCC data files received from the US Army Corps of Engineers (USACE-NED). Some of this area will contribute flow to Latham Brook in the modeled reach.

2. The assumption that Log Road washes out is quite severe. This washout almost doubles the flow Latham Brook sees upstream of the confluence between the North and South Forks. In fact, Nipsachuck Swamp currently serves to detain significant runoff events, and to eliminate large peak flows from the North Subwatershed Area.

3. The WCC HEC-2 model put the combined flow (North and South Subwatersheds) at the upstream end of the modeled reach, rather than specifying 370 cfs to the confluence and 660 cfs below the confluence.

4. The South Subwatershed Area is modeled as one contributing area. It contains culverts and wetlands that could significantly alter the runoff from the basin. However, detailed modeling of this subarea, other than using subbasin area ratios of the runoff peak, is beyond the scope of this evaluation.

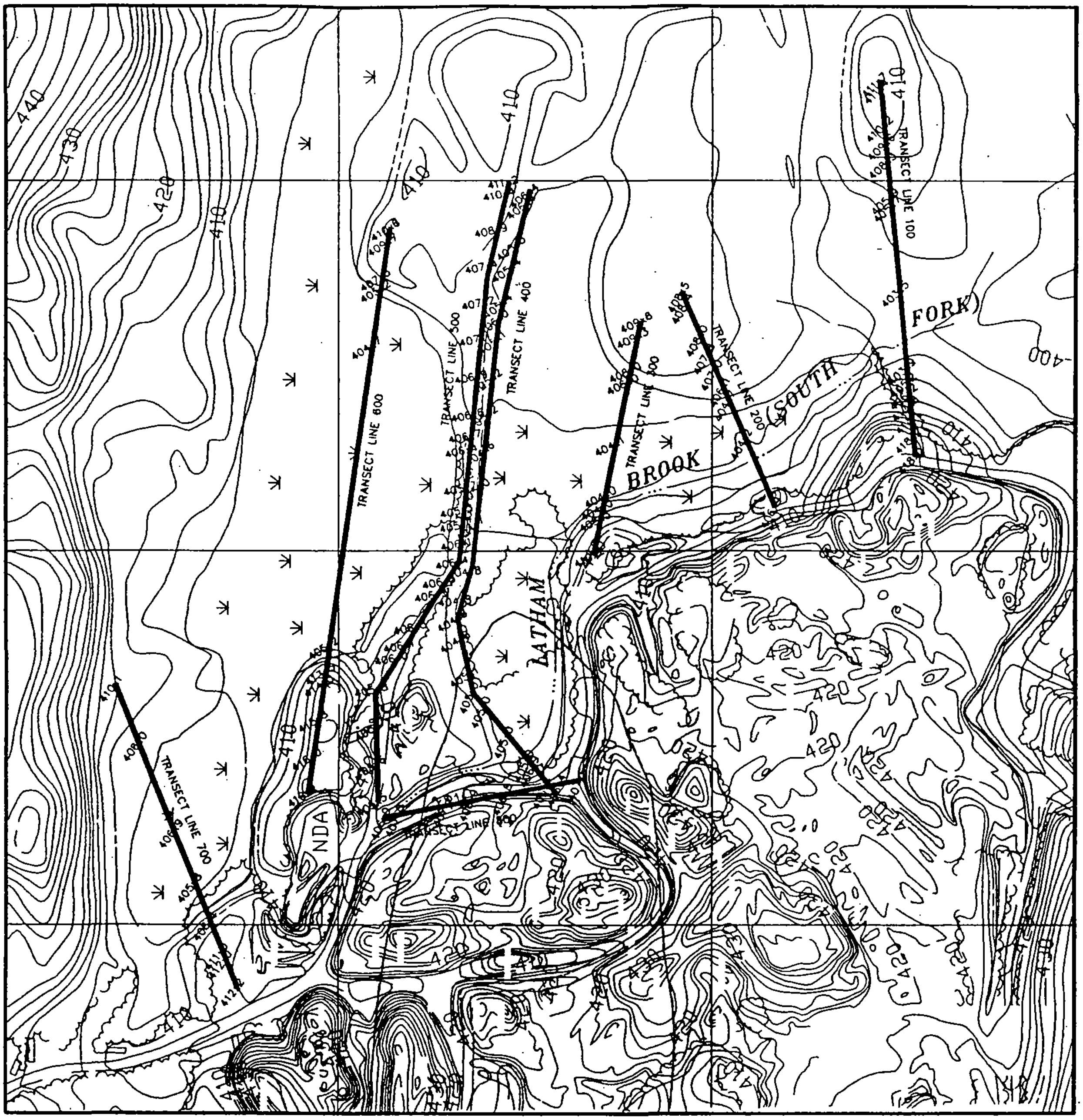
Observations 1, 2 and 3 above will have little or no effect on estimated water levels in the South Subwatershed Area (the treatment system siting area) as the South Fork drops over 6 feet before the confluence with the North Fork. However, they might affect the HEC-2 model runs for the 2, 10, and 25-year storm events (yet to be run) and thus, should be considered when evaluating the impact on the downstream culverts in the later evaluation task.

Observation 4 might be important in determining water elevations near the site of the proposed treatment plant. However, Ebasco has assumed a conservatively high flow contribution in this section of the South Fork, and the predicted water levels were well below the ground elevation in the vicinity of the proposed site.

6.2 Model Methodology Input Parameters and Assumptions

As part of this investigation, eight new transects were surveyed (Figure 6-4). Ebasco then developed the model of the South Fork of Latham Brook by (1) eliminating the transects in the WCC model above the confluence, and (2) adding the new surveyed transects along the South Fork of Latham Brook. The transect profiles surveyed for this investigations are shown in Figure 6-5.

The revised 1987 version of HEC-1 was used with the existing WCC data files following procedures outlined in the User's Manual (Hydrologic Engineering Center, 1987). The HEC-2 modeling was performed following procedures outlined in the User's Manual (Hydrologic Engineering Center, 1982) with an enhanced version of the updated August, 1991 version of HEC-2 that is commercially available (Boss, 1992).

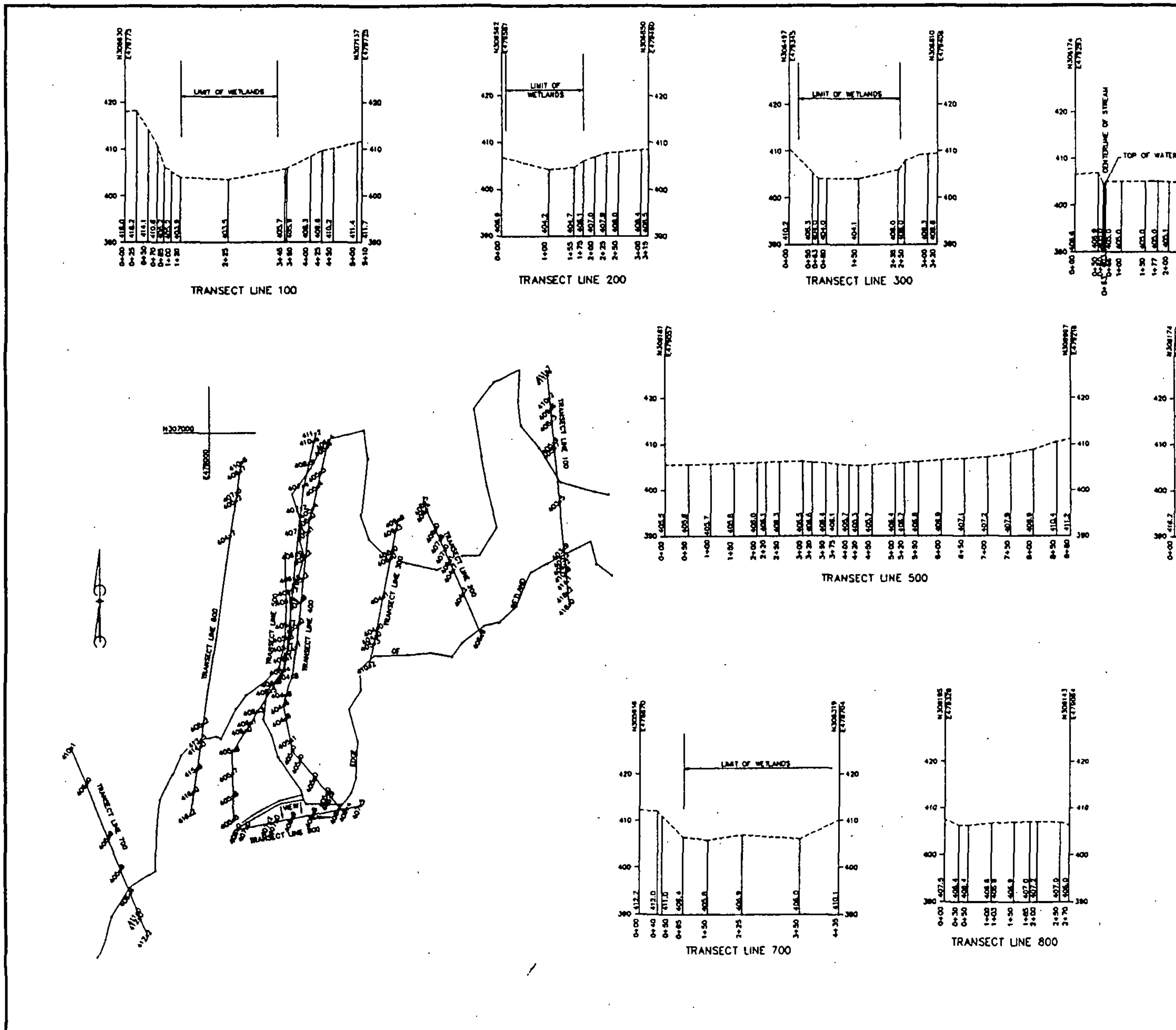


NOTE: INFORMATION IS BASED ON FIELD SURVEY PERFORMED BY MARC NYBERG & ASSOCIATES SEPTEMBER, 1993.

0 100 200 300 400
SCALE IN FEET

EBASCO SERVICES INCORPORATED	
U.S. ARMY CORPS OF ENGINEERS DAVIS LIQUID WASTE SITE SMITHFIELD, RHODE ISLAND	
SOUTH FORK TRANSECT LOCATIONS	
CONTRACT NO. DACW33-91-D-0005	FIGURE 6-4

DWG:DAVSITE3



SCALE: PLAN VIEW 1"= 100'
PROFILE: HORIZ. 1"= 100'
VERT. 1"= 10'

REVISED: NOVEMBER 3, 1993
(NOTE: THIS DRAWING HAS BEEN REDUCED – DO NOT SCALE DRAWING.)

NOTE: INFORMATION IS BASED ON FIELD SURVEY PERFORMED BY MARC NYBERG & ASSOCIATES SEPTEMBER, 1993

EBASCO SERVICES INCORPORATED
U.S. ARMY CORPS OF ENGINEERS DAVIS LIQUID WASTE SITE SMITHFIELD, RHODE ISLAND
SOUTH FORK TRANSECT PROFILES
CONTRACT NO. DACW33-91-D-0005 — FIGURE 6-5

DWG: DAVSITE8

This commercial version of HEC-2 required that <u>model</u> transect numbers increase in the upstream direction; thus, the model numbering system was based on the surveyed transect numbers. The corresponding numbering systems are shown in Table 6-1. In addition, it was necessary to add sections to improve model resolution in some areas. These sections were developed by using adjacent surveyed sections and adjusting for channel slope.

The system flows were then defined as (1) 100 cfs at the upstream section 1750 (area of the proposed treatment plant location), (2) 250 cfs at section 1400, (3) 290 cfs at section 1100, and (4) 660 cfs from the confluence to the downstream extent of the model. The incremental flows, totaling 290 cfs from the South Subwatershed Area, were developed by using ratios of the contributing subbasin area. Downstream of the confluence, a flow of 660 cfs was specified. This is identical to the downstream flow specified in the WCC study. Other assumptions used in the construction of this model are detailed in Section 6.5.

6.3 Maximum 100-Year Flood Surface Water Elevations

The model was then run, and 100-year water surface elevations calculated. The results are summarized in Table 6-1. The resulting 100-year flood plain is mapped on Figure 6-6. This map was constructed using the surveyed locations of the predicted elevations along the transects on the North Fork of Latham Brook. As can be seen in Figure 6-6, the GWTS is not within the boundaries of the 100-year floodplain. Therefore, no mitigative measures are required. However, the model does predict an overtopping of the cart path during this storm event.

6.4 Maximum 100-Year Flood Surface Water Elevations Including the Combined Discharge Flowrate

It was initially estimated that the treatment system would discharge up to 120 gpm (0.27 cfs). This flowrate is almost three times the presently calculated treatment system design flowrate of 45 gpm. However, based on schedule this treatment system higher flow rate was used in the 100-year flood evaluation. Two potential discharge locations were identified (Figure 1-1).

The revised HEC-2 model was run two more times with the higher incremental inflows of 0.27 cfs downstream from model sections 1,460 and 1,100, respectively. The treatment system discharge is small compared to the 100-year discharge in the South Fork of Latham Brook (estimated to be about 100 cfs in the vicinity of the proposed treatment system), and the effect of the additional discharge is less than 0.01 feet, which is the resolution reported in Table 6-1. Therefore, the results were not plotted or tabulated as there is no significant change in modeled water elevations.

6.5 Model Assumptions

The following assumptions were incorporated in the performance of the above work:

- The HEC-2 model for computation of steady-state water surface profiles was assumed suit the objectives of this investigation.
- Latham Brook flow was assumed to be non-uniform.
- The storm event rainfall distribution was assumed to be SCS Type III.
- The SCS method for storm event hydrography synthesis was assumed to apply to this watershed.

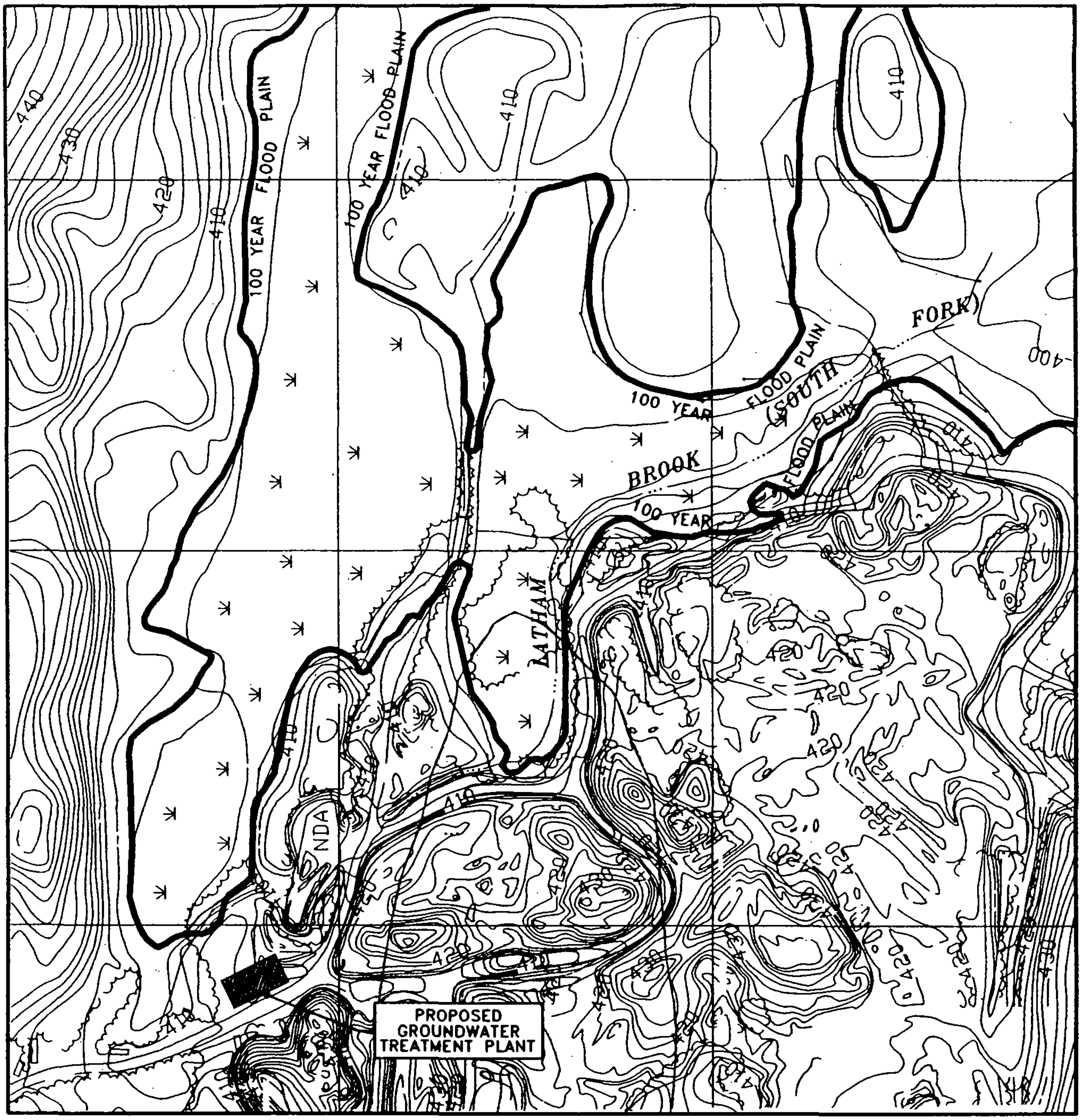
Table 6-1

100-Year Storm Elevations Computed from HEC-2 Analysis

Model Section Number	Surveyed Section Number	Heads (feet NGVD)
201		308.53
202		310.57
203		313.75
204		313.86
211	11	331.65
301		352.03
302		353.34
303		357.51
304		357.58
310	10	361.69
409	9	373.43
508	8	381.53
607	7	384.37
706	6	386.17
805	5	395.47
904	4	399.38
Confluence Between North and South Forks of Latham Brook		
1100	100	405.07
1200	200	405.89
1300	300	406.24
1350		406.26
1400		406.27
1450		406.27
1460	400	406.27
1490	500	406.42
1500	500	406.45
1600	600	406.45
1700		406.47
1750	700	406.89

Notes:

1. Surveyed Sections 4-11 from WCC (1992) study.
2. Sections 100-700 surveyed during this study.



0 100 200 300 400

SCALE IN FEET

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SMITHFIELD, RHODE ISLAND

100 YEAR FLOOD PLAIN
DELINEATION
SOUTH FORK, LATHAM BROOK

CONTRACT NO. DACW33-91-D-0005 | FIGURE 6-6

DWG:DAVSITE4

- All the subwatersheds' percent impervious areas were assumed to be 5 percent.

