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prepared for:

GENERAL ELECTRIC
Schenectady, New York

prepared by:

Dunn Geoscience Corporation

October 1984

REMEDIAL INVESTIGATION
GE/MOREAU SITE
11-CERCLA-30201

Prepared For:

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Schenectady, New York

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(Bound Separately)

EXECUTIVE SUMMARY

This remedial investigation was conducted for the General Electric Company, Schenectady, New York by Dunn Geoscience Corporation, Latham, New York. The remedial investigation consisted of an in-depth geohydrologic study of the area surrounding the GE/Moreau site located in the Town of Moreau, Saratoga County, New York.

The investigation, as of October 22, 1984, included test drilling, installation of 59 monitoring wells, collection of water level measurements and ground-water samples from 80 monitoring wells, collection and analysis of surface water samples from 14 locations, and the collection and analysis of ground-water samples from approximately 160 private residential wells. Field activities were initiated in April, 1984 and all field and laboratory procedures followed prescribed protocols detailed in the Work Plan approved by the U.S. Environmental Protection Agency.

Three major types of sediments were found in the study area: fine-grained glaciolacustrine sediments, deltaic sand deposits, and till. These deposits are divided into two geohydrologic units: The Moreau sand aquifer and an underlying confining bed. The Moreau sand aquifer consists of up to 88 feet of glaciodeltaic sand underlain by up to 28 feet of upper glaciolacustrine medium to fine sand with some silt. The confining bed consists of up to 25 feet of lower glaciolacustrine varved silt and clay underlain by till. The confining bed overlies dark gray argillaceous limestone bedrock.

On the average, the upper 75 percent of the Moreau aquifer is composed of the glaciodeltaic unit and the remainder is composed of the upper glaciolacustrine deposits. The Moreau sand aquifer occurs under unconfined, or water table conditions and, on the average is about 60 feet thick, but varies significantly.

Field falling-head hydraulic conductivity tests conducted on the upper glaciolacustrine deposits resulted in measured vertical hydraulic conductivity values in the range of 4.0 to 6.2 x 10^{-6} cm/sec. Laboratory constant head triaxial tests of upper glaciolacustrine deposits resulted in values of 1.7 x 10^{-4} and 1.2 x 10^{-5} cm/sec. Horizontal hydraulic conductivity tests utilizing slug tests in completed monitoring wells resulted in a hydraulic conductivity value of about 2.0 x 10^{-3} cm/sec.

Slug tests were also performed on the glaciodeltaic deposits. Calculated horizontal hydraulic conductivity values ranged from 2.4 x 10^{-3} to 2.1 x 10^{-2} cm/sec. The average vertical hydraulic conductivity measured from field tests was 1.6 x 10^{-3} cm/sec.

Both horizontal and vertical hydraulic conductivity vary with depth. On the average, horizontal hydraulic conductivity is between three and four times greater in the upper 75 percent of the aquifer then nearer the base. The vertical hydraulic conductivity in the lower 25 percent of the aquifer is about 300 times less than in the rest of the aquifer.

A ground-water mound exists in proximity to the GE/Moreau site causing ground-water flow toward the west, southwest, south and southeast. However, ground-water gradients to the west and southwest are very slight, generally being in the range of 0.0001 to 0.002 ft/ft. The major factor influencing ground-water flow is the topographic scarp southeast of the site that marks the edge of the Moreau sand aquifer. Near this area the gradient is up to 0.035 ft/ft and directs the principal flow of ground water to the southeast.

The average linear ground-water velocity for the upper portion of the aquifer is about 0.67 feet per day and about 0.27 feet per day for the lower portion.

Pumping wells show no apparent influence on the direction of regional ground-water flow based on data collected. Data indicate that the transmissivity is high enough that the influence from pumping wells is not significant enough to alter ground-water flow patterns.

Previous work and this investigation have shown evidence of contamination by organic compounds in the Moreau sand aquifer and indicates that the contamination is stratified within the aquifer. In addition, contamination was detected in some streams that make up the Fort Edward water supply. Although observed contamination includes a variety of organic compounds, trichloroethylene (TCE) is the most prevalent.

The areal extent of contamination of TCE in concentrations greater than 100 parts per billion (ppb) occurs in an essentially southeast trending plume approximately 4800 feet long and about 2000 feet wide at its widest point. The plume has its origin at the GE/Moreau disposal site and extends southeastward to the topographic scarp. The overall orientation of the plume follows the direction of ground-water flow. The downgradient limit of the plume appears to be controlled by ground-water discharge to springs and streams having their head waters at the foot of the escarpment.

Maximum organic levels occur in a relatively narrow, essentially southeast trending band. The band of maximum contamination coincides very closely with the ground-water flow path south of the GE/Moreau site. The data further show the tendency for TCE concentrations to be highest at intermediate and deep levels within the aquifer.

During the monitoring of residential wells in the area southwest of the GE/Moreau site, encompassing Terry Drive, Cheryl Drive, and Myron Road, low levels of five organic compounds were detected. Information collected from home owners concerning the reported depths of their wells indicates the horizon of contamination is between 35 and 55 feet below grade.

None of the contaminated wells in this area show a consistent or steady pattern of contamination. The low level contamination may be present in one or two rounds of sampling but not in others. Moreover, the wells of adjacent homeowners may have different low level contaminants present or no contamination at all.

Although low level TCE has been detected in a Cheryl Drive residence and four Terry Drive homes, insufficient data is currently available for identifying its source. The ratio of TCE to other organics in the residential ground-water samples does not conform to the general pattern directly downgradient of the disposal site. Given this condition and the dilution factors seen in wells along the paths of transverse dispersion, it is unlikely that the residential organic contamination came directly from the area of the disposal site or the defined plume.

Although the data suggest that the low levels of organic contamination in the Terry, Cheryl and Myron area are not associated with the defined plume, eight additional monitoring wells are being installed in an effort to better define the relationship, if any.

During the sampling of residential supply wells, water samples were collected from 16 wells reported to have been drilled into the bedrock aquifer. Chemical analysis of these samples did not detect any organic compounds.

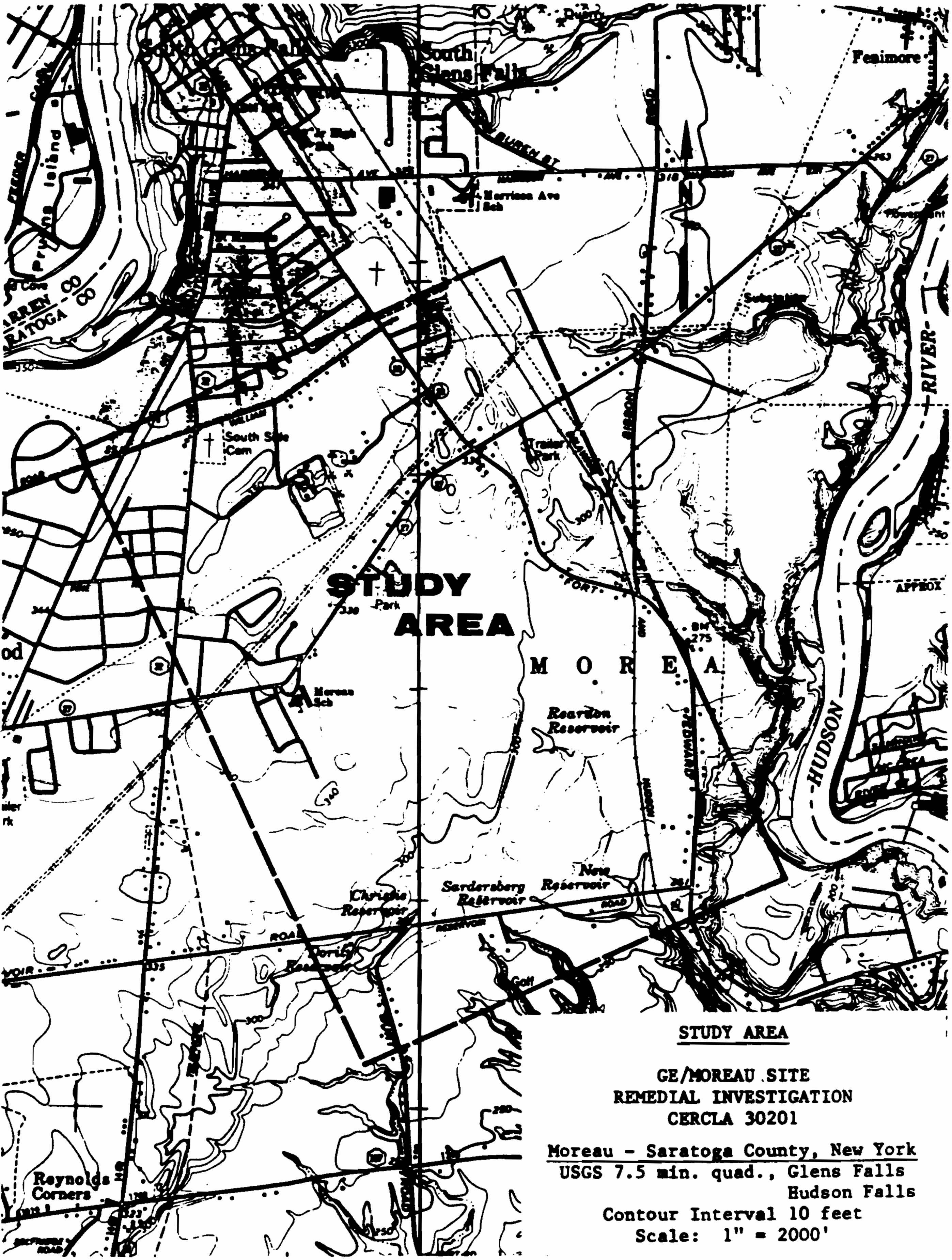
1.0 INTRODUCTION

1.1 General

This remedial investigation was conducted for the General Electric Company, Schenectady, New York by Dunn Geoscience Corporation, Latham, New York. The remedial investigation consisted of an in-depth geohydrologic study of the area surrounding the GE/Moreau site located in the Town of Moreau, just south of South Glens Falls, Saratoga County, New York. Figure 1.1 is a map of the site and surrounding area. The remedial investigation was conducted in response to Administrative Order No. 11-CERCLA-30201 issued to the General Electric Company by the United States Environmental Protection Agency, Region II pursuant to Section 106(a) of the Comprehensive Environmental Response, Compensation and Liability Act (CERCLA), 42 U.S.C. S9606(a). The Order was entered November 21, 1983 effective February 13, 1984.

The investigation was managed from the corporate offices of Dunn Geoscience corporation located 50 miles south of the project site. The project was conducted for Mr. T. Leo Collins, Manager, Environmental Programs and under the coordination of Dr. D. Wallace Magee, Manager, Environmental Quality Planning. The Dunn Geoscience project team consisted of the following:

William E. Cutcliffe, President - Corporate Advisor and Reviewer
D. Theodore Clark, Senior Hydrogeologist - Project Manager
James Narkunas, Senior Hydrologist - Geohydrology and Sample Collection



STUDY AREA

GE/MOREAU SITE
REMEDIAL INVESTIGATION
CERCLA 30201

Moreau - Saratoga County, New York
USGS 7.5 min. quad., Glens Falls
Hudson Falls
Contour Interval 10 feet
Scale: 1" = 2000'

Figure 1.1

Sander I. Bonvell, Chemist - Geochemistry and Sample Collection

William J. Miller, Hydrologist - Modeling and Sample Collection

Paul W. Hare, Hydrologist - Geohydrology and Sample Collection

Michael L. Ianniello, Geologist - Drilling Supervision and Data Collection

John M. Uruskyj, Geologist - Drilling Supervision and Geology

Rod Sutch, Geologist - Field Operations and Drilling Supervision

Jeffrey T. Wink, Geologist - Sample Collection

Additional support staff, including geologists and technicians were utilized as necessary. Surveying, cartography and drafting were done by Robert W. Shuey, Michael T. Maksymik, and Stewart Galloway.

Subcontract drilling services were provided by Warren George, Inc., Jersey City, New Jersey. Water and soil analytical laboratory services were provided by ERCO/Energy Resources Company, Inc., Cambridge, Massachusetts and Environmental Testing and Certification, Edison, New Jersey.

The report format is as follows. The introduction outlines the objectives, project scope, and conditions. This is followed by a methodology section which describes the field activities, schedules, and laboratory testing. Sections 3 and 4 address the geologic and hydrologic conditions, respectively. These sections establish the background information necessary to evaluate the specific data developed during the investigation. Sections 5 and 6, Influence of Pumping Wells and Water Budget, provide additional site specific hydrologic information. Section 7 describes the ground-water flow model, how it was applied, and the information it provides for further evaluation of regional and site conditions. Sections 8 through 11 describe the extent of ground water, surface water, soil and air contamination based on the results obtained from this and previous work. Sections 5, 7, 8, 10 and 11 are not yet complete in that additional data are being obtained. Once the compilation and evaluation of these data are completed an addendum to this report will be submitted.

This report consists of four items: the report text; two appendices; and a portfolio of plates.

1.2 Background and Site History

The GE Moreau Site (formerly Caputo Site) in the Town of Moreau, Saratoga County, New York was used as an industrial waste disposal site for waste materials generated by the General Electric Company reportedly from 1958 to 1968. In 1978, Town of Moreau and State officials began testing the air, surface water, ground water and soil at and near the disposal site. A plan was developed to remove some of the contaminated PCB soil and to cover the evaporation pit.

In September 1980, the site was included in an agreement between General Electric and NYSDEC whereby GE agreed to conduct remedial investigations at seven known waste disposal sites.

In the fall of 1982, it was determined that there were elevated concentrations of trichloroethylene (TCE) in the ground water. The Town of Moreau installed activated carbon filters in approximately 70 homes within the reportedly downgradient contaminated area. In the summer of 1983 after meetings with the Town of Moreau and State representatives, the United States Environmental Protection Agency (EPA) initiated negotiations with the General Electric Company to address the off-site contamination problem. The negotiations resulted in an agreement whereby the General Electric Company would conduct a remedial investigation and take necessary corrective action pursuant to Administrative Order No. 11 CERCLA-30201.

The following background information is taken from the April 1983 Hardick/Rich report entitled "Investigation of Ground Water Contamination in the Vicinity of the GE Moreau (Caputo) Site".

The GE Moreau Site was proposed for the EPA superfund list in December, 1982 and was ranked number 141 nationally (out of 400), and number 7 in New York State (out of 26). In December, 1978 the Town of Moreau removed approximately 100 cubic yards of contaminated material from the "evaporation pit" area and had it transported to a secure landfill site in Niagara Falls, New York. In May of 1979, through a joint effort by the New York State Department of Environmental Conservation (DEC), New York State Department of Transportation (DOT), and the Town of Moreau, a temporary cap was placed over the "evaporation pit" area. In March, 1982, the New York State Department of Health (DOH) tested water in private wells in the Bluebird Road area. The July report from the DOH entitled "An Assessment of Drinking Water Quality in the Area of the Caputo Inactive Hazardous Waste Site" indicated levels of contamination in some private wells.

In July, 1982, the Town of Moreau conducted water testing of private wells in residences in the vicinity of the GE Moreau site.

During July and August, 1982, 151 private wells were tested for volatile organic compounds. Test results indicated that 22 private wells had various levels of contamination of volatile organic compounds in excess of 1.0 ppb. Most of the wells tested were two-inch driven well points ranging from 26 to 87 feet in depth. The average depth of the 22 contaminated wells, (one-third of which are less than 40 feet in depth), is 44.3 feet.

In August, 1982, the Town of Moreau contracted C.A. Rich Consultants to conduct a geohydrologic investigation. The investigation was conducted to establish the extent of chemical contamination of the water in private wells and to confirm whether or not the contaminated wells were degraded due to previous dumping of liquid chemicals at the "evaporation pit".

As part of this investigation, seven shallow test wells were installed during September, 1982. These wells ranged from 24 to 37-1/2 feet deep and were used to establish a preliminary water-table contour map. In October, 1982, three deep test wells, ranging from 90 to 95 feet deep, were installed to determine soil conditions from the ground surface down to the clay layer. In December, 1982, two intermediate depth test wells were installed at depths of 53 and 58 feet. In January, 1983, FE-1, a flowing well, was installed at a spring area on land owned by the Village of Fort Edward.

The Hardick/Rich report concluded that the ground water had been degraded and contaminated with chlorinated hydrocarbon compounds throughout the designated study area. The report also concluded that the chemicals found in the "evaporation pit" are the same as those found in monitoring wells and private wells away from and downgradient from the pit. It was further concluded in the Hardick/Rich report that the "evaporation pit" area was the probable sole source of the chemical contamination and that there was a direct causal/effect relationship between reported past waste disposal practices at the GE Moreau Site and subsequent detection of contaminated ground water obtained from downgradient wells in the same aquifer that underlies the site.

1.3 Project Initiation

The Remedial Investigation which is the subject of this report was initiated in January, 1984 with a complete review of Administrative Order Index No. 11-CERCLA 30201.

Following the review and evaluation of the Administrative Order, numerous discussions were held with General Electric personnel and the EPA to define and discuss the specific activities required by the Order. During February and March, 1984, Dunn Geoscience aided General Electric in the preparation of an interim remedial plan for Part I, Immediate Corrective Action, of the Order and prepared a detailed work plan and implementation schedule for Part II, Remedial Investigation of the Order. The Part II Remedial Investigation Work Plan and Implementation Schedule was submitted to the EPA by Dunn Geoscience Corporation on March 28, 1984 (see Appendix A). During the 30-day EPA review period of the Work Plan and Implementation Schedule, site reconnaissance and field activities were conducted. Access to the initial soil boring and monitoring well location sites was obtained during April and drilling commenced on May 2, 1984.

1.4 Project Objectives

The objective of the remedial investigation was to define the nature and extent of ground-water contamination. As outlined in the Administrative Order, the geohydrologic investigation was to include the following:

- geohydrologic setting of the site including a characterization of the soils and definition of aquifer characteristics;
- ground-water gradients, velocity, and quality within the area of concern;
- location and influence of pumping wells on the movement of ground water;
- as practical, modeling of the drinking water aquifer to predict both rate and extent of ground-water contamination;
- determination of the vertical and lateral extent of contamination in soil, air, surface water, and ground water; and,
- appropriate health and safety plans for conducting the remedial investigation.

1.5 Scope and Conditions

The scope of the remedial investigation **allowed** a phased approach that would identify additional investigations and the development of supplemental data. The scope of the geohydrologic investigation was designed to provide sufficient information for data evaluation and to identify those areas requiring further investigation.

The scope of work outlined in the Work Plan followed the requirements of the Administrative Order and utilized field and laboratory protocols and methodologies that equaled or exceeded required standards.

The scope of Phase I called for a drilling program of soil borings and installation of monitoring well clusters at 13 locations. These locations were positioned throughout the study area to provide an overall evaluation of geohydrologic conditions, water quality, and the direction of ground-water movement. The initial water quality analyses and evaluation of water level data from these wells defined the positions for the second phase of drilling which included an additional 22 borings and monitoring wells at 9 locations. Subsequent water level measurements and ground and surface water quality analyses were conducted to expand the data base.

Ground-water samples were also collected from approximately 160 private residential wells and analyzed for volatile organics. The residential water quality samples were used to supplement the data obtained from the 80 wells monitored during this investigation.

At the 22 locations where monitoring wells were installed, permission to drill was readily obtained through complete cooperation of the respective property owners. At two proposed locations, permission could not be obtained. An alternate location a short distance away was obtained for one of these locations; the other location was dropped because a suitable alternate location could not be obtained. Physical access preparation was necessary at nine of the sites.

Field conditions were very good and resulted in no significant delays or impairment of activities or in the quality of data obtained. Subcontract drilling services and materials were as specified and the cooperation and quality of work was excellent.

The·cooperation and assistance offered by the Town of Moreau, the Moreau School, and the Village of Fort Edward was excellent and aided in the successful completion of the proposed field activity. Cooperation and assistance by the DEC on-scene coordinator was also excellent and further aided in meeting the objectives of the investigation.

Time constraints precluded extensive data collection over an extended period of time. However, the amount of data collected and the frequency of data collection exceeded that required by the Order as outlined in the Work Plan. The data obtained is reasonably conclusive; it is recognized that some additional data will be advantageous to the refinement of current observations and conclusions. The gathering of additional data is ongoing.

1.6 Previous Work

Previous work of significance at the site has been reported in separate reports and include the following (listed in chronological order):

- PCB's Removed From the Caputo Site on December 18 and 20, 1978, Town of Moreau, Saratoga County, New York, Hardick Associates (1978);
- Conceptual Engineering Study of Five Disposal Sites Known to have Received PCB Wastes, Wehran Engineering (1980);
- Caputo Site Engineering Report, O'Brien & Gere (1981);
- Caputo Site Engineering Report addendum, O'Brien & Gere (1982);
- Caputo Site Remedial Program Final Plan-Subsurface Investigation, O'Brien & Gere (1982);
- Investigation of Ground-Water Contamination in the Vicinity of the GE Moreau (Caputo) Site, Hardick Associates and C.A. Rich Consultants, (1983).

2.0 METHODOLOGY

2.1 General

All aspects of the remedial investigation were conducted using standard or accepted methods. Techniques and methods utilized during the field investigation and laboratory testing are described within this section. Specific analytical and numerical techniques used to interpret the geohydrologic data are addressed within the respective sections of the report.

2.2 Field Investigations

2.2.1 Drilling Program

During the two phases of drilling, from May 2, 1984 to June 18, 1984, and August 1, 1984 to August 30, 1984, a total of fifty-nine monitoring wells were installed at locations surrounding the GE Moreau site. Well clusters were numbered sequentially, based on the order of their completion. Individual wells in each cluster were further identified by their relative depth: shallow (S), intermediate (I), and deep (D). The locations of the wells are shown on Plate 1.

Test borings were advanced utilizing two drilling methods: hollow stem auger and mud-rotary. A CME-55 soil boring rig utilizing a four-inch, hollow-stem auger was used to install the first well (DGC-1) However, hydraulic, or flowing, sands encountered at a depth of 34 feet, rendered the hollow-stem auger method ineffective. Consequently, a mud-rotary rig was mobilized to the site on May 8, 1984 and, thereafter, all subsequent borings were advanced using mud-rotary techniques. To expedite the phase one drilling, a second mud-rotary rig was mobilized on May 15, 1984.

Mud-rotary drilling utilized bentonite drilling fluid and a 4-3/4-inch tri-cone roller bit. The drilling mud was mixed with water from two locations. Water was initially obtained from a well at the Town of Moreau Park and, later, from a municipal fire hydrant located on Williams Street in the Town of Moreau.

2.2.2 Soil Sampling and Classification

Standard and continuous soil samples were collected following ASTM standards for the split-barrel method. Continuous split-spoon samples were obtained from the deepest test borings at sites DGC-1, 2, 6, 11, and 15. Standard 5-foot interval samples were collected from the deepest test borings at sites DGC-3, 4, 5, 7, 8, 9, 10, 12, 13, 14, 16, 17, 18, 20, 21, 22, and boring 19S. No soil samples were obtained in borings DGC-14I, 14S, 15S, 18S, 20I, 20S, 21I, 21S, 22I, and 22S. Soil samples were obtained from a portion of the proposed screened interval in all remaining shallow and intermediate depth wells.

For all samples, blow counts were recorded for each six inches of soil penetrated by the split-spoon sampler as it was driven by a 140-pound hammer dropped from a height of 30 inches. Material recovered in the sampler was described using a modified version of the Burmister Classification System as well as the Unified Soil Classification System.

Representative portions of each split-spoon sample were placed in glass containers and retained by Dunn Geoscience Corporation for subsequent laboratory evaluation, as necessary. Soil boring logs, describing subsurface materials encountered in the test borings, are located in Appendix B. Borings were terminated when samples indicated that a borehole had penetrated lower glaciolacustrine, varved, clayey silts.

During the second phase of drilling, thin-walled, tube samples were also used to collect undisturbed glaciolacustrine samples from the deep test holes, DGC-14 and 15. The soils were recovered using three-inch diameter brass, open-tube samplers in accordance with ASTM standard methods.

Prior to collecting the tube samples, the borehole was cleaned out to the desired sampling depth. While the water level in the boring was kept at the naturally occurring ground-water level, the tube was pushed 24 inches into the soil using a rapid continuous motion. Before the tube was pulled, it was rotated at least twice to shear the sample off at the bottom. Upon removal of the tube, the sample recovery was measured and the disturbed material from the top of the tube and at least one inch of soil from the lower end of the tube were removed, described and then discarded. Both ends of the tube were then sealed with wax and fitted with end caps which were secured with tape. The taped end caps were then dipped in melted wax to prevent breaking the seals. Finally, the tube was labeled with the necessary information and placed in a container designed to reduce shock, vibration and disturbance during storage and shipment.

Soil samples for volatile organic analysis were collected in 40-ml vials from well clusters DGC-4 to DGC-13 and wells 1S, 1I, 2S, 3S, and 3I. The sample vials consist of 3 parts: a glass bottle, a teflon-faced septum, and a screw cap. The samples were representative portions of split-spoon samples collected during drilling. Each split-spoon sample was cut open, using a clean knife, and a representative soil sample taken from the center of the spoon. Two 40-ml vials were carefully filled approximately two-thirds full, capped, and labeled for future laboratory analysis. All samples were placed on ice and transported to the Dunn Geoscience office for refrigeration. Samples selected on a basis of preliminary field screening, described later, were then shipped in an insulated container via overnight courier to ERCO Laboratories in Cambridge, Massachusetts. Chain-of-Custody records were maintained for all samples sent to ERCO.

2.2.3 Volatile Organic Screening

As part of the drilling program for well clusters DGC-1 to DGC-13, Dunn Geoscience Corporation performed organic screening for volatile organic compounds on all split-spoon samples. An HNU Model PI-101 Photoionization Analyzer and Draeger detector tubes were used for the field screening.

The primary screening device utilized was a photoionization analyzer. Photoionization uses ultraviolet light to ionize many trace compounds (especially organics) and the model PI-101 employs this principle to measure the concentration of trace gases. In the PI-101, a chamber adjacent to the ultraviolet light source contains a pair of electrodes. When a positive potential is applied to one electrode, the field created drives any ions in the chamber to the collector electrode where the current is measured. The measured current is proportional to the concentration of organics sampled by the instrument's probe. The useful range of the instrument is from 0.1 to 2,000 ppm.

Drager detector tubes were also used on those samples in which the photoionization analyzer showed organic vapor concentrations exceeding 3.0 ppm. The Draeger tube method basically consists of a compound-specific detector tube (Trichloroethylene) and a hand-operated vacuum pump. The pump is used to draw a standard volume of air sample through the tube which undergoes a quantitative color change in the presence of the specific compound. The useful range of the detector tubes is from 2.0 to 200 ppm.

Representative portions of all split-spoon samples obtained from borings DGC-1 through DGC-13 were placed in clean, glass jars immediately after the split barrel sampler was opened. Although split-spoon sample recoveries varied, care was taken to prepare a standard quantity of sample. Each jar was sealed with metal foil and a screw cap labeled with the appropriate sample identification number. The sample was then heated to 40-degrees C. with a small portable heater. After 30 minutes, the sample was taken from the heater, the screw cap removed, and the metal foil pierced with the eight-inch extension to the photoionization probe. The headspace was tested for the presence of organic vapors and the results recorded (Appendix C). For selected samples, the jars were resealed using new metal foil, reheated, and retested using the Draeger detector tubes.

2.2.4 Vertical Hydraulic Conductivity Testing

Vertical hydraulic conductivity tests were performed in soil borings during drilling. In the deep borings, the test depths corresponded to the anticipated screen placements. In the shallow and intermediate borings, the tests were performed at, or near, the top of the proposed screened intervals.

Test preparation included driving four or five-inch casing to the selected depth. The bottom two feet of the casing was then driven into undisturbed soil to insure that a good seal was set between soil and casing. A tricone bit was used to carefully drill down to the bottom of the casing. The hole was then flushed by pumping clean water through a tremie pipe until the return flow was clear. Finally, the tremie pipe was withdrawn and the casing refilled to the top with clean water. The actual test was run by measuring the drop in water level within the casing over a period of time, usually 15 minutes. The difference in water level with respect to time was used to calculate the vertical hydraulic conductivity as shown in Appendix D.

2.2.5 Monitoring Well Installation

Following the completion of drilling, each borehole was thoroughly flushed clean of cuttings and drilling mud. The borings were backfilled with bentonite pellets to a depth within two feet of the proposed screen bottom. Two-inch I.D., mechanical flush-threaded, schedule 40, PVC riser and 10-slot or 20-slot screen was used to construct all monitoring wells.

The deep and intermediate wells in well clusters DGC-1 to DGC-3 were installed with five-foot screens, whereas ten-foot screens, designed to monitor the entire range of water-table fluctuation were installed in the shallow wells. Well clusters DGC-4 through DGC-22 were constructed to screen the entire saturated portion of the aquifer above the basal glaciolacustrine clayey silts. Size No. 2 Morie sand was emplaced in the annulus opposite, and extending two to three feet above the top of, the screens in well clusters DGC-1 and DGC-2. Size No. 1 Morie sand was used as filter pack material in all subsequent wells utilizing 20-slot screen. Wells constructed of 10-slot screen were completed with No. 1/2 Morie sand filter pack. A five-foot bentonite pellet seal was installed above each Morie sand pack. A cement-bentonite grout was then pumped into the remainder of the annulus. Lockable, steel, protective casings were cemented over the PVC riser extending above the land surface to prevent unauthorized access into the monitoring wells. The protective casings were then primed and painted with special-purpose, non-contaminating paint developed by Sherwin Williams. Well construction details are shown in Appendix B.

2.2.6 Well Development

All monitoring wells were developed using the air-lift or bailing methods. Well development is necessary for the following reasons:

- To remove residual drilling mud and formational silts and clays, thereby preventing turbidity during sampling that could potentially interfere with chemical analysis; and,
- To increase the hydraulic conductivity immediately around the well, which in turn reduces the potential of the well yielding insufficient volume of water during the sampling procedure.

Well development took place after the completion of each series of well installation. Well clusters DGC-1 through DGC-13 were developed from June 11, 1984 to June 15, 1984, and clusters DGC-14 through DGC-22 were developed from August 16, 1984 to September 4, 1984. Three methods of well development were used and each is briefly described below. Development waters were collected for most intermediate and deep wells and for eight shallow wells. Specific dates of completion, development methods, and water collection information are tabulated in Table 2.1.

