E.C.JORDAN CO. ENGINEERS & SCIENTISTS

NEW YORK STATE DEPARTMENT OF ENVIRONMENTAL CONSERVATION

SUPERFUND STANDBY CONTRACT

KINGSBURY LANDFILL SITE
Kingsbury
Washington County, New York
WORK ASSIGNMENT NO. D002472-1

HYDROGEOLOGIC REPORT
VOLUME I - TEXT

DECEMBER 1991

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KINGSBURY LANDFILL SITE
KINGSBURY, NEW YORK

Submitted to:

New York State Department of Environmental Conservation
Albany, New York

Submitted by:

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DECEMBER 1991

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EXECUTIVE SUMMARY

E.C. Jordan Co. (Jordan), under contract to the New York State Department of Environmental Conservation (NYSDEC), is submitting this Hydrogeologic Study Report for the Kingsbury Landfill Site, Washington County, New York. This report has been prepared in accordance with the requirements of the NYSDEC Superfund Standby Contract under Work Assignment No. D002472-1.

Closure activities were conducted at the Kingsbury Landfill between 1987 and 1989, including construction of a landfill containment system consisting of a circumferential slurry wall and cap. When the groundwater elevation rose above the southern portion of the slurry wall in 1988, a trench/drain system was installed to relieve what was considered a temporary condition caused by precipitation-generated groundwater mounding prior to the landfill cap installation. An Interim Leachate Treatment System (ILTS) has been treating leachate collected by the drain system since March 1989.

When it appeared that the elevated groundwater levels inside the Kingsbury Landfill persisted, the NYSDEC tasked Jordan (Task 2 of the April 1990 Kingsbury Site Work Plan) to conduct a Hydrogeologic Study to:

- identify the cause of high groundwater levels in the landfill;
- evaluate the seasonal fluctuations of the water table in the landfill; and
- determine the optimum groundwater level to mitigate future releases of landfill leachate.

The Hydrogeologic Study of the Kingsbury Site was conducted in three phases: (1) background information was collected on regional and site specific geologic and hydrogeologic conditions, (2) a field investigation was performed to supplement the existing site specific geologic and hydrogeologic data, and (3) the data were analyzed, and a conceptual hydrogeologic model was developed and incorporated into a numerical groundwater flow computer model.

Background geologic and hydrogeologic information was collected from documents and interviews with personnel from the New York Geologic Survey, from Soil Conservation Service documents, as well as pre- and post-construction engineering reports prepared by O'Brien & Gere Engineers, Inc. (1983) and Blasland and Bouck, Engineers, P.C., (1990). The data collected by Clean Harbors Environmental

Services Inc. as part of their ongoing groundwater monitoring program at the landfill was also used.

The purpose of the background data search was to gain a better understanding of the site geology, to facilitate proper location of monitoring wells, and to formulate a conceptual model of the site as a basis for groundwater modeling. The background documents included information regarding the geologic processes responsible for the deposition of soils at the site, and information pertaining to pre-closure hydrogeologic conditions, construction activities (i.e., clay cap and slurry wall emplacement), existing monitoring wells and pan lysimeters, and seasonal fluctuations in groundwater elevations.

The field effort further defined the site geology and hydrogeology by providing additional monitoring locations from which groundwater level measurements and groundwater samples could be obtained. In addition, the field effort was developed around an aquifer testing program that was used to support the computer modeling of groundwater flow conditions at the site.

The field portion of the project included installing and developing 28 observation wells and two pumping wells, collecting groundwater samples from selected wells, and conducting hydraulic stress tests to determine the aquifer hydraulic conductivity, transmissivity, and storativity. The information gathered included detailed lithologic logs from the newly installed wells, grain size analysis from several soil samples, groundwater level and chemical data, and aquifer stress test data to determine aquifer parameter values.