- The embankment at the outlet of Nipsachuck Swamp failed during the 100-year, 24-hour storm as in the WCC model.

- The outflow hydrographs from the North and South Subwatersheds were combined assuming that no downstream channel/floodplain routing effects would require hydrography time base offset. The East Subwatershed was excluded as in the WCC model.

- The South Subwatershed area upstream of the cart path was assumed to contribute 34.5 percent of the total subwatershed runoff.

- South Branch Latham Brook flow was assumed to be subcritical.

- No influence of the downstream culverts on upstream water surface elevations was assumed because of the super-critical flow conditions existing in the steep reaches below the confluence of the North and South Branches.

- The additional flow from the proposed treatment system was not assumed to be attenuated downstream of the point of discharge into Latham Brook.

- The northwest wetland culvert which crosses under the cart path has a maximum discharge capacity that is negligible relative to the South Branch Latham Brook 100-year discharge at this point. The culvert was assumed to be incapable of conveying the South Subwatershed flow during large storm events, and therefore most of the brook's stormwater would discharge over the cart path embankment.

7.0 PROPOSED TREATMENT System SITING AND COLLECTION PIPING CONFIGURATION

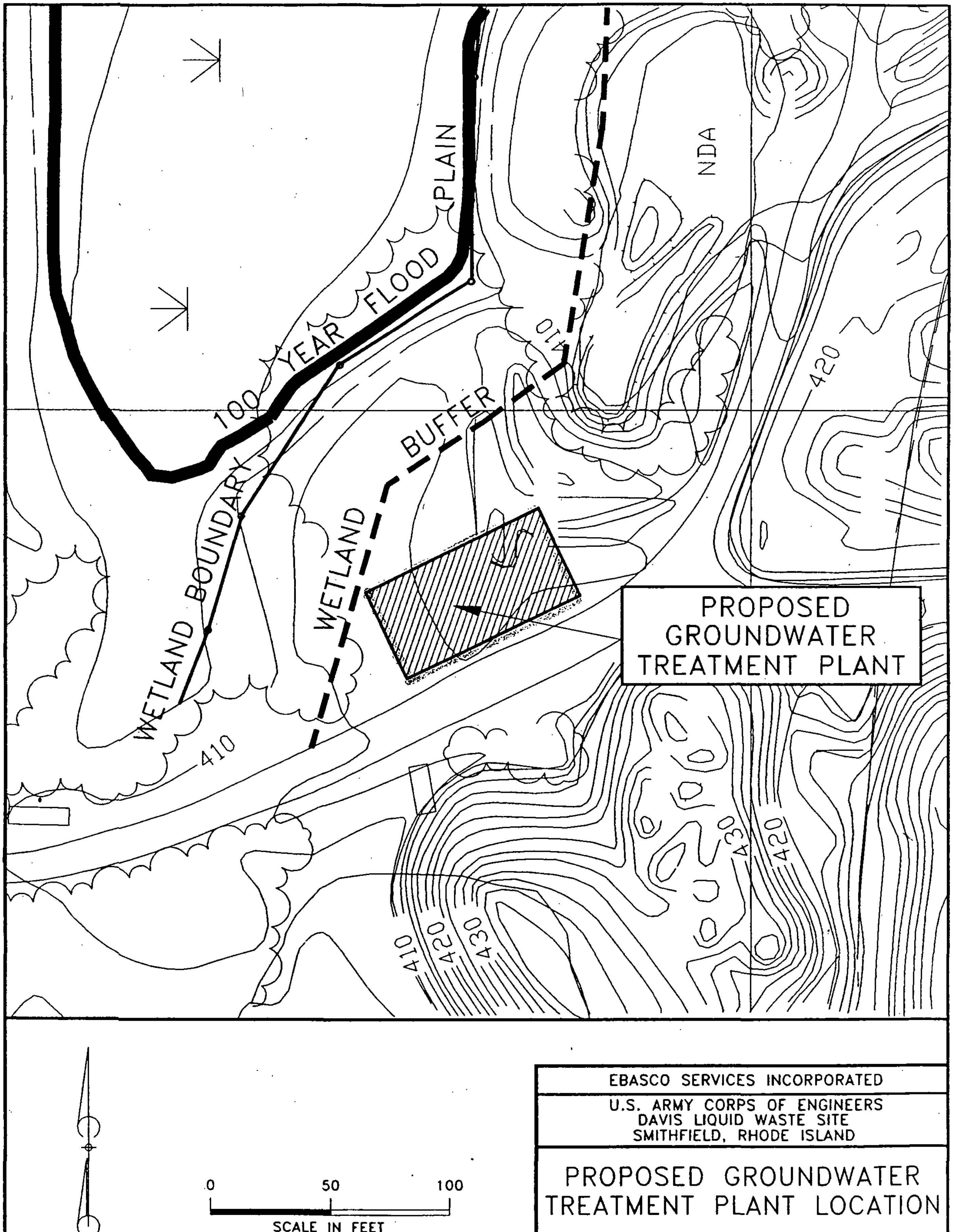
The Groundwater Treatment System consists of the Groundwater Treatment System (GWTS), 8 pairs of extraction wells and submersible pumps, and approximately 1,700 feet of influent header piping. The groundwater will be pumped from the extraction wells to the header system which will route the groundwater to the GWTS.

7.1 Groundwater Treatment System Siting

The proposed GWTS (Figure 7-1) will be located outside the 100 year flood plain in an area which is relatively level and clear of vegetation and debris. As a result, no mitigation measures with regard to construction within a 100 year flood plain are required. The GWTS is also located outside of the 50-foot perimeter wetland also regulated by RIDEM. The GWTS is anticipated to be no greater than 40' x 80' in size and will include metals pretreatment, air stripping with off-gas air treatment, and liquid-phase carbon adsorption polishing. The GWTS will also effectively house additional units that may be required to accommodate influent from supplemental extraction wells or wastewater resulting from source control measures. For further details on potential influent flow rates, see Section 4.0.

7.2 Collection Piping Runs and Pumping Configuration

The collection piping runs will consist of a minimum length of 1 inch piping from each well box which will then connect to one of three header pipes (2-4 inches in diameter) as shown in Figure 7-2. Header #1 will consist of well pairs 1, 3, 4 and 5; Header #2 will be made up of well pairs 2, 7 and 8; and Header #3 will connect well pair 6 to the GWTS. If supplemental extraction wells are added in the future it is envisioned that they will be included as an extension of Header #3. The piping will be buried below the frost line (approximately 4.5 feet) or, where necessary, appropriately insulated to prevent freezing. Where ever possible, the header piping will be routed along the existing service roads due to the heavy vegetation and tire piles present on site.



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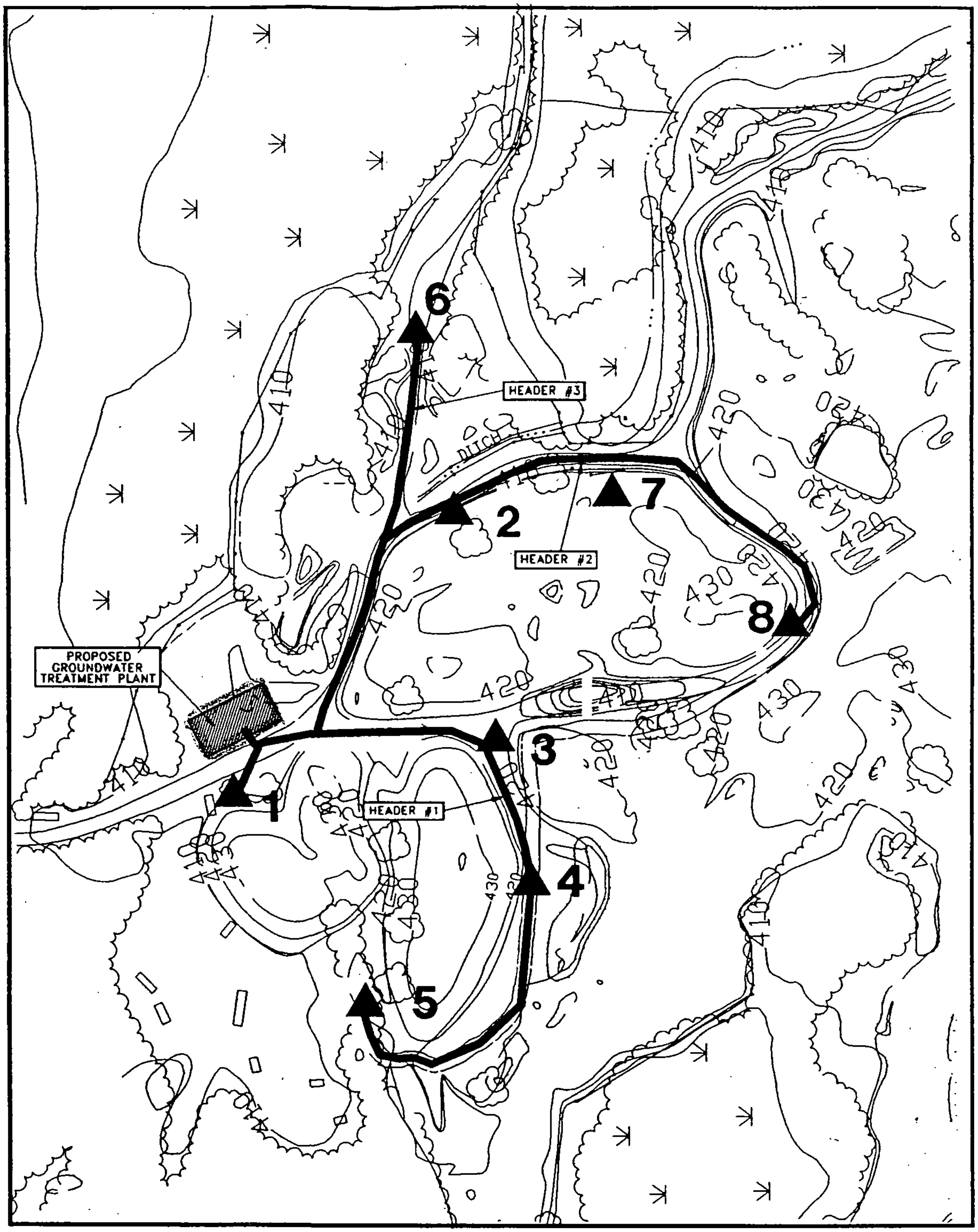
PROPOSED GROUNDWATER TREATMENT PLANT LOCATION

CONTRACT NO. DACW33-91-D-0005 | FIGURE 7-1

DWG:DAVSITE5



PIPING RUNS

2 ▲ EXTRACTION WELLS

N

0 100 200

SCALE IN FEET

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SMITHFIELD, RHODE ISLAND

EXTRACTION WELL LOCATIONS AND PIPING RUNS

CONTRACT NO. DACW33-91-D-0005 | FIGURE 7-2

DWG:DAVSITE8

8.0 ENGINEERING EVALUATION REPORT

As specified in the Statement of Work, three options were to be evaluated to determine the best method for discharging treated water from the GWTS. These options include a groundwater discharge at Alternate Area C, and two surface water discharge locations - Latham Brook and the wetlands directly upstream of the brook. Figure 8-1 identifies each of these locations and the site features which may affect implementation of the options including wetlands delineations, and 100-year flood plain location. A larger scale and more easily readable G-size plan is included in Appendix I. However, based on the groundwater modeling results of the ability of Alternate Area C to accept recharged groundwater, Alternate Area C was eliminated as a possible discharge option. A summary of this evaluation is presented in Section 8.1. Therefore, only the surface water discharge options are carried forward in this evaluation.

Sections 8.2 and 8.3 present the surface water discharge options and their characteristics, the criteria used in the evaluation, and the evaluation of the two remaining options based on these criteria.

8.1. Elimination of the Groundwater Discharge Option

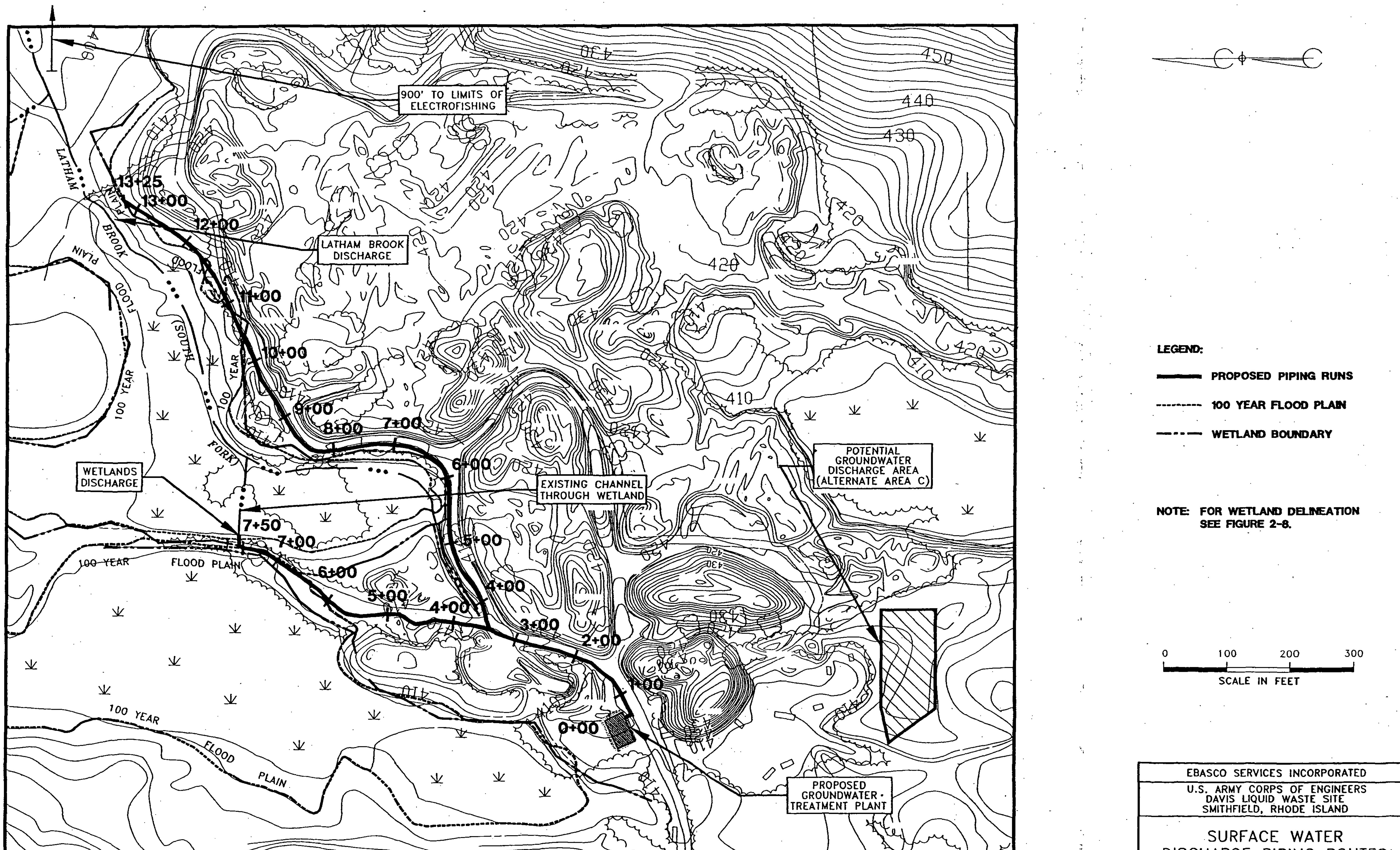
The groundwater discharge location identified as Alternate Area C was presented as a potential discharge option. This option would involve the discharge of treated effluent from the GWTS to a discharge gallery or leaching field structure installed at the site. The location of Alternate Area C is presented in Figure 8-1. As can be seen from the figure, Alternate Area C is essentially a plateau, bounded on the north by steep slopes leading into the Southern Disposal Area. The ability of the site to accept recharged groundwater, and the potential for groundwater breakout on the surface, the adjacent slopes or into the Southern Disposal Area was evaluated as part of the groundwater modeling effort completed previously, as described in Section 3.5.5 and Appendix D.