Method 1 - Air-Lift

The air-lift method involves pumping compressed air into the well forcing out water containing the undersirable fine sand and silt. The air is injected

Table 2.1.

Well Cluster	Method Of Development	Dates Of Development	Developmental Water Collected
DGC-1	1	6/11/84	
DGC-2	1	6/12/84	
DGC-3	1	6/11/84	
DGC-4	1	6/12/84	
DGC-5	1	6/8/84	
DGC-6	1,3S	6/13/84	I,D
DGC-7	1,3S	6/13/84	I,D
DGC-8	1,3S	6/14/84	I,D
DGC-9	1 I,D*	6/12/84	All
DGC-10	1	6/14/84	All
DGC-11	1	6/15/84	All
DGC-12	1	6/14/84	All
DGC-13	3	6/13/84	
DGC-14	2	8/20/84	All
DGC-15	2	8/17/84 (S,I) 8/20/84 (D)	All
DGC-16	2	8/16/84	S,D
DGC-17	2	8/17/84	D
DGC-18	2	8/20/84 (D) 8/21/84 (S,I)	All
DGC-19	3	8/23/84	
DGC-20	2	8/31/84	All
DGC-21	2	8/31/84	All
DGC-22	2	9/4/84	I,D

1 - air lift method

2 - modified air lift method

3 - hand bailed

S - shallow well

I - intermediate well

D - deep well

* The shallow well of cluster DGC-9 is dry.

into the well through a hose attached to an air compressor. The hose is cleaned with deionized water and lowered into the well until the lower end of the hose is positioned several feet above the top of the screened section. Positioning of the hose is important to prevent air from entering the sandpack where it might become trapped and possibly induce chemical changes in the water. A back-washing action is also accomplished by releasing short bursts of air capable of momentarily raising the water column. This surging motion helps release fine sand and silt trapped in the sand pack or on the surface of the borehole. Once released, these fine particles may travel through the screen and eventually be evacuated from the well. Discharging and back-washing are alternated until the discharge is relatively free of fine-grained sediment.

Method 2 - Modified Air-Lift

This method is an adaption of the basic air-lift method and provides the following advantages over Method 1:

- No air enters the well;
- Water is removed directly from the screened portion of the well;
- The coalescer unit reduces any possibility of introducing foreign substances into the well; and,

- Up to three wells may be developed simultaneously using one air compressor and one coalescer.

Five-foot sections of one-inch diameter PVC pipe are screwed together and lowered into the monitoring well until the end of the bottom-most section of pipe is positioned within the screened section of the well. Attached to the bottom of the pipe are two one-way check valves separated by about three inches of one-inch PVC pipe. Both check valves close in a downward direction. Two air compressor hoses are used. One connects the air compressor to the coalescer, and the other hose runs from the coalescer down the one-inch PVC pipe well development assembly unit to approximately five feet above the upper check valve. The orientation of the check valves allows the pipe to fill with water. Activation of the air compressor momentarily shuts the upper check valve and forces the trapped column of water up and out of the pipe. The release of the water lowers the pressure on the top of the check valve allowing water to again enter the pipe until the air pressure becomes sufficient to blow out the column of water. This process repeats itself if the water pressure (head) is capable of balancing the air pressure created by the compressor. In wells lacking adequately long water columns, the water pressure is incapable of reopening the check valve allowing a fresh column of water to enter. Manual control of the air pressure is necessary in these instances. The lower check valve assures that no air enters the monitoring well. In the majority of clusters developed using this method, the intermediate and shallow wells were developed simultaneously while the deep well was developed independently. To prevent cross-contamination between wells, the one-inch PVC pipe was washed with water before introduction into each well.

Method 3 - Bailing

Five shallow wells were developed by hand bailing. The modified air-lift method was attempted but proved ineffective due to the limited amount of water in these wells. Teflon point-source dedicated bailers, subsequently used in ground-water sampling, were utilized. The bailer served both as a surge-block device loosening the fine-grained material from the well annulus, and as a mechanism to remove the water and sediment from the well. The surging was accomplished by rapidly raising and lowering the bailer within the screened section. Bailing was continued until the water had sufficiently cleared or five well volumes of water had been removed (approximately 200 bailer volumes).

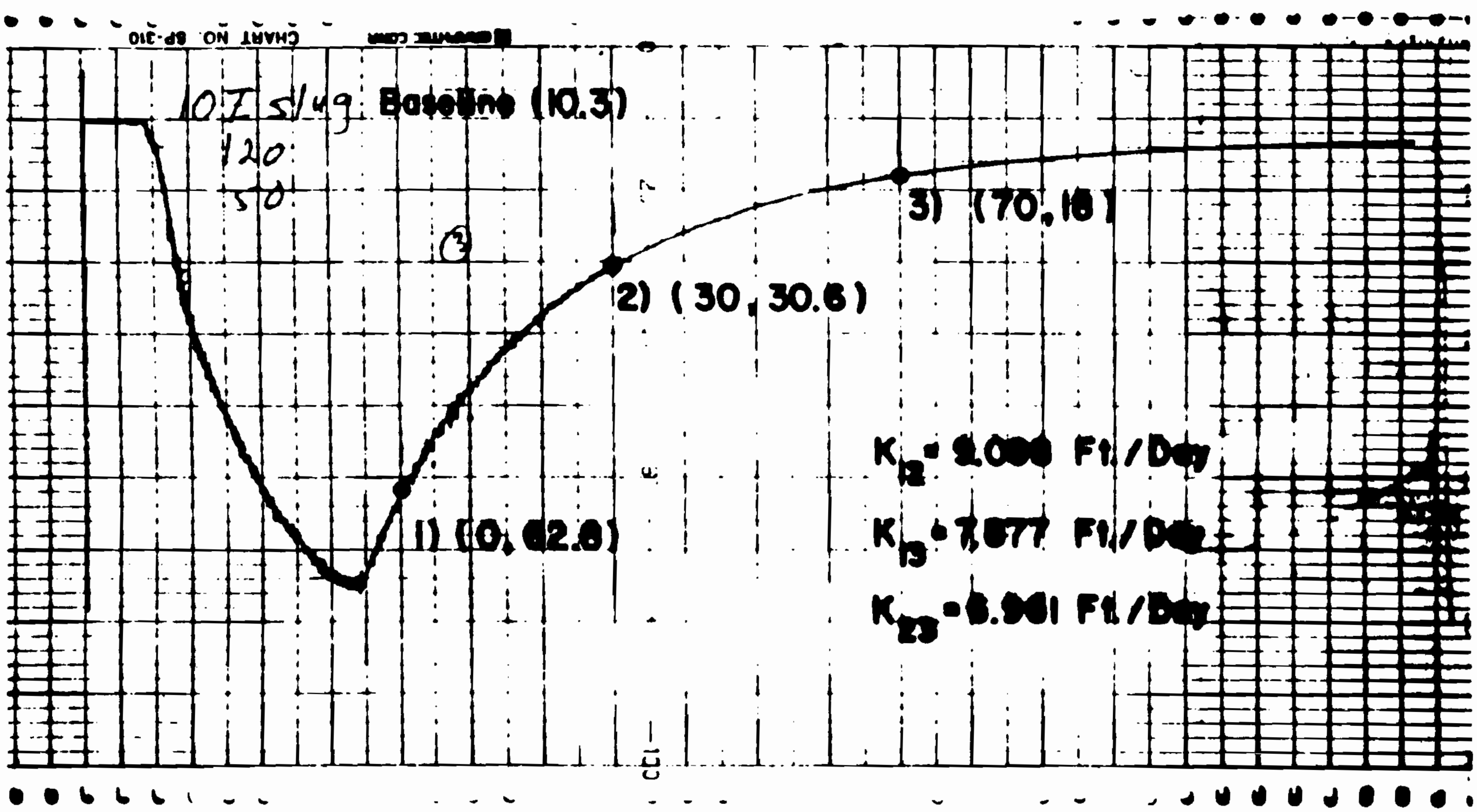
2.2.7 Horizontal Hydraulic Conductivity Testing

The horizontal hydraulic conductivity (K_h) of the Moreau Aquifer was estimated from the texture of its constituent deposits to range from 10^{-5} to 10^{-1} cm/sec (Freeze and Cherry, 1979, p. 29). This broad range was refined (10^{-3} to 10^{-1} cm/sec) through the use of field testing methods. Bail and slug tests performed on well clusters DGC-3, 4, 5, 8, 10, and 11 yielded quantitative information on the characteristics of the aquifer in the study area. Pump tests were not employed due to the large transmissivity values expected, the absence of close observation wells, and the limited discharge rates of pumps available for 2-inch monitoring wells.

Both slug and bail tests involve observing the recovery of water levels toward an equilibrium level after a known volume of water has been added to or removed from the well casing. During slug tests, 5 gallons (30.64 feet well equivalent) of deionized water were quickly introduced (10 to 15 seconds) into the well casing. During bail tests, a 2-foot (1.35-foot well equivalent) or 3-foot (2.00-foot well equivalent) dedicated teflon bailer was rapidly removed from below the static water level. In either test, a pressure transducer set 5 to 10 feet below the static water level was used to record water-level recovery on a strip-chart recorder (Enviro-Labs Model DL-240 Data Logger). Thus, a chart of pressure (at a specific measuring point) versus time was obtained for use in calculating the hydraulic conductivity (Figure 2.1).

Calculations of horizontal hydraulic conductivity (Kh) were based on the following equation (Dept. of the Navy, 1982):

$$K_h = \frac{R^2}{2L} \ln \left(\frac{L}{R}\right) \frac{\ln(H_1/H_2)}{(T_2 - T_1)}$$



EXAMPLE SLUG TEST - DGC Well 101

Time Scale = 120 mm/min.

Figure 91

where: K = horizontal hydraulic conductivity (L/t)
R = inside radius of casing/screen (L)
L = length of uncased/screened portion of well (L)
H = pressure/distance of water level from equilibrium value (chart units)
T = time expired from test start (t)

The method assumes that the aquifer tested is unconfined, homogeneous, and isotropic. The method is applicable to wells cased below the water table with uncased or screened extensions where L/R is greater than 8. It is, therefore, applicable to DGC intermediate and deep wells assuming a homogeneous and isotropic aquifer; it is not strictly applicable to shallow wells with uncased or screened portions above the water table, but it does serve as an approximation.

Results of the horizontal hydraulic conductivity testing are shown in Appendix D. As a check, the popular graphical method of Hvorslev (1951) was used to analyze the slug test data for well DGC-10I. The Hvorslev method yielded a hydraulic conductivity of 3.08×10^{-3} cm/sec (9.05 ft/day) compared to 2.82×10^{-3} cm/sec (7.98 ft/day) using the method described above.

Analysis of the results allows several generalizations:

- Although their results are within an order of magnitude, slug tests consistently yield lower hydraulic conductivity values than do bail tests. Generally, slug tests gave better estimates due to the greater volume of water displaced.

- The overall average horizontal hydraulic conductivity of the Moreau Aquifer based on all of the slug tests regardless of depth is 6.5 x 10 cm/sec (18.4 ft/day).

- The range in calculated hydraulic conductivity lies within two orders of magnitude (1.8 x 10^{-3} to 2.1 x 10^{-3} cm/sec for slug tests and 1.8 x 10^{-3} to 7.8 x 10^{-2} cm/sec for bail tests).

- The tests are repeatable.

- There is no apparent systematic spatial variation in hydraulic conductivity of the aquifer in the area tested.

- The deep wells show slightly lower hydraulic conductivity values than the intermediate wells (less than one order of magnitude difference).

- A sensitivity analysis performed by varying the value used for L (screen length) showed that the method was not very sensitive to the parameter L. Therefore, silting of the well screen would not significantly alter calculated hydraulic conductivity values.

2.2.8 Water Level Measurements

Water level measurements were obtained from observation wells on five dates: July 11, July 19, July 27, August 28, and September 26, 1984. Measurements were made on Dunn Geoscience Corporation monitoring wells and GE/Moreau Site wells (O'Brien & Gere 1, 2, and 3; B-28; and Jebco wells). In addition, levels were obtained from Town of Moreau monitoring wells and a Department of Transportation well cluster when representatives were available to provide access. Data was gathered as follows:

- July 11, 1984 measurements on DGC wells 1 to 13, and Town of Moreau and GE/Moreau Site wells;

- July 19, 1984 measurements on DGC wells 1 to 12 and GE/Moreau Site wells;

- July 27, 1984 measurements on DGC wells 1 to 13, and Town of Moreau, GE/Moreau Site, and Department of Transportation wells;

- August 28, 1984 measurements on DGC wells 1 to 20, and Town of Moreau and GE/Moreau Site wells; and,

- September 26, 1984 measurements on DGC wells 1 to 22, and Town of Moreau and GE/Moreau Site wells.

Measurements were obtained using a chalked stainless-steel tape which was cleaned prior to each measurement to prevent cross-contamination. The cleaning procedure involved rinsing the final four feet of the tape and weight with deionized water, then methanol, and a final deionized water rinse applied from squeeze bottles. Disposable laboratory gloves were worn by field personnel during water-level measurements.

The depth to water, indicated by a wetting line on the chalked section of the steel tape, was recorded for each measurement. This information was converted to water-level elevation with respect to mean sea level using the surveyed elevations of the measuring points (either top of PVC or steel casing). Water-level information is presented in Table 2.2.

The information in Table 2.2 was used to construct water-table contour maps and calculate hydraulic gradients discussed in a later section of this report.

2.2.9 Stream-Flow Measurement

In order to measure the volume of water lost to the diversion ditch, a V-notch weir equipped with a stilling well and water-level recorder was installed in the diversion ditch (Figure 2.2). This installation provided a continuous and reliable record of stream flow discharge over a period of approximately two months.

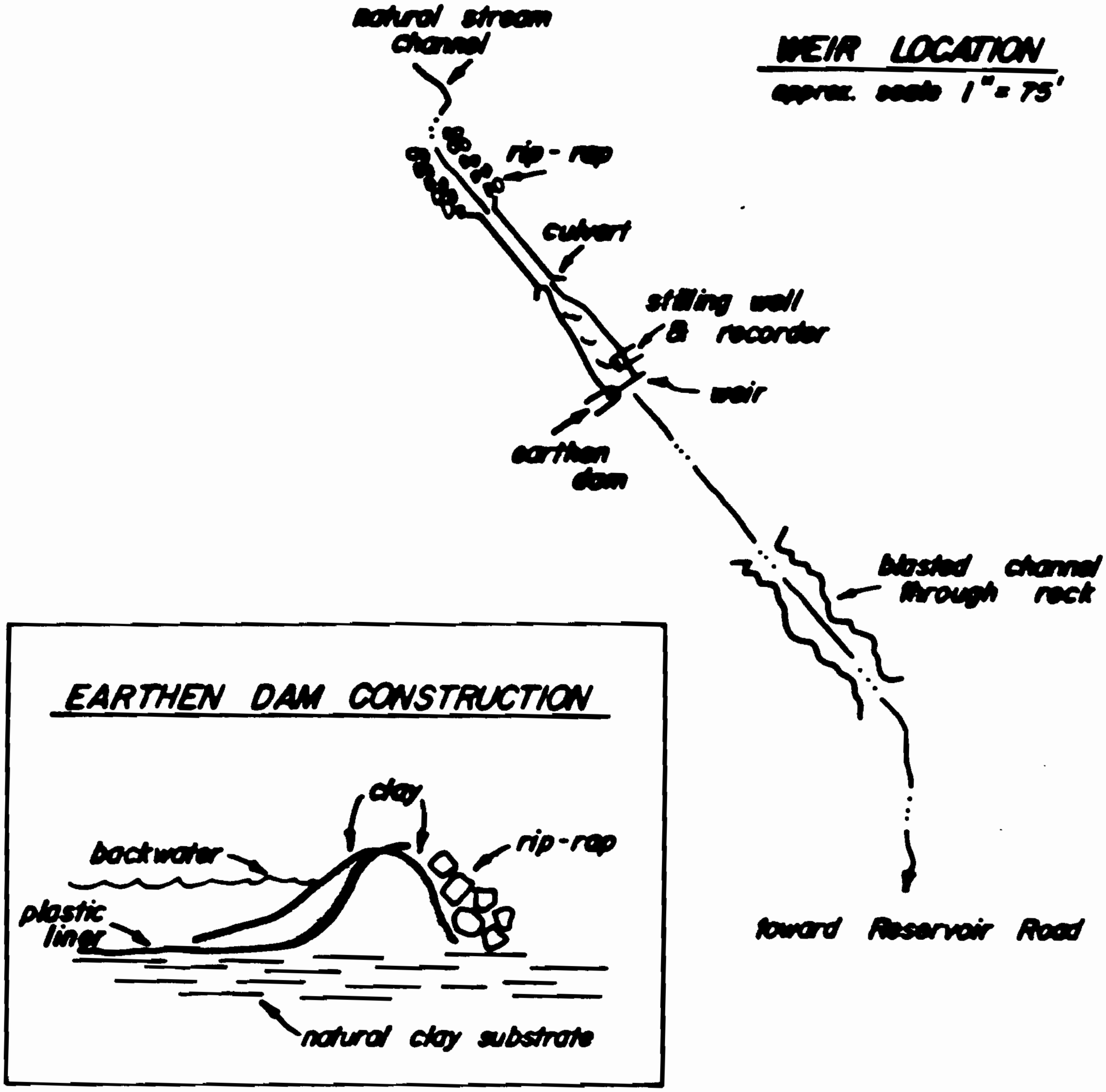
Table 2.2.

Water Level Information

Dunn Geoscience Wells	7/11/84	7/19/84	7/27/84	8/28/84	9/26/84
1S	296.72	296.64	296.56	296.25	295.98
1I	296.83	296.73	296.64	296.35	296.08
1D	297.25	297.23	297.20	296.82	296.54
2S	298.56	298.23	298.06	297.30	296.86
2I	295.50	295.39	295.27	294.96	294.69
2D	295.71	295.59	295.51	294.19	294.91
3S	295.56	295.48	295.39	295.11	294.89
3I	295.79	295.70	295.61	295.30	295.02
3D	295.76	295.67	295.59	295.29	295.01
4S	292.88	292.66	292.40	292.18	292.03
4I	292.71	292.53	292.29	292.07	291.91
4D	292.62	292.43	292.19	291.97	291.83
5S	304.15	304.06	304.09	303.67	303.35
5I	304.21	304.14	304.07	303.73	303.42
5D	305.76	305.71	305.69	305.20	304.85
6S	323.93	323.94	323.91	323.55	323.30
6I	323.93	323.94	323.91	323.55	323.28
6D	323.59	323.54	323.53	323.14	322.89
7S	324.46	324.41	324.41	324.10	323.76
7I	324.47	324.44	324.42	324.07	323.78
7D	324.06	323.37	323.95	323.53	323.27
8S	324.38	324.37	324.35	324.05	323.77
8I	324.41	324.39	324.39	324.04	323.77
8D	324.00	323.94	323.93	323.54	323.28
9S	Dry	300.68	Dry	Dry	Dry
9I	300.42	300.43	300.21	299.82	299.40
9D	300.36	300.22	300.12	299.72	299.31
10S	318.15	318.02	317.92	317.59	317.22
10I	318.20	318.08	318.04	317.63	317.28
10D	317.64	317.53	317.53	317.12	316.70
11S	324.25	324.24	324.26	323.85	323.53
11I	324.14	324.13	324.12	323.71	323.40
11D	323.63	323.65	323.67	323.14	322.74
12S	327.08	326.96	326.89	----	325.95
12I	323.15	324.73	322.91	322.53	322.17
12D	323.13	323.08	322.97	322.62	322.22
13	382.99	----	281.41	281.50	281.69
14S	----	----	----	320.83	320.57
14I	----	----	----	320.94	320.68
14D	----	----	----	320.83	320.56
15S	----	----	----	324.25	323.99
15I	----	----	----	323.99	323.74
15D	----	----	----	323.67	323.45
16S	----	----	----	318.69	318.33
16D	----	----	----	317.76	317.37
17	----	----	----	315.44	315.11
18S	----	----	----	324.99	324.63
18I	----	----	----	324.93	324.59
18D	----	----	----	323.64	323.27

Table 2.2
Water Level Information
Page 2

Dunn Geoscience Wells	7/11/84	7/19/84	7/27/84	8/28/84	9/26/84
19	----	----	----	324.55	324.30
20S	----	----	----	325.06	324.75
20I	----	----	----	324.74	324.65
20D	----	----	----	324.85	324.37
21S	----	----	----	----	324.88
21I	----	----	----	----	324.88
21D	----	----	----	----	324.67
22S	----	----	----	----	318.70
22I	----	----	----	----	319.03
22D	----	----	----	----	319.14
Sand Pit Stake	----	----	325.08	325.01	324.63
Town of Moreau Wells					
A	325.76	----	325.71	325.26	324.92
B	324.74	----	324.68	324.40	324.04
C	323.44	----	323.44	323.08	322.85
D	322.61	----	322.64	322.28	322.05
E	322.71	----	323.70	323.29	322.99
F	322.24	----	322.17	321.74	321.41
G	300.16	----	300.00	299.56	299.32
2	323.11	----	323.09	322.72	322.46
3	324.06	----	323.92	323.37	323.11
4	323.70	----	323.69	323.29	322.99
5	323.41	----	323.39	323.03	322.79
Caputo Site Wells					
OBG-1	325.74	325.64	325.43	325.12	324.82
OBG-2	325.76	325.69	325.50	325.16	324.83
OBG-3	325.78	325.77	325.58	325.16	324.81
B-28	325.57	325.52	325.46	325.13	324.82
Jebco 1.5	325.96	----	325.08	325.46	324.91
Jebco 2.0	325.13	----	325.05	324.39	323.78
Department Of Transportation Wells					
1	----	----	323.94	----	----
2	----	----	323.93	----	----
3	----	----	323.93	----	----



LOCATION OF WEIR
&
CONSTRUCTION DETAIL

Figure 2.2

After two attempts to place the weir in the natural channel upstream from the diverted channel failed due to severe piping, the weir was successfully placed approximately 75 feet below the culvert (Figure 2.2) on June 21, 1984. The wide man-made channel at this location required the construction of an earthen dam to constrict flow. This dam was constructed using clay from the stream bank and lined with plastic sheeting to help prevent seepage. The weir, a 90-degree, V-notch cut into a 3-foot by 8-foot sheet of 3/4-inch plywood, was then entrenched into the earthen dam and semiconsolidated clay substrate. Backfilling with clay and a bentonite-sand mixture prevented seepage around and under the plywood sheet. Finally, the downstream side of the weir and earthen dam were riprapped using rock from the blasted section of the channel downstream.

Measurements of the depth of flow (head) through the weir were gathered between June 21, 1984 and July 10, 1984. On July 10, 1984 an eight-inch stilling well and Steven's F-type water-level recorder were placed in the backwater 8 feet upstream from the weir. Head data were then collected continuously until weir failure occurred during a large storm on the evening of August 29, 1984. Head and time data, digitized from the recorder charts and corrected linearly to spot "control" measurements, were used to construct a weir hydrograph with discharge calculated as follows:

$$Q(\text{gal/min}) = 1094.17 \times H(\text{ft})^{2.5}$$

Plots of both weir head and discharge versus time are shown in Figures 2.3a and 2.3b.

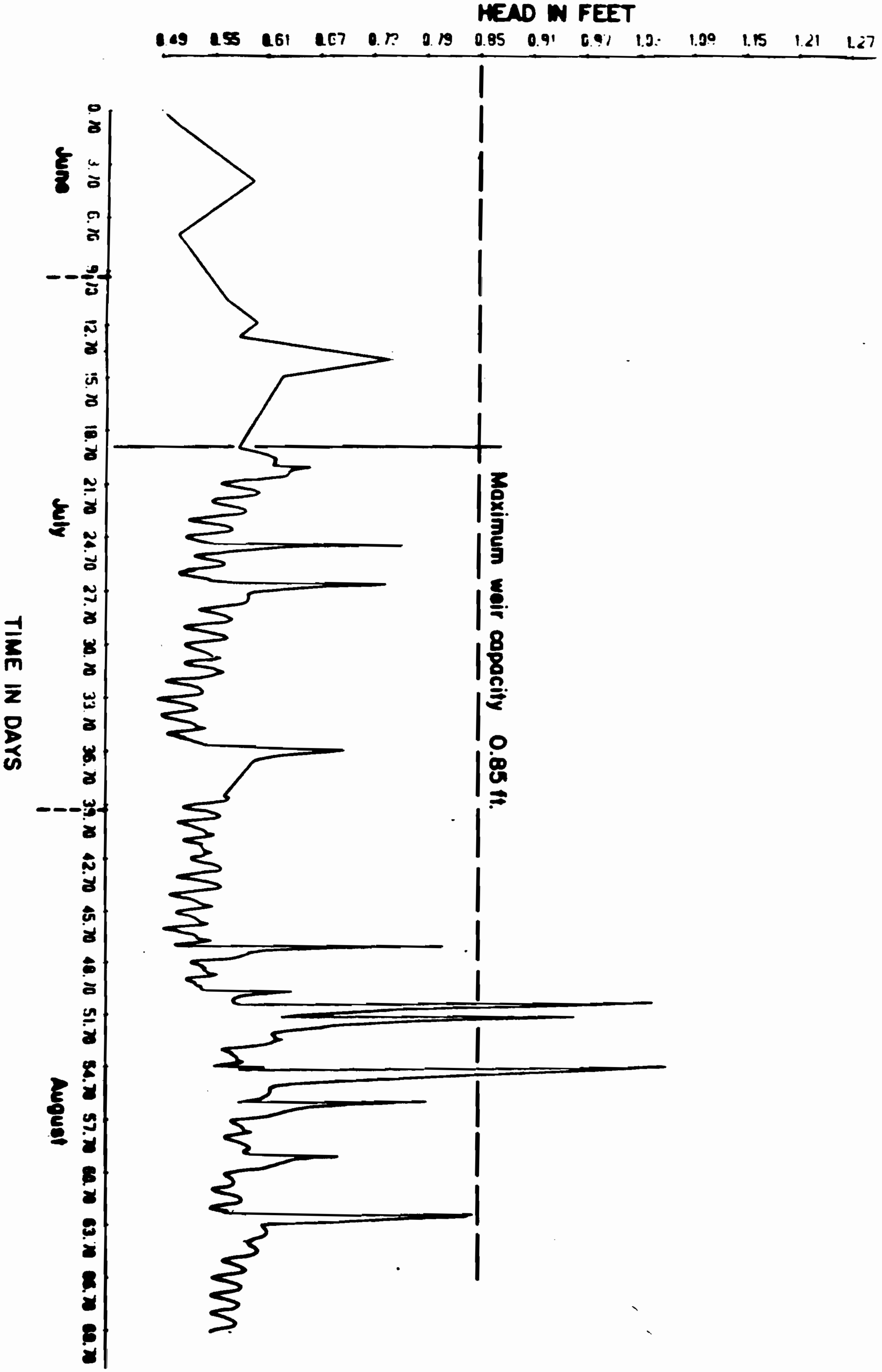

HEAD IN FEET
0.49
0.55
0.61
0.67
0.73
0.79
0.85
0.91
0.97
1.03
1.09
1.15
1.21
1.27
Maximum weir capacity 0.85 ft.
0.70
3.70
6.70
9.70
12.70
15.70
18.70
21.70
24.70
27.70
30.70
33.70
36.70
39.70
42.70
45.70
48.70
51.70
54.70
57.70
60.70
63.70
66.70
69.70
June
July
August
TIME IN DAYS

Figure 2.3a

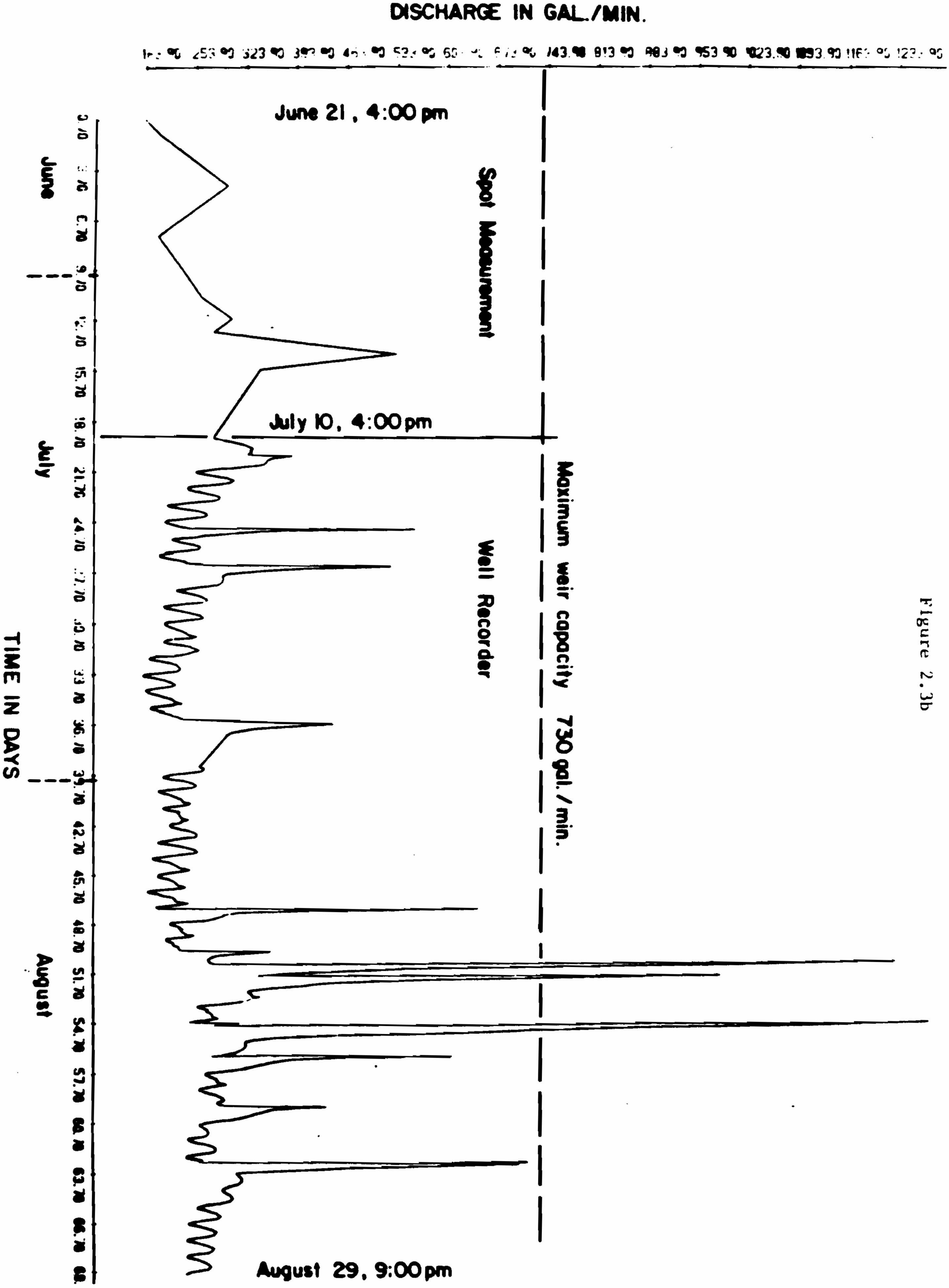

DISCHARGE IN GAL./MIN.
183.90
253.90
323.90
393.90
463.90
533.90
603.90
673.90
743.90
813.90
883.90
953.90
1023.90
1093.90
1163.90
1233.90
June 21, 4:00 pm
Spot Measurement
July 10, 4:00 pm
Well Recorder
Maximum weir capacity 730 gal./min.
August 29, 9:00 pm
0.70
3.70
6.70
9.70
12.70
15.70
18.70
21.70
24.70
27.70
30.70
33.70
36.70
39.70
42.70
45.70
48.70
51.70
54.70
57.70
60.70
63.70
66.70
69
June
July
August
TIME IN DAYS

Figure 2.3b

During the 69 days of weir operation, maximum and minimum flows were 1272 and 182 gallons per minute respectively. Average weir discharge was calculated as 287 gallons per minute. The hydrographs show that discharge is flashy, occurring in response to rainfall events. However, ground-water contributions to stream flow are significant and diurnal variations in flow due to evapotranspiration (ET) are apparent. Discharge is generally highest from 6:00 to 8:00 AM when ET demand is low; conversely, discharge is lowest from 4:00 to 6:00 PM when ET demand is high.