Following analyses of all the data, a hydrogeologic conceptual model of the site was developed. The conceptual model was the basis of a computer model used to simulate the groundwater flow conditions at the site. The computer model was used to evaluate the cause of high groundwater levels in the landfill, and to determine the optimum groundwater level to mitigate future releases of landfill leachate. Four scenarios were used in the evaluation of high groundwater levels within the landfill:

- Leakage through the base
- Infiltration through the cap
- Leakage through the slurry wall
- Existence of a mound within the landfill

Based on the model results and interpretations from the hydrogeologic data, the most likely cause of the elevated groundwater levels within the landfill is net leakage through the upgradient slurry wall, which could be expressed as a leaky or poorly-keyed slurry wall, or a sand seam beneath the wall which discharges within the upper slurry wall area.

Evaluation of groundwater levels inside the landfill indicated a decrease of approximately 0.6 to 1.0 foot during 1990 ILTS operations. The potential net leakage rate into the landfill appeared to be less than the average 1990 ILTS pumping rate of 6 gallons per minute (gpm). Preliminary data from 1991 indicates that the ILTS is continuing to lower the water table inside the landfill.

The optimum groundwater equilibrium level in the landfill depends on which of the four scenarios presented above are responsible for the elevated water table in the landfill. The strategy is to maintain groundwater levels which are several feet below the top of the slurry wall's southern end while minimizing head gradients which would increase flux into the slurry wall's northern end and out of the slurry wall's southern end.

1.0 INTRODUCTION

E.C. Jordan Co. (Jordan) under contract to the New York State Department of Environmental Conservation (NYSDEC), is submitting this Hydrogeologic Study Report for the Kingsbury Landfill Site, Washington County, New York. This report has been prepared in accordance with the requirements of the NYSDEC Superfund Standby Contract and Work Assignment No. D-002472-1.

1.1 SCOPE

This report presents the findings of the Hydrogeologic Study described under Task 2 of the April 1990 Kingsbury Site Work Plan which was initiated by the NYSDEC to address elevated groundwater levels within the Kingsbury Landfill slurry wall and cap containment system. Components of the Task 2 Hydrogeologic Study are:

- Subtask 2.1 - Review and Evaluation of Historical Information;
- Subtask 2.2 - Provide Recommendations for New Monitoring Wells;
- Subtask 2.3 - Installation of New Monitoring Wells;
- Subtask 2.4 - Collection of Groundwater Data; and
- Subtask 2.5 - Development of Groundwater Flow Model and Report.

Task 3 will evaluate potential groundwater control alternatives and will be presented in a subsequent report.

1.2 OBJECTIVES

The objectives of the Kingsbury Hydrogeologic Study are:

- to identify the cause of high groundwater levels in the landfill;
- to evaluate the seasonal fluctuations of the water table in the landfill; and

- to determine the optimum groundwater level to mitigate future releases of landfill leachate.

This Hydrogeologic Report consists of 7 sections. Section 1.0 establishes the scope and objectives of the Hydrogeologic Report. Section 2.0 provides background and historical closure information on the landfill, and Section 3.0 describes the environmental setting of the Kingsbury Site. The hydrogeologic investigation activities are described in Section 4.0, and Section 5.0 presents the results of the investigation. The development and use of the groundwater flow model are described in Section 6.0. Section 7.0 presents evaluations of the Hydrogeologic Report's objectives.

2.0 BACKGROUND

The following subsections provide background information on the Kingsbury Site.

2.1 SITE LOCATION AND DESCRIPTION

The Kingsbury Site is located in the southwestern part of the Town of Kingsbury, Washington County, New York. Access to the site is from Burgoyne Avenue and Feedertow Road (Figure 2-1). The Feedertow Canal, which runs approximately parallel to Feedertow Road, eventually drains into the Old Champlain Canal located southeast of the landfill. The Kingsbury Site is an inactive landfill approximately 17 acres in area (Figure 2-2). The landfill has been inactive since 1987.