As discussed in section 3.5.5, the modeling predicted that with a recharge rate of 26.2 gpm (the initial estimate of groundwater extraction), under steady state conditions breakout would occur at the ground surface. The estimated discharge rate from the GWTS is projected at between 17 and 45 gpm, with an average of 30 gpm projected. Based on the modeling results, a preliminary determination was made that using Alternate Area C would not be feasible due to its inability to receive the total discharge volume expected from the GWTS. Subsequent to the preliminary determination which was based on the groundwater modeling results, additional evaluations were performed based on compliance with RIDEM regulatory criteria under various hydraulic conductivity values. A summary of this evaluation follows.

The RIDEM Groundwater Section has no specific design criteria. Applicants may either satisfy the RIDEM Individual Subsurface Disposal System (ISDS) Guidelines that are used primarily in the design of septic systems or else provide a design based on a groundwater mounding analysis. Common to either approach are the following requirements:

- There must be an initial three foot separation between historic seasonally high groundwater and the bottom of the discharge structure.
- The resultant mound cannot exceed the bottom elevation of the discharge structure or break out laterally on the ground surface.
- The discharge effluent must meet groundwater discharge chemical standards.

Average ground surface elevation in the Alternate Area C discharge area is 416.5 feet msl. A seasonally high groundwater elevation was selected as 407 feet msl based on a measured elevation in OW56 of 406.5



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SMITHFIELD, RHODE ISLAND

SURFACE WATER DISCHARGE PIPING ROUTES

CONTRACT NO. DACW33-91-D-0005 | FIGURE 8-1

DWG:DS023

feet during the RI and the observed standing water in the Southern Disposal Area at surface elevations greater than 407 feet msl. Using Rhode Island guidelines for frost protection of 4.5 feet and a discharge structure design of 4-inch (0.33 feet diameter) pipe with 0.5 feet of gravel beneath, a minimum structure depth is 5.33 feet. Subtracting 5.33 feet from the ground elevation of 416.5 feet msl, the bottom of the structure is at elevation 411.17 feet msl. When compared to the seasonal high groundwater elevation of 407 feet, the 3-foot separation criteria is met and the maximum allowable discharge mound height beneath the discharge area is 4.17 feet. On the floor of the Southern Disposal Area, 95 feet north of the discharge bed center, the surface elevation is 408 feet msl and the maximum allowable mound height would be 1 foot. A majority of the Southern Disposal Area is at this elevation (408 msl) or lower.

Given the highly restrictive loading rates allowed under the ISDS regulations, a groundwater mounding approach was undertaken to further evaluate discharge location feasibility. The Hantush mounding calculation for a rectangular area was selected (Bouwer, 1978). Data parameters are listed in tables presented on Figure 8-2.

Using a 70 x 187 foot area to represent an estimated available usable discharge area of 13,100 ft^2, the 45 gpm discharge flow rate was simulated for a range of hydraulic conductivities. Hydraulic conductivity in this area may be in the range of 16 to 70 ft/day. A 70 ft/day value represents an estimated upper bound of hydraulic conductivity and an optimal value could be 100 ft/day. Simulation results are plotted in Figure 8-2 and show that over the assumed range of hydraulic conductivity (16 to 70 ft/day), the mound height in the center of the discharge area exceeds the maximum allowable discharge mound height and is thus, regulatorily unacceptable. At the optimal hydraulic conductivity value of 100 ft/day, although the maximum allowable discharge mound height at the bed center is not exceeded, the discharge will still create a mound 2.50 feet high in the Southern Disposal Area (95 feet north of the discharge bed center point) and cause surface breakout (by exceeding a mound height of 1 foot) and thus, is also regulatorily unacceptable. Further calculations show that a hydraulic conductivity value greater than 260 ft/day is required to provide an acceptable mound height of less than 1 foot for this Southern Disposal Area location.

The same approach and calculations were performed as above using a 30 gpm discharge flow rate (see Figure 8-3), and the 1 foot maximum allowable discharge mound height for the SDA floor also was exceeded over the expected range of hydraulic conductivity (16 to 70 ft/day). Under the assumed and homogeneous optimal hydraulic conductivity of 100 ft/day, it appears that breakout will also occur in the Southern Disposal Area where a mound height of 1.67 feet still exceeds the minimum acceptable mound height criteria of 1 foot.

Therefore, based on the results of the above mounding analysis and considering the previous modeling results, the groundwater discharge location for Alternate Area C is regulatorily unacceptable. Therefore, Alternate Area C was eliminated from consideration as a possible discharge option.

8.2 Development of Surface Water Discharge Options

In this section, the two on-site surface water discharge options are further developed. Selection of specific locations for the two surface water discharge options was completed during the site walkdown and is presented in section 2.6. Details of implementing these options are presented in the following sections.

Figure 8-2 Alternate Area C Mounding Analysis 45 gpm

Data for Mounding at Alternate Area C

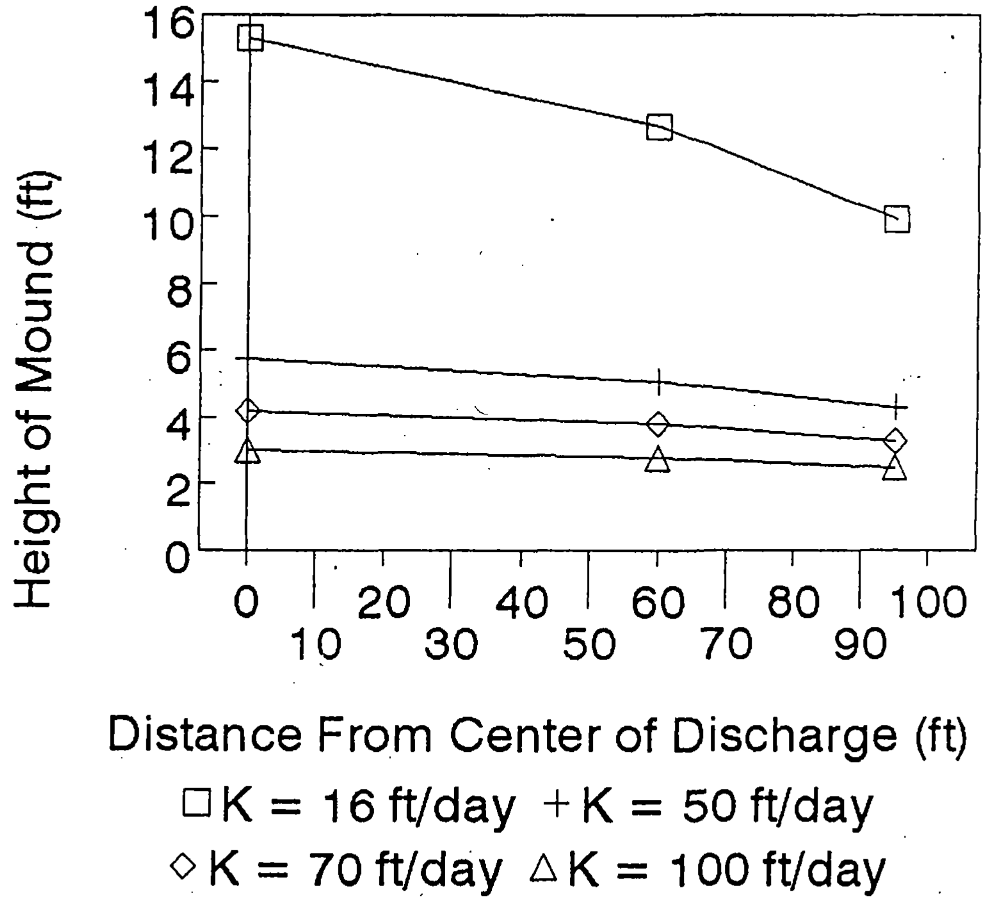
K (ft/day)	Height of Mound Generated: Bed Center (0,0)	Slope (0,60)	SDA Floor (0,95)
	0	60	95
16	15.31	12.68	9.94
50	5.75	5.07	4.30
70	4.22	3.81	3.29
100	3.03	2.79	2.50

Data Inputs	
Saturated Thickness	17 ft
Porosity	0.25 dim
Discharge Volume	45 gpm
Time Elapsed	365 days
Bed Length	70 ft
Bed Width	187 ft
Initial Water Table	0 ft

Critical Mound Height Calculation	
Ground Surface Elev.	416.50 ft msl
Structure Depth	-5.33 ft
Structure Bottom Elev.	411.17 ft msl
Seasonal High GW Elev	-407.00 ft msl
Max. Allowable Mound	4.17 ft

Structure Depth Basis	
Frost Protection Depth	4.50 ft
Pipe Size (4-inch)	0.33 ft
Bottom Stone Depth	0.50 ft
Structure Depth	5.33 ft

Alternate Area C Mounding Analysis (Hantush Method)



AC94-033
3/14/94 3:16pm

Figure 8-3 Alternate Area C Mounding Analysis 30 gpm

AC94-033
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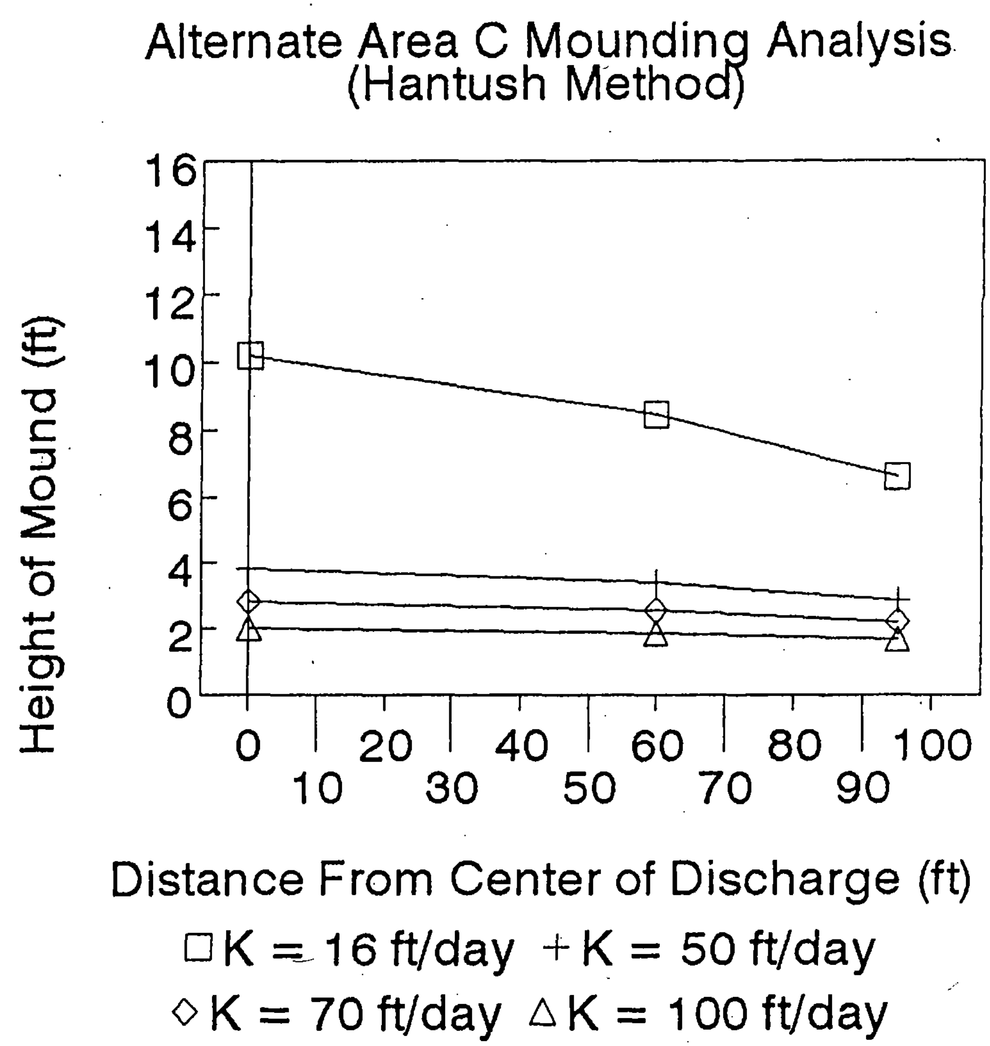
Data for Mounding at Alternate Area C

K (ft/day)	Height of Mound Generated Bed Center (0,0)	Slope (0,60)	SDA Floor (0,95)
	0	60	95
16	10.21	8.45	6.62
50	3.83	3.38	2.87
70	2.81	2.54	2.20
100	2.02	1.86	1.67

Data Inputs	
Saturated Thickness	17 ft
Porosity	0.25 dim
Discharge Volume	30 gpm
Time Elapsed	365 days
Bed Length	70 ft
Bed Width	167 ft
Initial Water Table	0 ft

Critical Mound Height Calculation	
Ground Surface Elev.	416.50 ft msl
Structure Depth	-5.33 ft
Structure Bottom Elev.	411.17 ft msl
Seasonal High GW Elev	-407.00 ft msl
Max. Allowable Mound	4.17 ft

Structure Depth Basis	
Frost Protection Depth	4.50 ft
Pipe Size (4-inch)	0.33 ft
Bottom Stone Depth	0.50 ft
Structure Depth	5.33 ft



8.2.1 Latham Brook Discharge

The location selected for discharge to Latham Brook, along with the piping route proposed to convey treated groundwater from the GWTS to the brook, is presented in Figure 8-4. Based on this route, an estimated discharge pipeline profile was developed, and it was determined that gravity discharge is feasible. The profile is presented in Figure 8-5, and a conceptual design of the outfall structure is presented in Figure 8-6. Based on the proposed GWTS location, the minimum head available at all times is approximately 413 feet, with the stream discharge location at approximately 405 feet.

As shown in Figure 8-4, the proposed piping run exits the GWTS toward the site access road, follows the road to avoid disruption of the tire storage area, bears east at the fork, exits the road just before the rise at the eastern edge of the figure, and will be routed to the brook as shown. Following this route, it is expected that the required cover for frost protection could be maintained for the entire length of the pipeline without significantly impacting the existing contours. Under this scenario, the discharge piping will exit via a concrete outfall structure (shown in Figure 8-6) which would be built into the side of the small hill. The discharge water would then be directed toward the stream utilizing a 25-foot long riprap apron for energy dissipation and to direct the flow into Latham Brook. The estimated length of pipe run for this discharge option is 1300 feet.