2.2.10 Water Quality Sampling

Ground water and surface water samples were collected in accordance with the protocols outlined in attachments 2 and 4, respectively, of the March 28, 1984 Remedial Investigation Work Plan and Implementation Schedule. The only modification was that surface water samples were collected by totally immersing the collection vial and capping it under water making sure to eliminate head space and air bubbles.

Two rounds of ground-water sampling and analysis were performed. The first took place between June 25 and July 9, 1984 encompassing well sites DGC-1 through DGC-13. The second round took place between September 5 and September 17, 1984 encompassing well sites DGC-1 through DGC-22, Town of Moreau wells TM-A through TM-G and TM-2 through TM-5 and Fort Edward well, FE-1.

Twelve rounds of surface water collection/analysis were performed between May 2 and October 4, 1984.

Results for both ground and surface waters will be discussed in later sections.

Residential well sampling followed the protocol presented in Appendix E.

Two major rounds of ground-water sampling from residential, domestic wells were performed during April 23 to May 8, 1984 and July 30 to August 8, 1984. More frequent periodic sampling was performed on a monthly or bi-monthly basis at selected residences.

All water samples (residential, surface, ground) were identified and given a sample number, logged into a chain-of-custody procedure, kept chilled, and subsequently shipped to the laboratory (via overnight express).

2.2.11 Surface Soil Sampling

On August 14, 1984, Dunn Geoscience Corporation recommended to General Electric Company a reconnaissance protocol to identify potential sites of PCB-contaminated soils. The protocol was modified slightly and submitted that day to EPA. The submitted protocol, with very minor additions, was accepted by EPA in a letter dated September 28, 1984. A copy of this letter and the protocol is found in Appendix E.

The soil sampling/analysis program is still in progress. The only phase totally completed is the staking of soil sampling locations, although a finished site map(s) has not been finalized. Staking the sites (according to the site map submitted to EPA) took place during the first three weeks in September, 1984. Since that time minor additions/corrections have been made to the initial map but have not altered the approved protocol.

The sites of soil sampling constitute the general areas around and to the east of the disposal site, bounded by Fort Edward Road. This was the area of access to the site during its time of operation.

The initial phase of sampling is still in progress, although as of the preparation of this report sample collection was suspended pending resolution of authorized access onto the site. This phase constitutes sampling all sites between a depth of four and eight inches below grade. Each site is dug with a hand shovel and the soil is transferred to glass bottles with aluminum foil-lined metal caps. Shovels are wiped clean with a paper towel, rinsed in deionized water, trisodium phosphate solution, and a final water rinse between samples. All samples are returned to Dunn Geoscience Corporation for analysis with a McGraw-Edison PCB field test kit.

It is the objective of this study to sample sites for PCBs in the soils at the grade that existed during the time of operation of the Caputo Site. In several areas this "original" grade has been disturbed by excavation or covered with "fill" materials. Sites where the original grade is not well defined or delineated were noted in the daily log and will be returned to at the completion of the 6-inch (4" to 8") sampling phase. Depths for sampling at these sites will be determined by visual inspection of excavations or soil core samples and surrounding land that does appear to be at original grade.

Field soil sampling was suspended on September 21, 1984 due to problems in obtaining access.

2.2.12 Aerial Photography

Aerial photography services were provided by Lockwood Support Services of Rochester,, New York. The study area and surrounding region was flown on April 9, 1984. A mylar manuscript, with a scale of one inch equals 200 feet and a 5-foot topographic contour was later supplied to Dunn Geoscience Corporation.

2.2.13 Surveying

Surveying of well clusters DGC-1 through DGC-13 was carried out by Dunn Geoscience Corporation during June and July, 1984. Clusters DGC-14 through DGC-22 were surveyed in September, 1984. All well elevations and locations were tied to United States Coast and Geodetic Survey bench marks. C.T. Male Associates, Latham, New York assisted Dunn Geoscience Corporation in resolving initial well elevation discrepancies created by inaccurate information on USC&GS benchmark elevations.

2.2.14 Cartography

The final base map was prepared by Dunn Geoscience Corporation from the mylar manuscript provided by Lockwood Support Services. The base map covers the entire study area shown in Figure 1.1. To produce a more functional map for purposes of the report, the original 200 feet to the inch scale was reduced to 400 feet to the inch.

2.3 Laboratory Testing

2.3.1 Grain-Size Distribution Analyses

Grain-size distribution analyses were conducted on 65 samples collected during test-hole drilling. Samples selected for analysis represent a wide range of locations and depths within the deposits. In addition, one sample of the well filter pack sand was also analyzed. All samples were analyzed according to ASTM tests C-117, C-136, and D-422 Standards.

The grain-size distribution analysis method separates the soil particles into size groups which were used to check the descriptions of the soil samples as described in the field. Mechanical separation was performed by sieving the samples through graded sieves down to a particle diameter of about 0.07 mm. Additionally, still finer particles of five samples were separated and classified using the hydrometer, or sedimentation, method. The grain-size distributions of the tested samples are included in Appendix F.

2.3.2 Triaxial Cell Testing

Empire Soils Investigations, Inc., conducted laboratory triaxial cell tests on three shelby tube samples collected during test-hole drilling. The tests were conducted to determine the vertical hydraulic conductivity (K_v) of the samples.

The test is performed in a triaxial cell, where the test specimen is enclosed in a latex rubber membrane, sealed at the base and cap with rubber O-rings. Filter paper, porous stones, and drainage leads facilitate the application of hydrostatic pressure to both ends of the specimen.

Pressure is generated by self-compensating mercury columns capable of delivering pressures up to about 140 pounds per square inch. Three mercury columns are used. One provides a backpressure to ensure saturation of the test specimen. The second column is set at a higher pressure to produce a hydrostatic gradient across the length of the specimen. The third column provides the cell confining pressure, which acts on the rubber membrane to prevent passage of water up along the sides of the specimen.

After the apparatus is assembled, and the triaxial cell is filled with degassed water, cell pressure, gradient pressure, and backpressure is gradually applied until the preset pressure levels on the self-compensating mercury columns are reached.

To minimize consolidation effects, net confining pressure of 10 pounds per square inch was selected for this series of tests in conjunction with a backpressure of 80 pounds per square inch. It was judged that this net confining pressure would approximate conditions in the field, while the backpressure would assure saturated conditions.

A hydrostatic gradient is established across the length of the sample by increasing the pressure at the base of the specimen while maintaining the backpressure at the top. The rate of flow through the specimen is determined by burette readings, at timed intervals, on a volume change unit inserted in the gradient pressure line. The flow rate is routinely determined for two different gradient pressures. For this study, gradient pressures 0.4 to 10 pounds per square inch higher than the backpressure were used. At each gradient pressure, the hydraulic conductivity test stage is conducted for a sufficient period of time to accurately determine the flow rate.

The Permeability Test Report forms are located in Appendix D and contain all pertinent sample identification and specimen data. Graphical presentations are given of the data upon which the flow rate determination was made at the respective gradient pressures, as well as the vertical hydraulic conductivity at the respective hydraulic gradients applied during the test. Individual test results are tabulated in the Appendix.

2.3.3 Chemical Analyses: Soil Borings

Selected soil boring samples obtained during well construction were sent to the laboratory for chemical analysis of volatile (purgeable) halogenated hydrocarbons by gas chromatography. Sample preparation and analysis followed a procedure developed by the New York State Department of Health. A copy of this procedure is included in Appendix E. The organic compounds analyzed for are listed below:

Bromodichloromethane
1,2-dichloropropane
Trans-1,3-dichloropropylene
Trichloroethylene
Dibromochloromethane
Cis-1,3-dichloropropylene
1,1,2-trichloroethane
Bromoform
1,1,2,2-tetrachloroethane
Tetrachloroethylene
Chlorobenzene
2-chloroethyl vinyl ether

Chloromethane
Bromomethane
Vinyl chloride
Chloroethane
Methylene chloride
1,1-dichloroethylene
1,1-dichloroethane
1,2-trans-dichloroethylene
Chloroform
1,2-dichloroethane
1,1,1-trichloroethane
Carbon tetrachloride

Table 2.3 lists the individual sample site and date, pertinent location information and analytical comparisons to HNU and Draeger tube field testing. Laboratory data on all subsurface soil samples is located in Appendix J.

2.3.4 Chemical Analyses: Near-Surface Soils

A PCB Field Test Kit, manufactured by McGraw-Edison, was used to estimate the concentration of polychlorinated biphenyls (PCBs) in near-surface soil. Ten percent of these samples were rechecked by laboratory analysis.

The field kit testing procedure basically consists of the following four steps:

- Extraction of PCB molecules from the soil;

Table 2.3.

SOIL BORINGS

VOLATILE ORGANICS

Sample No.	DGC Site	Depth (ft.)	Sampling Date	HNU	DRAEGER	Parameter	Concentration, ug/L
812	5	50-52	5/24/84	29	-	ND	
828	5	60-62	"	140	-	ND	
827	5	65-67	"	130	-	ND	
820	5	70-72	"	21	-	ND	
813	5	85-87	"	4.0	-	ND	
879B	7	45-47	5/31/84	6.8	0	ND	
880B	7	55-57	"	56	60	1,2-trans-dichloroethylene	530
						Trichloroethylene	1600
677	6	58-60	"	6.0	0	ND	
912	7	0-2	"	27	0	ND	
886	7I	46-48	"	12.0	2.0	ND	
884A	6	96-98	"	4	0	ND	
913	8	25-27	6/7/84	7.6	-	ND	
915	8	55-57	"	7.2	0	ND	
918	8	45-47	"	4.2	-	ND	
920	8	35-37	"	7.2	0	ND	
923	8	75-77	"	5.2	0	ND	
928	9	30-32	"	4.9	0	ND	
932	9	75-77	"	3.4	0	ND	
951	9	55-57	"	4.4	2.0	ND	
946	10	35-37	6/8/84	6.2	0	ND	
947	10	40-42	6/8/84	11.2	2.0	Trichloroethylene	99
114	10S	26-28	6/12/84	3.2	0	ND	
862	11	28-30	6/8/84	5.3	0	Methylene Chloride	100
893	11	38-40	6/8/84	6.3	2.0	ND	
898	11	48-50	6/8/84	4.2	2.0	ND	
890B	11	58-60	6/8/84	2.3	30.0	1,1-dichloroethane	<110
						1,2-trans-dichloroethylene	<110
						Chloroform	<110
118	11	74-76	6/8/84	2.8	20	Trichloroethylene	1100
113	11I	46-48	6/11/84	3.8	0	Trichloroethylene	<76
145	12	35-37	6/13/84	3.0	0	ND	

ND - Not detected for all parameters in list

- Extraction of chloride substituents from the PCB molecules;
- Measurement of the chloride ion concentration using a specific ion electrode (probe); and,
- Relating chloride level to original PCB level in the soil.

The testing procedure is outlined for a single soil sample, although in practice groups of five samples were tested simultaneously. All testing equipment, including vials and chemicals for which trade names alone were given, were supplied by the Kit manufacturer. For the first 35 samples, calibration of the probe was carried out after each batch of five samples. Adjustments were rarely needed at this frequency, with two millivolts being the maximum correction required. Subsequently, the calibration frequency was reduced to every two batches (10 samples).

A soil sample jar is uncapped, foil top removed, and the contents emptied into a 12-inch by 9-inch by 2-inch metal baking pan. A small spatula is used to take six scoops of soil from various locations throughout the pan and fill an empty vial one-third to one-half full. The remaining soil is transferred back to the jar, fresh aluminum foil added, cap screwed on, and stored for possible further analysis. The pan is cleaned out using a paper towel in preparation for the next sample. The spatula is wiped clean with a paper towel and rinsed in deionized water. A volume of Soil

Extract Solvent, equivalent in weight to the soil in the vial, is added to the soil vial, capped, and shaken for 30 seconds. Mixing of the soil and extraction solvent will release a minimum of 25 percent of the soil's PCB molecules to the solvent.

A second vial, called a Reaction Vial, containing a premeasured amount of solvent has one milliliter (ml) of Reaction Fluid added to it. The resultant solution is capable of altering chemical bonds in PCB molecules, thereby releasing chloride ions. One ml of the PCB-containing solvent from the soil vial is transferred to the Reaction Vial using a 1-ml pipetor with a disposable tip. The reaction vial is shaken for twenty seconds. Five ml of a chloride Extraction Fluid is then added to the Reaction Vial. The vial is capped and shaken for ten seconds. After one or two minutes the liquid separates into two distinct layers, with the lower aqueous layer containing the chloride ions.

The probe is removed from a Rinse, in which it rests, and wiped with a fresh lab tissue. The probe is inserted through the top layer and into the bottom layer of the liquids in the Reaction Vial. Probe equilibration is obtained in less than two minutes at which time the response is recorded on the record sheet. The probe is wiped clean with a fresh lab tissue, swirled in a beaker of deionized water, wiped dry with another clean tissue, and replaced in the rinse until the next usage.

The recorded probe response (in millivolts) is related to PCB concentration (ppm) by a pair of measurement charts provided with the Kit. One chart is to be used when dealing with PCB as Aroclor 1242, and the other is to be used when dealing with PCB as Aroclor 1260. The method detection limit for Aroclor 1242 in soil is 36 ppm with this Kit. The results presented in Section 10 assume PCBs are present as Aroclor 1242, a reasonable assumption since Aroclor 1242 was used in capacitor manufacture. This assumption may over-estimate the actual concentration of PCBs present in a given sample. The charts demonstrate an inverse proportionality between the logarithm of PCB concentration (ppm) and probe response (mV). The charts are designed for direct reading when analyzing PCBs in transformer oil. Knowing that at least 25 percent of the PCB molecules are released from the soil after adding the Soil Extract Solvent, the maximum PCB content is four times the value indicated on the chart. The calculated value is then recorded on the record sheet.

All soil samples collected have been analyzed with the PCB test kit. Duplicate analyses on ten percent of the samples are in progress. Soil samples containing detectable concentrations of PCB, as well as a number of selected samples, totaling a minimum of ten percent of all collected samples, were sent to Environmental Testing and Certification, Edison, New Jersey, for EPA SW-846 (8.08) analysis.

2.3.5 Chemical Analyses: Water

All aqueous samples - surface, ground and residential - were analyzed for volatile organic compounds by the same general chromatographic methods, either EPA Method 601 or EPA Method 624. Residential well and surface water samples were run individually or by combining several samples as a composite. Three 40 ml vials of water were collected from each sampling point. The first of these would be used to make a composite, consisting of no more than five samples. The laboratory was instructed to analyze the individual samples making up a composite if the concentration of one or more chemicals in the composite sample exceeded the trigger levels listed below:

TRIGGER LEVELS

Trichloroethylene	13.5/n ppb
Any Individual Organic Chemical	45.0/n ppb
Total Organic Chemicals	90.0/n ppb
Vinyl Chloride	4.5/n ppb

where n is the number of individual samples making up a composite.

If the concentration of one or more chemicals in an individual sample as measured by Method 601 falls within 10 percent of the action levels specified in the protocol, the sample was analyzed by Method 624. In this case the result of the Method 624 analysis will determine if action levels, listed below, had in fact been exceeded.

ACTION LEVELS

Trichloroethylene	15 ppb
Any Individual Organic Chemical	50 ppb
Total Organic Chemicals	100 ppb
Vinyl Chloride	5 ppb

EPA Test Method 601 is a purge and trap gas chromatographic method applicable to the determination of purgeable halocarbons as provided under 40 CFR 136.1. EPA Test Method 624 is a purge and trap gas chromatographic/mass spectrometer (GC/MS) method applicable under the same 40 CFR 136.1 criteria.

Priority pollutant analyses were run on a few monitoring well samples.

The laboratory contracted to perform the analyses of all water samples was ERCO/Energy Resources Company, Inc., of Cambridge, Massachusetts. Their Quality Assurance Program plan is found in Appendix G.

3.0 GEOLOGY

3.1 Regional Description and Geomorphology

The greater Glens Falls region is an area of diverse geology and topography. Elevations range from about 1500 feet above mean sea level in the Luzerne Mountains at the western edge of the region to about 110 feet in the Hudson River floodplain south of the Village of Fort Edward. The total relief of the area is about 1400 feet.

East of the Luzerne Mountains a broad low relief plain, composed predominantly of sand, stretches east, roughly to the position of the Hudson River. The Hudson River, flowing eastward through the Luzerne Mountain gap, meanders across a relatively wide floodplain over the deltaic sandplain. To the east and northeast of the relatively flat-topped deltaic sandplain, low lying lacustrine clays deposited in glacial Lake Albany and its successors are observed. Northeast of the Glens Falls-Hudson Falls area, till hills or drumlins can be observed rising up above the lacustrine clays. The drumlins exhibit a roughly northeast-southwest orientation corresponding to the direction of glacial ice movement responsible for the till deposition.

East of the deltaic plain, lacustrine clays can be found extending out to the Taconic front, located about four miles east of the Village of Fort Edward. The Taconic Region is predominantly a till covered highland reaching elevations in excess of 1000 feet above mean sea level.

The major surface water body in the region is the Hudson River and the associated canal system. The Hudson flows primarily to the east from the Luzerne Mountain gap to Hudson Falls where a southerly flow is initiated. Clendon Brook as well as numerous unnamed intermittent streams draining the region, flow into the river.

The Hudson River flow ranges from a maximum daily discharge, as measured at the Fort Edward gauge station, of 35,000 cubic feet per second, to a minimum daily discharge of 1,000 cubic feet per second. (Note: 1 cubic foot per second equals 449 gallons per minute). The yearly mean flow of the Hudson is 4,981 cubic feet per second.

Located within the study area are four reservoirs; New, Sanderspree, Dority, and Christie. These reservoirs, located in the Town of Moreau, provide water to the Fort Edward Water District.

3.2 General Stratigraphy

Unconsolidated deposits of glacial origin overlie bedrock as shown in Table 3.1. The glacial deposits reach an observed thickness in excess of 120 feet. The vertical distributions of these deposits is shown with a vertical exaggeration of five to one in four cross-sections (Plates 3, and 4). Glacial deposits observed in the study area were deposited by or in conjunction with the Laurentide Ice sheet which covered much of northern North America during the Late Wisconsin glaciation. Three major types of sediments were found in the study area: fine-grained glaciolacustrine sediments, deltaic sand deposits, and till. Over most of the site, the unconsolidated glacial deposits are overlain by solum (topsoil).

Table 3.1

GENERALIZED STRATIGRAPHY AND GEOHYDROLOGY

THICKNESS (Feet)	STRATIGRAPHIC UNIT	DESCRIPTION	GEOHYDROLOGIC UNIT
5 to 88	Glaciodeltaic	Light brown to brown, coarse to fine sand with a trace of silt. Discontinuous layers of coarse sand and fine gravel. Occasional black-red sand seams.	Moreau Sand Aquifer
0 to 28	Upper Glacio-lacustrine	Gray medium to fine sand with some silt. Frequent silt and clay seams.	
2 to 25+	Lower Glacio-lacustrine	Gray varved silt and clay. Frequent seams of fine sand in the upper section.	Confining Bed
3± to ?	Lodgement Till	Dark gray sand and gravel in a clayey silt matrix	
?	Bedrock	Medium to dark gray, thinly bedded argillaceous limestone.	Bedrock Aquifer

3.3 Bedrock

The Glens Falls region is underlain by three major bedrock types; metamorphic crystalline rocks, shelf carbonates, and basinal shales. The bedrock structure in the area is dominated by a series of roughly northeast-southwest trending high-angle normal (block) faults.

The Luzerne Mountains, which comprise the western border of the region are composed exclusively of intensely deformed high-grade Pre-Cambrian metamorphic rocks.

Late Cambrian to Late Middle Ordovician shelf carbonates are found in the area from South Glens Falls north to Glen Lake. The carbonates are composed of interbedded limestones and dolostones with occasional sandstone and siltstone members. The carbonate sequence is wholly contained in the Beekmantown, Black River, and Trenton groups. The total thickness of the carbonate sequence is in excess of one-thousand feet in some areas. The carbonates extend south of the Hudson River in the vicinity of Glens Falls, the regional dip is gentle, generally less than 5 degrees, and to the south.

Based on bedrock exposures in the Town of Moreau, the uppermost carbonate unit, Glens Falls limestone, appears to completely underlie the study area. The Glens Falls limestone is best described as a medium to dark gray, thinly-bedded limestone. Information from a local well drilling contractor indicates that the top of bedrock was encountered at a depth of 125 feet in drilling a well at the Moreau Elementary School. Below the Glens Falls limestone are other, older limestone formations which collectively form a relatively thick carbonate sequence.

In the area south of the Moreau study area, the Glens Falls limestone is overlain by Late Middle Ordovician basinal shales. Near the faulted contact with the limestone, the thickness of the shale is probably a few hundred feet. The Snake Hill shale extends south to Northern Albany County where it is up to 1300 feet thick. Unlike the underlying carbonate sequence, which is gently folded, the shale is moderately to intensely folded.

3.4 Glacial Till

The oldest unconsolidated deposit observed in the study area was a three-foot thick layer of glacial lodgement till. The till was found in the one boring, DGC-9D, which penetrated the overlying glaciolacustrine silts and clays. The dark gray till observed is composed of sand and gravel in a clayey silt matrix and overlies a relatively unweathered bedrock surface.

The lodgement till observed underlying the site is the product of deposition from a continental glacier. The till was deposited by the Hudson-Champlain lobe of the Laurentide ice sheet of Late Wisconsin Age. The Hudson-Champlain lobe represents the last of the four major North American glacial stages. As the Hudson-Champlain lobe advanced south, it scoured out older unconsolidated deposits and weathered rock down to a fresh bedrock surface. Subsequently, the dark gray till observed in DGC-9D was deposited over the relatively fresh bedrock surface.

Although the overlying glaciolacustrine silts and clays were completely penetrated by only one boring, the depositional processes, which deposited the till observed in that boring, were operating over the entire study area. Based on the mode of deposition, it is most probable that glacial till directly overlies bedrock throughout the site.

3.5 Lower Glaciolacustrine Deposits

Lower glaciolacustrine sediments observed during the drilling are primarily gray, soft, varved silt and clays. Silty seams and layers containing some fine sand are frequently encountered and are typically one-sixteenth inch to one-inch thick. The frequency of these seams and layers appear to generally decrease with depth.

Plate 5 illustrates the top of the lower glaciolacustrine clay unit beneath the study area and was developed from well log information obtained during test-hole drilling. The map indicates an irregular top of clay surface with approximately 40 feet of relief. The clay surface reaches a maximum observed elevation of 282 feet in the area of clusters DGC-10 and DGC-16. From this high point, the clay surface appears to slope down to the north, south, and west. No information for the area east of the high is available. In the region of the erosional escarpment, the clay begins to climb from a low elevation of approximately 250 feet to about 270 feet east of the escarpment. Although borings do not fully penetrate the lower lacustrine deposits, it is likely that the clay surface roughly corresponds to the buried bedrock surface.

The lower glaciolacustrine silts and clay overlie till and/or bedrock in the study area. The upper contact with the upper glaciolacustrine sediments is, by nature, gradational. The silts and clays observed are the product of predominantly vertical sedimentation in Glacial Lake Quaker Springs, the successor to Lake Albany. Fine-grained sediments introduced into the lake basin by the growing Glens Falls deltaic complex were suspended in the waters of the lake. The rhythmic nature of the sediments, varving, is probably indicative of relative lake energy due to seasonal changes. During the "summer" period, lake water was fairly agitated allowing primarily silt to settle to the lake floor. "Winter" conditions, probable ice cover, allowed clay-sized particles to settle in the calm lake waters.

Based on data from DGC-9 and the Moreau Elementary School well, it appears that the thickness of lower glaciolacustrine silt and clay varies from at least 3 to 25 feet in the study area.

3.6 Upper Glaciolacustrine Deposits

Upper glaciolacustrine deposits of varying compositions overlie the lower glaciolacustrine silts and clays. The composition is variable but generally consists of gray medium to fine sand with some silt. Gray silt and clay seams are frequently observed in the upper glaciolacustrine deposit.

The contact with the lower glaciolacustrine deposits is gradational as is the upper contact with glaciodeltaic deposits. With the exception of DGC-9D, where no upper lacustrine deposits were observed, the thickness of the deposits range from about 28 feet in cluster DGC-15 to a minimum of 5 feet observed at cluster DGC-16. Although the thickness is quite variable, a few generalizations can be made. Based on the boring information, the upper glaciolacustrine deposits appear to be thickest over the northwestern portion of the study area. The southern, central portion of the study area, surrounding cluster DGC-10 appears to have the thinnest accumulation of upper lacustrine deposits. The deposit then appears to thicken both to the east toward DGC-4, and south toward cluster DGC-1. Although data are sparse to the west of DGC-9, it appears likely that the upper glaciolacustrine deposits thicken to the west as well.

The upper glaciolacustrine deposits, when present, are always found between lower glaciolacustrine silt and clay and glaciodeltaic sand deposits. The fine sand and silt which comprises this deposit owes its character and origin to both the lacustrine and deltaic environments. The upper glaciolacustrine environment is best thought of as transitional from deltaic to true lacustrine deposition. Deltaic fine sand and silt are interbedded with silt and clay of lacustrine origin.

3.7 Glaciodeltaic Deposits

The uppermost stratigraphic unit overlying the glaciolacustrine silts and clays consists primarily of light brown to brown, coarse to fine sand with a trace of silt. Discontinuous layers of coarse sand with a trace to some medium fine gravel are sometimes found associated with the brown sand deposits. Occasional medium and fine gravel dropstones are also found within the glaciodeltaic sand deposits. Red-black sand seams, composed predominantly of garnet and other heavy minerals are frequently encountered.

The thickness of glaciodeltaic deposits varies from a maximum of 88 feet in DGC-15 to a minimum of 5 feet in DGC-13. In general, the glaciodeltaic deposits thicken to the west-northwest over the site. DGC-13 lies east of the roughly north-south trending erosional escarpment. As such, it is likely that some amount of glaciodeltaic sand has been eroded from the area. The sand reaches its lowest observed topographic position of 254 feet above mean sea level at DGC-14. The contact with the underlying upper glaciolacustrine deposits is gradational and is usually expressed as increasing silt, decreasing sand, color change from brown to gray, and the occurrence of silt and clay seams.

The glaciodeltaic deposits are part of the Glens Falls deltaic complex. The delta began forming as the Hudson-Champlain glacial lobe retreated north past the Luzerne Mountain gap. The gap is located approximately 6 miles west of the study area and separates the Luzerne Mountains and the Palmertown Range. During the northward retreat, the ice maintained the level of glacial Lake Albany at about 430 feet above sea level. Deglaciation of the Luzerne Mountain gap allowed glacial meltwaters derived from deglaciating uplands to flow through the gap into Lake Albany. During the initial stages of delta development, the study area was still ice covered.

Around the time the study area was deglaciated, Lake Albany lowered to about 350 to 360 feet in the Moreau area. Lowering of Lake Albany to Lake Quaker Springs level exposed the Luzerne Mountain gap. Delta building shifted to the east of the gap. Sand influx into the study area was minimal and lake bottom silts and clays are the major deposits. Successive progradation of the delta into the basin increased the amount of sand available for deposition over the site. The delta advanced over the site during Lake Quaker Springs time, depositing up to 88 feet of deltaic sands and gravels.

The deltaic deposits generally coarsen upward and gravel seams and lenses are confined to the upper one-half to one-third of the delta.

The generally upward coarsening of the deposit is characteristic of delta deposition. The coarsening was caused by the increasing energy available as the delta prograded and water and depositional waters became shallower.

Lowering of Lake Quaker Springs to a lower (280 feet) Lake Coveville level signaled the end of deltaic deposition over the study area. Erosion of the Glens Falls delta and the subsequent formation of the roughly North-South trending escarpment near the eastern edge of the study area occurred at this time. The escarpment is actually an erosional terrace caused by the erosive action of glacial Lake Coveville on the previously deposited glaciodeltaic deposits. To the east of the terrace, in the area of DGC-13, nearly all the deltaic sands have been removed. Sand thickness of only 5 feet was observed. Further to the south and east the sand completely disappears and glaciolacustrine deposits are exposed.