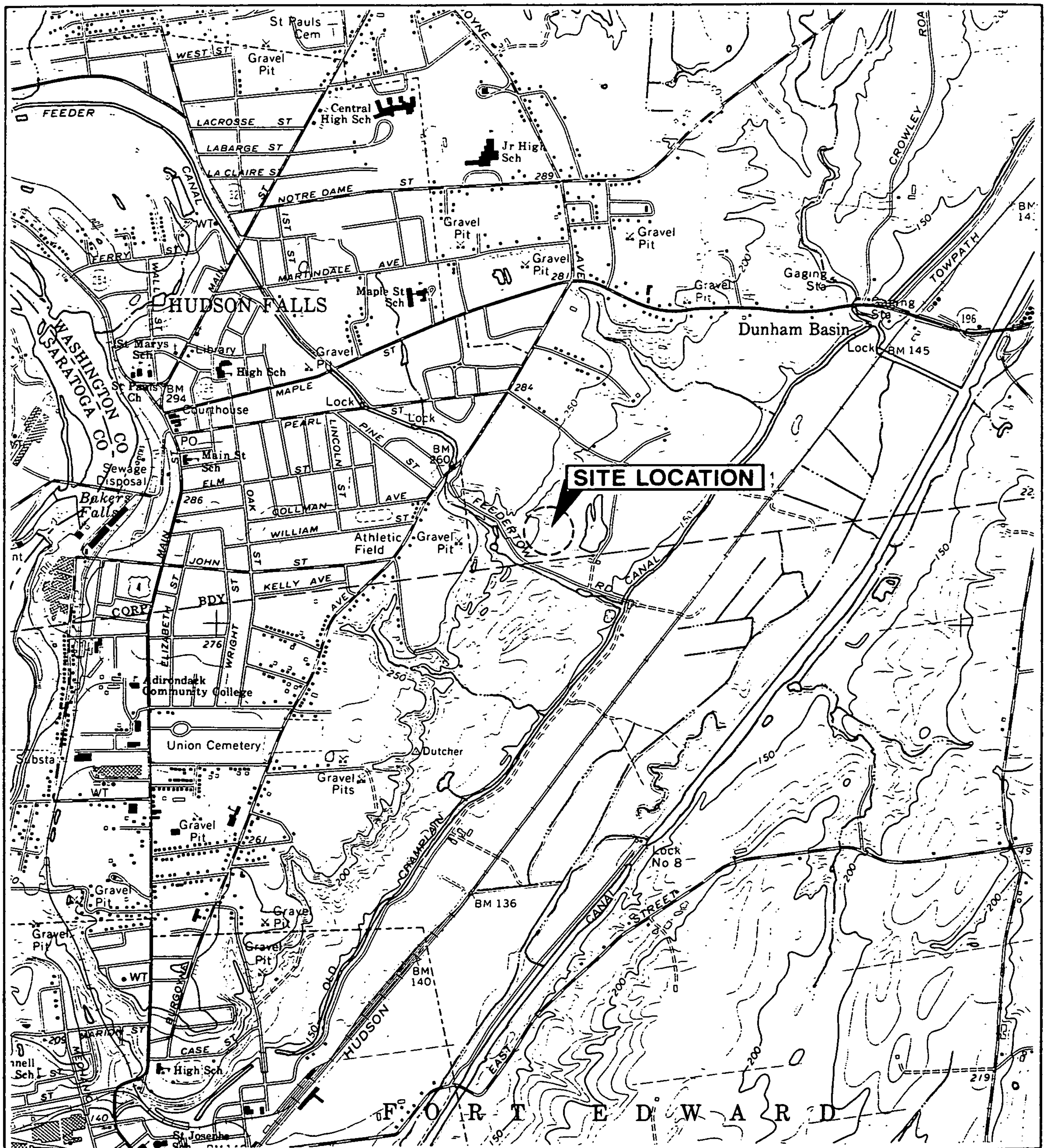
The top and the toe of the Kingsbury Landfill are at elevations of approximately 280 feet and 210 feet mean sea level (MSL), respectively. Leachate within the landfill is contained by a soil/bentonite cutoff wall surrounding the landfill; the cutoff wall is keyed into a glacial clay deposit at a depth of 25 to 85 feet below ground surface. A soil cap of low permeability material covering the top of the landfill inhibits infiltration of precipitation. The precipitation runoff is collected in drainage swales at the top of the landfill. Surface water in the swales eventually flows into the Feedertow Canal.