8.2.2 Wetlands Discharge

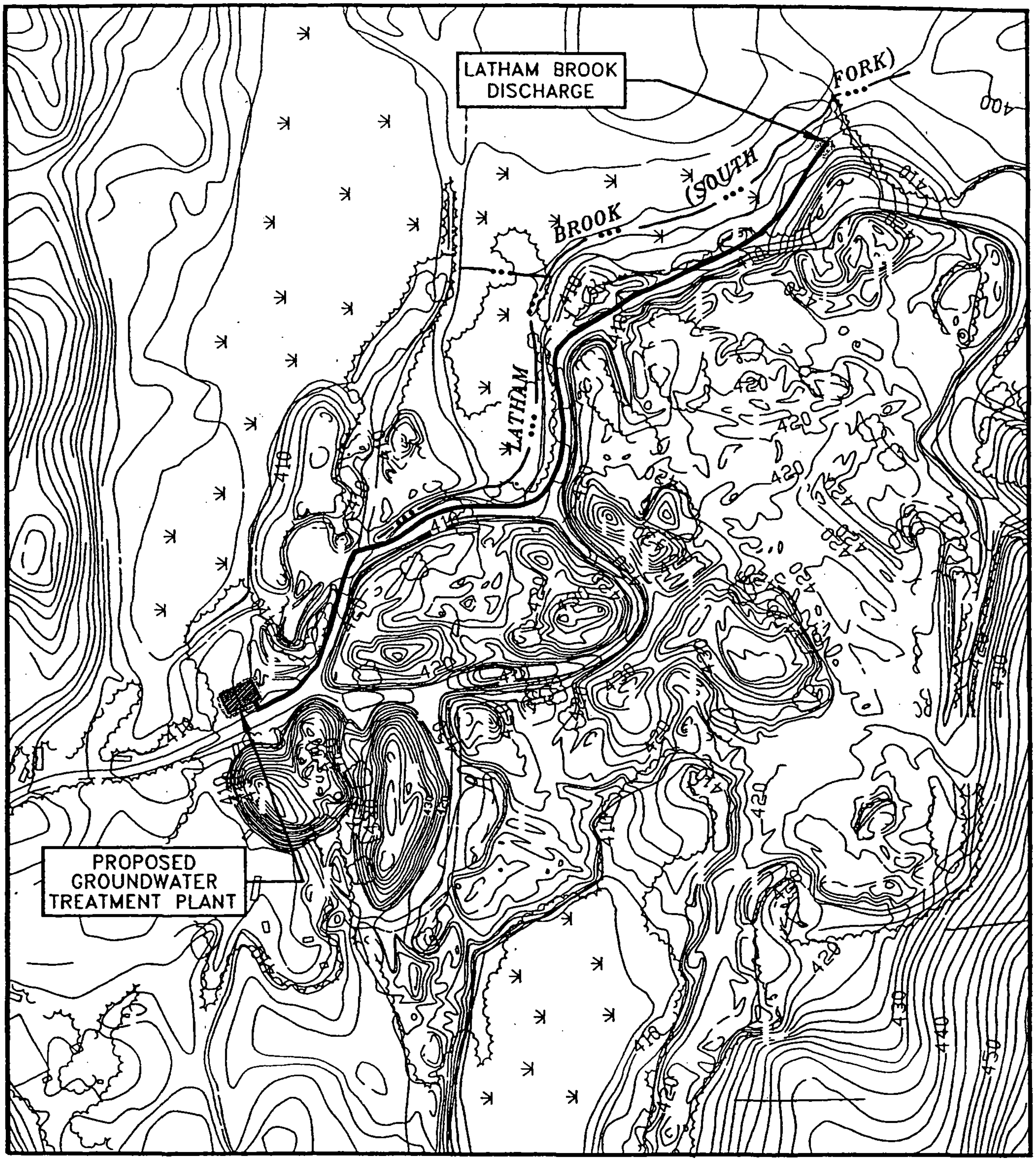
The location selected for discharge to the wetlands and the associated piping route proposed to convey treated groundwater from the GWTS to the wetlands discharge area are presented in Figure 8-7. As with the discharge to Latham Brook, a discharge piping profile has also been developed and is presented in Figure 8-8. The outfall structure for this option will be similar to the Latham Brook structure, as conceptually presented in Figure 8-6. The elevation difference between the GWTS, at approximately 413 feet, and the discharge point, at approximately 405 feet, is similar to that for the Latham Brook discharge point. The estimated length of piping, estimated at 750 feet, is less than the Latham Brook piping.

As with the brook discharge piping, the wetlands piping follows the site access road, bears north at the fork, and ends prior to the existing culvert under the access road. At this point, a discrete channel exists in the wetlands, into which the GWTS discharge will exit.

It is anticipated that discharge to this location will also be by gravity flow. Although the section of the site along which this piping is routed is relatively flat, it is expected that the head in the discharge storage tank will be sufficient to sustain the discharge flow. However, due to the existing contours, it will be difficult to maintain the 4'-6" depth of cover necessary for freeze protection near the outfall structure. Therefore, the design for this portion of the discharge line should consider freeze protection measures for the pipe, the acceptable vehicle loading, and potential for frost heave.

8.3 Evaluation of Discharge Options

Each of the criteria utilized in the evaluation is described, and the ability of each discharge location to meet the desired objectives is discussed in this section. The two discharge alternatives are similar; therefore, their evaluation against the established criteria is also similar. The evaluations are presented together in the following sections, with any significant differences noted.



KEY:

PROPOSED PIPING RUN

0 100 200 300 400

SCALE IN FEET

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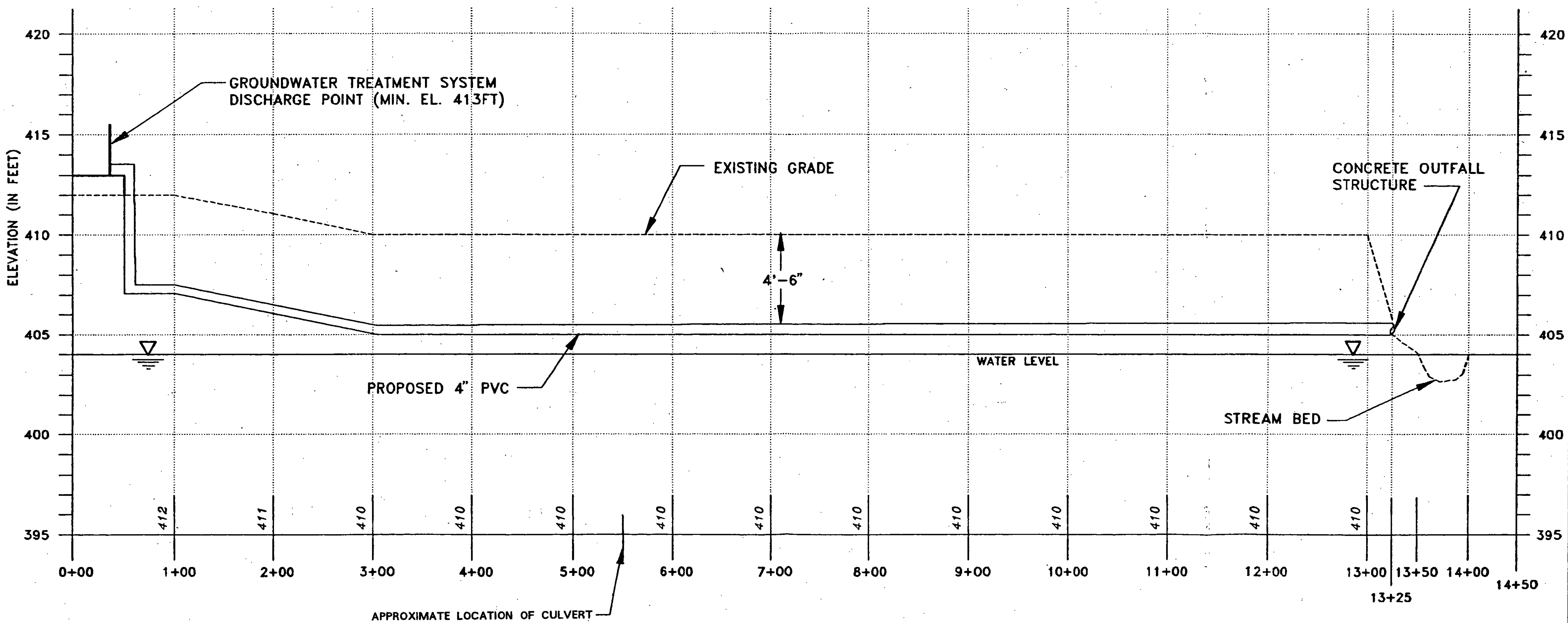
DAVIS LIQUID WASTE SITE
SMITHFIELD, RHODE ISLAND

PIPING ROUTE
AND DISCHARGE LOCATION
FOR LATHAM BROOK DISCHARGE

CONTRACT NO. DACW33-91-D-0005

FIGURE 8-4

DWG:DS018



SCALE: 1" = 100'(HORZ.)
1" = 5'(VERT.)

NOTES:

1.) BEDROCK ELEVATIONS ≤ 395 FEET FOR ENTIRE LENGTH OF PIPE TRENCH. GROUNDWATER ELEVATION 404 FEET ± 0.5 FEET.

2.) FOR STATION LOCATIONS SEE FIGURE 8-1.

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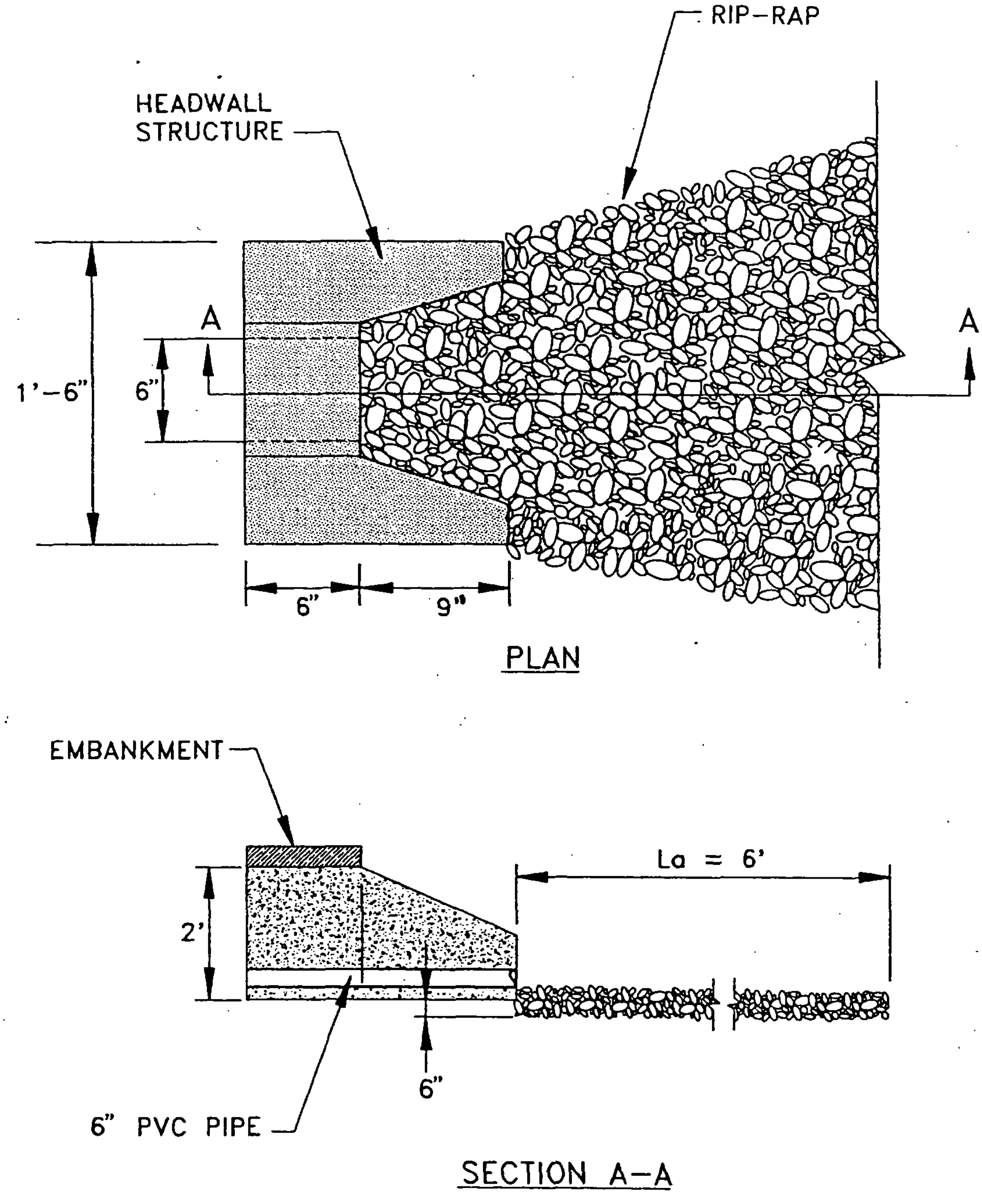
SMITHFIELD, RHODE ISLAND

DISCHARGE PIPELINE PROFILE
LATHAM BROOK
DISCHARGE OPTION

CONTRACT NO. DACW33-91-D-0005 | FIGURE 8-5

DWG:DS022



- NOT TO SCALE -

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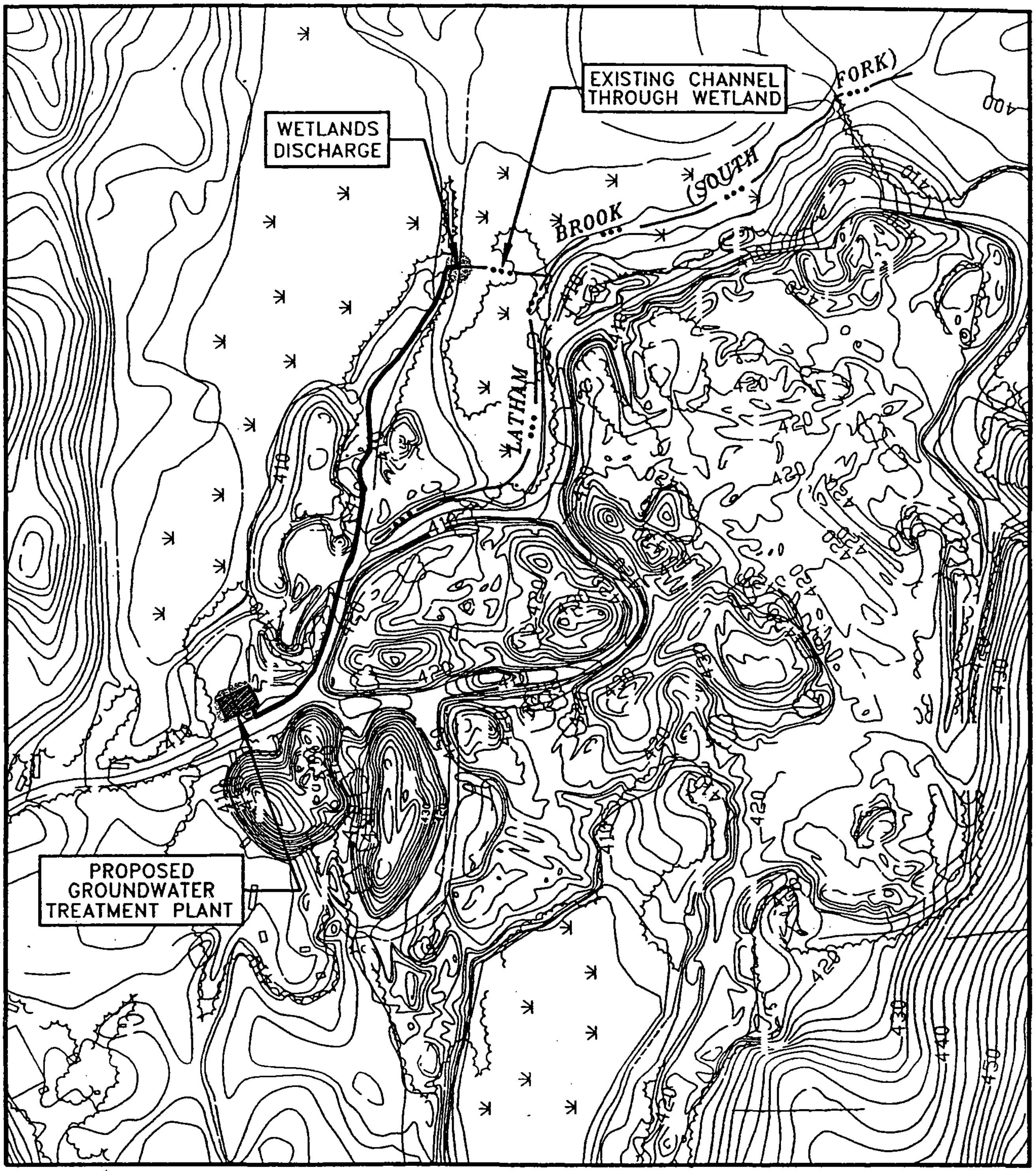
U.S. ARMY CORPS OF ENGINEERS DAVIS LIQUID WASTE SITE SMITHFIELD, RHODE ISLAND
TYPICAL DISCHARGE STRUCTURE
CONTRACT NO. DACW33-91-D-0005 / FIGURE 8-6