4.0 GEOHYDROLOGY

4.1 Geohydrologic Units

The stratigraphic units described in the section on geology can be grouped into three distinct geohydrologic units on the basis of hydraulic conductivity and other hydrologic properties. Their grouping into broader geohydrologic units does not suggest that the individual stratigraphic units are distinctly homogeneous. Variations in the lithology, texture, thickness and extent of the individual stratigraphic units are expected and do occur. However, the variations do not necessarily result in large differences in hydraulic characteristics, so that it is possible to combine them.

The three geohydrologic units in the study area are the Moreau sand aquifer, a semi-confined, artesian, bedrock aquifer, and a confining bed which separates them. These three units are shown opposite the corresponding stratigraphic units in Table 3.1.

Because the scope of the remedial investigation focused on the Moreau sand aquifer, neither the bedrock aquifer nor the confining bed were investigated in detail. In addition, the necessity to avoid unintentionally providing conduits through which contaminants might enter the bedrock aquifer, precluded test drilling to any significant depth below the Moreau sand aquifer. Nevertheless, brief descriptions of the bedrock aquifer and its confining bed precede the more detailed discussion of the Moreau sand aquifer below.

4.1.1 Semi-Confined Bedrock Aquifer

The semi-confined bedrock aquifer underlies the two other geohydrologic units present in the study area. It is composed of a sequence of carbonate rocks most of which are calcareous limestones. The aquifer lies directly beneath a confining bed which separates it from the overlying Moreau sand aquifer. Data from DGC-9, the only test boring completed into the bedrock aquifer, shows the elevation of the top of aquifer at this point is about 275 feet above sea level. Other data collected during the drilling of a well at the Moreau Elementary School indicates that the surface of the bedrock aquifer occurs approximately 125 feet below land surface, or at an elevation of about 215 feet. Very limited data from three domestic bedrock wells drilled along Fort Edward Road suggest that the top of the aquifer decreases from about 280 feet at the intersection with Bluebird Road to about 240 feet at the intersection of Sisson Road. Although these few points are too sparse to approximate the configuration of the aquifer's surface, they do demonstrate that the top of the bedrock aquifer is irregular and expresses considerable relief within the study area.

Ground water within the bedrock aquifer occurs along the bedding planes and in the joints, fractures, and solution cavities of the carbonate rocks. A quarrying operation approximately 1.5 miles northeast of the GE/Moreau site has exposed carbonate rocks which comprise a portion of the bedrock aquifer. Observations made there suggest that the bedrock aquifer is generally dense and exhibits low hydraulic conductivity. However, well records from Northern Saratoga LCounty show that well yields from the carbonate bedrock aquifer range from 4 to 300 gallons per minute with a median yield of about 22 gallons per minute. This information indicates that the bedrock aquifer, in at least some locations, has a moderate potential for development. In comparison, the yields of wells completed in a shale bedrock aquifer south of the Moreau study area range from two to six gallons per minute.

Due to the inaccessibility of the relatively few bedrock wells in the study area, no water-level measurements were made in the bedrock aquifer. However, the static water level in a Moreau Elementary School well, shortly after its completion, was reported as being above the top of the aquifer. The difference between the top of the aquifer and the reported water level indicates that the bedrock aquifer is artesian, and that water levels measured in wells completed solely within the aquifer represent a potentiometric surface.

4.1.2 Confining Bed

The lower glaciolacustrine and lodgement till stratigraphic units described in the geology section collectively comprise a confining bed overlying the bedrock aquifer as shown in Table 3.1. Both stratigraphic units exhibit lower hydraulic conductivity than the overlying glaciodeltaic sand deposits due to their higher content of silt and clay. Therefore, the confining bed tends to retard the vertical flow of ground water through it.

Because the lower glaciolacustrine deposits comprise the upper portion of the confining bed, the clay surface illustrated on Plate 5 also depicts the top of the confining bed. As indicated on Plate 5, and as supported by the geologic depositional history of the region, the confining bed is continuous over the study area.

Based on the limited information available, the confining bed within the study area varies from about 3 to 25 feet thick. At most locations within the study area, the confining bed is presumed to include both the lower glaciolacustrine deposits and the lodgement till. However, the lodgement till is reportedly absent at the Moreau Elementary School well, whereas the lower glaciolacustrine deposits are absent at well DGC-9. Consequently, in such areas, the confining bed is comprised only of the single stratigraphic unit present.

In order to preclude the introduction of contaminants into the bedrock aquifer, no test holes were drilled into the lodgement till. Consequently, no field or laboratory tests were conducted to determine the hydraulic conductivity of the till. However, the till is composed of an unsorted mixture of gravel, clay, silt, and sand-sized particles; the unsorted nature characteristic of till generally tends to make it a poor transmitter of ground water.

Laboratory triaxial testing conducted on the shelby tube samples of the lower glaciolacustrine sediments obtained in DGC-14, indicate a vertical hydraulic conductivity of about 5.1 x 10^{-7}cm/sec. Field horizontal hydraulic conductivity tests were not performed on the lower lacustrine deposits. The presence of observed silt and fine sand seams, especially in the upper portion of the deposit, indicate that horizontal hydraulic conductivity is probably higher than the vertical hydraulic conductivity determined in the laboratory.

4.1.3 Moreau Sand Aquifer

The Moreau sand aquifer is the uppermost geohydrologic unit in the study area. The aquifer is comprised of the upper glaciolacustrine stratigraphic unit and the saturated portion of the glaciodeltaic unit as shown in Table 3.1. On average, the upper 75 percent of the aquifer is composed of the glaciodeltaic unit, and the remainder composed of the upper glaciolacustrine deposits.

The Moreau sand aquifer is the study area's most productive aquifer where it occurs north and west of the erosional scarp.

Ground water within the Moreau sand aquifer occurs under unconfined, or water-table, conditions. The base of the aquifer coincides with the top of the confining bed and is, therefore, depicted by Plate 5.

The top of the aquifer occurs at the water table and is free to rise or fall in response to ground-water recharge and discharge. The top of the aquifer was determined during the remedial investigation by measuring water levels in shallow wells located within the study area. Its position between July and September, 1984 is shown on Plates 6, 7, and 8. The three maps show that the elevation of the Moreau sand aquifer ranged from about 325 feet near the GE/Moreau site to about 285 feet at the erosional escarpment. The configuration of the aquifer surface was nearly constant during that time as evidenced by only slight changes in the contour lines on the plates.

On average, the aquifer is about 60 feet thick, but varies significantly. The aquifer is thickest in the northern and western portions of the study area reaching between 81 and 89 feet at wells DGC-14, 15, 20, and 21. Although data is unavailable, the history of geologic deposition in the area would suggest the aquifer may be thicker to the northwest. Aquifer thickness declines steadily toward the southeast, until the aquifer is only about 40 feet thick immediately north and west of the erosional scarp. The scarp represents the southeastern boundary of the aquifer in the study area since the thickness of sand decreases abruptly east of this position. Although section A-A' on Plate 3 shows that the two stratigraphic units comprising the Moreau sand aquifer extend beyond the scarp, their combined thickness has been reduced significantly such that their ability to store and transmit water has been greatly diminished. The decreasing aquifer thickness is not unexpected and reflects the processes of deposition and subsequent erosion described in Section 3.0.

In one isolated area west of the erosional escarpment, the aquifer thickness lessens to between 36 and 38 feet. This area is located near wells DGC-10, 16, and 17 and coincides with the mound in the underlying confining bed shown on Plate 5.

Field falling-head hydraulic conductivity tests were conducted on upper glaciolacustrine material in two boreholes, DGC-8D and DGC-6D. Results show that observed vertical hydraulic conductivity values are in the range of 4.0 to 6.2 x 10^{-6} cm/sec. Laboratory constant head triaxial tests were also performed on the shelby tube samples submitted from boring DGC-15D. The tube contained two types of upper glaciolacustrine material, gray fine sand, and gray silt. Laboratory testing indicates upper glaciolacustrine hydraulic conductivity from 1.7 x 10^{-4} cm/sec for fine sand to 1.2 x 10^{-5} cm/sec for upper glaciolacustrine silt. Horizontal hydraulic conductivity was measured for upper glaciolacustrine sediments in DGC-11D and DGC-5D. Utilizing the slug method, a horizontal hydraulic conductivity value of about 2.0 x 10^{-3} cm/sec (5.7 ft/day) was obtained. Based on the field and laboratory testing, horizontal hydraulic conductivity appears to be greater than vertical hydraulic conductivity by approximately three orders of magnitude. This large difference in hydraulic conductivity is not unexpected. Silt and clay seams, which occur sporadically within the upper glaciolacustrine reduce vertical hydraulic conductivity while having little or no effect on horizontal hydraulic conductivity.

Field horizontal hydraulic conductivity tests were performed on the glaciodeltaic deposits in well clusters DGC-3, 4, 5, 8, 10, and 11. The calculated horizontal hydraulic conductivities range from 2.4×10^{-3} to 2.1×10^{-2} cm/sec, with an average of 7.4×10^{-3} cm/sec (21 ft/day). A total of 25 field falling head hydraulic conductivity tests were conducted on glaciodeltaic deposits. The range of calculated values lies from 4.4×10^{-6} to 2.0×10^{-2} cm/sec. The calculated average vertical hydraulic conductivity for glaciodeltaic sediments is 1.6×10^{-3} cm/sec. Based on the calculations, the horizontal hydraulic conductivity is approximately 4.5 times greater than the vertical hydraulic conductivity. This relationship is not unusual in stratified deposits such as the glaciodeltaic sands.

These test results indicate that both horizontal and vertical hydraulic conductivity vary with depth, coinciding with the two stratigraphic units which comprise the aquifer. On average, horizontal hydraulic conductivity is between three and four times greater in the upper 75 percent of the aquifer than nearer the base. However, the largest difference between the upper portion of the aquifer and near its base occurs with respect to vertical hydraulic conductivity. In this regard, the vertical hydraulic conductivity in the lower 25 percent of the aquifer is more then 300 times less than in the rest of the aquifer.

These differences are attributed to the increased percentage of fine-grained sediments, and greater degree of stratification that occurs closer to the aquifer base.

The short period of time allowed to complete the remedial investigation prevented collection of water-level measurements for more than three months. In addition, the irregular and infrequent measurements available from documented sources prevents the construction of a hydrograph for even one complete year. Nevertheless, enough previous data are available to recognize that ground-water levels in the Moreau sand aquifer follow the classic annual cycle common to much of the northeast.

Ground-water level fluctuations in the Moreau sand aquifer are due to a net change in the amount of water stored within the deposits. The storage change results from the interaction of ground-water recharge and discharge in the study area.

Recharge to the Moreau sand aquifer is derived primarily from the downward seepage of rain or melted snow which occurs throughout the area. The main components of recharge in the study area are infiltration and percolation of part of the area's total precipitation. A detailed analysis of recharge is the subject of the water budget presented in Section 6.0. The three processes involved in recharge to the aquifer are as follows:

- infiltration of the water from the ground surface into the soil;

- percolation or downward movement of the water through the vadose zone; and,

- arrival of the water at the water table where it enters the aquifer.

The relationship between recharge and total precipitation at the site is governed by many factors among which are type of precipitation, storm characteristics, soil cover, soil moisture conditions, topography, and vegetative cover. These factors determine how much precipitation will infiltrate the soil to move downward as percolation or return to the atmosphere by evapotranspiration. Therefore, since recharge is clearly a residual value, the amount of precipitation falling on the study area is not, by itself, an accurate indication of ground-water level changes. Only a portion of the water infiltrating the soil, for example, actually reaches the water table and enters the aquifer.

Once in the aquifer, ground water moves towards the erosional escarpment where it seeps out as springs or into streams in a process called discharge. Movement of the water in this process occurs under the influence of gravity and is in the direction of the hydraulic gradient.

The relationship between recharge and discharge in the study area can be considered generally in the context of the equation of hydrologic equilibrium. The equation is a statement of the fundamental principle of ground-water hydrology that recharge is equal to discharge plus or minus changes in ground-water storage. In symbolic form, it is:

$$R = D \pm \Delta S$$

where: R is ground-water recharge

D is ground-water discharge

ΔS is the change in ground-water storage

Under natural conditions, the aquifer storage as represented by the zone of saturation, tends to remain in balance with recharge and discharge. Recharge occurs intermittently during and immediately following periods of precipitation. Discharge, on the other hand, occurs continuously as long as the water table stands at a higher level than the discharge area which, in this case, is at the erosional escarpment. Since the aquifer is unconfined, the zone of saturation is free to expand during periods of recharge and to contract during the intervening periods. During periods when recharge exceeds discharge, water is added to storage in the void spaces of the deposits; consequently, the zone of saturation expands, and the water table rises. During the remainder of the time, discharge at the escarpment, which occurs more or less continuously, depletes the water in storage; gravity drainage of the interstices occurs causing the zone of saturation to contract, and water levels to decrease.

This process operates in the study area as previous data suggests that water levels in the aquifer follow a fairly rhythmic seasonal pattern reflecting the net change of water stored in the aquifer due to the interaction of recharge and discharge. During the summer, the first part of precipitation absorbed from each rainfall event replaces the soil moisture previously depleted by plant growth. Consequently, as water levels fall, little, if any, excess water moves downward through the vadose zone to the water table to offset discharge to the springs and streams. During the winter and early spring, there is relatively little moisture deficiency in the soil zone so that most precipitation absorbed by the soil ultimately reaches the zone of saturation. Since the amount of water reaching the aquifer is greater than that being discharged there is a rise in the water table as storage increases and the zone of saturation becomes thicker. The high water levels are normally maintained throughout the spring months until increased evapotranspiration results in reduced percolation accompanied by falling water levels during the summer. This cycle is repeated each year with only slight variations in the range of water level.

A first-cut analysis of the ground-water balance in the Moreau aquifer was made by utilizing the steady-state equation of hydrologic equilibrium. Ground-water discharge on August 28, 1984 was calculated for a cross-sectional area of the aquifer between well clusters DGC-1 and DGC-4, a lineal distance of about 2700 feet. Based on the local hydraulic gradient on that day (0.03 ft/ft), the average thickness of the aquifer (48 ft), and average hydraulic conductivity values for the upper and lower portions of the aquifer (21 ft/day and 5.7 ft/day, respectively), the daily discharge was estimated at about 500,000 gpd.

The average daily volume of recharge was determined by applying the recharge rate (.0029 ft/day) calculated by the water budget described in Section 6.0 to the area (256A.) overlying that portion of the aquifer thought to discharge between wells DGC-1 and DGC-4. The calculated volume was about 243,000 gpd.

For the 48-day period leading up to the August 28 water-level measurements, the average decline in the water-table throughout the aquifer was 0.53 feet. Assuming an effective porosity of 30 percent, this decline represented about 0.16 feet of water per square foot of aquifer, or an average change in storage of about 278,000 gpd over the area of interest. Applying the equation of hydrologic equilibrium, the total volume represented by recharge and change in storage is about 520,000 gpd compared to 500,000 gpd for ground-water discharge. Given the inherent variability common to the three terms in the equation, the balance is very close.

4.2 Ground-Water Regime

4.2.1 Ground-Water Flow Network

Water levels measured on July 27, August 28, and September 26, 1984 were used to construct water-table contour maps presented as Figures 6, 7, and 8. Because only wells screened at the water table can be considered representative, only water-level elevations from shallow wells were used (total of 34 wells). Over the three months of observations, water levels have declined nearly one foot.

The data indicate that a ground-water mound exists in proximity to the GE/Moreau Site causing flow toward the west, southwest, south, and southeast. However, gradients to the west and southwest are very slight, generally being in the range of 0.0001 to 0.002 ft/ft. Moreover, the major factor influencing ground-water flow in the study area is the northeast-southwest trending topographic scarp marking the edge of the Moreau Aquifer and located approximately two-thirds of a mile south of the GE/Moreau Site. Consequently, flow lines toward the west and southwest change direction to the south and southeast. The ground-water gradients near the scarp are high — up to 0.035 ft/ft — and direct ground waters to the southeast. Thus, the principal gradients from the GE/Moreau Site are toward thd south and southeast.

Wells comprising each well cluster are screened at different depths. This construction enables the evaluation of vertical, as well as horizontal, flow direction. In general, the area surrounding the GE/Moreau Site is a zone of ground-water recharge as indicated by lower water elevations in progressively deeper wells at any one cluster. In contrast, the areas next to, and south of, the topographic scarp are ground-water discharge areas as indicated by higher water levels in the deeper wells than associated shallower wells. Discharge is further indicated by numerous seeps and springs at the scarp base and the presence of a flowing well (FE-1).

With the exception of well clusters DGC-2, 5, 12, and 18, water levels in shallow, intermediate, and deep wells do not differ greatly (generally less than 0.60 feet difference). This condition suggests nearly horizontal flow in the aquifer. At well cluster DGC-12, however, the water level in the shallow well is 4 feet higher than in the intermediate and deep wells, indicating an anomalous ground-water mound. It is important to note that this mound does not alter the flow of ground water in the intermediate and deep zones of the aquifer which is to the southeast. One possible explanation for the high water level in the shallow well is the existence of a layer of less permeable material in the 30 to 40-foot depth range. Withdrawal of ground water by the trailer park well at this location is from the intermediate and deep zones of the aquifer while recharge via the septic systems would be at the surface. A less permeable layer would allow the ground-water table in this area to rise under these conditions.

4.2.2 Ground-Water Flow Rate

Recognizing the differences in horizontal hydraulic conductivity within the Moreau sand aquifer, separate flow rates are calculated for the upper and lower portions of the aquifer. In both cases, however, the rate of ground-water flow is estimated by modifying Darcy's Law to account for the porosity of the aquifer:

$$V = KI/n;$$

where: V = average linear velocity of ground water
K = average horizontal hydraulic conductivity
I = average hydraulic gradient
n = effective porosity

The hydraulic conductivity values are the averages of the values determined by field testing and are 21 feet per day and 5.7 feet per day for the upper and lower portions of the aquifer, respectively.

The hydraulic gradient is the difference in water-table elevation between two points on a flow line divided by the length of the flow line separating the points. The gradient used in both estimates disregards vertical flow components, if any, and is calculated based on a flow line interpreted from equipotentials illustrated on the August 28, 1984 water-level contour map (Plate 7). It represents a decline in head from 325 feet to 285 feet over a distance of 4200 feet.

The effective porosity is assumed, on the basis of sample descriptions, to be 30 percent in the upper portion of the aquifer and 20 percent in the lower portion. The lower portion of the aquifer was assigned a lesser effective porosity due to a higher percentage of silt and clay at this level.

Based on these values, the average linear velocity is about 0.67 feet per day for the upper portion of the aquifer and about 0.27 feet per day for the lower portion. Therefore, the time of travel is about 18 years for ground water flowing entirely within the upper portion of the aquifer from the GE/Moreau site to a discharge point at the erosional escarpment near well DGC-3, a distance of approximately 4400 feet.Similarly, the time of travel is about 45 years for ground water following the same flow line primarily within the lower portion of the aquifer. In actuality, some flow would occur in the upper portion of the aquifer.

It should be noted that these results represent average values for the study area. Actual velocities are likely to vary throughout the aquifer due to heterogeneities within the system. Consequently, the results presented here should be recognized as reasonable estimates, based on a necessarily generalized model of the aquifer, and around which actual values will most likely be distributed.

5.0 INFLUENCE OF PUMPING WELLS

With the exception of the properties to the north served by South Glens Falls water districts, all the residential properties surrounding the site are supplied with water from individual, private wells. Major single well pumping centers within the study area include the Bluebird Terrace Trailer Park and the Moreau Elementary School. The trailer park draws its supply from a 2-inch well screened in the intermediate to deep sections of the unconsolidated aquifer. The Moreau Elementary School well is a deep bedrock well that draws its supply from the underlying bedrock aquifer.

To determine the influence of pumping wells on the movement of ground water, a continuous recording water level recorder was installed on monitoring well DGC-12S, which is located in the center of the trailer park approximately 100 feet from the supply well. For a period of one week, water levels in the shallow portion of the aquifer were continuously recorded. The straight-line trend data suggest that the trailer park pumping well has no influence on the surrounding shallow aquifer. The recorder was checked and found to be operating properly.

The recorder was transferred to the intermediate well adjacent to DGC-12S. Water levels monitored for a period of one week exhibited a slightly decreasing straight-line trend, attributed to anticipated seasonal water level decline.

Preparations were made to install the recorder on monitoring well TM-C, which is completed in the shallow upper portion of the unconsolidated aquifer. Well TM-C and adjacent intermediate and deep wells TM-5 and TM-2 are located adjacent to the Terry and Cheryl Drive residential area. Due to the number of homes (58), the area was considered as a center of pumping that could influence ground water flow. The significant difference between this area and the trailer park is that the water supply is drawn from individual wells distributed throughout the area, each with an average estimated yield of 150 gallons per day.

Observed activities at the well sites raised questions as to the security of the well and water level recorder. It was decided not to install the recorder on any of the wells at this site until modifications could be made to the recorder housing to provide better security for the well. The modifications were in progress at the time of the preparation of this report.

Evaluation of the data obtained over the two week period at DGC-12S and DGC-12I indicates that the transmissivity of the aquifer material is high enough that the influence of pumping the trailer park production well does not extend far enough to reach the DGC-12 well cluster. Supporting this observation is the description of the aquifer material logged at DGC-12 and the short term pumping periods that may preclude the formation of a significant cone of depression around the production well.

6.0 WATER BUDGET

Mean annual precipitation in the area near the GE/Moreau Site is 35.21 inches. This figure represents a volume of 612 million gallons of water per square mile per year. As shown in Figure 6.1, the distribution of mean monthly precipitation is relatively uniform throughout the year. Mean monthly precipitation is approximately three inches per month except in January and February when it is slightly lower.

Potential evapotranspiration (PET), as calculated by the Hamon (1961) Equation is 24.61 inches annually. The concept of PET differs from actual evapotranspiration (ET) in that PET assumes an unlimited availability of water for the evapotranspiration process; nevertheless, PET values serve as reasonable approximations of ET in most instances. Figure 6.1 also shows the variation of PET as calculated from weather data collected at the Glens Falls FAA Airport, located approximately four miles northeast of the GE/Moreau Site. This graph is representative of PET at the GE/Moreau Site. Because PET is primarily a function of air temperature, the shape of the PET curve bears a close resemblance to a graph of mean monthly temperature.

The interaction of the various components operating within the hydrologic cycle may be summarized by a water budget of the study area. In a water budget, mean monthly PET and overland runoff, if any, are subtracted from mean monthly precipitation to obtain a water surplus or water deficit value, and ultimately, a ground-water recharge estimate.

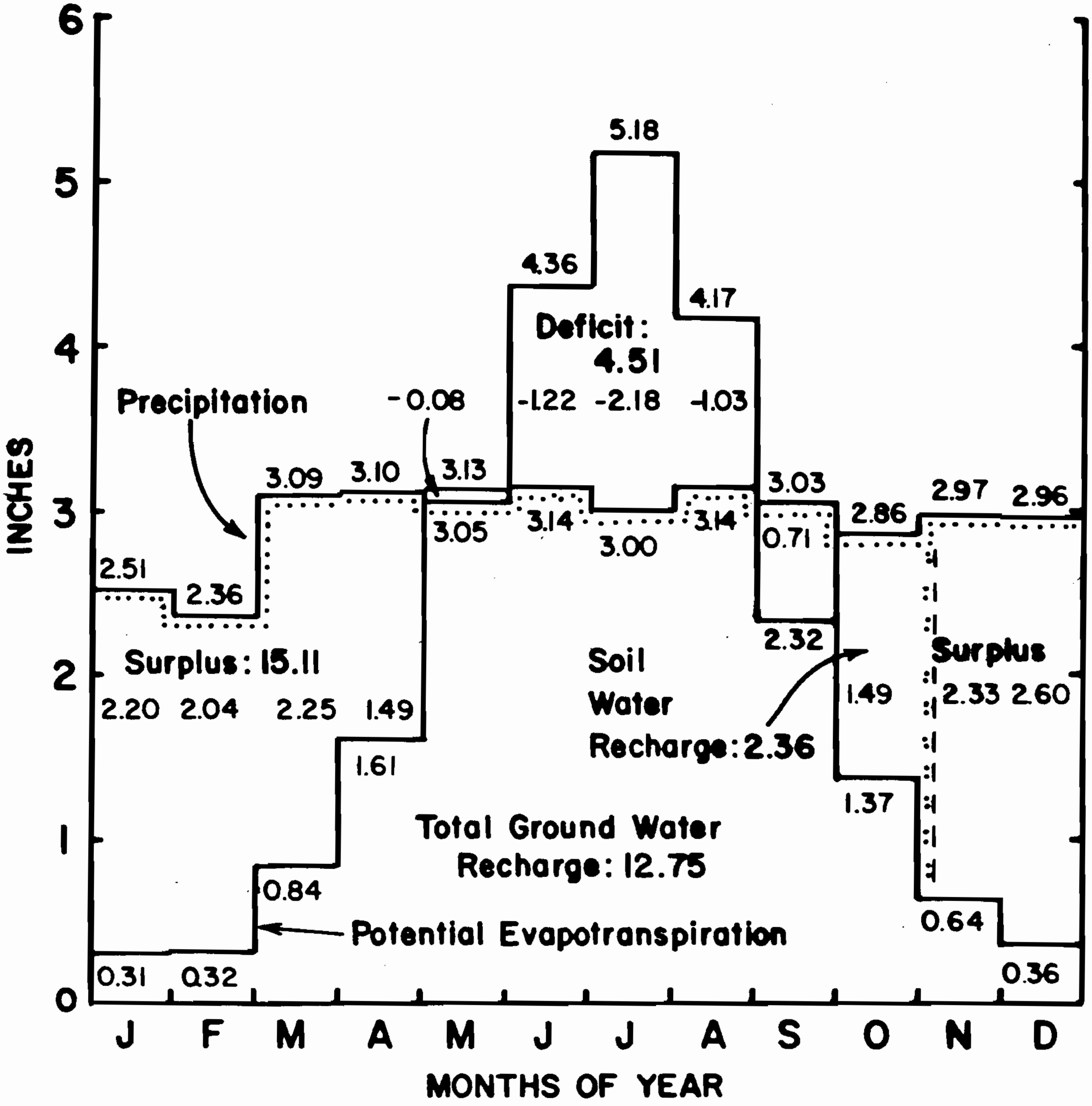


Figure 6.1 WATER BUDGET FOR THE AREA NEAR THE GE MOREAU SITE

Figure 6.1 illustrates a water budget for the area near the GE/Moreau Site. Examination of the graph reveals that a water surplus exists from January through April and from September through December and during which times ground-water recharge occurs. During the warmer summer months, PET is high due to increased insulation and vegetative demands, and recharge ceases.

During this period of high PET, the moisture content of the soil drops below field capacity, creating a soil moisture deficit. Before recharge can occur following the summer months, the soil must be returned to its field capacity. Only when this condition has been met will surplus water be available as recharge.

In this example, 35.21 inches of water enters the hydrologic system annually. At the site, the absence of streams indicates that overland flow rarely occurs in important quantities. Therefore, overland flow is assumed to be negligible for the purposes of this calculation. The streams that are present south of the site are assumed to represent baseflow from the ground-water reservoir and are assigned to total ground-water discharge which occurs primarily as seeps and springs at the foot of the topographic scarp. Consequently, PET alone removes much of the water entering the study area, resulting in an annual water surplus of 15.11 inches. The soil moisture deficit of the soil at this site was determined to be about 2.36 inches. Therefore, subtracting this value from the annual surplus leaves 12.75 inches of water available as ground-water recharge to the water-table aquifer. This estimate of ground-water recharge represents 36 percent of the area's annual precipitation or about 222 million gallons of water per square mile per year.

7.0 GROUND WATER MODELING

7.1 Introduction

A ground-water model was developed to aid the geohydrologic investigation in the vicinity of the GE/Moreau site. The model was used as a tool to help interpret and predict ground-water flow and contaminant movement. It was developed to simulate ground-water flow in the Moreau aquifer on a regional scale. The model calculates ground-water elevations within the Hudson River meander north of Reservoir Road (see Plate 9 and 10). This area is larger than that covered on the Dunn Geoscience field investigation base map. This regional perspective, however, is necessary for the following reasons:

- to locate the position of a regional ground-water divide that should exist in the Moreau aquifer in proximity to the site;
- to include the true aquifer boundaries in the model; and,
- allow an evaluation of the influence of distant aquifer boundaries on the ground-water flow in proximity to the GE/Moreau site.

In general, a ground-water model is designed to represent reality by quantitatively and qualitatively mimicking the physical and hydraulic aspects of the actual aquifer system. The modeling provides a powerful quantitative tool that is used to synthesize existing data, indicate data gaps, assess site geohydrology, and evaluate proposed corrective actions by predicting their effect on the movement of ground-water contaminants. Models, however, will always be less complex than the real systems they represent.

The results of a model are constrained by the quality of field data necessary as model input. As such, emphasis on model assumptions, field conditions, and data limitations is essential. In this report, emphasis has been placed on defining the regional ground-water flow pattern while at the same time scrutinizing model assumptions and data limitations.

7.2 The Numerical Model

The two-dimensional numerical model, as applied in this study, simulates the drainage of water through the water-table aquifer surrounding the GE/Moreau Site. The boundaries for the modeled aquifer system are the prescribed head and the prescribed flux. The information gained from the model is the distribution of hydraulic head.

The model used is the United States Geological Survey (USGS) two-dimensional (2D) finite-difference ground-water flow model. It was written and documented by Trescott, Pinder, and Larson (1976). It is used to solve the two-dimensional ground-water flow equation. The flow equation is a form of the continuity equation (principle of conservation of mass) which states that:

Inflow - Outflow = Rate of Accumulation or Depletion

For a water-table aquifer with two-dimensional flow and assuming alignment of the coordinate axes with the principal components of the hydraulic conductivity (k) tensor, the flow equation may be expressed as:

$$\frac{\partial}{\partial x}\left(K_{xx}\, b \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(K_{yy}\, b \frac{\partial h}{\partial y}\right) = S_y \frac{\partial h}{\partial t} + W(x,y,t) \qquad (1)$$

where:

h is the hydraulic head (L);
x and y are cartesian coordinates (L);
Kxx and Kyy are the principal components of the hydraulic conductivity tensor (L/t);
S_y is the specific yield of the aquifer (dimensionless);
b is the saturated thickness of the aquifer (L);
t is time (t);
W is the volumetric flux of recharge or withdrawal per unit surface area of the aquifer (L/t).