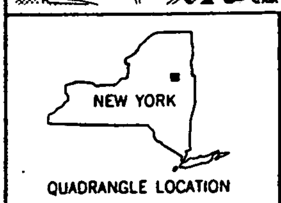
Leachate from the landfill flows into collection trenches at the southeast corner of the landfill. The leachate is processed through a temporary treatment plant located on site. The treatment plant effluent is discharged to the Feedertow Canal.

2.2 SITE HISTORY

The Kingsbury Landfill is reported to have been used as a disposal site since the 1930's (O'Brien and Gere, 1983). The landfill reportedly received municipal and industrial wastes, including polychlorinated biphenyls (PCBs) and industrial solvents until 1987 when the landfill was closed.

Complaints of leachate migrating into a surface water body that was used as a domestic drinking water supply by an adjacent downgradient property owner resulted in legal action against the Town of Kingsbury between 1967 and 1972. The Town of





SOURCE: USGS QUADRANGLE, HUDSON FALLS N.Y.. DATED 1966, 7.5 MINUTE SERIES.

SCALE IN FEET

0 2,000 4,000

FIGURE 2-1
SITE LOCATION MAP
HYDROGEOLOGIC INVESTIGATION
KINGSBURY LANDFILL

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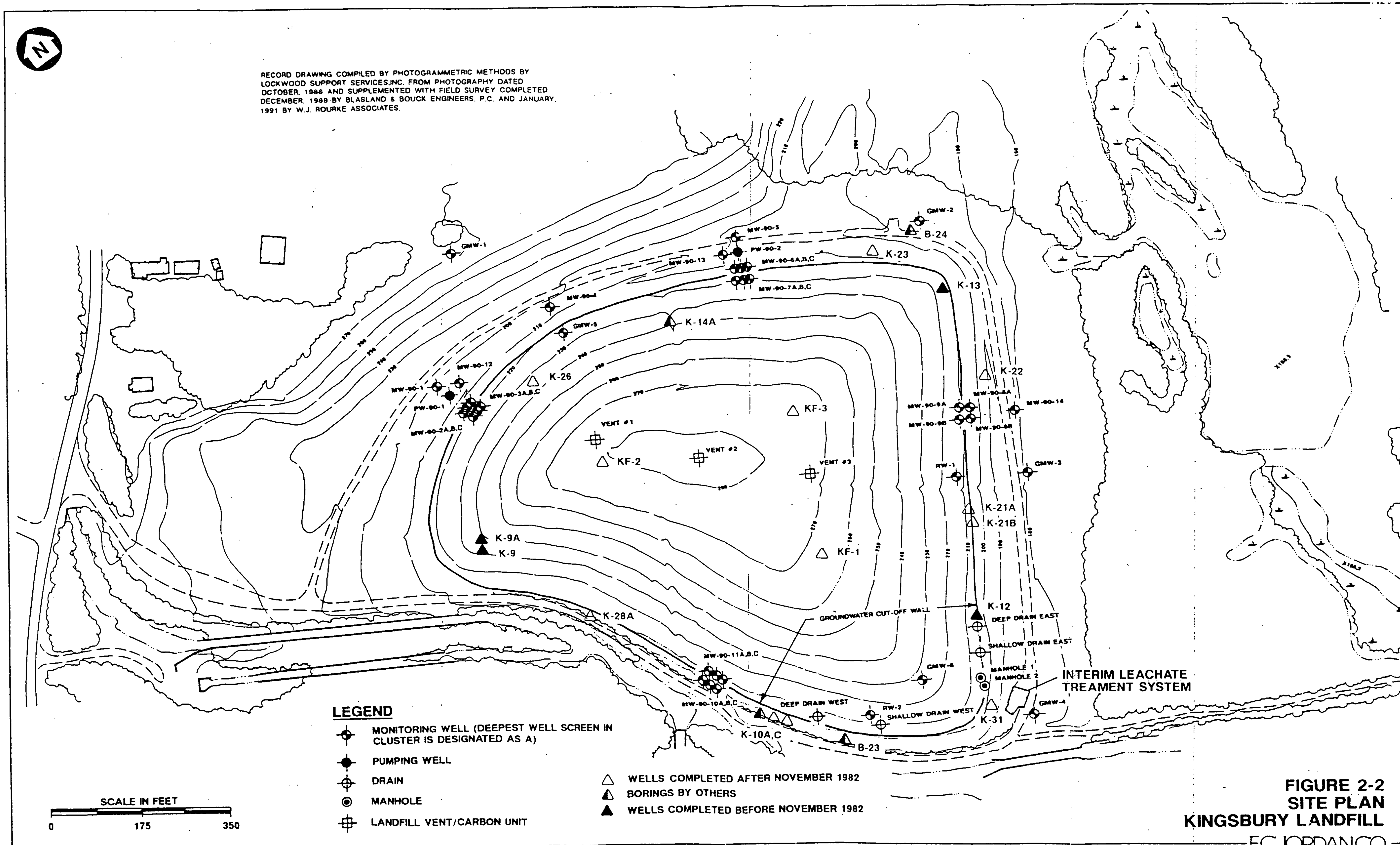