DWG:DS024



KEY:

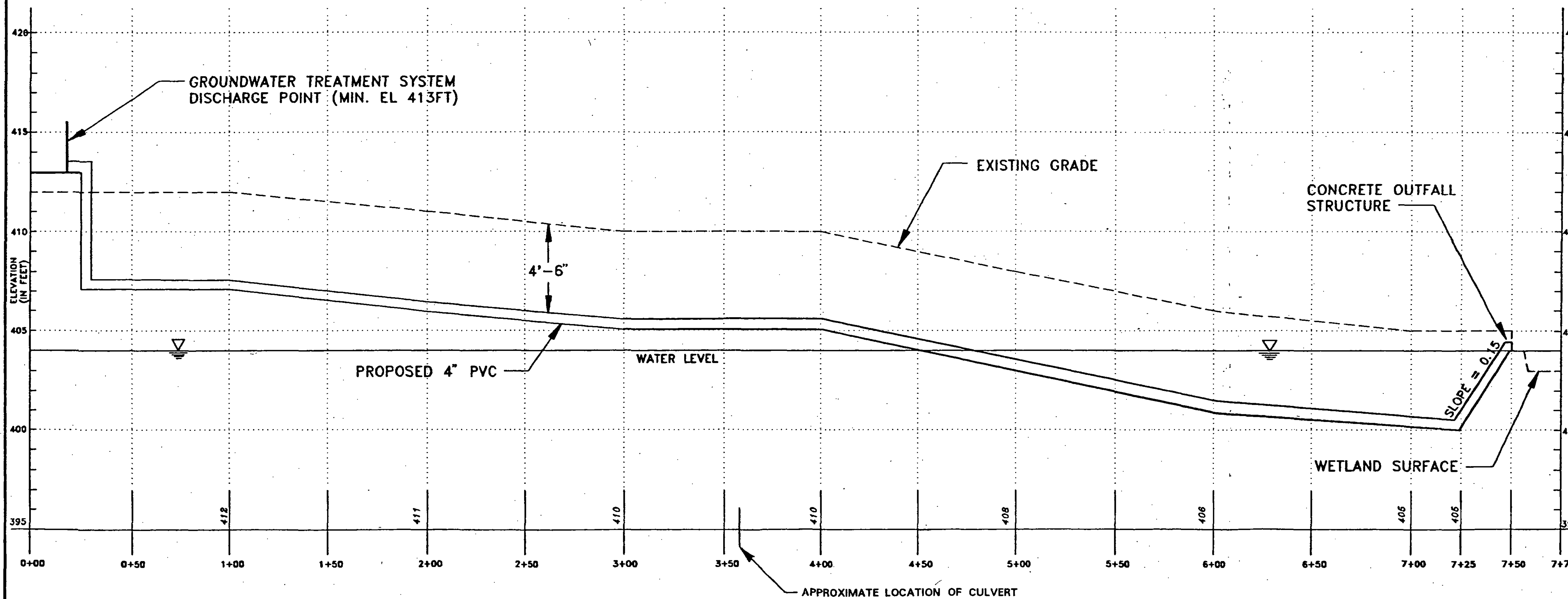
PROPOSED PIPING RUN

0 100 200 300 400

SCALE IN FEET

EBASCO SERVICES INCORPORATED	
U.S. ARMY CORPS OF ENGINEERS DAVIS LIQUID WASTE SITE SMITHFIELD, RHODE ISLAND	
PIPING ROUTE AND DISCHARGE LOCATION FOR WETLANDS DISCHARGE	
CONTRACT NO. DACW33-91-D-0005	FIGURE 8-7

DWG:DS019



SCALE: 1" = 50'(HORZ.)
1" = 5'(VERT.)

NOTES:

1.) BEDROCK ELEVATION $\leq$ 395 FEET FOR ENTIRE PIPE TRENCH.
GROUNDWATER ELEVATION 404 FEET $\pm$ 0.5 FEET.

2.) FOR STATION LOCATIONS SEE FIGURE 8-1.

DWG:DS021

EBASCO SERVICES INCORPORATED
U.S. ARMY CORPS OF ENGINEERS DAVIS LIQUID WASTE SITE SMITHFIELD, RHODE ISLAND
DISCHARGE PIPELINE PROFILE WETLANDS DISCHARGE OPTION
CONTRACT NO. DACW33-91-D-0005 — FIGURE 8-8

The evaluation criteria are:

- Treatment Requirements. This criteria evaluates the ability of each discharge option to meet the anticipated treatment goals, and assesses the difference in treatment methods between them.

- Operations and Maintenance Requirements. The need for operator attention and GWTS system O&M is evaluated with this criterion.

- Protection of Human Health and the Environment. This criterion evaluates the potential impact of the discharge options on the wetlands and the 100 year flood plain.

- Implementability. The ability of the option to be constructed, given the existing physical site constraints, is assessed.

- Regulatory Feasibility. The ability of the option to meet the regulatory guidelines is evaluated with this criterion.

- Cost Effectiveness. This criterion examines the cost differential between the discharge options.

8.3.1 Treatment Requirements

An evaluation of the treatment requirements for each discharge option was performed to determine if the requirements are achievable and to develop a conceptual treatment system design approach which would achieve these goals. Based on published Rhode Island regulations and additional guidance provided by Rhode Island regulators, expected discharge requirements for the two proposed surface water discharge locations were developed. Although both locations discharge to surface, the wetlands discharge area is defined as a Class A water body, while Latham Brook is a Class B water body. These classifications can impact the required treatment criteria, and each are discussed in the following sections. Under this criteria, the receiving water body which is regulated by the least stringent discharge effluent quality criteria would be preferable since the treatment requirements would be reduced.

8.3.1.1 Chemical Discharge Criteria

The Rhode Island regulations (RIDEM, 1988b) state that "Waters shall be free from chemical constituents in concentrations or combinations which could be harmful to human, animal, or aquatic life for the appropriate most sensitive and governing water class use or unfavorably alter the biota." These regulations do not differentiate between Class A and Class B water bodies, and therefore, the chemical discharge criteria will be the same for both the brook and wetlands discharge locations. The criteria which define this concentration are provided in the RIDEM Ambient Water Quality Guidelines (Appendix B of the Water Quality Regulations for Water Pollution Control). These guidelines provide criteria for some priority pollutants. However, for the priority pollutants for which guidelines are not provided, the Method Detection Limit is required by the Rhode Island Department of Environmental Management "unless the discharger demonstrates to the satisfaction of the Director that a higher concentration will not adversely effect the most sensitive use of the water body". Additionally, the regulations state that "The limits prescribed by the United States Environmental Protection Agency will be used where not superseded by more stringent State requirements". This criteria is utilized for the metals Aluminum and Iron, which are not regulated by RIDEM. Chemical discharge criteria have been developed based on the above approach and are summarized in Tables 8-1, 8-2, and 8-3 for volatile organics, semivolatile organics, and metals respectively. These tables include the projected influent concentrations as calculated with the mass balance approach presented in Section 5.0. Table 5-4 presents estimated influent

Table 8-1

Davis Projected Volatile Treatment Requirements
Surface Water Discharge
Ambient Water Quality Criteria and RIDOH Detection Limits

COMPOUND	ESTIMATED INFLUENT ug/l	DISCHARGE STANDARD ug/l	DISCHARGE CRITERIA ug/l (1)	REFERENCE (2)
Acetone	644			
Benzene	55	5.9	4.72	A
Bromodichloromethane	0			
Bromoform	0			
Bromomethane	0			
MEK/2-Butanone	333			
Carbon disulfide	1			
Carbon tetrachloride	0			
Chlorobenzene	4	18	14.4	A
Chloroethane	7			
Chloroform	376	32	25.6	A
Chloromethane	0			
Dibromochloromethane	0			
1,1-Dichloroethane	142			
1,2-Dichloroethane	2	131	104.8	A
1,1-Dichloroethene	32	13	10.4	A
1,2-Dichloroethene (cis)	3834			
1,2-Dichloropropane	0			
cis-1,3-Dichloropropene	0			
trans-1,3-Dichloropropene	0			
Ethylbenzene	2410	36	28.8	A
Methylene chloride	132	214	171.2	A
MIBK	8			
Styrene	0			
1,1,2,2-Tetrachloroethane	0			
Tetrachloroethene	856	5.3	4.24	A
Toluene	4462	14	11.2	A
1,1,1-Trichloroethane	2159			
1,1,2-Trichloroethane	2	20	16	A
Trichloroethene	3185	43	34.4	A
Vinyl Acetate	0			
Vinyl Chloride	188			
Xylenes (total)	7929			
4-Methyl-2-Pentanone	59			
3-Chloroethylvinylether	0			

(1) Discharge Criteria = Discharge Standard * Allocation Factor (0.8)
(2) Reference standards were selected based on the following priority:
A) RI Ambient Water Quality Criteria (Chronic);

NOTE: Limits are listed for Priority Pollutants only.

Table 8-2

Davis Projected Semivolatile Treatment Requirements
Surface Water Discharge
Ambient Water Quality Criteria and RIDOH Detection Limits

COMPOUND	ESTIMATED INFLUENT ug/l	DISCHARGE STANDARD ug/l	DISCHARGE CRITERIA ug/l (1)	REFERENCE (2)
Phenol	8.0	5.6	4.48	A
bis(2-Chloroethyl) ether	4.0	14.0	11.2	A**
1,3-Dichlorobenzene(m)	1.0	8.7	6.96	A
1,4-Dichlorobenzene(p)	14.0	1.2	0.96	A
Benzyl Alcohol	2.0			
1,2-Dichlorobenzene(o)	137.0	1.8	1.44	A
2-Methylphenol	6.0			
4-Methylphenol	13.0			
Isophorone	7.0	130.0	104	A
2-Nitrophenol	4.0			
2,4-Dimethylphenol	10.0			
Benzoic Acid	224.0			
1,2,4-Trichlorobenzene	24.0	1.7	1.36	A
Naphthalene	32.0	2.6	2.08	A
4-Chloroaniline	1.0			
2-Methylnaphthalene	6.0			
Dimethylphthalate	3.0	37.0	30	A
Pentachlorophenol	11.0	0.10	0.08	A
Di-n-butylphthalate	6.0		96000	A**
Butylbenzylphthalate	8.0	1.9	1.52	A
Bis(2-ethylhexyl)phthalate (3)	12.0	12.0	9.6	A

(1) Discharge Criteria = Discharge Standard * Allocation Factor (0.8)
(2) Reference standards were selected based on the following priority:
A) RI Ambient Water Quality Criteria (chronic limit);
A**) RI - Human Health Criteria;
(3) Typically considered a laboratory contaminant.

NOTE: Limits are listed for Priority Pollutants only.
Only those semivolatiles historically detected at the Davis Site are included in this list.

Table 8-3

Davis Projected Total Metals Treatment Requirements
Surface Water Discharge
Ambient Water Quality Criteria and RIDOH Detection Limits

COMPOUND	ESTIMATED INFLUENT ug/l	DISCHARGE STANDARD ug/l	DISCHARGE CRITERIA ug/l (1)	REFERENCE (2)
Aluminum	34414	87	69.6	C
Antimony	11	10	8	A
Arsenic (III)	18		1.12	A**
Barium	801			
Beryllium	3	0.2	0.16	A
Cadmium	11	0.3	0.24	A*
Calcium	18701			
Chromium (III)		53.81	43.05	A*
Chromium (VI)		11	8.80	A
Chromium (Total)	38	64.8	51.84	A
Cobalt	40			
Copper	79	2.3	1.84	A*
Iron	62665	1000	800	C
Lead	23	0.3	0.24	A*
Magnesium	14302			
Manganese	4335			
Mercury	2	0.012	0.5	B
Nickel	52	30.96	24.8	A*
Potassium	23023			
Selenium	0	35	28	A
Silver	0	0.0033	0.2	B
Sodium	12585			
Thallium	1	1	0.8	A
Tin	1			
Vanadium	82			
Zinc	570	20.8	16.64	A*

(1) Discharge Criteria = Discharge Standard * Allocation Factor (0.8)
(2) Reference standards were selected based on the following priority:
A) RI Ambient Water Quality Criteria (Chronic);
A*) RI Ambient Water Quality Criteria (Chronic), adjusted for Hardness = 14.6 mg/l
A**) RI - Human Health Criteria;
B) RIDOH Method Detection Limit (Amended 12/24/92).
C) Federal Ambient Water Quality Criteria (Chronic);

NOTE: Limits are listed for Priority Pollutants only.

concentrations for both total and dissolved metals based on the results of the most recent WCC sampling event. Criteria for certain metals are required to be developed based on the hardness of the receiving water body, which was calculated to be 14.67 mg/l in PDER II (Woodward Clyde, 1993b). Discussions with RIDEM have confirmed that there will be no discharge limits for non-priority pollutants except those regulated by EPA (i.e., Aluminum and Iron) (Liberti, 1993 and 1994).

Discharge limits for a surface water are established by multiplying the above discharge criteria by an allocation factor for the particular stream, and by a dilution factor developed based on the 7Q10 flow (the one-in-ten-year seven-consecutive-day low flow) of the receiving water body. For discharges to Latham Brook, however, the base flow of the stream is so low that no dilution factor will be allowed. RIDEM (Liberti, 1994) has indicated that an allocation factor of 0.8 will be applied to the discharge. The listed discharge criteria will be the discharge limits for both Latham Brook and the wetlands discharge locations.

The review of the Rhode Island regulations which regulate discharge requirements (RIDEM, 1988b) resulted in the establishment of the expected discharge criteria presented above. As noted, there is no difference in the RIPDES chemical criteria to discharge to Class A (the site wetlands) or Class B (Latham Brook) waterbodies for priority pollutants. Therefore, the two chemical contaminant reduction treatment requirements for the discharge options would be identical.

For pH, the regulations (RIDEM, 1988b) differentiate between Class A and Class B water bodies. The pH requirements are more stringent for discharge to a Class A water body (the wetlands), than a Class B water body (Latham Brook). Class A waters are required to maintain the pH "as naturally occurs", while Class B waters are allowed a range of 6.5 to 8.0 S.U. (standard units) or "as naturally occurs." The pH of the discharge effluent for each option will need to comply with the above requirements. Therefore, the pH of the wetlands discharge must be "as naturally occurs", which for the proposed wetlands discharge location has been recently measured in the range of 6.0 to 6.3 S.U. (see Table 2-3, Electrofishing Survey Data, Stations 3 and 4. Note that Stations 1 and 2 are ponded areas which are not representative of the receiving water body at the wetlands discharge location). The allowable pH for the Latham Brook discharge is 6.5 to 8.0 S.U. or "as naturally occurs". Electrofishing data indicate that the Latham Brook pH is also in the range of 6.3 S.U., resulting in an allowable pH range of 6.3 to 8.0. Because the proposed GWTS will raise the pH to approximately 8.5 for the second stage to cause precipitation of metals, then lower the pH with acid to achieve discharge requirements, the wetlands discharge option will require the use of more acid to achieve the lower discharge limit for pH. Therefore, the Latham Brook discharge location is preferable for this criteria.

8.3.1.2 Thermal Impact

The Rhode Island regulations (RIDEM, 1988b) also establish a limit for "allowable temperature increases". For Class A water bodies the limit is "None other than of natural origin", while for Class B "only such temperature increases that will not impair any usages specifically assigned to this Class" are allowed. This requirement is more specifically defined by the statement "the temperature increase shall not raise the temperature of the receiving waters above the recommended limit on the most sensitive receiving water use, and in no case exceed 83°F. In no case shall the temperature of the receiving water be raised more than 4 F°." Additional limits are placed on designated cold water habitats, however neither Latham Brook nor the wetlands are designated as cold water habitats.