Most aquifer systems have variable properties and complex boundary conditions, and the aquifer analyzed in this study is no exception. Due to the variability of the aquifer materials as well as the non-linearity of equation 1, an exact analytical solution to the partial differential ground-water flow equation cannot be obtained directly. Therefore, approximate numerical methods were employed. In this case, the numerical methods involve the substitution of finite-difference approximations for the partial derivatives in the flow equation. To do this, the area of interest is subdivided into a number of smaller subareas in which the aquifer properties are assumed uniform.

In this study, a variably spaced finite difference grid is used to subdivide the project area into rectangular blocks (shown in Plate 9). The point at the center of each block is called the node, and the nodes are located by the (i,j) indices (see Figure 7.1). The hydraulic head at a given node is assumed to be the average head over the area of the block. Time dependence of the hydraulic head is handled by dividing time into increments or steps; the head at a given node is treated as constant within each time step, and it is assumed to vary in stepwise fashion from one time step to the next.

Using this pattern, the continuous partial derivatives in equation 1 are replaced by finite-difference approximations for the derivatives at each node for a given time step. The result is N equations and N unknowns, where N is the number of blocks representing the aquifer, and the unknowns are the head values at the nodes per time step. The finite-difference equation for unconfined ground-water flow at node (i,j) is:

$$\frac{1}{\Delta x_j}\left[K_{xx(i,j+\frac{1}{2})}b\,\frac{(h_{i,j+1,k}-h_{i,j,k})}{\Delta x_{j+\frac{1}{2}}} - K_{xx(i,j-\frac{1}{2})}b\,\frac{(h_{i,j,k}-h_{i,j-1,k})}{\Delta x_{j-\frac{1}{2}}}\right]$$

$$+\frac{1}{\Delta y_i}\left[K_{yy(i+\frac{1}{2},j)}b\,\frac{(h_{i+1,j,k}-h_{i,j,k})}{\Delta y_{i+\frac{1}{2}}} - K_{yy(i-\frac{1}{2},j)}b\,\frac{(h_{i,j,k}-h_{i-1,j,k})}{\Delta y_{i-\frac{1}{2}}}\right] \qquad (2)$$

$$= \frac{S_{y(i,j)}}{\Delta t}(h_{i,j,k}-h_{i,j,k-1}) + W_{i,j,k}$$

where

$h_{i,j,k}$ is the hydraulic head at time-level k for node (i,j) (L);

$K_{yy(i+\frac{1}{2},j)}$ is the hydraulic conductivity in the y-direction between node (i,j) and (i+1,j) (L/t);

S_y is the specific yield at node (i,j) (dimensionless);

b is the saturated thickness of the aquifer at node (i,j) (L);

Δx_j, Δy_i are the space increment in the appropriate direction (L);

Δt is the time increment (t);

$\Delta x_{j+\frac{1}{2}}$ is the distance between node (i,j) and node (i,j+1) (L);

i is the index in the y-direction;

j is the index in the x direction;

k is the time index.

In this model application, the source term W(x,y,t) includes both evapotranspiration and precipitation.

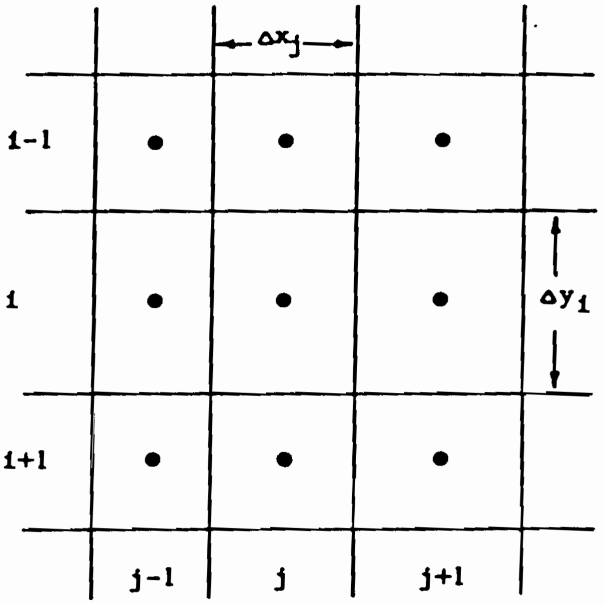


FIGURE 7.1

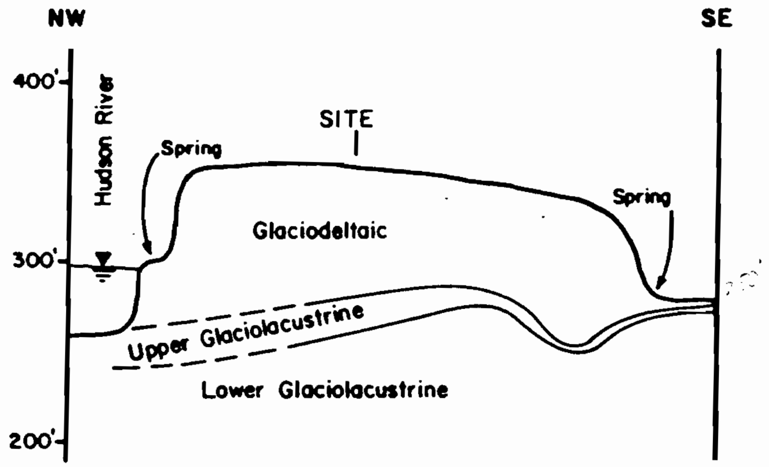
Index Scheme for Finite-Difference Grid Written for Node (i,j) (Trescott, Pinder, and Larson, 1976)

7.3 Conceptual Model, Boundary Conditions, and Data Requirements

The application of ground-water models involves three main phases: 1) system conceptualization, 2) history matching or model calibration, and 3) prediction. The system conceptualization involves organizing available information on the hydrogeology and the site engineering design into an internally consistent framework. This framework is the backbone of the conceptual model that qualitatively describes the behavior of the hydrogeologic ground-water system. This conceptualization is then translated into mathematical terms such as boundary conditions and hydraulic coefficients.

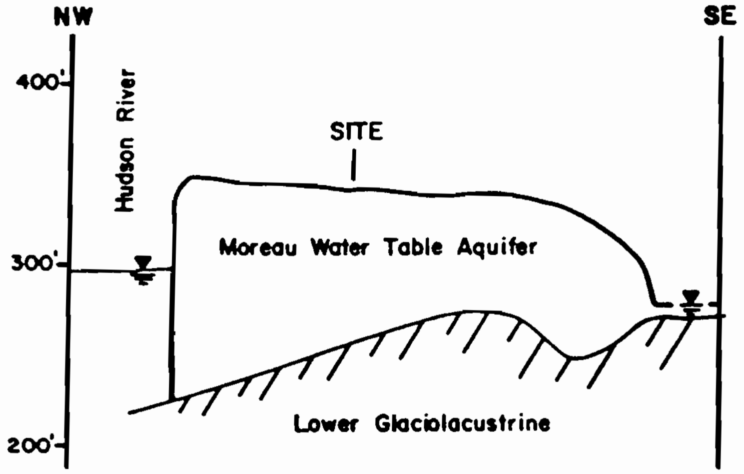
The conceptual model of the ground-water flow system in the vicinity of the GE/Moreau Site is illustrated in Figure 7.2. In order to represent this flow system as shown, a number of simplifying assumptions had to be made. The site specific assumptions inherent in this conceptual model are:

- the shallow ground-water system comprises the glaciodeltaic and upper glaciolacustrine deposits and it is a water-table aquifer;
- the hydraulic properties of the shallow aquifer are isotropic and homogeneous;
- all ground-water flow in the zone of saturation is horizontal, that is, two-dimensional;
- the shallow water-table aquifer is bounded by constant head boundaries to the west, north, and east of the study area. These constant head boundaries correspond with the ground-water elevations at the base of the sand cliffs along the Hudson River (west and north of



A) Natural flow system in the Moreau water table aquifer.

- Glaciodeltaic seidments; precipitation infiltrates vertically to recharge water table; ground-water discharges as springs eventually reaching the Hudson River and the Ft. Edward Reservoirs.
- Upper Glaciolacustrine sediments; ground-water flows to discharge as springs eventually reaching the Hudson River and the Ft. Edward Reservoirs. These sediments are 3 to 4 times less permeable than the glaciodeltaic sediments.
- Lower Glaciolacustrine sediments; semi-permeable sediments



B) Simulated flow system of the Moreau water table aquifer.

- Glaciodeltaic and Upper Glaciolacustrine sediments comprise water table aquifer with ground-water flowing horizontally and discharging as springs; the ground-water head in the vicinity of the spring is constant.
- Lower Glaciolacustrine sediments compris the impermeable basal boundary of the aquifer.

Figure 7.2 - Conceptual Model of the Moreau water table aquifer.

the site) and at the base of the small scarp along the 300-foot land surface elevation contour east and southeast of the site. The southwestern model boundary does not correspond with the true aquifer limit. This boundary was first represented as a no-flow boundary (zero flux) then it was represented as a constant head boundary. The shape of the ground-water contours in the southwest portion of the modeled area changes depending on the choice of the southern boundary condition. However, since this boundary is over a mile away from the primary area of interest, its impact on the ground-water flow patterns within the study area is minimal;

- the lower glaciolacustrine deposit and lodgement till underlying the glaciodeltaic sediments serve as an impermeable boundary, that is, no-flow;
- water in the shallow water-table aquifer is derived from precipitation and aquifer storage; and,
- water in the shallow water-table aquifer is discharged as springs and through evapotranspiration.

Modeling this conceptualized ground-water flow system requires certain hydrogeologic information in order to simulate the observed water-level distributions and the effects of proposed remedial measures. The data arrays used to simulate the ground-water flow in the shallow water-table aquifer in the vicinity of the GE/Moreau Site are as follows:

- a 42 x 43 rectangular finite difference grid;
  the upper elevation of the lower glaciolacustrine sediments;
- an initial water-level distribution;
- the water-bearing zone hydraulic conductivity;
- precipitation and evapotranspiration rates;
- land surface elevation.

7.4 Model Calibration

The goal of model calibration is to adjust model input until a reasonable match between observed and computed water levels are achieved. During calibration it is imperative to constrain the input parameters to realistic values which are best if derived from site specific field testing. Another important guideline for calibration is to never make input more complex than available data warrant; the model can always be updated as new data become available. These general guidelines were followed in calibrating the numerical model of the shallow ground-water flow in the vicinity of the GE/Moreau site.

The ground-water flow model was preliminarily calibrated under steady state conditions. The calibration involved adjusting the hydraulic conductivity, precipitation, evapotranspiration, and the boundary conditions. As yet, an exact match to the observed ground-water elevation contours has not been achieved. However, the general trend of the observed contours has been simulated. Plate 11 compares the ground-water elevations simulated in scenario six to the observed 9/26/84 ground-water elevations. A more refined calibration is possible but not necessary until further data is available.

During calibration, a variety of simulations were made using hydraulic conductivity values ranging from 5.6 to 22.4 ft/day to calculate ground-water elevations. Precipitation was adjusted in each simulation until ground-water elevations in the vicinity of the site were in the range of 325 feet above the mean sea level. In some simulations the boundary conditions were altered to evaluate impacts on the water table configuration in the immediate vicinity of the site.

In every simulation two important features in the shape of the water table are encountered. First, a ground-water divide trending northeast-southwest occurs northwest of the site. In every scenario modeled, this divide persists and remains northwest of the site. Second, ground-water mounds exist both northeast and southwest of the site. Thus, according to model the site overlies a saddle shaped portion of the water table surface. This saddle shape is important because it indicates a ground-water mound exists south of the Myron Road area and trends to the southwest. This mound inhibits ground-water movement to the southwest. Similarly, the mound northeast of the site inhibits ground-water movement in that direction. Thus, the model indicates water table highs inhibit ground-water movement from the GE/Moreau site to the southwest, west, north, and northeast. This conclusion is in accord with the observed movement of contaminants as can be seen by the shape and orientation of the plume shown in Plate 12.

Model calibration was focused where water level data was available; that is, downgradient and in proximity to the site. As such, calibration of the entire modeled area was not attempted. Some specific calibration simulations will be discussed next. The model results are presented both in the regional (total grid) point of view and locally, i.e., illustrating only the area covered by the Dunn Geoscience field investigation base map.

The first two scenarios (1 and 2) would be identical except the northeast boundary of the model was changed in scenario 2. Approximately 40 grid blocks were added to the model thus enlarging the size of the simulated aquifer. In both scenarios uniform values of hydraulic conductivity, precipitation, and evapotranspiration were used. These values were K = 13 ft/day, P = 20 inches/year, and ET = 9.8 inches/year. The south-southwest boundary condition was zero flux (no flow). All other boundary blocks were constant head. These constant head boundaries varied from 300 to 295 to 290 to 285 feet going around the model grid clockwise from the southwest. These boundaries were assumed to correspond with ground-water elevations along the Hudson River and along the base of the erosional scarp east of the site. The northeastern constant head boundary that was expanded, in scenario 2, was set at 285 feet in both scenario 1 and 2. The simulated ground-water elevations and the boundary conditions for scenarios 1 and 2 are shown in Figures 7.3 through 7.8. As can be seen, no change in the ground-water contours is perceptible in the vicinity of the GE/Moreau site (compare Figures 7.5 and 7.8).

In the next two scenarios (3 and 4), the hydraulic conductivity and the precipitation rate were increased. Also, the southeastern constant head boundaries were lowered to an

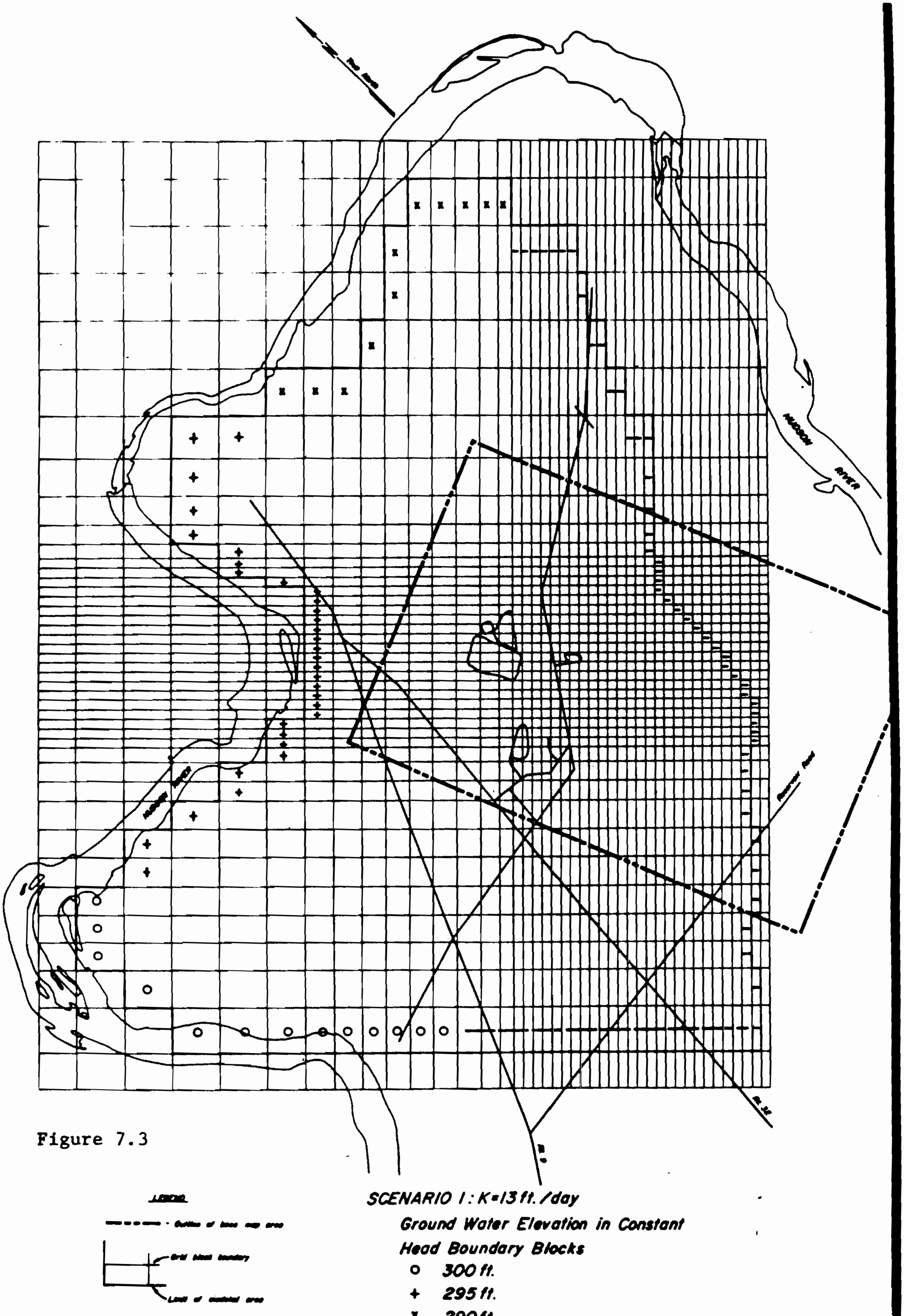


Figure 7.3

SCENARIO I: K=13 ft./day

Ground Water Elevation in Constant
Head Boundary Blocks

o 300 ft.
+ 295 ft.
x 290 ft.
\- 285 ft
——— No outward flow

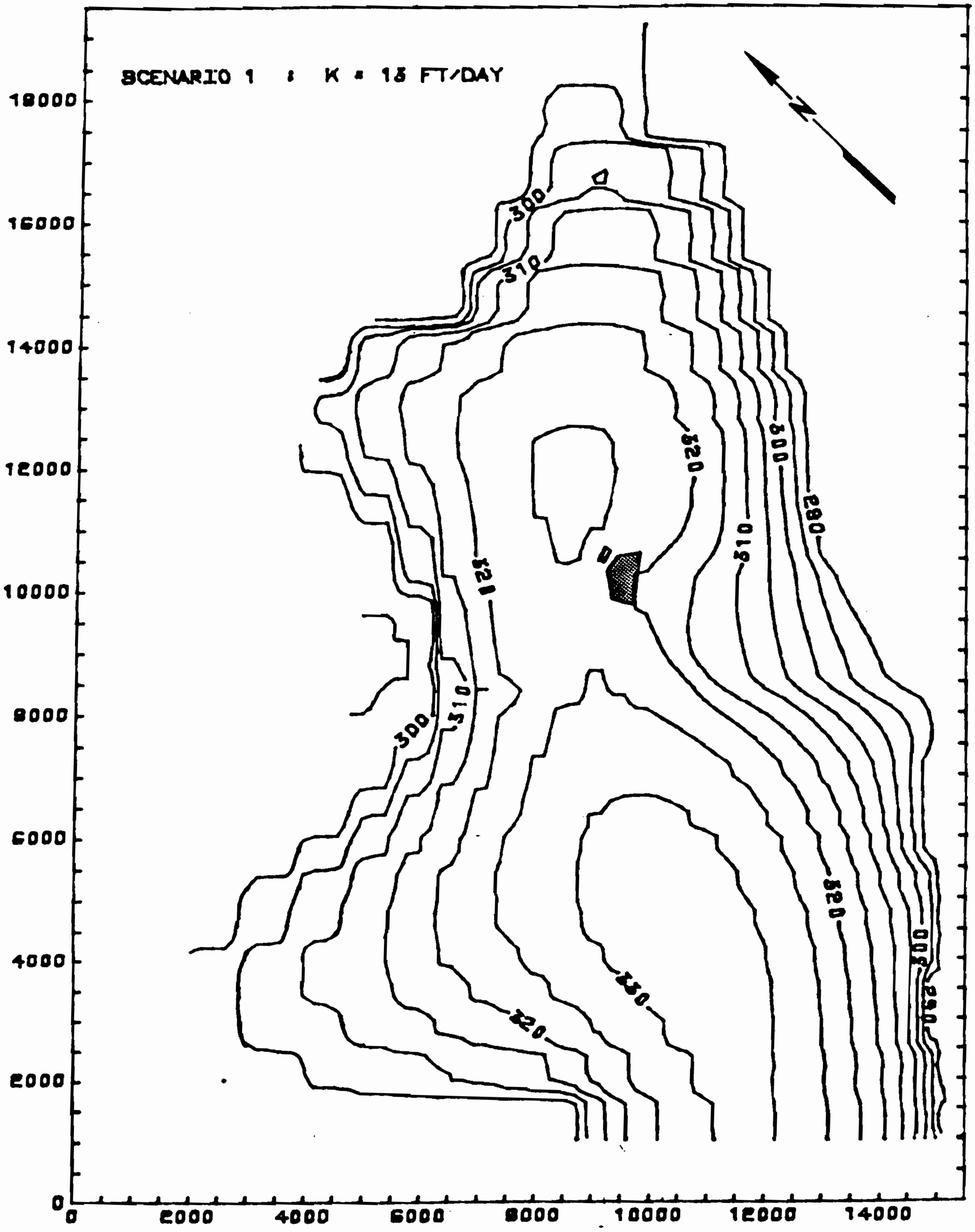


Figure 7.4 - Regional perspective of simulated ground water elevations for scenario 1; P = 20 inches/year, ET = 9.8 inches/year, K = 13 ft/day; stipled pattern illustrates the site location; coordinate units are feet.

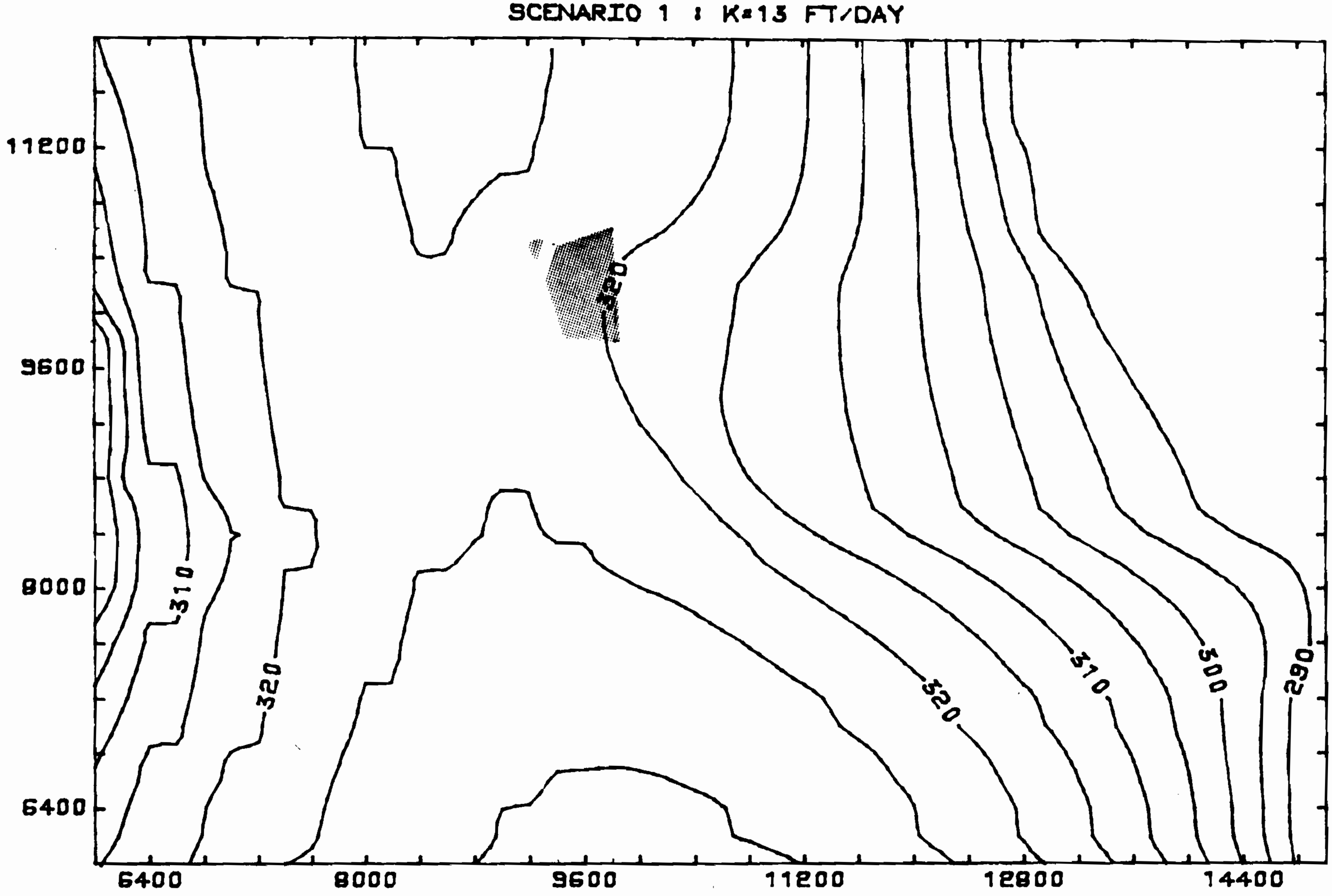


Figure 7.5 - Local perspective of simulated ground water elevations; stipled pattern illustrates site location; the coordinates are in feet.

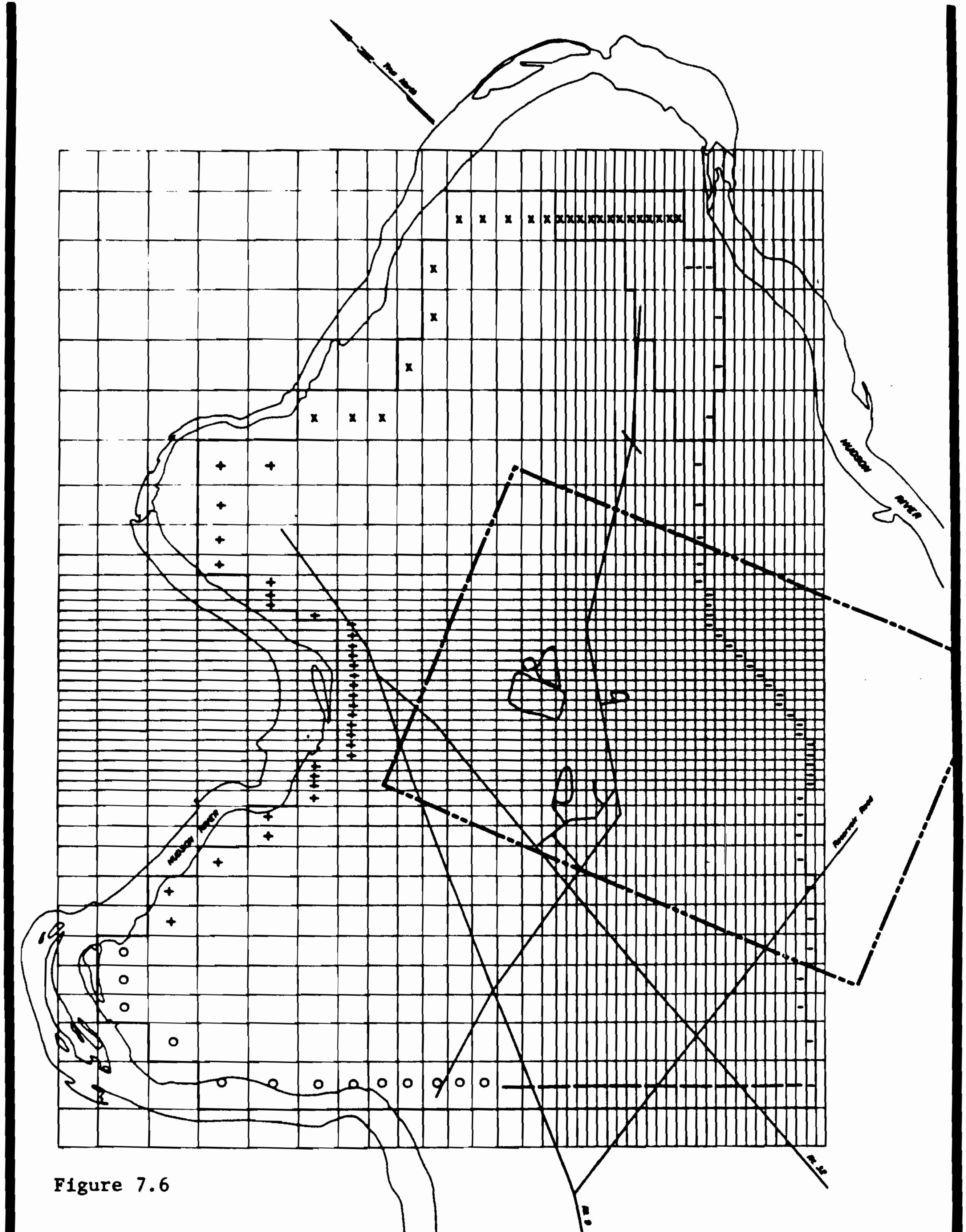


Figure 7.6

Legend

- - - - Outline of base map area

Grid block boundary

Limit of modeled area

SCENARIO 2: K=13 ft./day

Ground Water Elevations in Constant
Head Boundary Blocks

○ 300 ft.

+ 295 ft.

x 290 ft.

– 285 ft.