FIGURE 2-2
SITE PLAN
KINGSBURY LANDFILL

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Kingsbury attempted unsuccessfully to divert surface water runoff and leachate migration toward the Feedertow Canal by constructing several drainage ditches. In 1975, during hearings before NYSDEC, the General Electric Company (GE) acknowledged that they disposed of 1900 tons of industrial waste material at the landfill, including PCBs in the form of scrap capacitors.

2.3 PREVIOUS INVESTIGATIONS

Following the disclosure by GE of its involvement in the disposal of hazardous materials in the Kingsbury Landfill, the NYSDEC and the New York State Department of Health (NYSDOH) initiated a sampling and analysis program of surface water, leachate, and soil sediment. The results of the preliminary investigation indicated that PCBs were present in the surface water in the vicinity of the landfill.

Subsequent to the initial findings of the preliminary investigation, Weston, Inc. and Wehran Engineering, P.C. were contracted by NYSDEC to conduct additional investigations at the Kingsbury Site between 1977 and 1979. NYSDEC completed additional sampling and analysis in 1980 and 1981.

The results of these investigations revealed elevated levels of PCBs and other contaminants prompting closure of the Kingsbury Landfill. An agreement between NYSDEC and GE, signed on September 24, 1980, outlined GE's responsibilities for the final remedial closure plan. Under the terms of the agreement, GE conducted a hydrogeologic investigation and prepared remedial action plans. A revised engineering report detailing the hydrogeologic investigation and proposed remedial alternatives was issued in November, 1982 (O'Brien & Gere, 1982). A report addressing the theoretical post construction groundwater hydraulic conditions was issued in February, 1983 (O'Brien & Gere, 1983). The Closure Plan was implemented and details of the construction activities are provided in the Engineering Report (Blasland & Bouck, 1990).