Utilizing the watershed subbasin areas and flow rates provided in the Draft Remedial Investigation Report (CDM, 1986), flow rates were calculated for each month of the year for both the wetlands and Latham Brook. Stream temperature data was obtained from published data on a river in Rhode Island, and the effluent discharge temperature was estimated at 55°F based on a groundwater temperature of 52°F and a temperature increase of 3 F° through the treatment system. Utilizing the mixing formula, and discharge flow rates of 30 gpm (average flow rate) and 50 gpm (a conservative maximum flow used at the time of this calculation), the combined receiving water body temperatures were calculated for both the wetlands and Latham Brook discharge locations. The formula and complete data are included in Appendix K.

These effects were plotted, and are presented in Figures 8-9 and 8-10 for the wetlands and Latham Brook respectively. These figures are identical, since the estimated stream and wetland temperature and flow are the same. It is evident that temperatures at the discharge point will be elevated during cold weather months, while the receiving water body temperature will decrease during warm weather. With either the maximum (estimated at 50 gpm) or average (30 gpm) discharge, the temperature increase in the stream would be greater than 4 F° at the discharge location for both the wetlands and Latham Brook discharge points only in the winter months (December through April for 50 gpm, and December through March for 30 gpm), as calculated with a mixing formula (See Appendix K).

Following the initial evaluation of the thermal impact, the downstream impacts were determined. The Statement of Work requested that "thermal dispersion" modeling be performed to determine the extent of the impact on the downstream reaches of the brook. However, based on the low flow rates in both Latham Brook and the wetlands, and the fact that the wetlands discharge would be into a discreet channel, it was determined that a thermal dissipation calculation would be more appropriate. The methodology used for the calculations is included in Appendix K, and was based on methods provided by an EPA Water Quality Assessment document (EPA, 1985).

Two downstream locations were selected for thermal evaluation. The first was the confluence of the South Fork of Latham Brook and the flow from Nipsachuk Swamp (North Fork) (Station #6 of Figure 2-9), representing the greatest additional flow into the brook on site and could potentially return the temperature to the required level through mixing. However, initial calculations did not support this. The second downstream location was at the limits of electrofishing shown in Figure 2-9 (Station #8). No fish were located downstream in Latham Brook to that point. Plots summarizing the results of these evaluations at the limits of electrofishing are provided in Figures 8-11 and 8-12. The results indicate that:

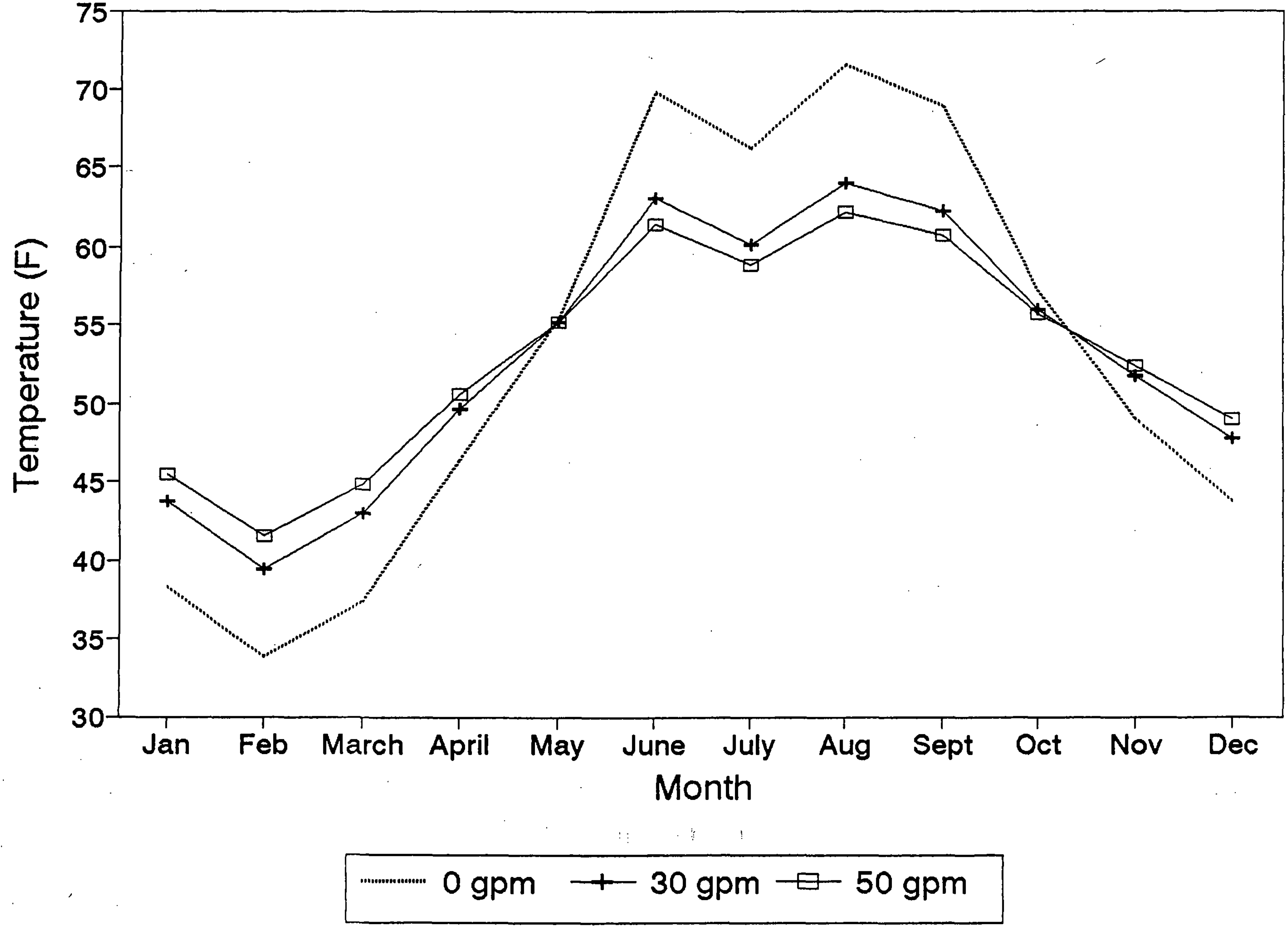
- With either the maximum (estimated at 50 gpm) or average (30 gpm) discharge, the temperature increase in the stream would be less than 4 F° at the limits of electrofishing for both the wetlands and Latham Brook discharge points throughout the year.
- Base minimum flow periods are expected to occur only in the months when the discharge temperature is less than the stream temperature, therefore no temperature increase will occur during base minimum flow periods.
- Since the discharge temperature will remain constant throughout the year, the discharge will tend to moderate the stream temperature. Stream temperatures will decrease in the summer, and increase in the winter.

The following sections discuss the thermal impact on the receiving waters at each of the discharge locations.

8.3.1.2.1 Latham Brook Thermal Impact

The treated water discharge could have some thermal effects on Latham Brook and adjacent wetlands immediately downstream of the discharge location. The thermal modeling of the surface water discharge predicted that the point of discharge would have a water temperature of 55°F. The estimated average monthly ambient temperatures in Latham Brook range from 33.8°F to 71.6°F. With the maximum discharge (modeled at 50 gpm), the Latham Brook temperatures would be from 41.5°F to 62.2°F. The

Figure 8-9 Thermal Effects of Wetlands Discharge
Temperature Profile at the Discharge Point



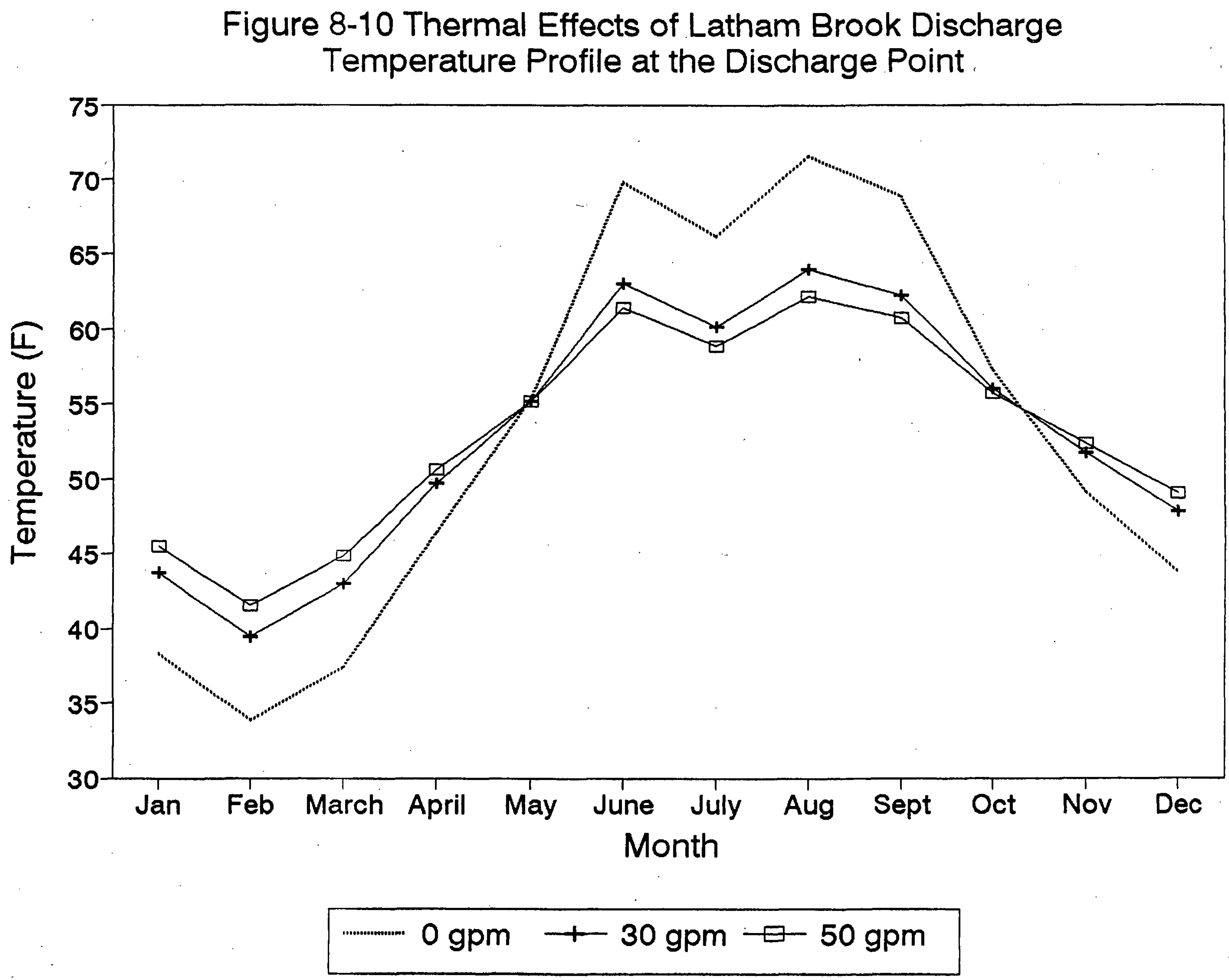
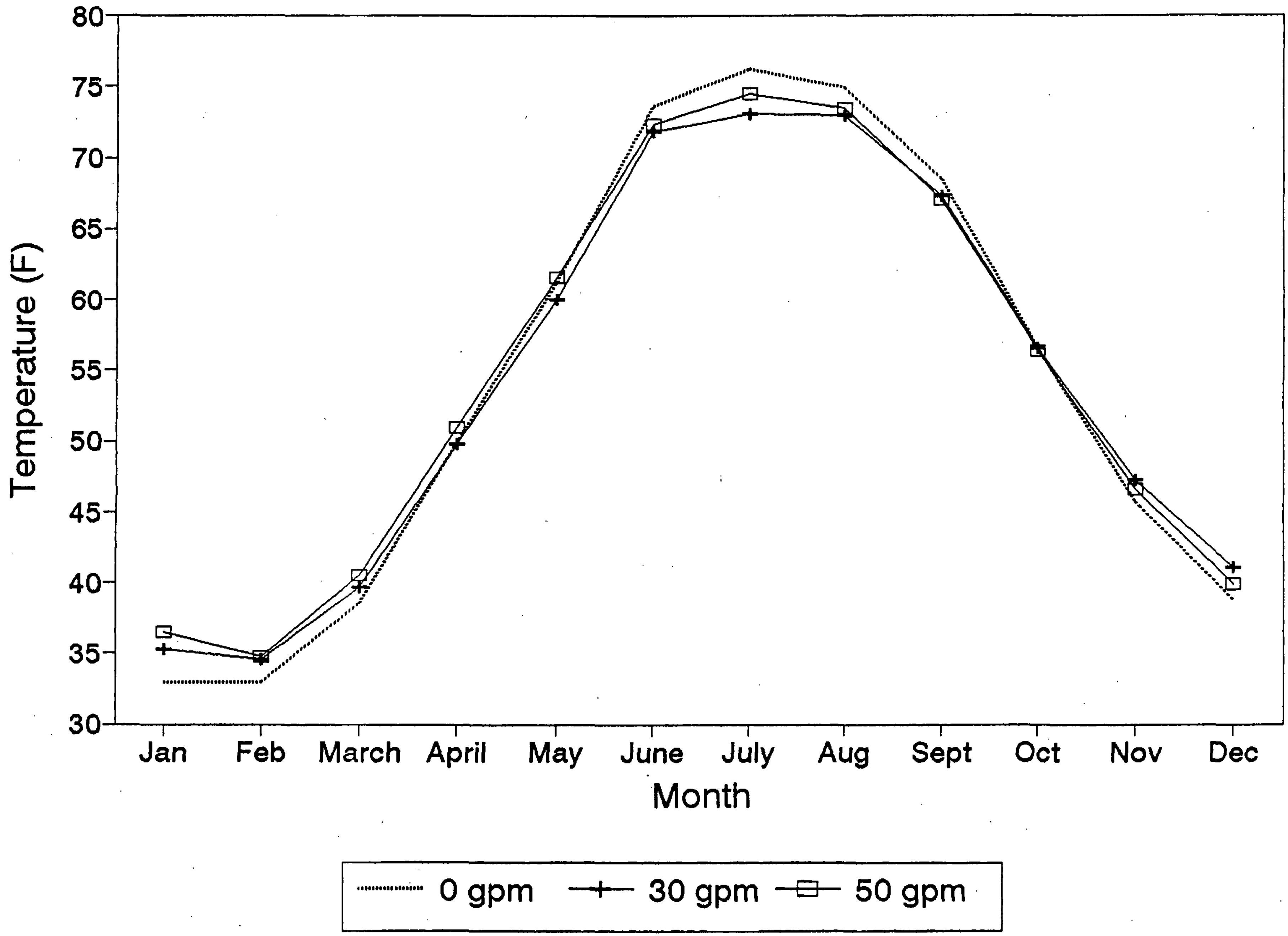


Figure 8-10 Thermal Effects of Latham Brook Discharge
Temperature Profile at the Discharge Point
Temperature (F)
75
70
65
60
55
50
45
40
35
30
Jan
Feb
March
April
May
June
July
Aug
Sept
Oct
Nov
Dec
Month
0 gpm
30 gpm
50 gpm

Figure 8-11 Thermal Effects of Wetlands Discharge
Temperature Profile at the Limits of Fish Shocking