-·——— No outward flow

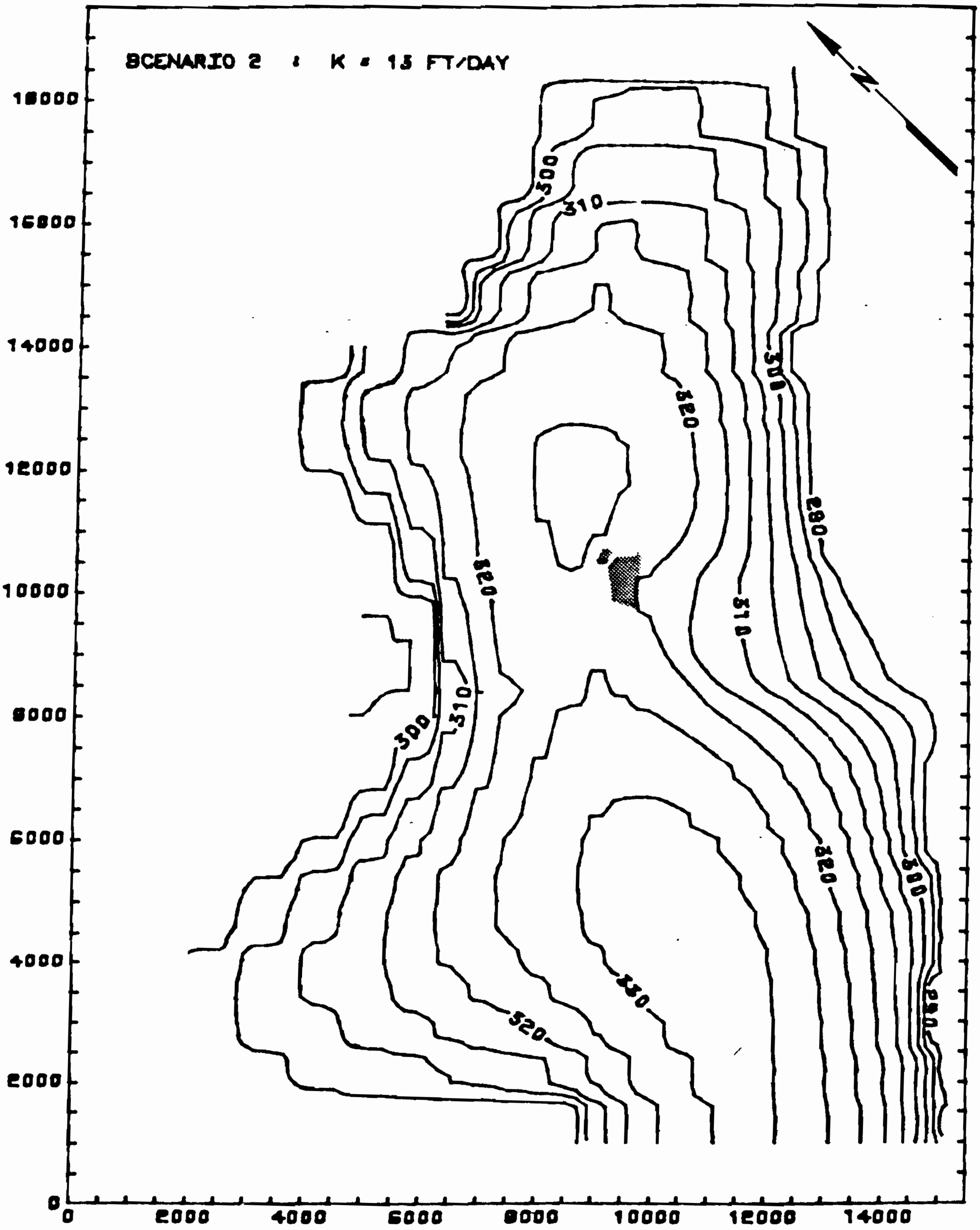


Figure 7.7 - Regional perspective of simulated ground water elevations for scenario 2
P = 20 inches/year, ET = 9.8 inches/year, K = 13 ft/day.
Note the change in contours in northeast corner of plot when compared with Figure 7.4.

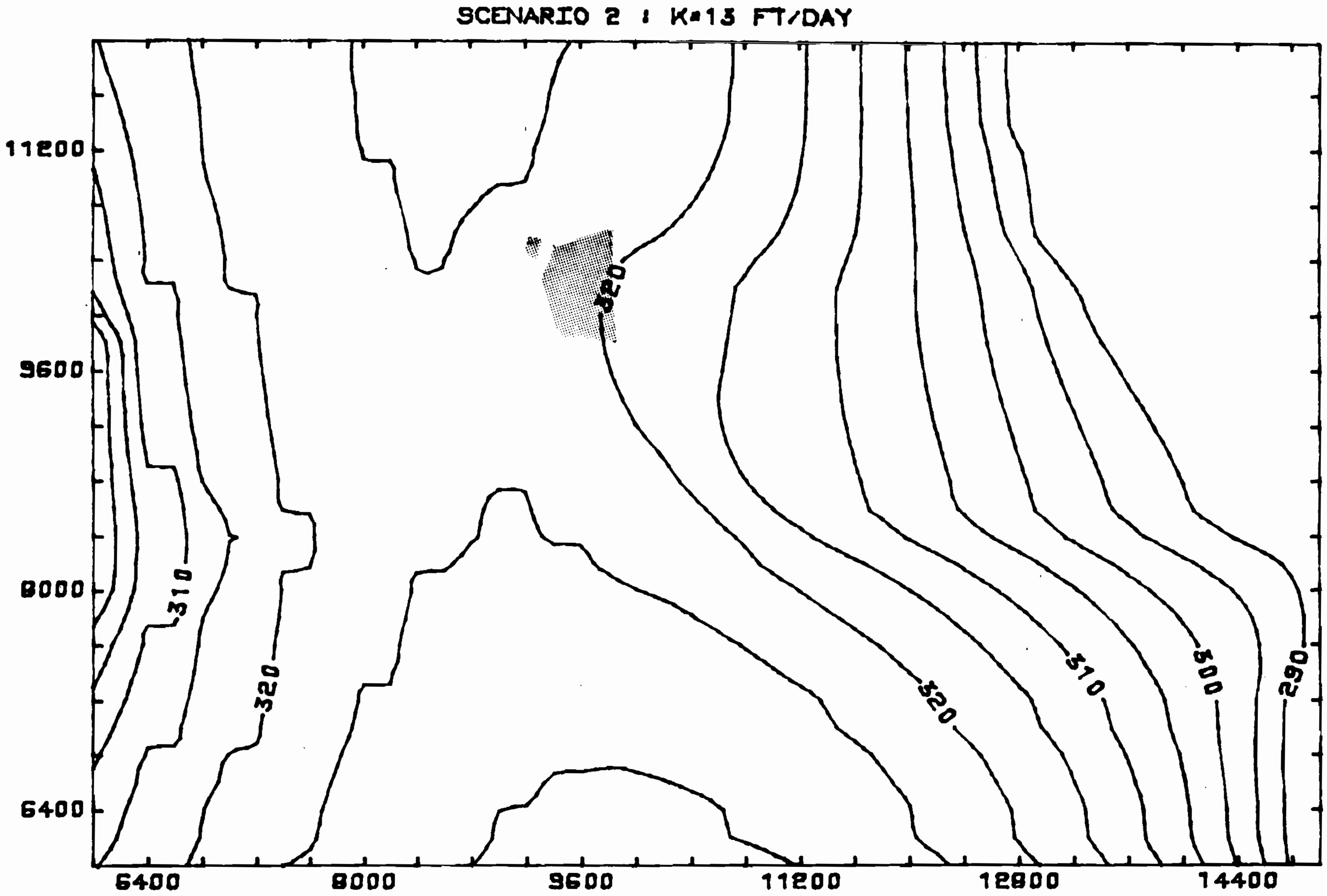


Figure 7.8 - Local perspective of ground-water elevations for scenario 2.
Note the lack of change in the contours when compared to Figure 7.5

As mentioned earlier, the calibration of this model is not completed. Despite the incomplete calibration, the model has proven very useful as a tool in understanding the ground-water flow system in the Moreau aquifer. Considering the shape of the Moreau aquifer (see Plates 3 and 4, and Figure 7.2, the conceptual model), it is apparent a ground-water divide exists in proximity to the site. The model indicates that a divide occurs northwest of the site. Unfortunately, the observed ground-water elevation data are not sufficient to locate the divide. But the location and shape of the observed contaminant plume (Plate 12) supports the model results; a ground-water divide northwest of the site inhibits ground-water and any entrained contaminants from moving to the west and northwest.

The model also exhibits two ground-water elevation "highs" or mounds located on the ground-water divide northeast and southwest of the site. Again, ground-water elevation data are not sufficient to substantiate this mounding. the model calibration is considered incomplete because the observed ground-water elevations in DGC-14 and the DOT well at Myron and Gansevoort Roads do not indicate mounding. Changing the hydraulic conductivity values and precipitation rates did not appreciably alter the shape or position of the ground-water divide or mounds in any model run. Altering the boundary conditions did not change the model results either. The scenarios just discussed describe a broad spectrum of geohydrologic conditions. The hydraulic parameters used in those scenarios span the range of conditions expected for the Moreau aquifer.

This model is intended as a guidance tool to indicate general flow paths and to highlight data deficiencies. It should be considered as a precursor to a local scale model which should include solute transport. The southwestern geohydrologic conditions would need further definition in order to prescribe boundary conditions for a local model. The modeling work performed to date positively supports the existence of ground-water elevation highs on three sides of the site; northeast, northwest, and southwest. These highs will only allow ground-water movement to the southeast. This is supported by the observed contaminant plume.

7.5 Model Limitations

As stated earlier, a model attempts to represent reality to the extent possible when dealing with a complex aquifer system. The proper application of this model is strongly dependent upon the user's knowledge of the model limitations and the implications of such limitations. The limitations associated with this model are as follows:

- possible inaccuracies in the conceptualization of the shallow water table flow system, especially the physical shape and extent of the aquifer;

- the lack of hydrogeologic data north, south, and west of the site;

- the inherent mathematical errors associated with the numerical solution scheme (Strongly Implicit Method - SIP) utilized in the USGS two-dimensional flow model.

Limitations of Conceptual Model

The limitations of our conceptualization of the ground-water flow in the shallow water table aquifer lie in our simplifying assumptions and our choice of boundary conditions, specifically, the assumptions of horizontal flow, isotropy and homogeneity. The aquifer was modeled as a single-layered two-dimensional porous medium. In reality, the glaciodeltaic and the upper glaciolacustrine sediments have substantially different hydraulic conductivities (K = 21 ft/day vs. K = 5.7 ft/day, respectively). Also, the southern and basal boundary conditions may not exactly depict the real aquifer boundaries. However, given the lack of data to describe the system, these assumptions and boundary conditions are reasonable.

Limitations Due to Lack of Data

The lack of data on the extent of the aquifer system north, south, and west of the GE/Moreau site prevents detailed model input concerning the base of the aquifer, the aquifer hydraulic properties and water levels. The hydrogeologic investigation was a localized study in the immediate vicinity of the site. The model was a regional perspective of the ground-water flow in the Moreau aquifer. Thus, detailed geohydrologic data was only available for the immediate site vicinity (where the model grid was finer and more detail was used for model input). The lack of physical aquifer data forces the use of assumed aquifer base elevations and hydraulic properties anywhere beyond the Dunn Geoscience field investigation project area. In addition, a long term historical water level record is not available. This limits the model calibration to conditions recorded at several recent points in time.

Despite the data deficiencies, the model is very useful for predicting the general configuration of the water table in the Moreau aquifer. This provides insight to conditions controlling ground-water flow directions.

Limitations Due to the Numerical Solution Procedure

Finite difference techniques, as used in the model, are subject to two major types of error. The first is the error due to replacing the differential equations describing ground-water flow by a set of algebraic equations. The exact solution of the algebraic equations differs somewhat from the solution of the original differential equations. This is termed truncation error.

The second error associated with a numerical solution scheme is round-off error. This error is a result of the finite accuracy of computer calculations, i.e., numbers are rounded-off. Both of these errors are usually negligible compared to the error associated with system conceptualization.

7.6 Ground Water Modeling Summary-to-Date

The task of modeling the ground-water flow has not been completed. The model still needs calibration work. As illustrated in Plate 11, the simulated water levels indicate a ground-water mound should exist beneath the Myron Road area and extend southward. In that area, the observed data (DOT and DGC-14) does not support the simulated results. As such, calibration focused on a boundary condition evaluation is being continued. Specifically, an evaluation of lowering the elevations of the constant head boundaries west and northwest of the site is underway. Also, in light of the water balance

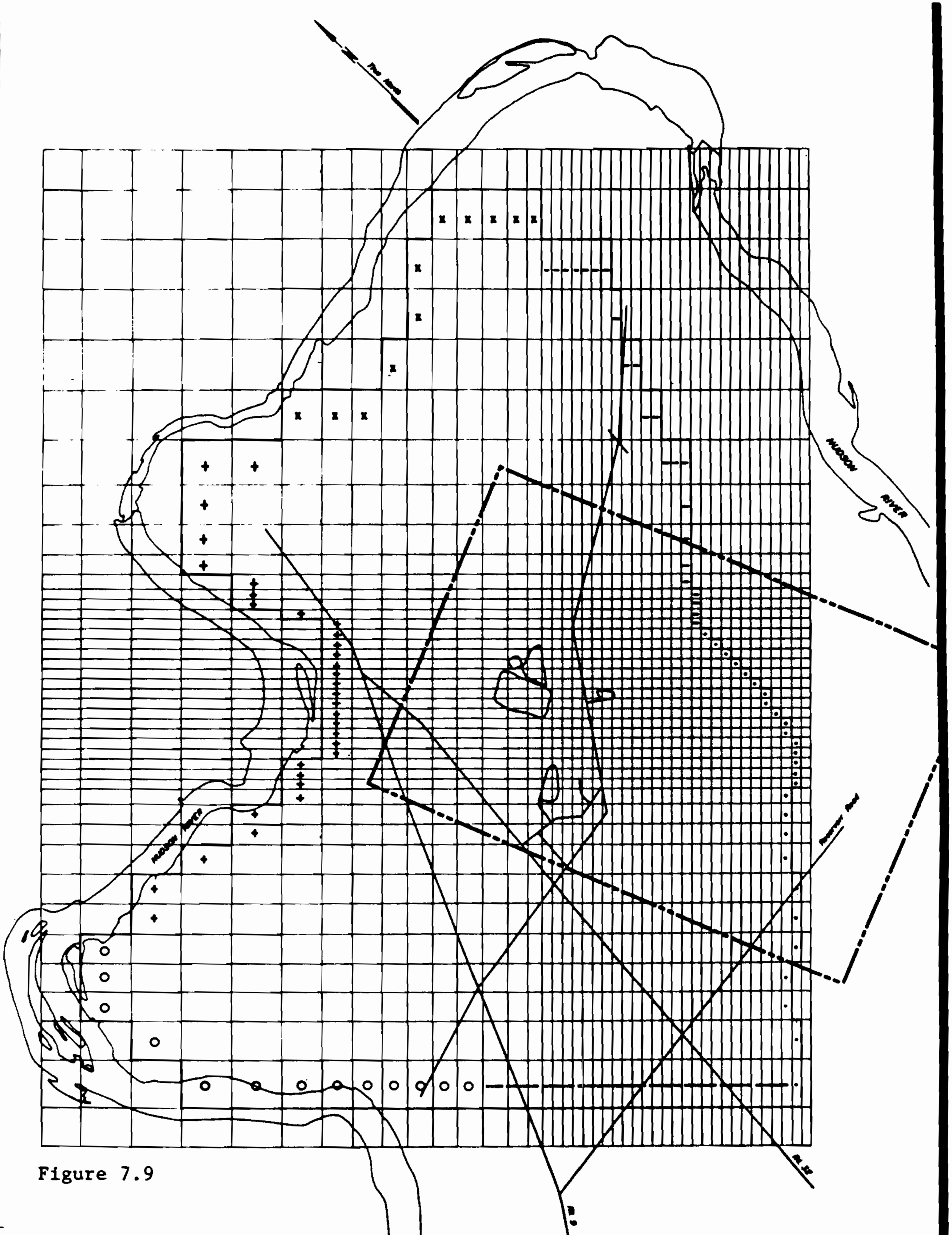
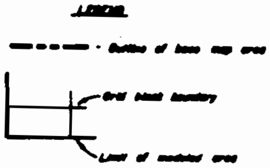


Figure 7.9



SCENARIO 3 : K = 22.4 ft./day

Ground Water Elevations in Constant Head Boundary

○ 300 ft. ——— No outward flow

\+ 295 ft.

x 290 ft.

\- 285 ft.

· 280 ft.

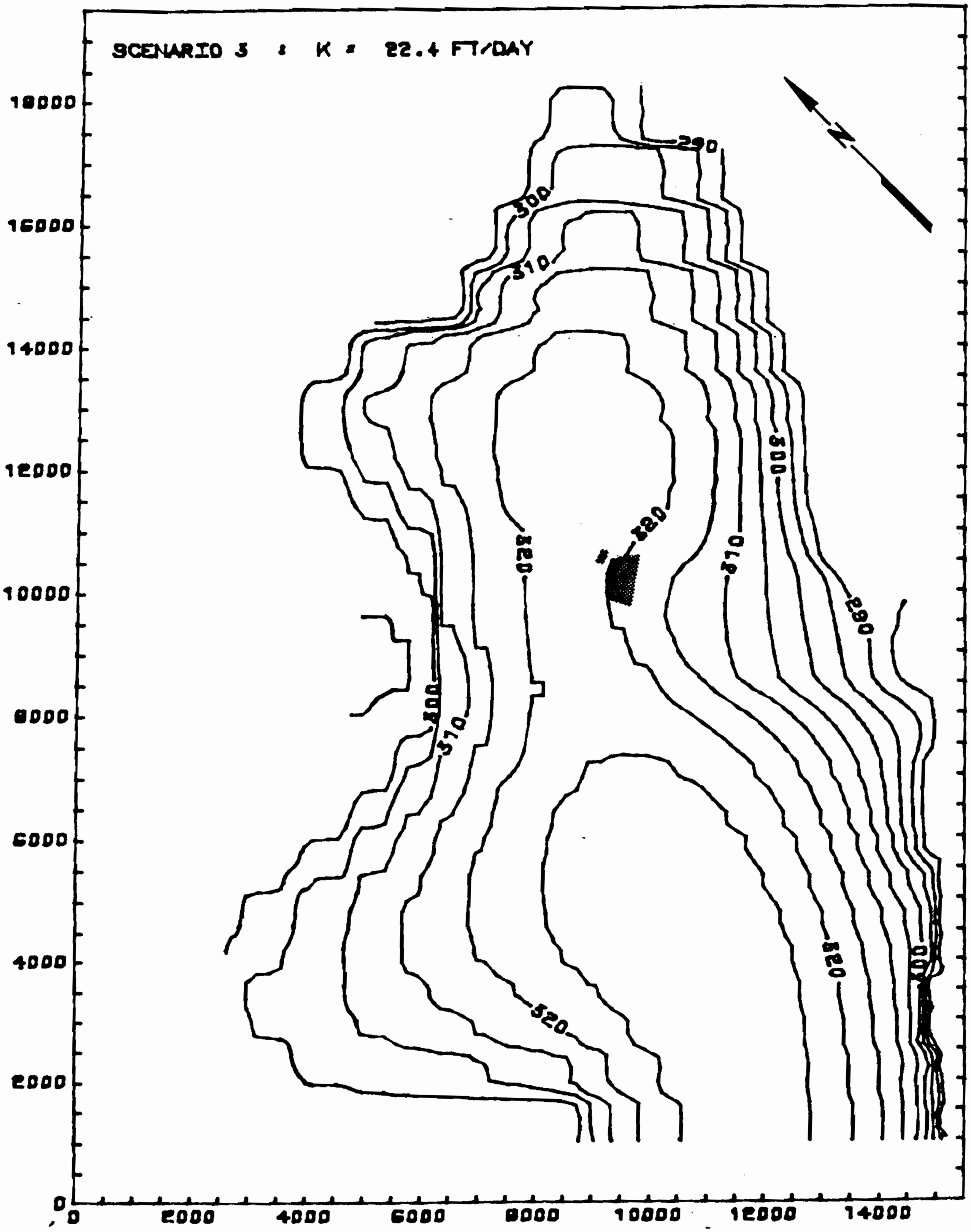


Figure 7.10 - Regional perspective of simulated ground water elevations for scenario 3;
P = 30 inches/year, ET = 9.8 inches/year, K = 22.4 ft/day.

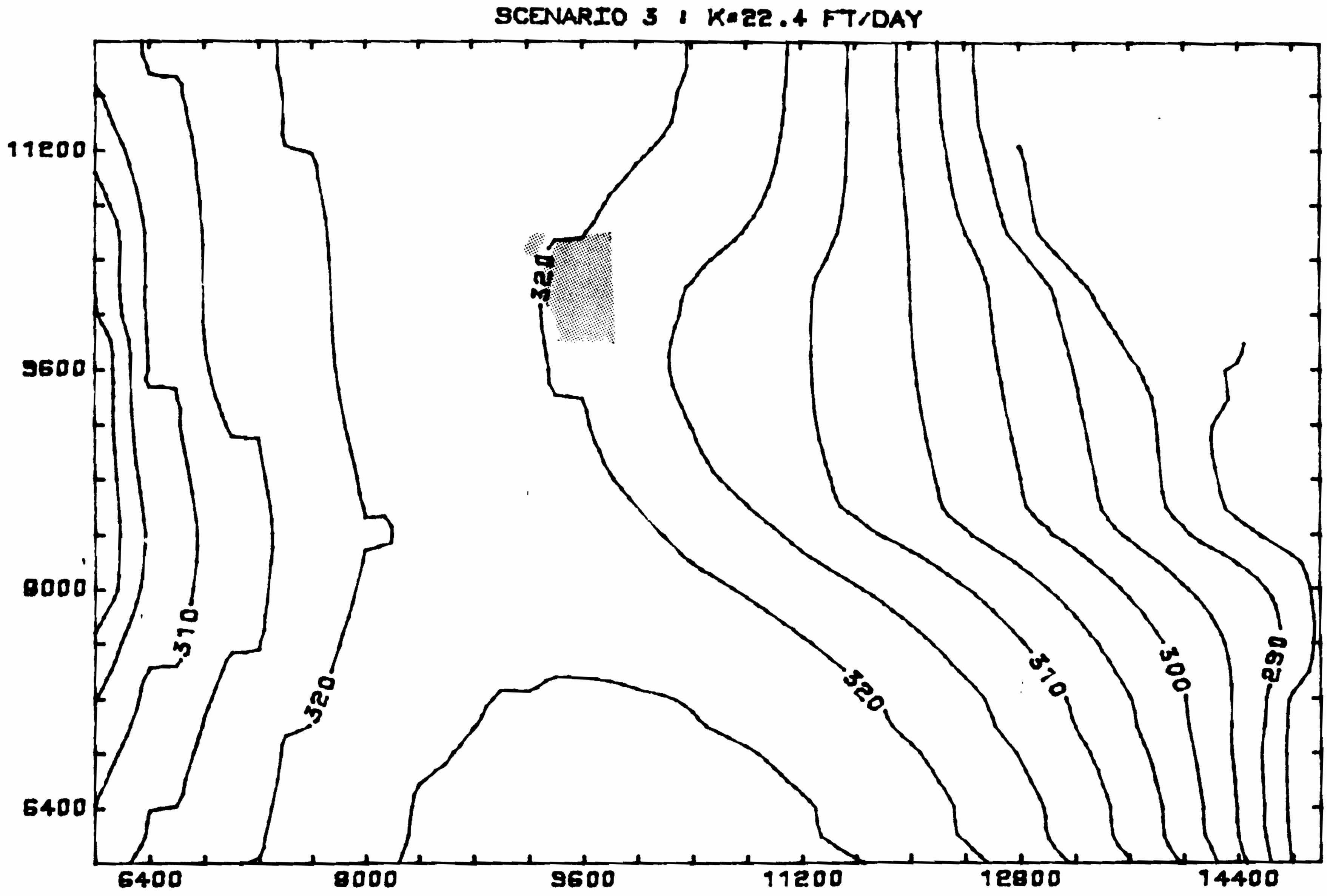


Figure 7.11 - Local perspective of simulated ground-water elevations for scenario 3.

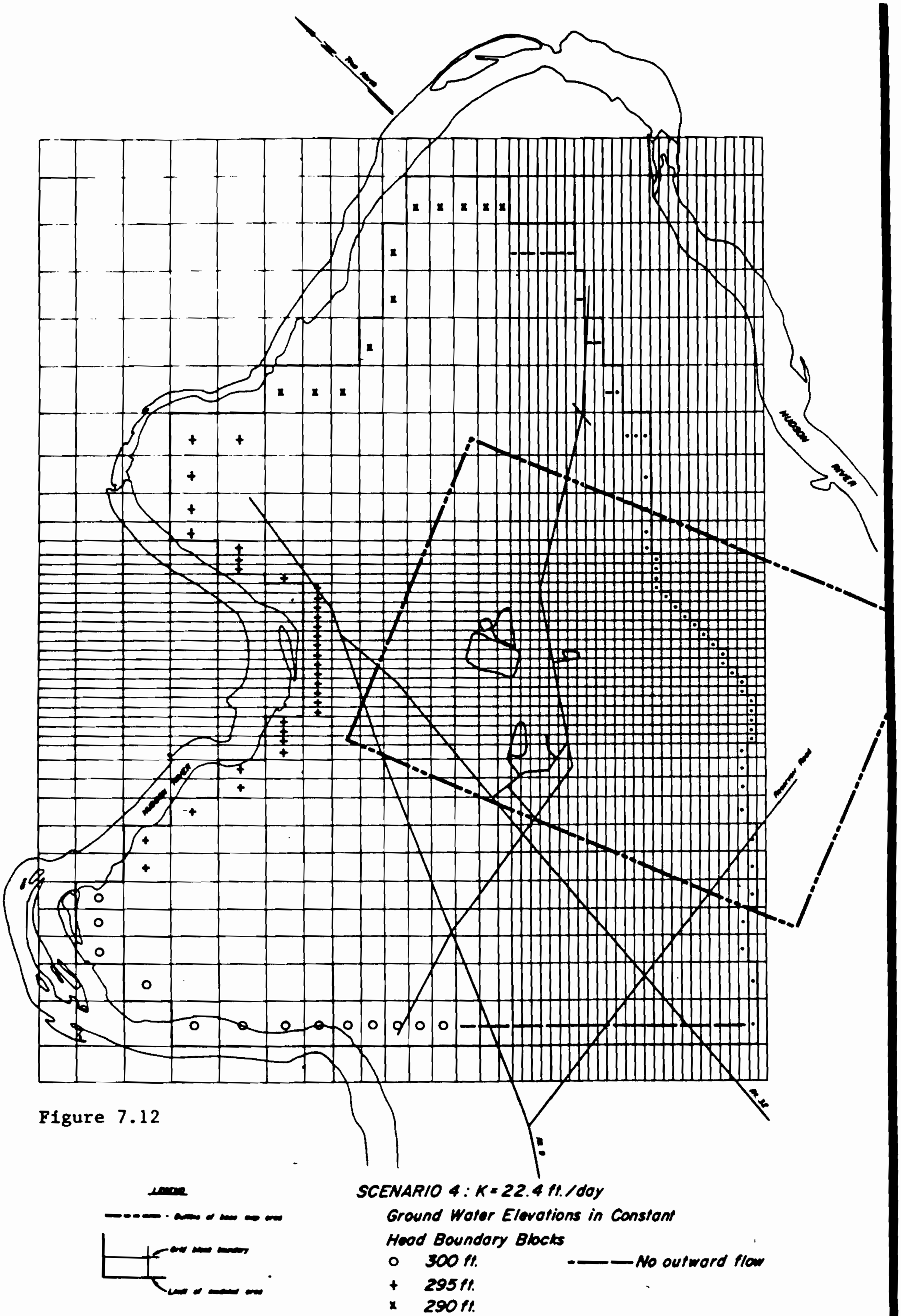


Figure 7.12

SCENARIO 4 : K = 22.4 ft./day

Ground Water Elevations in Constant Head Boundary Blocks

- ○ 300 ft.
- + 295 ft.
- x 290 ft.
- \- 285 ft.
- · 280 ft.

——— No outward flow

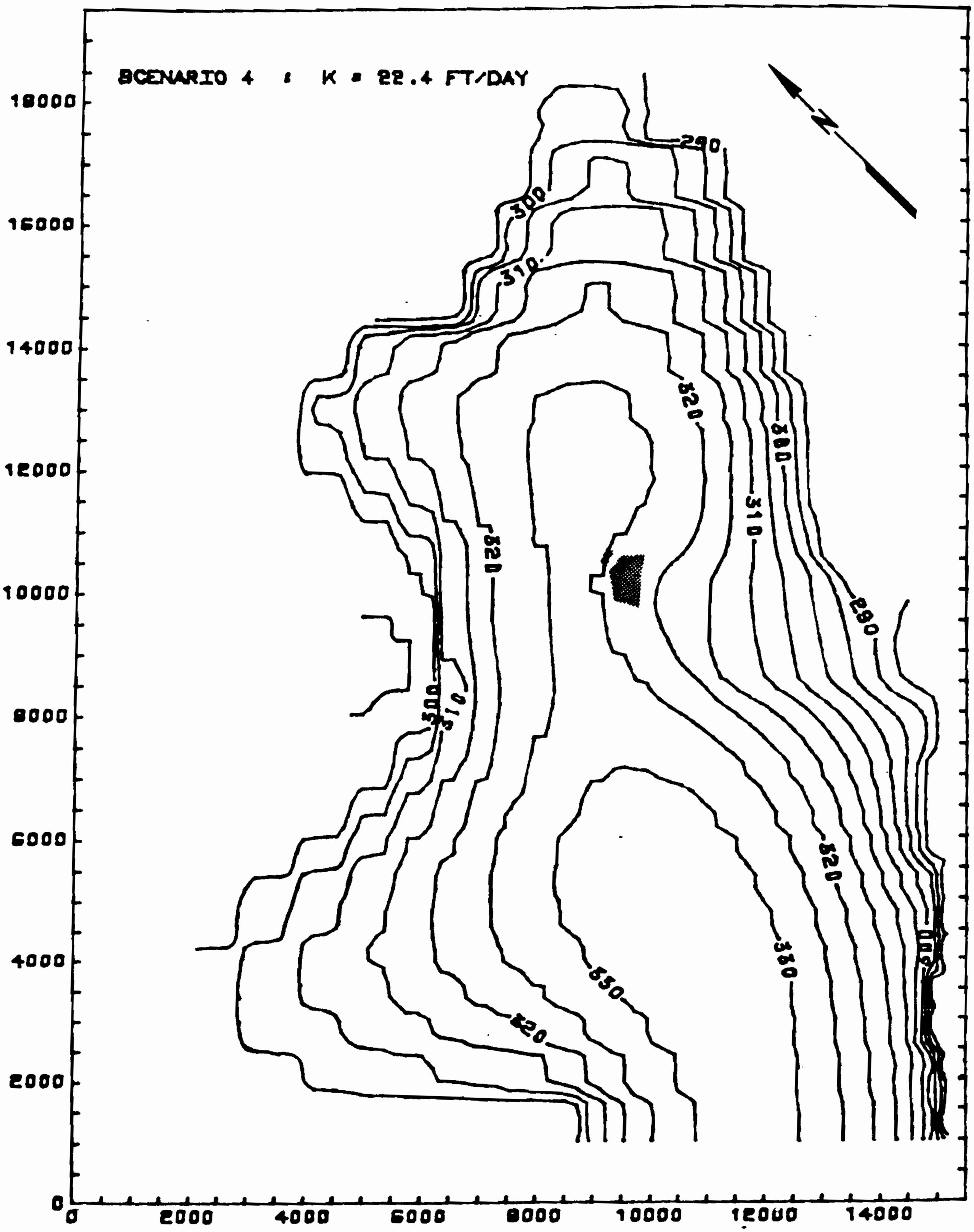


Figure 7.13 - Regional perspective of simulated groundwater elevations for scenario 4; P = 35 inches/year, ET = 9.8 inches/year, K = 22.4 ft/day. Note the increased mounding when compared to Figure 7.10.