2.4 LANDFILL CLOSURE DESIGN AND CONSTRUCTION

The specifications of the proposed closure design were detailed in the Kingsbury-Fort Edward Sites Engineering Report -- Revised (O'Brien & Gere, 1982). The landfill closure design specified the following components:

- a graded and compacted clay cap with a saturated hydraulic conductivity of 1 X 10^{-7} centimeters per second (cm/sec) or less;
- a vegetative cover over the clay cap;
- a groundwater cutoff slurry wall with a hydraulic conductivity of 1 X 10^{-7} cm/sec or less extending around the perimeter of the landfill and keyed into the top of an underlying lacustrine clay deposit;
- a perimeter surface water drainage system;
- a gas vent system; and
- a monitoring and relief well system.

The purpose of these components was to effectively isolate the waste material from the surrounding environment and minimize the production of leachate and its migration from the site. Closure activities were conducted between 1987 and 1989, including the installation of the clay cap and groundwater cutoff wall. Construction related services were provided by Blasland & Bouck Engineers, P.C. (Blasland & Bouck, 1990).

Following completion of the groundwater cutoff wall, a seep area was observed in the southern portion of the site approximately 10 feet inside the cutoff wall. The seep was the result of rising groundwater levels caused by the restriction of natural flux (damming effect) by the slurry wall (Blasland & Bouck, 1988). Following completion of the cutoff wall (late summer 1987), but prior to the completion of the clay cap (mid-June 1988), Blasland and Bouck conjected that precipitation continued to contribute to a groundwater mound within the landfill. As a result of water being released (draining) from storage (the mound), the groundwater elevation rose above the cutoff wall along the south side of the landfill; the overflow was expressed at the ground surface as leachate seeps. O'Brien & Gere (1983) had earlier estimated that the groundwater mound would dissipate during the construction activities provided that the cap and cutoff wall were completed during the same field season.

To address what was considered a temporary condition, a trench/drain system was installed in the southern portion of the landfill during October and November 1988 to reduce the groundwater elevation within the containment system and prevent further discharge over the cutoff wall. The leachate was pumped from the trench/drain to a newly constructed Temporary Leachate Treatment System (TLTS) where the leachate was treated for metals, volatile organics, and PCBs. A second shallow drain system was installed in April 1989. From March through December 1989, the ILTS treated and discharged approximately 2 million gallons to the Feedertow Canal. Between January and April 1990, the TLTS was upgraded and renamed the Interim Leachate Treatment System (ILTS). The ILTS operated during 1990 from June through December, treating and discharging a total of approximately 1.5 million gallons into the Feedertow Canal under NYSDEC effluent criteria.

3.0 ENVIRONMENTAL SETTING

The following subsections provide an overview of the regional and site specific hydrogeologic conditions.

3.1 REGIONAL GEOLOGY AND HYDROGEOLOGY

The Kingsbury Site is located in the physiographic province referred to as the Hudson - Mohawk Lowlands (Cadwell and others, 1986). The bedrock underlying the region is composed of Ordovician age shale, sandstone and limestone. The lowland area was formed as a result of erosional processes that chemically and physically disintegrated the Ordovician age rocks at a rate more rapid than the older, more resistant formations in the Adirondack Highlands to the west, and the New England Uplands to the east. The process took several hundred million years and occurred prior to glaciation. The regional landscape was further modified during the past two million years by glacial erosion and deposition.

Glacial erosion was more pronounced in the lowland areas because the ice sheets were thicker in the valleys. As the tremendous mass of ice successively advanced and retreated in the valleys, the relatively soft bedrock was ripped up, disintegrated and subsequently deposited as unconsolidated material during periods of glacial melting. Several glacial advances and retreats in the region have been documented, although each successive advance essentially destroyed the geologic record of the preceding event. The glacial imprint of the region is principally the result of the last ice sheet covering the area. The last episode of continental glaciation, referred to as the Late Wisconsin glaciation, began approximately 30,000 years ago and reached to its maximum extent on Long Island, New York approximately 20,000 years ago.