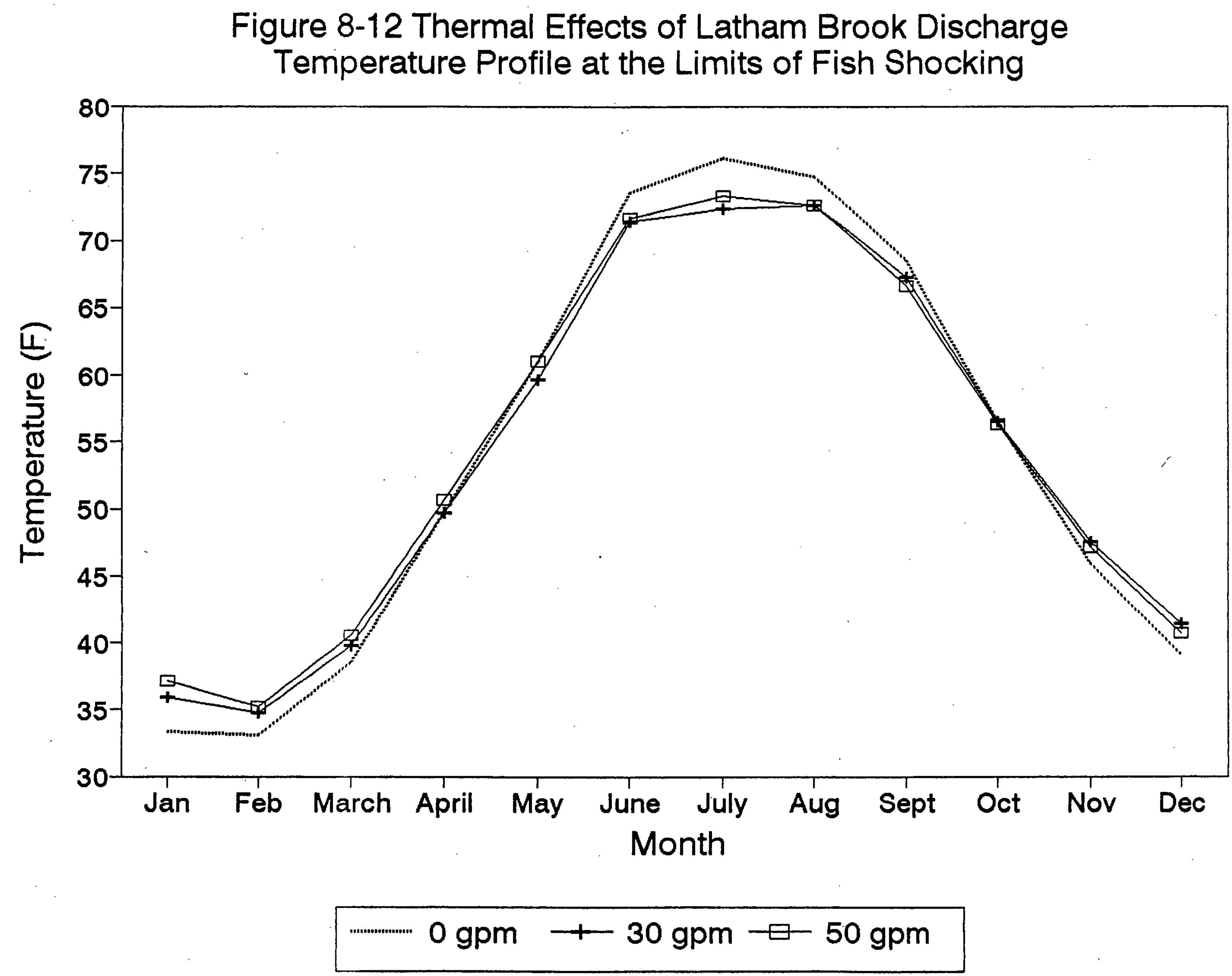


Figure 8-12 Thermal Effects of Latham Brook Discharge
Temperature Profile at the Limits of Fish Shocking
Temperature (F)
80
75
70
65
60
55
50
45
40
35
30
Jan
Feb
March
April
May
June
July
Aug
Sept
Oct
Nov
Dec
Month
0 gpm
30 gpm
50 gpm

discharge would result in a predicted average annual temperature increase of less than 4 F° in the brook. The brook downstream of the limit of the fish sampling would continue to approach the ambient water temperatures and not be affected by the discharge into the brook. In the summer the cooler discharge temperatures would potentially reduce the temperature of the brook downstream in the immediate area by a predicted maximum of 9.4 F°. These cooler summer water temperatures would have higher dissolved oxygen (DO) levels than the ambient temperatures. These higher DO levels may benefit aquatic life. Aquatic life favoring cooler water temperatures in the summer would benefit. Aquatic species preferring cooler waters may become more abundant. Additionally, most if not all of the aquatic life presently inhabiting the brook and adjacent wetlands would probably tolerate the cooler water temperatures and continue to exist in these habitats.

In the winter, the discharge into Latham Brook would warm the water temperature by a predicted maximum of 7.7 F° in the immediate area of the discharge point. The warmer water temperatures in the brook and adjacent wetlands downstream in the immediate area of the discharge point would probably not have a measurable effect on aquatic life hibernating or overwintering in these habitats.

8.3.1.2.2 Wetlands Thermal Impact

Similar to the Latham Brook discharge, in the summer, the cooler discharge temperatures in the wetlands would potentially reduce the water temperature of the wetlands and nearby brook downstream in the immediate area of the discharge point by a predicted maximum of 9.4 F°. The slightly cooler summer water temperatures would have higher DO levels which may benefit aquatic life in both the wetlands and Latham Brook. Aquatic life preferring cooler water temperatures in the summer would benefit. Aquatic life preferring cooler waters may become more abundant. However, most, if not all, of the existing wetland and brook aquatic life would tolerate the cooler water temperature regime and continue to exist in these habitats.

In the winter the discharge into the wetlands would warm the water temperature in the wetlands and nearby brook in the immediate area of the discharge point by a predicted maximum of 7.7 F°. The warmer water temperatures would not have a measurable effect on aquatic life hibernating or overwintering in these habitats. The wetlands and brook downstream of the fish sampling limits would continue to approach the ambient water temperatures and would not be affected thermally by the discharge in the wetlands.

8.3.1.2.3 Thermal Impact Summary

Although the RIPDES regulations limit thermal impacts to both Class A and Class B waterbodies, initial discussions with RIDEM have indicated that thermal limitations are only applied when surface water is withdrawn from a water body, heated, and discharged into the water body. Additionally, it could be possible to redefine the thermal mixing zone to extend to the limits of electrofishing. Therefore, the thermal impact is considered equal with discharge to either location.

It should also be noted that discharge to either location will enhance the aquatic habitat in Latham Brook. The continuous discharge of treated groundwater will promote a more steady flow in the brook, particularly during summer low flow conditions.

8.3.1.3 Air Discharge

The proposed Groundwater Treatment System at the Davis will involve the discharge of air from the Air Stripper. With the current scenario, the vapor phase discharge from the Air Stripper will be treated utilizing Activated Carbon or some other emissions control device. Under Rhode Island DEM Air

Pollution Control (APC) Regulation No. 9, paragraph 9.3.1(f) (RIDEM, 1988a), a Minor source permit is required for the construction, installation or modification of "any stationary source which has the potential to increase emissions of a listed toxic air contaminant by greater than the minimum quantity for that contaminant..." Table 8-4 lists the air contaminants expected to be found at the Davis site which are also regulated by APC Regulation No. 9, Appendix A. Of the regulated contaminants, Chloroform, Tetrachloroethene, and Trichloroethene could be present at levels above the limits listed in APC Regulation No. 9 prior to treatment with an emissions control device (such as Vapor Phase Activated Carbon). However, none of the contaminants will be present in concentrations exceeding those in APC Regulation No. 9 following the emissions control device. Clarification was requested of RIDEM concerning whether the increase in emissions was determined before or after the control device. RIDEM responded that the regulation applied to emissions before the control device (Marcarcio, 1994). Note that a limit for Vinyl Chloride, which is difficult to remove with activated carbon, is not included in the list of regulated air toxics.

Clarification of APC No. 9, paragraph 9.3.2(a)(3) was also requested of RIDEM. This paragraph exempts from the provisions of 9.3.1(h) (the need for permitting an emissions control system and appurtenances) systems where "The emissions control system is used to treat emission of air contaminants generated from a groundwater cleanup operation and the emissions control system will reduce the emissions of VOC by at least 95%." This would seem to apply to the Davis site, however, RIDEM stated that this exemption was written to cover gasoline station cleanups which were dealing with small quantities, and did not have the lead time to go through the entire air discharge permitting process. Based on this clarification, RIDEM stated that the exemption would not apply to a facility at a hazardous waste site (Marcarcio, 1994).

If meeting the criteria of a Minor Source permit is required, the regulations in APC No.22, Air Toxics, also apply. Air toxics regulations require that the discharge following the control device must meet the listed levels for "Ambient Air Quality". The Rhode Island Regulations define "Ambient Air Quality" as "the maximum allowable ambient air concentration of a listed toxic air contaminant contributed by a stationary source, at or beyond that facility's property line". The "Acceptable Ambient Levels" for the regulated Air Toxics are listed in Table 8-5. Clarification of the term "property line" was requested, and RIDEM stated that the definition is the legal property boundary, whether or not there are residents within the property boundary (Marcarcio, 1994).

"Acceptable Ambient Levels" are determined through air dispersion modeling, but RIDEM typically would perform a screen model once a permit application has been filed. RIDEM stated that this model will predict the location of maximum impact, and that if this location is outside the property boundary, additional control measures must be taken (i.e., raising the stack height, etc.). The model uses the contaminant concentrations following the emissions control device. Therefore, if it can be determined that the concentrations discharged from the emissions control device are below AALs, no modeling will be required. To decrease the time required to obtain approval to construct, some applicants perform their own air dispersion modeling. Preliminary calculations indicate that the discharge from the Davis emissions control device will be several orders of magnitude greater than the AALs.

In summary, for the situation at the Davis site, the following would apply:

- Since the discharge prior to the air control device (carbon unit or other treatment unit) is expected to exceed the limits listed in Appendix A of APC Regulation No. 9, meeting the criteria of a Minor Source permit will be required;

- Subsequently, since meeting the criteria of a Minor Source permit is required, the regulations in APC No.22, Air Toxics, will also apply. Because the discharge from the

Table 8-4

RIDEM Air Pollution Control Regulation No. 9
Minimum Quantities and Projected Davis Discharge

(1)	Pounds Per Hour (2)	
	RIDEM Limit	Projected Discharge
Acrylonitrile	0.0005	
Aniline	0.04	
o-Anisidine	0.001	
Antimony dust and fumes	1.14	
Arsenic	0	
Benzene	0.005	0.0012
Benzidine	0	
Benzotrichloride	0	
Benzyl chloride	0.001	
Cadmium dust and fumes	0	
Carbon tetrachloride	0.002	
Chloroform	0.002	0.0085
Chromium dust, fumes, and mist	0	
3,3'-Dichlorobenzidine	0.0004	
Dioctyl phthalate (DOP, DEPH)	0.02	
Diphenyl (biphenyl)	0.02	
Diphenylamine	1.14	
Epichlorohydrin	0.04	
Ethylene dichloride (1,2 dichloroethane)	0.002	0.00004
Ethylene oxide	0.0006	
Hydrazine	0	
Hydrogen chloride	1.14	
Hydrogen fluoride	0.1	
Manganese dust and fumes	0.01	
Methyl cellosolve	1.14	
Methylene bisphenyl isocyanate (MDI)	0.003	
4,4'-Methylene bis(2-chloroaniline) (MOCA)	0.05	
Methylene chloride (dichloromethane)	0.1	0.003
Nickel dust and fumes	0.0001	
5-Nitro (o-anisidine)	0.004	
2-Nitropropane	0.01	
Perchloroethylene (tetrachloroethylene)	0.002	0.019
Styrene	1.14	
Toluene	1.14	0.10
Toluene-2,4-diisocyanate (TDI)	0.001	
o-Toluidine	0.002	
1,1,2-Trichloroethane	0.3	0.00004
Trichloroethylene	0.02	0.072
Triethylamine	1.14	
Xylenes	1.14	0.18

(1) ░░░ Indicates Minimum Quantities may be exceeded.

(2) The listed projected discharge assumes 100% volatilization, and was calculated using influent data from Table 8-1 at 45 gpm using the following formula:

$$\text{Concentration (ug/l)} * \text{Flow Rate (gal/min)} * 3.7854 \text{ l/gal} * 2.2 \text{ lb/kg} * 10^{-9} \text{ kg/ug} * 60 \text{ min/hr}$$

Table 8-5

RIDEM Air Pollution Control Regulation No. 22
Acceptable Ambient Air Levels
(ug/m³)

	1 Hour Average	24 Hour Average	1 Year Average
Acrylonitrile			0.07
Aniline		3	
o-Anisidine		1	0.02
Antimony dust and fumes			40
Arsenic			0.0002
Benzene			0.1
Benzidine			0.00002
Benzotrichloride			0.0007
Benzyl chloride	20		0.01
Cadmium dust and fumes			0.0006
Carbon tetrachloride			0.03
Chloroform			0.04
Chromium dust, fumes, and mist			0.00009
3,3'-Dichlorobenzidine			0.002
Dioctyl phthalate (DOP, DEPH)		200	0.5
Diphenyl (biphenyl)		7	0.4
Diphenylamine			200
Epichlorohydrin		200	0.8
Ethylene dichloride (1,2 dichloroethane)			0.04
Ethylene oxide			0.01
Hydrazine			0.0003
Hydrogen chloride	2000	600	
Hydrogen fluoride	30		
Manganese dust and fumes	2		
Methyl cellosolve		100	
Methylene bisphenyl isocyanate (MDI)		0.2	
4,4'-Methylene bis(2-chloroaniline) (MOCA)		1	
Methylene chloride (dichloromethane)			0.2
Nickel dust and fumes			0.002
5-Nitro (o-anisidine)			0.08
2-Nitropropane			0.2
Perchloroethylene (tetrachloroethylene)			0.05
Styrene			30
Toluene		2000	400
Toluene-2,4-diisocyanate (TDI)		0.2	0.03
o-Toluidine			0.04
1,1,2-Trichloroethane			7
Trichloroethylene			0.3
Triethylamine		300	20
Xylenes		700	

emissions control device is not expected to meet the AALs at the end-of-pipe, it will be necessary to perform air dispersion modeling to determine the ambient air quality at the property line.

Since there is no difference in air discharge requirements, and no difference in the quality of air emissions to be treated for either the stream or wetlands discharge option, the two discharge locations are considered equal for air discharge requirements.

8.3.2 Operations and Maintenance Requirements

The envisioned GWTS design will include a control and autodialer alarm system which will allow for the facility to operate with minimal operator attention and the discharge piping/structures are anticipated to require no operator attention other than monitoring and sampling. It is not expected that a full-time operator will be present on-site, but an operator will periodically visit the facility to perform routine maintenance and respond to alarms. The routine activities would include mixing batches of chemicals which feed into the precipitation/flocculation unit, maintaining the mechanical equipment, monitoring and adjusting the controls, and performing routine sampling.

Consistent with the treatment requirements, there are minimal differences between the operations and maintenance requirements for the two discharge options. The proposed treatment system is identical for both discharge locations. The more stringent pH requirements for the wetlands discharge will result in minimally higher chemical usage. However, other than a minor cost, there will be no difference in the O&M implementation requirements.

8.3.3 Protection of Human Health and the Environment

As presented in Section 8.3.1, with proper treatment, the required discharge limits will be met equally by both discharge alternatives; therefore, they will be equally protective of human health and the environment from the chemical discharge perspective. This section evaluates the hydraulic and construction impacts of the discharge on both the wetlands and 100-year flood plain for each alternative. Additional criteria, such as short term worker risk, noise, and proximity to residences was assumed as equal between the options and will not be discussed.

8.3.3.1 Wetlands Impact

Construction of the GWTS will occur outside both the delineated wetlands and the 50-foot perimeter wetland. Most of the discharge piping installation will occur outside of the delineated wetlands, but within the 50-foot perimeter wetland as defined by RIDEM. Some excavation in Wetland A will be required to place the discharge pipe and outfall structures for the wetland discharge location, while the outfall structure for the Latham Brook discharge location would be outside the delineated wetlands, but within the 100-foot riverbank wetland of the brook (which is less than 10 feet wide) as defined by RIDEM (RIDEM Freshwater Wetlands Act, Appendix 4, paragraph D.2(c)). The wetland discharge option excavation would result in minor disturbance of the wetland and localized and temporary increases in turbidity in the wetland during the excavation and backfilling of the discharge pipe trench. These would likely be considered "Insignificant Alterations". Implementation of standard erosion and sedimentation control methods would minimize construction impacts to the wetlands. Minimal wetland construction impact would occur during placement of the riprap apron at each location.

The maximum flowrate of the surface water discharge is expected to be 45 gpm. For the wetlands discharge option, the increased water flow would result in slightly increased water levels in Wetland A downstream in the immediate area of the discharge location. However, no significant impact on the vegetation and animal life in the wetlands is expected to occur as a result of the somewhat wetter conditions. No significant increase in the size of Wetland A would result from the increased water levels. Most of the discharge into the wetlands would eventually reach Latham Brook. As a result, there would be an increase in the existing flowrate of the brook. The impact on aquatic life in the brook from either discharge location would be beneficial since there would be a slight increase in aquatic habitat due to additional flow and a year-long sustained flow. The Latham Brook option is preferable due to the lower wetland impact during construction. This option will also require less wetlands restoration (approximately 1250 ft^2) than the wetlands discharge option (approximately 5000 ft^2).