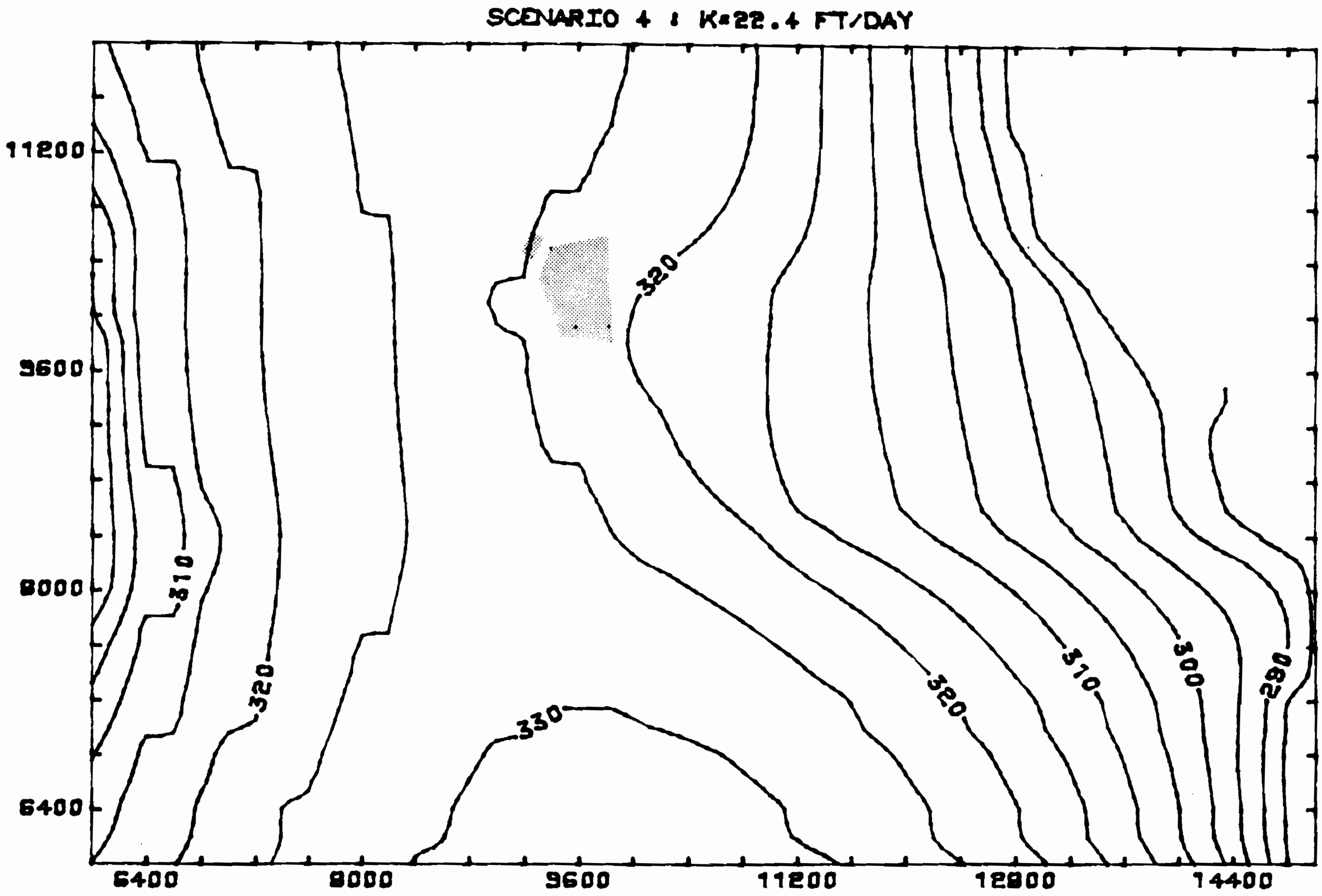


Figure 7.14 - Local perspective of simulated ground water elevations for scenario 4. Note the increased ground water elevation and the consistancy in the ground-water divide location when compared to Figure 7.11 (scenario 3).

elevation of 280 feet. Again hydraulic conductivity, precipitation, and evapotranspiration are maintained uniform across the model grid. In scenario 3, these values are K = 22.4 ft/day, P = 30 inches/year, and ET = 9.8 inches/year. In scenario 4, the precipitation is increased to 35 inches/year. The constant head boundary values for both scenarios range from 300 to 295 to 290 to 285 to 280 going clockwise around the model grid from the southwest. The south-southwest boundary is zero flux (no flow). The simulated ground-water contours and boundary condition descriptions for scenarios 3 and 4 are shown in Figures 7.9 through 7.14. As can be seen, the extra 5 inches/year in scenario 4 causes increased mounding of the water table but it does not change the location or shape of the ground-water divide or the ground-water mounds.

The last two scenarios (5 and 6) discussed are almost identical. In both, precipitation and evapotranspiration are uniform over the model grid (P = 35 inches/year and ET = 9.8 inches/year). In addition, two values of hydraulic conductivity (K = 22.4 ft/day and K = 5.6 ft/day) were used. The lower value was assigned to the 5 blocks bordering the southeastern portion of the model grid. The higher value was assigned to all the other blocks. This caused the contours along the scarp to get close together providing a better match to observed contours. As in scenarios 3 and 4, the constant head boundaries varied from 300 to 295 to 290 to 285 to 280 going clockwise around the model grid from the southwest. The only difference between scenario 5 and scenario 6 is the south-southwest boundary condition which is no flow in scenario 5 but is constant head (parabolic distribution) in scenario 6. Quite a difference exists between contour plots of scenario 5 and scenario 6 but again the ground-water divide and the presence of ground-water mounds still persist in location and shape.

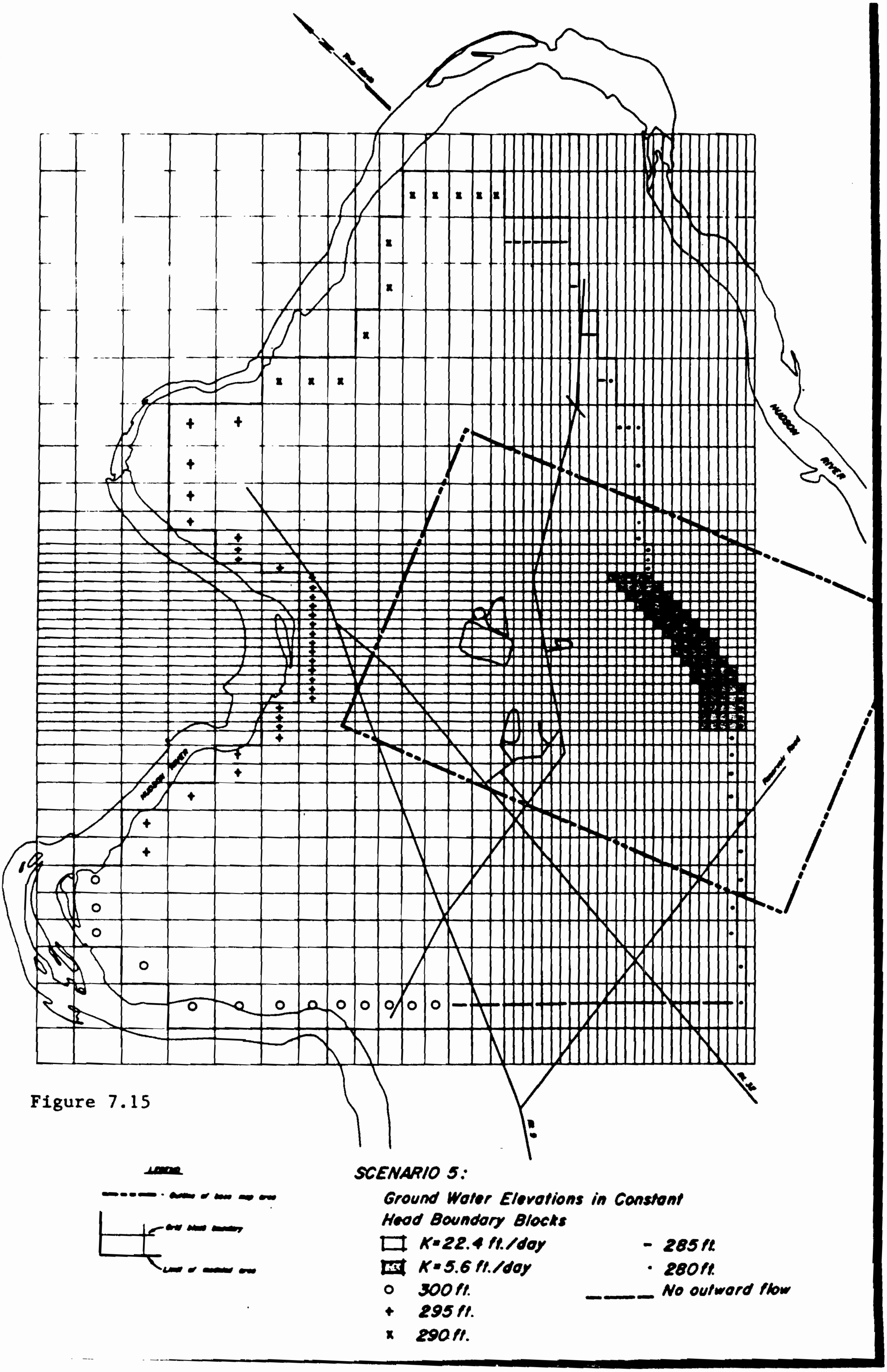


Figure 7.15

SCENARIO 5:

Ground Water Elevations in Constant Head Boundary Blocks

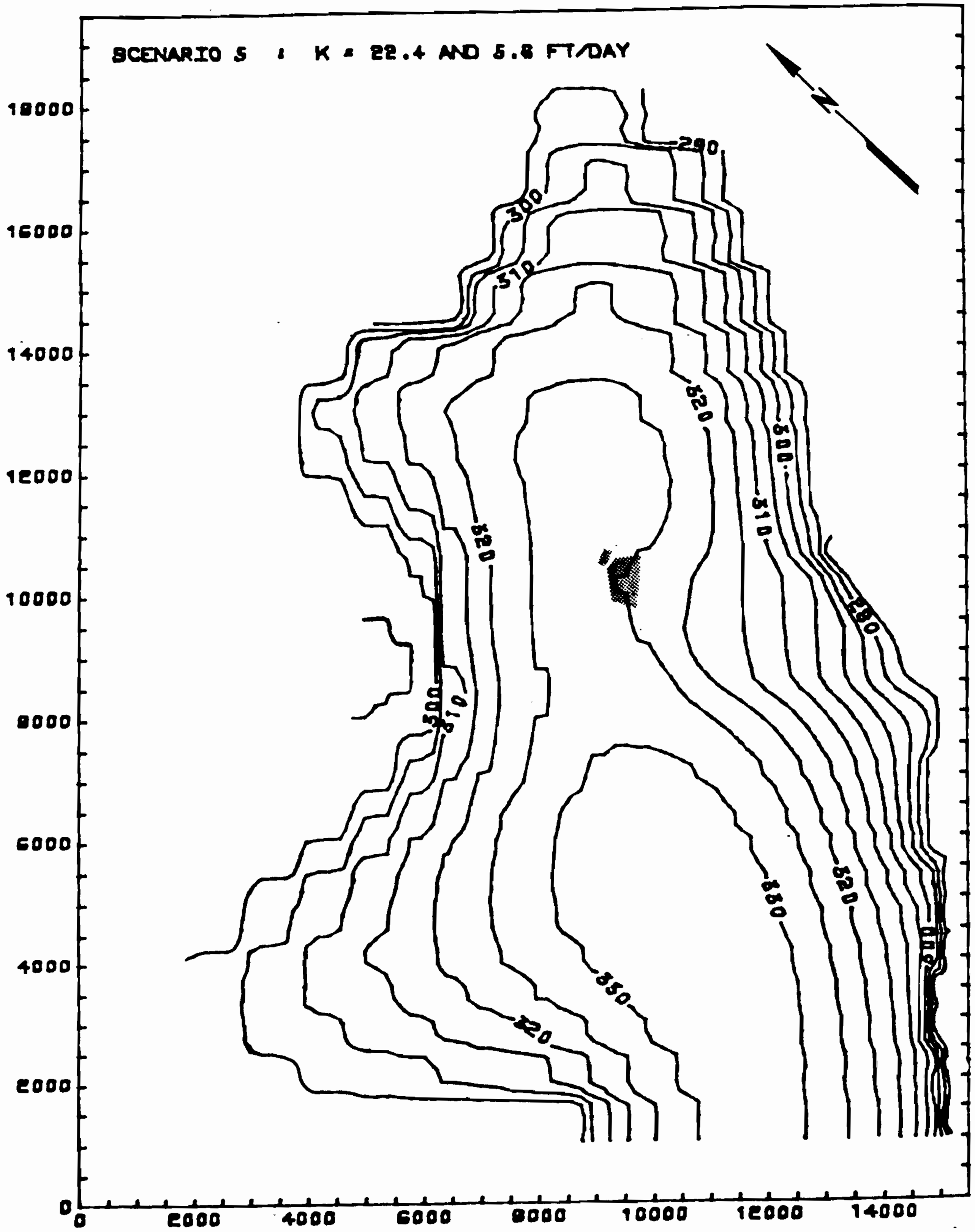


Figure 7.16 - Regional perspective of simulated water levels for scenario 5; P = 35 inches/year, ET = 9.8 inches/year, K = 22.4 and 5.6 ft/day.

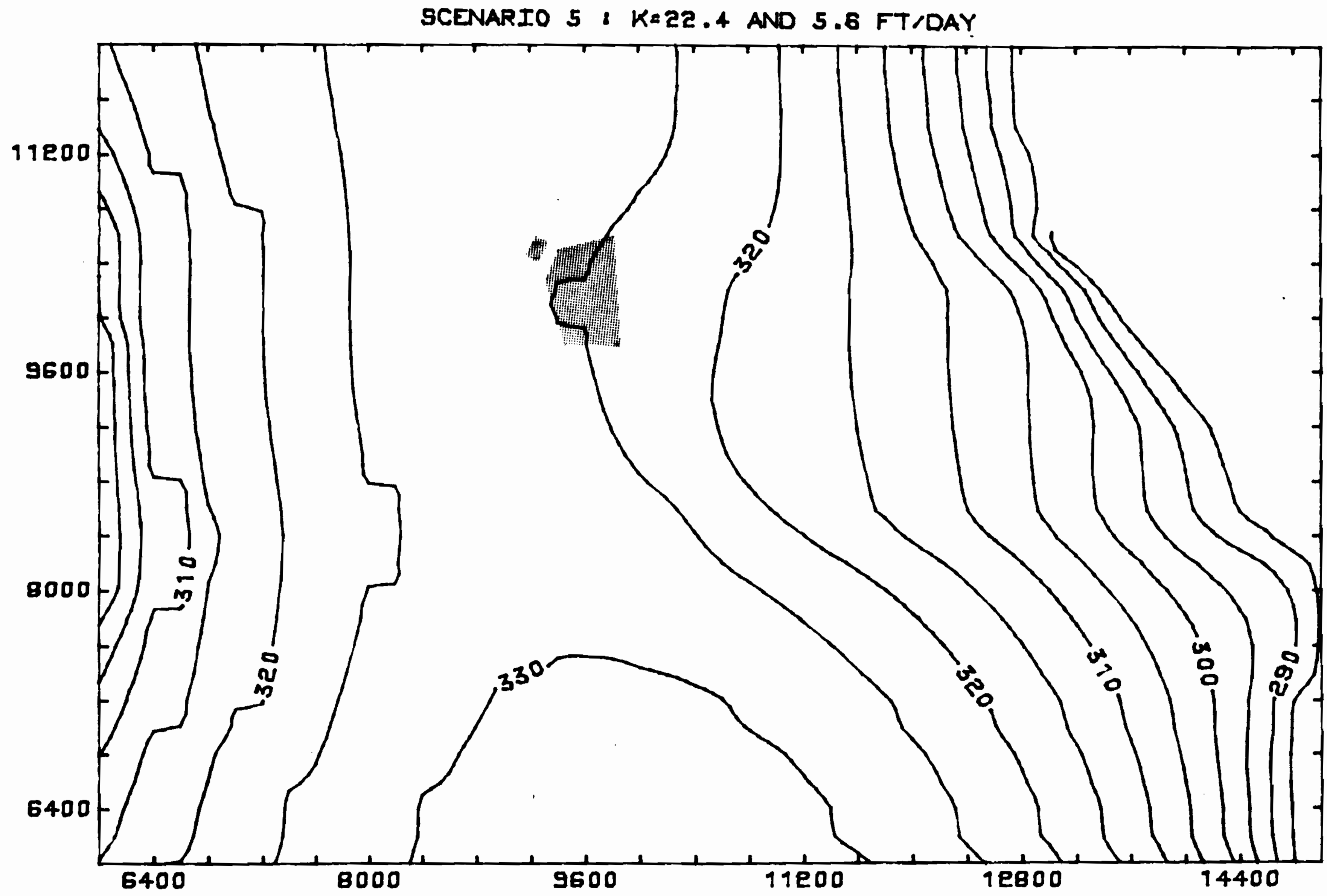


Figure 7.17 - Local perspective of simulated ground water elevations for scenario 5. Note the contours (300 to 295) southeast of the site (right of the site on plot) are closer together when compared to the previous scenarios (1-4).

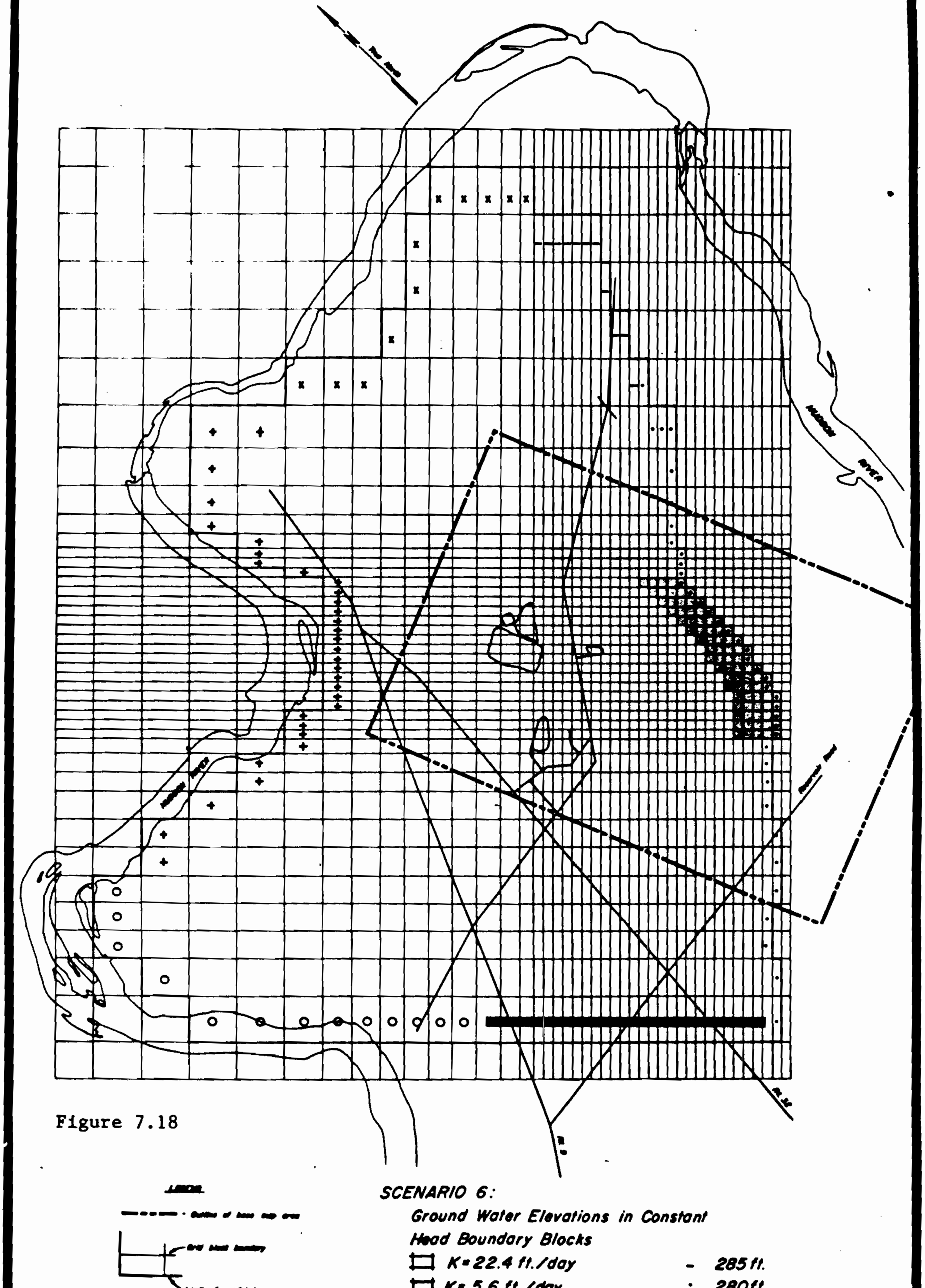


Figure 7.18

SCENARIO 6:

Ground Water Elevations in Constant Head Boundary Blocks

⌶	*K = 22.4 ft./day*
⌶	*K = 5.6 ft./day*
o	*300 ft.*
+	*295 ft.*
x	*290 ft.*
-	*285 ft.*
·	*280 ft.*
■	*Parabolic Shape Maximum elevation 324*

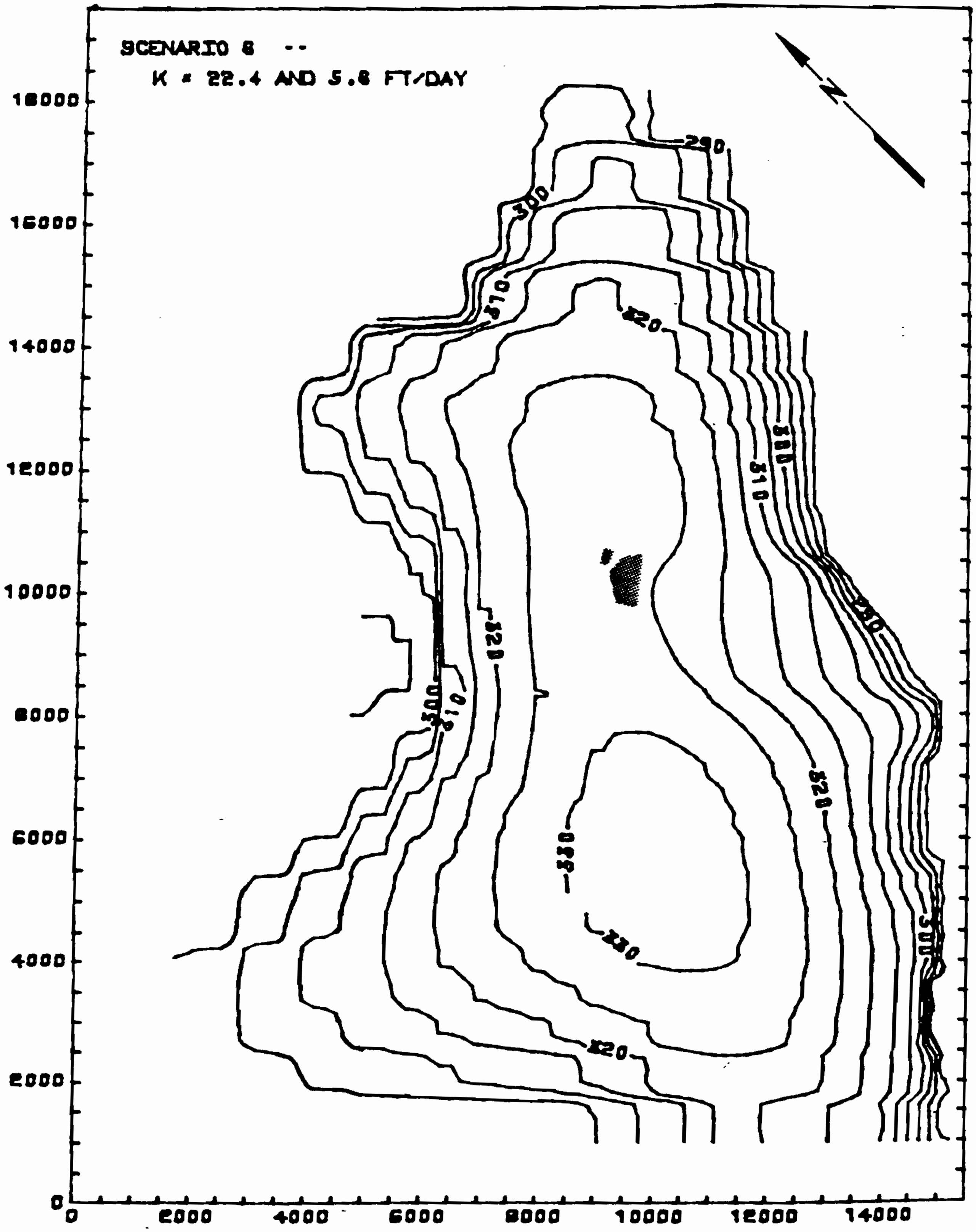


Figure 7.19 - Regional perspective of the simulated ground water elevations for scenario 6. P = 35 inches/year, ET = 9.8 inches/year, K = 22.4 and 5.6 ft/day. Note change in the southwestern mound when compared to figure 7.16.

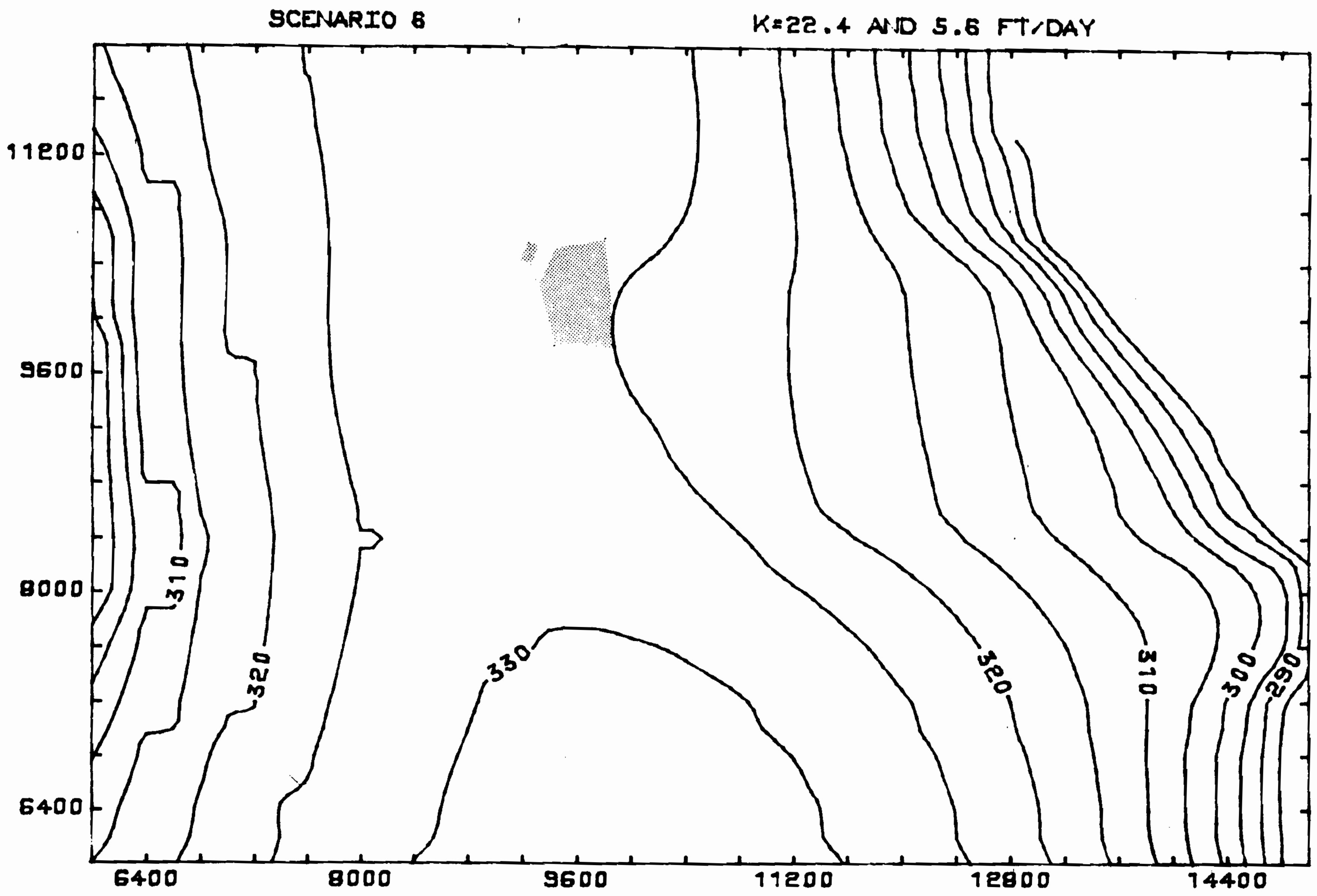


Figure 7.20 - Local perspective of simulated ground elevations for scenario 6. Note the slight increase in mounding when compared to Figure 7.14 (scenario 5).