As the glacier advanced to the southeast, large quantities of soil and bedrock were carried with it. Basal till deposits were deposited beneath the advancing glacier. The gradual melting and subsequent retreat of the ice sheet resulted in the release of the captured rock, gravel, sand, silt, and clay. As the glacier melted, the upland areas were exposed first, with ice lobes remaining in the lowland areas. The valley lobes often dammed meltwater streaming from the thawing glacier. As a result, proglacial lakes were formed and lake (glaciolacustrine) sediments began to accumulate. The Hudson - Champlain Glacial Lobe created Glacial Lake Albany, which was the largest of the Pleistolene lakes to cover parts of Washington County

(Cushman, 1953). Eventually the ice lobe retreated and stagnated to the north of the present Kingsbury Site. During this period, Glacial Lake Albany reached its maximum length of approximately 140 miles and width of 8 to 12 miles.

The ice dam that created Glacial Lake Albany was located approximately 10 miles north of the Kingsbury Site in the vicinity of Fort Ann, New York where it remained intact for approximately 5,000 to 8,000 years. During this period, silt and clay particles were transported by rivers and deposited into the deep, still environment of Glacial Lake Albany. Deltas consisting of gravel to fine sand were being formed where rivers entered the lake. One such delta is located approximately eight miles to the west of the Kingsbury Site. The delta formed as the glacial Hudson River, which was transporting glacial debris from the adjoining Adirondack Highlands, flowed into Glacial Lake Albany and deposited its load. Over time, the delta prograded outward from its source, gradually covering the deeper deposits of silts and clays. Finer-grained sand formed the stratified bottomset beds of the delta. According to Cushman (1953), the terminus of the delta was in the vicinity of the Kingsbury Site.

The grain size of the sediments composing the delta deposits range in size from fine sand and silt to coarse gravel. The grain size typically becomes coarser in the direction of the delta source (Cushman, 1953). Groundwater occurs in both the bedrock and glacial deposits. The porosity and permeability of the delta deposits are relatively high, making them the most productive aquifers in the region (Cushman, 1953). The underlying glaciolacustrine silts and clays, while having high porosity, have low permeability making them low yielding in terms of aquifer production. Flow of groundwater through the bedrock is controlled primarily by joints and cleavage planes. The joint openings in the bedrock are small thereby reducing storage, permeability, and yield, particularly at depth.

3.2 SITE GEOLOGY AND HYDROGEOLOGY

3.2.1 Soils

A soil survey map of Washington County indicates that the Kingsbury Landfill Site overlies lacustrine and delta derived soils known as the Vergennes-Kingsbury and Oakville soil series, respectively (U.S.D.A., 1972).

The Oakville series soils, which consist of fine to medium sand and silt, represent the bottomset beds of the delta. The Vergennes-Kingsbury soils consist of calcareous, varved-glaciolacustrine silt and clay deposits. The bedrock beneath the site has been mapped as the Snake Hill Formation, a black, non-fossiliferous shale of Ordovician age (Cushman, 1953).

Soil borings installed by O'Brien & Gere (1982 and 1983) revealed that the Kingsbury Site is located within a deposit of brown, fine to medium sand and silt. The thickness of the sediments varies between zero and sixty feet; the deposit thickens in a easterly direction. A deposit of gray silt and clay was encountered below the sand (delta) deposits. The soils identified by O'Brien and Gere were found to be consistent with the description of the delta and lacustrine deposits that are provided in the Soil Survey (1972) and Groundwater Resources (1953) of Washington County.

An investigation into the possible presence of vertical sand dikes within the underlying clay unit was completed as part of this project. Several geologists knowledgeable of the Kingsbury vicinity were contacted including Professor Parker Calkin from the State University of New York (SUNY) at Buffalo, Dr. Robert LaFleur from RPI, Dr. David Franzi from SUNY-Plattsburg, and Mr. Donald Cadwell of the New York State Geologic Survey. Mr. Cadwell, who has performed extensive mapping projects in the Kingsbury region, reported no knowledge of vertical sand dikes within the clay unit in this region (Cadwell, 1991).