8.3.3.2 100-Year Flood Plain Impact

The flow from the GWTS discharge is so low (approximately 0.10 cfs) compared to the 100-year flood stream flow (approximately 100 cfs in the vicinity of the GWTS) that the affect of the discharge on the 100-year flood plain is negligible (less than 0.01 feet) for either the wetlands or the Latham Brook discharge options.

8.3.4 Implementability

The implementability of the discharge option is defined as the technical feasibility of implementing the option. For the evaluation of discharge options this includes an assessment of the availability/reliability of technology, supplies, expertise to install the discharge piping/structure, and the treatment technology necessary to achieve the treatment objectives; an evaluation of the impacts to off-site culverts resulting from the discharges; and an assessment of the constructability of each option.

8.3.4.1 Technology Assessment

The technologies required to design, install, and construct the discharge piping and outfall structure are proven and readily available. There are no site constraints which would require the use of specialized construction techniques for either discharge location. The technologies proposed for treatment of the extracted groundwater are proven, commercially available technologies. Operated properly, the proposed system will be able to meet the criteria for volatile organics and semivolatile organics, except for Pentachlorophenol and 1,2,4-Trichlorobenzene at both discharge locations. In the most recent sampling data, the presence of Pentachlorophenol was noted only in one duplicate sample from one well during the 1991 WCC Predesign sampling. Therefore, the probability of Pentachlorophenol being present in the influent should be evaluated prior to finalizing a design basis. 1,2,4-Trichlorobenzene was only slightly above the discharge limit, and should be further evaluated during future sampling and/or startup. Both of these semivolatiles are adsorbable on carbon, but removal to the expected discharge limits could result in more frequent replacement or regeneration of the liquid phase carbon. As discussed in Appendix L, an additional metals removal step may be required to achieve the very low discharge requirements for Arsenic and Mercury, as well as for certain metals such as Cadmium, Chromium, and Lead, if they are present in the soluble form in the influent.

The complete evaluation of inorganic treatment requirements and presentation of the proposed conceptual treatment system is presented in Appendix L. The results of this evaluation indicate that the treatment system would be composed of the following processes:

- Equalization;
- Hydroxide Precipitation;
- Filtration;

- Optional Metals Polishing;
- Air Stripping and Vapor Phase Carbon Adsorption;
- Liquid Phase Carbon Adsorption; and
- pH Adjustment.

Based on the theoretical evaluation of the removal efficiencies and an evaluation of the Woodward Clyde treatability study data for volatile organics, semivolatile organics, and metals it appears that the proposed system, with the assumptions presented in Appendix L, could potentially achieve the discharge limits for regulated organic contaminants, except as noted above.

However, several uncertainties with the projected influent characteristics and projected removal efficiencies exist particularly with respect to metals removal. Two actions are recommended to reduce these uncertainties. First, the possibility of relaxing the proposed discharge limits should be investigated, since much of the uncertainty is associated with achieving the very low projected discharge requirements. Specific metals for which relaxed discharge requirements would be desirable include Arsenic, Beryllium, Cadmium, Chromium, Copper, Lead, Mercury, Thallium, and Zinc. Relaxation of the limits for Arsenic, Beryllium, Cadmium, Chromium, Copper, Lead, Mercury, and Thallium could eliminate the need for a polishing step. Relaxation of the limits for Zinc could result in the elimination of the need for sulfide precipitation.

Additionally, bench scale studies should be considered prior to proceeding with a full scale design. Uncertainties associated with the ability to meet the low discharge requirements, speciation of metals, the interaction between the contaminants, the effectiveness of the proposed coagulants and flocculants, and determination of design parameters such as mixing rates and settling times could be obtained from such studies. It would also be possible to determine the need for sulfide precipitation and/or the optional polishing step.

Both discharge options will require the same treatment and are considered equally technically feasible. Should the limits at one of the discharge locations be relaxed due to higher background levels, this assessment may change.

8.3.4.2 Culvert Impact

The off-site culverts downstream from the discharge area already impacted by flood flows including the 2-year event (Section 2.9). The impact of the proposed discharge flowrate on the downstream culverts is considered insignificant relative to the flood flows. Although, some dissipation of flow may occur under the wetland discharge option (when the wetland is in a recharging mode), this will be minimal, since the proposed wetlands discharge is into a discrete channel which directly feeds into Latham Brook. The additional flow into Latham Brook will, therefore, be the same for both discharge options. Thus, any impact on downstream culverts is considered equal for each discharge option.

8.3.4.3 Constructability

The constructability criterion evaluates the physical constraints under which the alternative must be constructed. For the discharge options at the Davis site, several of these constraints result from the physical limitations imposed by current land use on the site. Additional limitations may be imposed by the depth of bedrock along the piping runs on site, and the elevation of the groundwater table. A comparison of each alternative with respect to constructability is presented in the following sections.

8.3.4.3.1 Physical Limitations Imposed by Current Land Use

The land available for implementation of a remedial action at the Davis site is extremely limited. A significant area is currently used for tire storage, and will not be available for installation of wells, piping systems, the GWTS or support systems. Additionally, areas of heavy contamination (Northern and Southern Disposal Areas) are also not appropriate for implementation of the action. The remaining site areas consist of either heavily wooded areas, wetlands, or site access roads. The layouts for the discharge piping were limited to routes along the site access roads to minimize required clearing and grubbing, minimize wetlands impact, and avoid disruption of current land use. It is assumed that these constraints will not be altered.

Each of the options can be accomplished with minor impact on current land use. Each of the piping routes avoids both the tire storage piles and the contaminated areas on site. Because each route follows site access roads, it is likely that these roads may be closed to traffic during a portion of the trenching and pipe installation. Although traffic is minimal, it is significant to the landowner's current operations. This is particularly true for the wetlands discharge option, since wetlands directly abut the site access road, requiring trenching to occur within the roadway to minimize wetland impact. For the Latham Brook option, although challenging, the trenching could potentially occur along the edge of the site access roads, allowing traffic flow to be maintained.

Additionally, the portion of the wetlands piping that is in the site access road near the outfall structure may have less than the required soil cover as the piping slopes upward to ground surface (see Figure 8-8), and thus will be subject to loads from vehicular traffic and freezing. During design, the allowable loading on this pipe and appropriate freeze protection must be evaluated and protective measures taken where necessary.

Each option will require some clearing and grubbing to remove heavy vegetation along the route. This will be slightly more extensive for the Latham Brook discharge option because a portion of the piping exits the roadway, running toward the brook. It is anticipated that the wetland discharge option will have a greater impact on current site use due to the potential need to close the site access road during construction of the outfall structure and installation of a portion of the piping.

8.3.4.3.2 Groundwater Elevation

Figures 8-5 and 8-8 show profiles of the proposed pipe routes, along with groundwater elevations. The pipe route to the Latham Brook discharge location remains above the groundwater table for its entire length. However, the pipe route for the wetlands discharge location is below the groundwater table for approximately 300 feet of its 700 foot length. Installation of the piping in this area may require significant trench dewatering during installation of the pipeline and construction of the outfall structure. Therefore, the Latham Brook discharge option is more easily constructable with regard to groundwater concerns.

8.3.4.3.3 Bedrock Impact

Neither of the discharge piping routes are expected to require removal of bedrock, based on the bedrock contours provided in the Draft Remedial Investigation for the Davis Liquid Site (CDM, 1986). Therefore, the options are equally implementable from this aspect.

8.3.5 Regulatory Feasibility

The regulatory guidelines which apply to discharges to water and air have been discussed in section 8.3.1. As was stated, the proposed treatment system will be designed to achieve the required chemical limits. The treatment train design will be the same for both the wetlands and Latham Brook discharge locations. The thermal limits at the discharge location cannot be achieved at either discharge location, unless engineering measures are employed or the mixing zone is redefined to extend to the limits of electrofishing. Both locations do achieve compliance at the limits of electrofishing. The wetlands discharge location must be reviewed by the RIDEM Wetlands division in addition to the Water Quality section to assess long term impact to the wetlands from the additional discharge flow. Additionally, wetlands protection guidelines must be followed during construction of the collection piping, GWTS, discharge piping, and outlet structures for each option. ~~Due to the additional review necessary for the wetlands discharge option, the regulatory feasibility of this option is judged to be lower~~ than that of the Latham Brook discharge.

8.3.6 Cost

A detailed presentation of the Comparative Life Cycle Cost Analysis is provided in Section 9.0 and Appendix M. This analysis indicates that with an implementation cost of at least $2 million, the cost differential between the options will be less than 0.1%. This amount is negligible considering the accuracy of estimates based on conceptual design, and ~~the alternatives are determined to be equally cost-effective.~~

9.0 COST ANALYSIS

The Comparative Life Cycle Cost Analysis of the two surface water discharge options was prepared as a "delta" analysis. The surface water discharge to Latham Brook was considered the base case and surface water discharge to the wetlands was considered Alternative 1. Only dissimilar components of each of the discharge options were included. The Groundwater Treatment System can be considered to be comprised of three major components for the purpose of this analysis; 1) Collection system, 2) Treatment system and 3) Discharge system.

The collection system includes costs associated with the construction of the extraction wells and collection piping up to the entrance of the treatment building and the operation and maintenance of the same. The key assumptions for the analysis of this component include the following:

- Construction costs for the extraction wells and collection piping are identical for the base case and Alternative 1.

- Extraction well and collection piping maintenance are identical for the base case and Alternative 1.

The treatment system includes costs associated with the construction of the treatment system building, process equipment, internal piping and electrical instrumentation and controls within the building proper and the operation and maintenance of the same. The key assumptions for the analysis of this component include the following:

- Pretreatment equipment selected is common to both alternatives.

- Filters, backwash water and carbon filter sizing are identical for both alternatives.

- The needs for potable water are identical for both alternatives.

- Identical controls will be utilized for both alternatives.

- Building size will be identical for both alternatives.

- All utility costs are similar for both alternatives.

- There are no piping differences inside the treatment system.

- Operation and Maintenance cost differences between the two alternatives for the treatment system are minimal.

The discharge system includes costs associated with the construction of the discharge piping from the treatment system building and the discharge structures and the operation and maintenance of the same. The key assumptions for the analysis of this component include the following:

- Permitting equivalency costs are considered the same for both alternatives. However, the wetlands' discharge option could require additional regulatory interface due to its identification as a "Class A" water body.

- The surface water discharge to the wetlands will be in the location indicated in Figure 8-4.

- The surface water discharge to the stream will be in the location indicated in Figure 8-7.

These assumptions were used to develop the "Delta" analysis for both capital costs and operation and maintenance costs.

9.1 Capital Costs

In order to identify the construction components which would be included in the "Delta" analysis, the components of each discharge option were reviewed. Based on this review and the assumptions previously described, a list of items was developed utilizing the HTRW Remedial Action Work Breakdown Structure (WBS). Preliminary quantities were established for each of these work items for each alternative. The direct costs for these items were estimated using version 5.20H of Micro-Computer Aided Cost Engineering System (MCACES). The direct cost for each work item for each alternative was compared and a "Delta" cost was established using the discharge to Latham Brook as the base case.

The three major components of the system were analyzed. It was determined that the collection system and the treatment system for each discharge option contained similar work items which would effectively yield insignificant cost difference between the two discharge options. The need for a small amount of additional acid to adjust pH for wetland discharge was the only difference. The third major component of the system, the discharge system, did contain several work items which yield different costs for each discharge option. The differences in direct cost for the two alternatives were estimated for five WBS Level 2 work items, which are listed below including a description of the specific work items.

Table 9-1

Delta Capital Cost

WORK ITEMS xx.xx HTRW Code of Accounts (specific work items)	"DELTA" COST ($) (Estimated cost for Wetlands Discharge Option - Estimated Cost for Latham Brook Discharge Option)
33.02 Monitoring, Sampling, Testing (Laboratory Chemical Analysis - TCLP for excavated soil)	(1,265)
33.03 Site Work (Clearing and Grubbing, Earthwork, Roads and Field Engineering)	(12,760)
33.05 Surface Water Collection and Control (Sediment Barriers - Silt Fence and Straw Bales)	(3,040)
33.06 Groundwater Collection and Control (Trenching, Piping and Fittings, Backfill, Disposal Structure)	704
33.20 Site Restoration (Wetlands Restoration)	14,785
TOTAL "DELTA" COST	(1,576)

The comparative cost differences identified are primarily due to the different discharge locations. ~~The additional distance required to locate the discharge to the stream is the primary difference which causes the four negative "Delta" costs (higher alternative cost) for this option.~~ It should be noted that although the brook discharge piping is approximately 600 feet longer than the wetlands, a portion of the piping for each will be placed in the same trench as the collection piping, effectively reducing the differential. The only positive "Delta" cost (higher base cost) occurs due to the additional wetland construction and restoration work required for the wetland discharge and additional dewatering.

Backup documentation used to develop these comparative costs are contained in Appendix M - Cost Analysis.

9.2 Operation and Maintenance Costs

In order to identify the operation and maintenance (O&M) components which would be included in the "Delta" analysis, the O&M components of each component of the Groundwater Treatment System were reviewed for each discharge alternative. Based on this review and the previously described assumptions, it was determined that the O&M components for the collection and discharge systems contained identical work items which would effectively yield no cost difference between the two discharge alternatives. The third component of the system, the treatment system did contain one difference in O&M components required for the wetlands discharge option.

A preliminary cost was developed for the additional sulfuric acid (H_2SO_4) required to adjust the pH of the wastewater from 8 to 6 within the clearwell tank (T-02) to meet the wetlands discharge criteria. This cost was then estimated for a 5, 10 and 20 year life cycle. A discount factor of 7% was used to calculate the present worth for each of these periods. The table listed below contains the present worth value of the "Delta" cost for additional O&M costs associated with the discharge to the stream.

Table 9-2

Delta Operations and Maintenance Cost

OPERATION AND MAINTENANCE LIFE CYCLE	"DELTA" COST ($) (Present Worth [Wetlands Discharge Option - Latham Brook Discharge Option])
5 Years	512
10 Years	878
20 Years	1,324

The comparative cost differences identified are primarily due to the pH adjustments required to discharge to different water classes. ~~Discharge to the wetlands requires additional sulfuric acid and is therefore the primary difference which causes the "Delta" costs (higher alternative cost) for this option.~~

Backup documentation used to develop these comparative costs are contained in Appendix M - Cost Analysis.

9.3 Summary

Based on the Life Cycle Cost Analysis for the two discharge alternatives it appears that the discharge option to the wetlands is more cost effective on a capital cost basis, while the Latham Brook discharge option is more cost effective on an operations and maintenance cost basis. The above cost differentials are so low (estimated to be less than 0.1% of the total cost) that the differentials are considered negligible.

10.0 RECOMMENDATION OF DISCHARGE OPTION

Table 10-1 has been prepared to summarize the results of the evaluation of discharge options presented in this report. As seen in the table, there is negligible difference between the two options for many of the criteria evaluated. However, the wetlands discharge location has several marginal disadvantages, including:

- The need to meet more stringent Class A discharge requirements, resulting in additional pH adjustment, with a resultant minimally greater O&M requirement;
- A greater impact to the site wetlands during construction activities;
- The need for additional trench dewatering during construction due to pipe being installed below the groundwater table; and,
- Approval may be required from an additional regulatory body (RIDEM Wetlands division) to allow discharge at this location.

Therefore, the Latham Brook discharge option is recommended over the wetlands discharge option.

Table 10-1

Comparison of Discharge Options

Evaluation Criteria	Latham Brook	Wetlands
Treatment Requirements		
Chemical: Priority Pollutants Organics	0	0
Priority Pollutants Inorganics	0	0
pH	+	-
Thermal Discharge Limits	0	0
Air Discharge	0	0
Operations and Maintenance	0	-
Protection of Human Health and the Environment		
Wetlands Impact: Discharge Flow	0	0
Construction	0	-
100 Year Flood Plain Impact	0	0
Implementability		
Treatment Technology	0	0
Culvert Impact	0	0
Constructability: Land Use Constraint	0	-
Groundwater	+	-
Bedrock Impact	0	0
Regulatory Feasibility	0	-
Cost: Capital	0	0
O&M	0	0

Key: 0 No Appreciable Difference
+ Advantage
- Disadvantage

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