As mentioned earlier, the calibration of this model is not completed. Despite the incomplete calibration, the model has proven very useful as a tool in understanding the ground-water flow system in the Moreau aquifer. Considering the shape of the Moreau aquifer (see Plates 3 and 4, and Figure 7.2, the conceptual model), it is apparent a ground-water divide exists in proximity to the site. The model indicates that a divide occurs northwest of the site. Unfortunately, the observed ground-water elevation data are not sufficient to locate the divide. But the location and shape of the observed contaminant plume (Plate 12) supports the model results; a ground-water divide northwest of the site inhibits ground-water and any entrained contaminants from moving to the west and northwest.

The model also exhibits two ground-water elevation "highs" or mounds located on the ground-water divide northeast and southwest of the site. Again, ground-water elevation data are not sufficient to substantiate this mounding. the model calibration is considered incomplete because the observed ground-water elevations in DGC-14 and the DOT well at Myron and Gansevoort Roads do not indicate mounding. Changing the hydraulic conductivity values and precipitation rates did not appreciably alter the shape or position of the ground-water divide or mounds in any model run. Altering the boundary conditions did not change the model results either. The scenarios just discussed describe a broad spectrum of geohydrologic conditions. The hydraulic parameters used in those scenarios span the range of conditions expected for the Moreau aquifer.

This model is intended as a guidance tool to indicate general flow paths and to highlight data deficiencies. It should be considered as a precursor to a local scale model which should include solute transport. The southwestern geohydrologic conditions would need further definition in order to prescribe boundary conditions for a local model. The modeling work performed to date positively supports the existence of ground-water elevation highs on three sides of the site; northeast, northwest, and southwest. These highs will only allow ground-water movement to the southeast. This is supported by the observed contaminant plume.

7.5 Model Limitations

As stated earlier, a model attempts to represent reality to the extent possible when dealing with a complex aquifer system. The proper application of this model is strongly dependent upon the user's knowledge of the model limitations and the implications of such limitations. The limitations associated with this model are as follows:

- possible inaccuracies in the conceptualization of the shallow water table flow system, especially the physical shape and extent of the aquifer;

- the lack of hydrogeologic data north, south, and west of the site;

- the inherent mathematical errors associated with the numerical solution scheme (Strongly Implicit Method - SIP) utilized in the USGS two-dimensional flow model.

Limitations of Conceptual Model

The limitations of our conceptualization of the ground-water flow in the shallow water table aquifer lie in our simplifying assumptions and our choice of boundary conditions, specifically, the assumptions of horizontal flow, isotropy and homogeneity. The aquifer was modeled as a single-layered two-dimensional porous medium. In reality, the glaciodeltaic and the upper glaciolacustrine sediments have substantially different hydraulic conductivities (K = 21 ft/day vs. K = 5.7 ft/day, respectively). Also, the southern and basal boundary conditions may not exactly depict the real aquifer boundaries. However, given the lack of data to describe the system, these assumptions and boundary conditions are reasonable.

Limitations Due to Lack of Data

The lack of data on the extent of the aquifer system north, south, and west of the GE/Moreau site prevents detailed model input concerning the base of the aquifer, the aquifer hydraulic properties and water levels. The hydrogeologic investigation was a localized study in the immediate vicinity of the site. The model was a regional perspective of the ground-water flow in the Moreau aquifer. Thus, detailed geohydrologic data was only available for the immediate site vicinity (where the model grid was finer and more detail was used for model input). The lack of physical aquifer data forces the use of assumed aquifer base elevations and hydraulic properties anywhere beyond the Dunn Geoscience field investigation project area. In addition, a long term historical water level record is not available. This limits the model calibration to conditions recorded at several recent points in time.

Despite the data deficiencies, the model is very useful for predicting the general configuration of the water table in the Moreau aquifer. This provides insight to conditions controlling ground-water flow directions.

Limitations Due to the Numerical Solution Procedure

Finite difference techniques, as used in the model, are subject to two major types of error. The first is the error due to replacing the differential equations describing ground-water flow by a set of algebraic equations. The exact solution of the algebraic equations differs somewhat from the solution of the original differential equations. This is termed truncation error.

The second error associated with a numerical solution scheme is round-off error. This error is a result of the finite accuracy of computer calculations, i.e., numbers are rounded-off. Both of these errors are usually negligible compared to the error associated with system conceptualization.

7.6 Ground Water Modeling Summary-to-Date

The task of modeling the ground-water flow has not been completed. The model still needs calibration work. As illustrated in Plate 11, the simulated water levels indicate a ground-water mound should exist beneath the Myron Road area and extend southward. In that area, the observed data (DOT and DGC-14) does not support the simulated results. As such, calibration focused on a boundary condition evaluation is being continued. Specifically, an evaluation of lowering the elevations of the constant head boundaries west and northwest of the site is underway. Also, in light of the water balance calculations, precipitation and evapotranspiration rates will be further evaluated. The continued modeling and calibration process will be enhanced by data collected in the continuing geohydrologic field study. The modeling work performed to date supports the existence of ground-water elevation highs on three sides of the GE/Moreau site; northeast, northwest, and southwest. These highs limit ground-water movement to the southeast, as exemplified in the observed water quality data.

8.0 GROUND WATER - EXTENT OF CONTAMINATION

Prior to commencing the remedial investigation, only three monitoring wells, TMF, TMG, and FE-1 were located south of Bluebird Road. Each well, on at least one occasion, had shown evidence of contamination by organic compounds. The various depths at which the wells were completed and in which organic compounds were detected, suggested that contamination was stratified within the aquifer. In addition, contamination was detected in streams which flow into the Fort Edward water system reservoirs.

Previous investigations had concluded that ground water flowed in a general southward direction from the GE/Moreau waste disposal site. Examination of USGS 7.5-minute topographic maps and field inspection of the area's topography supported this conclusion.

Based on these limited water quality results and on a general knowledge of ground-water flow, a first phase monitoring well network for the remedial investigation was designed. The purpose of the network was to provide information for a preliminary assessment of the areal and vertical extent of contamination, and subsequently for the location of additional wells.

The first phase of drilling resulted in the installation of 37 wells. At 12 locations, the wells were installed as three-well clusters. Site 13 is a single well. Clusters were utilized to determine to what degree the contaminants varied with depth within the aquifer. Clusters DGC-1 to DGC-5 were drilled first in a line parallel to, and just west of, the erosional escarpment. The clusters' function was to aid in determining whether contaminants had reached the downgradient limit of the Moreau sand aquifer. If so, the well spacing along the length of the scarp would indicate the width of any contamination at the aquifer boundary.

The second series of wells to be installed during this phase included well clusters DGC-6 to DGC-8. These wells were located southwest of the GE/Moreau site to monitor ground-water levels and quality between the site and the private residences on Cheryl and Terry Drives.

The third series of cluster wells in the first phase of drilling were located at sites DGC-9 to DGC-13. This series of wells was oriented in the general direction of ground-water flow and extended from just south of the GE/Moreau site to southeast of the Moreau sand aquifer boundary. After this series of wells had been completed, the first round of water quality samples were collected.

In the second phase of drilling, 22 wells were installed. The wells included six three-well clusters, one well pair, and two individual wells. Two well clusters were installed on each side of well cluster DGC-10. They served as sampling points for determining the width of the contaminated zone at its longitudinal midpoint. Two other clusters were installed east and west of the existing wells just south of the site. These wells were installed to determine the width of the contaminated zone near the GE/Moreau site. The remaining wells were installed north of the site to determine if there was any ground-water flow towards the north from the site and, if so, whether organic compounds had migrated to these locations.

Table 8.1 includes only those sites where organics were detected; results are given for both the June/July and September, 1984 rounds of sampling. The second round of water samples was collected after the phase two wells had been completed.

Plate 12 shows the approximate areal extent of ground-water contamination based on the analyses of ground-water samples collected in September, 1984. The map was prepared by considering the highest level of trichloroethylene (TCE) found in any well cluster or in any

Table 8.1

Summary of Analytical Results
Ground Water Monitoring

Purgeable Halocarbons, EPA Method 601

Well No.	Trichloroethylene	Trans-1,2-dichloroethylene	Vinyl Chloride	1,1,1-trichloroethane	1,1-dichloroethylene	Tetrachloroethylene	Methylene Chloride	Chloroform	Chlorobenzene	Dichlorobromomethane
DGC 1S	ND/ND			ND/ND		ND/ND				
1I	ND/4.3			ND/<1		ND/ND				
1D	ND/4.6			ND/<1		ND/3.3				
3S	ND/ND	ND/ND					ND/ND	ND/ND		
3I	54/250	ND/1.0					ND/ND	ND/<1		
3D	2700/16000	11/2.2					39/ND	6.1/1.9		
*3D	1400	6.6					6.5	3.6		
4S	ND/ND							ND/ND		
4I	4.0/<1							ND/ 1		
4D	ND/ND							ND/1.4		
5S								ND/ND		
5I								ND/4.2		
5D								ND/ND		
6S	31/120	ND/2.7						ND/<1		
6I	69/30	ND/1.4						ND/<1		
6D	ND/ND	ND/ND						ND/ND		
7S	800/1400	130/81	4.3/ND			<1/ND		ND/10		
**7I	/12000	/5000	/ND		/13	/5.5		/2.6		
7D	3700/3200	480/550						ND/<1		6.9/ND
8S	13/56	ND/2.7								
8I	290/450	ND/ND								
8D	1.3,1.8/3.8	ND/ND								
***9S										
9I	47/32	ND/1.2								
9D	1300/1200	450/450								
10S	110/1200	ND/8.9		ND/ND				ND/1.7		
10I	700/1300	81/52		ND/ND		ND/2.6		ND/6.7		
10D	81/2300	<1/56		ND/9.3		ND/47		ND/12		
11S	3400/970	70/11	5.8/ND	ND/3.8		<1/32		ND/2.1		
**11I	/28000	/1500	/41	/4.1	/6.4	/80	/20	/4.8	/93	
11D	8300/22000	160/730	ND/ND	ND/1.3	ND/3.1	1.7/9.8	19/25	1.8/4.9		

Table 8.1
Summary of Analytical Results
Ground Water Monitoring
Page 2

Well No.	Trichloroethylene	Trans-1,2-dichloroethylene	Vinyl Chloride	1,1,1-trichloroethane	1,1-dichloroethylene	Tetrachloroethylene	Methylene Chloride	Chloroform	Chlorobenzene	Dichlorobromomethane
12S	ND/ND									
12I	ND/ND									
12D	2.2/310									
13	ND/<1									
15S	ND	ND						ND		
15I	ND	ND						ND		
15D	35	1.6						<1		
19	2.4					1.2		<1		
21S	120	<1		1		12				
21I	4.4	ND		ND		ND				
21D	9.3	ND		ND		ND				
22S	22	ND						<1		
22I	7600	180					93	<1		
22D	15000	320					220	<1		
TM-A	5.8	<1								
TM-B	8.5	1.2								
TM-D								2.3		
TM-F	690	8.8								
TM-G	120	5.4								
TM-3	52	2.3								
TM-4	81000	46000	510		140	11	200	750		
FE-1	600	6.9					16	3.6		

Legend to Table

All results in ug/L (ppb).

ND None detected.

Values for wells DGC-1 - DGC-13 are given for both rounds of sampling, 6/25/84 - 7/9/84 and 9/5/84 - 9/17/84 as XYZ/ABC, respectively.

* Results of a second sample in the first round of sampling.

** Purgeable organic compounds in first round were determined by EPA Method 624. See Table 8.2

*** Well 9S contains insufficient water for sampling.

Contaminants were not detected in wells 2S, 2I, 2D (both rounds) nor in 14S, 14I, 14D; 16S, 16D; 17: 18S, 18I, 18D; 20S, 20 I, 20D; TM-C; TM-E; TM2; TM5 (second round).

individual well if it is not part of a cluster. Although observed ground-water contamination includes a variety of organic compounds, TCE is most prevalent and, therefore, was chosen to be an indicator of overall contamination.

Table 8.2 lists the results of the wells (DGC-3D, 7I, 11I) analyzed for priority pollutants.

Isopleths showing concentrations of 100, 500, and 10,000 ppb of TCE are illustrated on Plate 12. However, since the plate shows the maximum concentration of TCE detected at each single well or well cluster regardless of depth, or length of sampling interval, the isopleths cannot represent strict interpolations between data points. Nevertheless, this does not diminish the usefulness of the isopleths in depicting the relative degree of aquifer contamination. Reported concentrations of TCE less than 100 ppb were not contoured, but are included on the map for completeness.

The areal extent of contamination representing TCE concentrations greater than 100 ppb occur in an essentially southeast trending plume approximately 4800 feet long and about 2000 feet wide at its widest point. The plume has its origin at the GE/Moreau disposal site and extends southward to the erosional escarpment. The downgradient limit of the plume appears to be controlled by ground-water discharge to springs and streams having their head waters at the foot of the escarpment.

The overall orientation of the plume follows the general direction of ground-water flow indicating that advection is the primary mechanism of contaminant migration. Maximum organic levels occur in a relatively narrow, essentially southeast trending band that includes within it wells or clusters DGC-7, DGC-11, TM4, DGC-22, and DGC-3. Similar to the overall plume the orientation of this band of high TCE

Table 8.2

Results Summary
Priority Pollutant Analysis*
Ground Water Monitoring

Parameter	DGC 11I	DGC 7I	DGC 3D**
Nickel	6	<5	<5
Aroclor 1242	1.0	0.3	ND
Trichloroethylene	4100/4300	5300	
Trans-1,2-dichloroethylene	1300/1800	3000	
Vinyl Chloride	190/ND	31	
1,1-Dichloroethylene	ND/ND	6	
1,2-Dichloroethane	ND/ND	5	
Benzene	12/ND	31	
Toluene	7/ND	7	
Chloroform	ND/ND	77	
Acetone	2000/220	72	
1,2,4-Trichlorobenzene	<10	<10	ND
1,2-Dichlorobenzene	<10	ND	ND
1,4-Dichlorobenzene	<10	ND	ND
Benzofluoranthene	ND	ND	<10

* Purgeable organic compounds by Method 624/results are on first round samples except 11I which includes results of a second round analysis for purgeables.

** A Method 624 analysis was not run on this sample. Refer to Table 8.1 for purgeable halocarbons determined by Method 601.

Priority pollutant compounds not included in the above table were not detected.

All results in ug/L (ppb).

levels coincides very closely with ground-water flow paths south of the GE Moreau site.

The extent of contamination is also affected to a lesser degree by dispersion as indicated by the detection of lower-level organics in wells east and west of the band of highest TCE concentrations. Dispersion is responsible for lateral spreading of the contaminants as they migrate in response to ground-water flow; however, its influence on the migration of contaminants is much less than that of advection This conclusion is supported by the length-to-width ratio of the plume, the relative concentrations of TCE in the central versus outer portions of the plume, and the sharp decrease in concentration over a relatively short transverse distance.

Plate 12 shows a cross-section line drawn through the most contaminated portion of the plume. The corresponding cross-section, E-E', illustrates the vertical distribution of TCE in the Moreau sand aquifer and is shown on Plate 13. It extends from well TM3 southeastward to well DGC-3, located near the aquifer's boundary. The cross-section includes wells located within the zone of high TCE concentrations and other selected wells which have been projected onto the line. The GE/Moreau site is not shown on the cross-section, but is located generally northward of well TM3.

The water table on the section slopes from northwest to southeast and represents the general direction of ground-water flow in this area. Consequently, TCE and any associated organic compounds introduced into the sand aquifer at the GE/Moreau site will be transported in this direction by advection.

The most significant aspect of contaminant distribution illustrated by the section is the tendency for TCE concentrations to be highest at intermediate and deep levels within the aquifer.

The most likely scenario depicted by the distribution of TCE in the cross-section is as follows: Chemicals disposed of at the GE/Moreau site infiltrated the unsaturated soil beneath the site and percolated to the water table. Since TCE has a low solubility and is heavier than water, the separate TCE phase continued to sink under the influence of gravity. Upon entering the aquifer, low concentrations of dissolved chemicals were transported in the general direction of ground-water flow by advection. Simultaneous to its migration toward ground-water discharge points at the erosional escarpment, the plume may have moved downward following a flow path characteristic of recharge areas. Since the hydraulic gradient near the GE/Moreau site is very slight, horizontal ground-water flow occurred slowly and sufficient time was available for the chemicals to sink to intermediate or deep levels at the site. As it migrated southeastward, the plume continued to sink until it encountered the underlying confining bed. The low vertical hydraulic conductivity of the confining bed retarded further downward movement and, at this point, the plume flowed along the base of the aquifer in the direction of ground-water movement.

Insufficient ground-water flow and water quality data have been generated to define the area outside the plume which comprises sites TMD, DGC-14, Moreau Elementary School, and DGC-5. The first and last of these contained anomalous chloroform traces; however, the school and well cluster 14 directly adjacent to it, have been clean.

Four "upgradient" monitoring wells showed the presence of TCE at low to significant levels (2.4 - 120 ppb). These sites (DGC-15, 19, 21, TMA) are located either on or adjacent to, a ground-water mound or divide situated at the GE/Moreau site and the sand pit adjacent to the site with its bodies of surface water. Not enough hydrologic information has been obtained to define ground-water flow in a northerly direction. As of the writing of this report, resampling of the upgradient wells is in progress and the installation of additional upgradient monitoring wells has been scheduled.

During the monitoring of residential wells in the area southwest of the GE/Moreau site, encompassing Terry Drive, Cheryl Drive, and Myron Road, low levels of five organic compounds were detected: 1,1,1-trichloroethane, tetrachloroethylene, chloroform, trichloroethylene, and methylene chloride.* Information collected from home owners concerning the reported depths of their wells indicates the horizon of contamination is between 35 and 55 feet below grade. Documented well construction information is not available for a more accurate delineation of the vertical extent of residential well contamination.

None of the contaminated wells in this area shows a consistent or steady pattern of contamination. Low level contamination of a given well may be detected in one or two rounds of sampling but not in others. Moreover, it is not unusual to obtain results which show that at a given time the wells of adjacent homeowners have different low level contaminants present.

One of the contaminants in the residential wells is 1,1,1-trichloroethane, which showed concentrations of 9.4 and 5.0 ppb at a Myron Road residence and 1 to 10 ppb at two Cheryl Drive residences. However, transverse dispersion from the center line of the defined plume (Plate 12) shows great attenuation over a very short distance. No well sampled within the defined plume had a concentration of 1,1,1-trichloroethane high enough to make it likely

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*A trace (less than 1 ppb) of 1,1-dichloroethane was also reported in one residential well sample in one of the sampling rounds.

that the 1,1,1-trichloroethane found in a few residential wells is a result of dispersion from the plume. In addition, equipotential lines showing ground-water flow due south/southeast of the GE/Moreau site indicate no flow component to the west. Given this condition and the dilution factors seen in wells along the path(s) of transverse dispersion, it is unlikely that the 1,1,1-trichloroethane found in residential wells comes from the area of the disposal site or the defined plume.

The same reasoning can be applied to tetrachloroethylene which also was detected in the residential wells at the 1-2 ppb level. Monitoring wells in the defined plume which contained significant levels of tetrachloroethylene (e.g. DGC-10D, 11I; 47 and 80 ppb, respectively) also contained very high levels of TCE, which were not detected at the expected significant level in the residential wells. Given the dilution necessary to cause a concentration gradient of tetrachloroethylene from, say, 80 ppb at DGC-11I to 1 ppb at the residential wells, we would expect to see TCE in the residential wells show a corresponding drop in concentration. Since DGC-11I contained 28000 ppb TCE we would expect accordingly about 350 ppb TCE in the residential wells. However, of the three residential wells where tetrachloroethylene was detected in any of the sampling events (1.3 and 2.4 ppb; 1 ppb; 1 and 1.4 ppb), TCE was only detected at one home at very low levels (3.1 and 1.8 ppb, corresponding to the home with 1.3 and 2.4 ppb tetrachloroethylene).

The fact that elevated TCE levels have never been found, combined with knowledge of the general ground-water flow and the flow path and dimensions of the plume, make it unlikely that the plume is the cause of the low level organic contaminants found in these residential wells.

Only one residence (Cheryl Drive) had a trace of chloroform (1.1 ppb). However, as mentioned, downgradient of the homes, wells TM-D and DGC-5I also showed traces of chloroform as the only organic contaminant present (2.3 and 4.2 ppb, respectively). The same considerations of transverse dispersion of contaminants from the plume to these wells applies to chloroform as well as to those organic compounds just mentioned. TM-4 is the only well that contains a high concentration of chloroform. TM-4 also contains high concentrations of other contaminants, including trichloroethylene. Since the only contaminant present in the residential well on Cheryl Drive, in TM-D and in DGC-5I is chloroform, it is unlikely that the plume is the cause.

Although low level TCE has been detected in a Cheryl Drive residence and four Terry Drive homes, insufficient data is available for identifying its source. The ratio of TCE to other organics in the residential waters does not conform to the general pattern of contamination in wells directly downgradient of the disposal site.

No significance is attached to the finding of methylene chloride (1 ppb) in one Myron Road residence due to its relative absence everywhere else and its notoriety as a ubiquitous laboratory contaminant. Also, the absence of TCE in this well rules out the plume as the probable cause.

In addition to the evidence cited above, the absence of contamination in the Town of Moreau cluster TM-2, TM-5 and TM-C suggests that the low levels of organic contamination in the Terry, Cheryl and Myron area are not associated with the plume. Eight additional monitoring wells will soon be installed in an effort to confirm this. The eight additional wells will sample pathways to that area not fully covered by the existing network.

During the sampling of residential water supplies, water samples were collected from 16 wells reported to have been drilled into the bedrock aquifer. Chemical analysis of these samples failed to detect any organic compounds. These results indicate that the bedrock aquifer is uncontaminated in those general areas where the samples were collected. Plate 14 shows the locations of these wells along with their reported depths.

9.0 SURFACE WATER - EXTENT OF CONTAMINATION

The stream surface water contamination in the wooded area downgradient of the topographic escarpment leading from Moreau into Fort Edward is extremely consistent and nearly constant. The data in Table 9.1 support this contention. Of the twelve sampling rounds undertaken, results from nine are available and tabulated.

The two collection boxes and the four reservoirs have never had organic contaminants detected in them since monitoring began. The pump house located at the Fort Edward Water Treatment Plant was sampled four times (May 2 to June 13) and chloroform and dichlorobromomethane were detected. The pump house was dropped from the sampling program since it was thought that chlorination of the water generated these trihalomethanes. The two compounds were never detected at the stream sites (X-4 to X-7), where trichloroethylene and trans-1,2-dichloroethylene were found.

The clear well located at the Water Treatment Plant was added to the sampling program on June 27, 1984. Water at the clear well is only slightly chlorinated and the results indicate that contamination is non-existent at this point.

Trans-1,2-dichloroethylene may be present in the water as an impurity formed during the manufacture of trichloroethylene or as a result of the degradation of trichloroethylene.

Table 9.2 lists the ratios of trichloroethylene plus trans-1,2-dichloroethylene concentrations at different sites along the major stream path (X6 + X7, X4, X5) to portray the concentration decrease that occurs downstream due to dilution, turbulent flow, evaporation, etc.

Table 9.1

Stream and Reservoir Analytical Results
Fort Edward, New York

Site	Code	May 2	May 16	June 4	June 13	June 27	July 12	July 25	August 8	August 22	Range	Mean	Standard Deviation
New Reservoir		ND		ND	ND	ND	ND	ND	ND	ND			
Collection Box 1		ND	TCE,*	ND	ND	ND	ND	ND	ND	ND			
Collection Box 2		ND	<1	ND	ND	ND	ND	ND	ND	ND			
Christie Reservoir		ND		ND	ND,ND	ND	ND	ND	ND	ND			
Sanderspree Reservoir		ND		ND	ND	ND	ND	ND	ND	ND			
Dority Reservoir		--	--	--	--	ND	ND	ND	ND	ND			
X-4	A	130	140	140	120	140	170	120	150	140	120-170	138.9	14.5
	B	4.5	5.3	8.2	4.0	7.5	11	ND	5.9	8.6	4.0-11	6.3	3.1
X-5	A	46	51	57	45	59	63	38	63	69	38 - 69	54.6	9.6
	B	1.2	<1	1.6	<1	1.8	3.8	ND	ND	8.5	ND -8.5	4.22	3.3
X-6	A	260	240	250	210	240	260,240	190	250	220	190-260	234.4	21.7
	B	13	20	27	14	28	29,32	5.1	28	22	5.1-32	20.8	8.1
X-7	A	35	38	42	46	48	45	68	56	73	35 - 73	50.1	12.3
	B	ND	ND	ND	ND	ND	ND	ND	ND	ND	NA	NA	NA
Pump House	C	54	50	27,26	2.9,4.5	--	--	--	--	--			
	D	4.3	2.5	3.1,2.9	ND,ND	--	--	--	--	--			
Clear Well	C	--	--	--	--	ND	<1	ND	ND	ND			

ND = None Detected

Code:
A = Trichloroethylene
B = trans-1,2-dichloroethylene
C = Chloroform
D = Dichlorobromomethane

Note: Samples were also collected on 9/5, 9/19, and 10/4/84. Results are pending.

* Represents composite of all five samples.

Table 9.2

	5/2	5/16	6/4	6/13	6/27	7/12	7/25	8/8	8/22	mean	S.D.
(X6+X7)÷X4	2.29	2.05	2.15	2.18	2.14	1.80	2.19	2.14	2.12	2.12	0.13
X4 ÷ X5	2.85	2.85	2.53	2.76	2.43	2.71	3.16	2.47	1.92	2.63	0.33
(X6+X7)÷X5	6.52	5.84	5.44	6.00	5.20	4.87	6.92	5.30	4.06	5.24	1.13

NOTE: The above figures represent the reduction in total organics (trichloroethylene plus trans-1,2-dichloroethylene) in going from one site to another, as indicated.

Taking the point source as the fork at sampling points X-6 and X-7, the reduction in concentration is about two fold by the time water reaches sampling point X-4; between X-4 and X-5 a concentration reduction of just over two and one-half times occurs. The overall reduction from the fork to X-5 is approximately five and one-quarter fold. The first stage (fork to X-4) is very constant as evidenced by the very low standard deviation. This factor increases and the precision of the concentration reduction decreases as water flows downstream, i.e. the standard deviation for the reductions of the two stages and the overall effect, are approximately 6, 12, and 21 percent, respectively.

As the stream ending at sampling point X-6 begins with a seep from the escarpment, it builds up momentum and volume as it moves downstream until finally it merges with the stream that ends at sampling point X-7. From the fork to X-4 the water is very shallow (3"-6") and slow moving; sometimes it is underground. This section is in heavily shaded forest. At X-4, the forest ends and an open marsh begins. The section of the marsh containing X-4 has heavy vegetation, mostly over six feet high. In the summer time this area was very hot, sunny and humid, causing an increase in stream temperature which would also increase volatility. X-5 is in a partially sunny, wooded area, unlike the other two sites.

In an attempt to close the data gap between the last well in the defined plume (DGC-3) and the earliest surface water site (X-6), a sample of water was collected from the toe of the escarpment where water emanating from the ground forms the stream that ends at X-6. This was collected on October 4; results are pending from the laboratory.

The consistency of the levels of surface water contamination may offer a mechanism of ground-water monitoring - levels in the surface water may act as indicators of levels in the ground water. If the relationship between ground-water contamination and surface water contamination can be linked by a constant parameter, then concentration information of one may be used to predict levels in the other, and vice-versa.

It is interesting to note that the stream containing site X-7 contributes only trichloroethylene and not trans-1,2-dichloroethylene. The stream at X-6 contains the highest concentration of both chemicals, the percent of trans-1,2-dichloroethylene relative to trichloroethylene being 8.87%.' Although the stream containing X-7 has the same topography as the stream ending at X-6, it is interesting that we do not see at X-7 the 4-5 ppb trans-1,2-dichloroethylene expected from the 50 ppb (mean) of trichloroethylene.

10.0 SOILS - EXTENT OF CONTAMINATION

Due to time and access limitations of the soil sampling/analysis program, there is insufficient data to substantiate the reported widespread PCB contamination. Of the approximately 160 sites that have been analyzed by the field kit, only four have shown the possible presence of PCBs: The sites are identified on a map included in Appendix E.

Site	PCB as Aroclor 1242 (ppm)
A1b	36, 10
A1c	224, 180(216), 88
A2c	232, 162(180)
A4c	106, 232(252)

Note: Replicate figures represent different extractions on different days. The number in parentheses is a repeat millivolt reading of the second solution after six hours.

The following samples have been sent to the ETC Laboratory for PCB confirmation by gas chromatography at a 1-2 ppm method detection limit; results are pending.

A1b
A1c
A2b
A2c
A3c
A4c
A5c
A14c
A19c
C8
J9
K2
M3

As soon as access to the site of proposed soil sample collection has been obtained and modifications to the sampling protocol are approved, site soil sampling will continue. Field kit and laboratory analyses will continue as samples become available. An addendum to this section of the report will be prepared and submitted following review and evaluation of the data.

11.0 AIR - EXTENT OF CONTAMINATION

The NYS Department of Environmental Conservation has established two air quality monitoring stations in the vicinity of the GE/Moreau Site. Their locations are shown on a map attached to the Work Plan located in Appendix A. The stations were activated in August, 1983 and maintained operation until mid November, 1983 approximately one month after remedial operations at the disposal site were interrupted. The stations were reactivated in mid June, 1984 prior to renewed remedial activities at the site. As of the preparation of this report, the stations were still operational with anticipated termination approximately mid November, 1984. Checking and sample/data collection from each station was done weekly by the on-scene coordinator from DEC with the exception of the initial 3-week period when operational checks were made every two or three days.

The stations monitor PCBs, trichloroethylene, benzene, methylene chloride, and total suspended particulate. In addition, the north station also monitors temperature, wind speed, and wind direction.

All of the results during the 1983 monitoring period were below detection limits with one exception. On November 2, 1983, the 24-hour average benzene concentration was found to be 18 ug/m^3 at the station near Terry Drive. The 1983 data report is located in Appendix K.

Communication with the Division of Air indicates that the evaluation and reporting of the 1984 data will not be available until the test results from the final samples are available and subsequent data evaluation, report preparation and review have been completed. The estimated availability of the report is early 1985.

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