3.2.2 Groundwater

Groundwater at the site exists throughout the glacial deposits. The elevation of the water table in wells installed by O'Brien and Gere (1982) indicate that groundwater is flowing in a south-southeasterly direction. The majority of the flow is limited to the lacustrine sand deposits (O'Brien & Gere, 1982). Average pre-construction groundwater recharge was estimated by O'Brien & Gere to be 23,500 gallons per day (gpd) entering the Kingsbury Site. This value was calculated based on a site area of 35 acres and a net groundwater recharge value of 9 inches per year (approximately 25% of the yearly precipitation). Groundwater flux was estimated to be 20,000 gpd at a velocity of 0.67 feet per day (ft/day) based on the following conditions:

- cross sectional area = 20,000 ft^2;
- hydraulic gradient = 0.04 ft/ft;

- hydraulic conductivity = 25 gpd/ft^2 = 1.2 E-3 cm/sec;
- effective porosity = 20%.

The difference of 3,500 gpd between the recharge rate and discharge rate was believed to be caused by discharge to surface water as base flow but is also probably well within the limits of accuracy provided by the estimated aquifer parameter values. Based on further investigation, these figures were later amended (Blasland et. al., 1983). The pre-remediation groundwater flow through the site was estimated to be 78,000 gpd where:

- cross sectional area = 31,500 ft^2
- hydraulic gradient = 0.033 ft/ft
- hydraulic conductivity = 75 gpd/ft^2 = 3.5 E-3 cm/sec

A post construction groundwater flow analysis was performed prior to remediation to assess the influence of the design (Blasland & Bouck, 1983). The analysis recognized that groundwater could flow into and out of the site via three paths:

- the site cap;
- the groundwater cutoff wall; and
- the underlying glaciolacustrine deposits at the base of the site.

The calculations were based on the assumptions that steady-state conditions existed and groundwater levels measured in December 1982 were representative of long term water levels in and around the landfill. It was also assumed that the geologic formations were homogeneous and isotropic and remedial construction was performed according to the design.

The analysis concluded that post-construction groundwater flow through the site would be 1,200 gpd, which is a 98% reduction from pre-construction conditions. The flow would occur through the walls and the underlying glaciolacustrine deposits; the cap would essentially be impervious. It was also determined that a groundwater equilibrium level would be established at an elevation between 193 and 195 feet MSL within the landfill containment system.

3.2.3 Surface Water

Surface water flow in the vicinity of the Kingsbury Site is present within the Feedertow Canal, which is located adjacent and to the west. The Feedertow Canal empties into the Old Champlain Canal which is located approximately 2,000 feet southeast of the site. Surface water from the site also drains into an unnamed pond located 500 feet to the southeast, and into a drainage ditch located along the northern toe of the landfill slope. The Feedertow Canal discharge volume varies and is mechanically controlled by the New York State Department of Transportation (NYSDOT). Baseflow, presumed to be fed by groundwater discharge, appears continuous throughout the year. Outflow from the manmade pond is controlled by a vertical culvert pipe in the southwest end of the pond. Surface water flow in the drainage swale on the north side of the landfill is continuous throughout the year indicating that the water table is at the ground surface in this area.

4.0 HYDROGEOLOGIC INVESTIGATION ACTIVITIES

Investigative methods used during this study are described in the following subsections. Results for each investigative activity are presented in Section 5.0.

4.1 SOIL BORING AND MONITORING WELL INSTALLATION

To verify the presence and depth of the overburden units and to provide monitoring locations for aquifer testing, 30 soil borings and 28 monitoring wells were installed from October to November 1990.

The locations of the monitoring wells were selected to evaluate: (1) the horizontal and vertical distribution of head both inside and outside the groundwater cutoff wall; (2) the degree of hydraulic connection through the clay base of the landfill; and, (3) the degree of hydraulic connection across the groundwater cutoff wall. Further rationale for the placement of the wells was provided in the Hydrogeologic Field Investigation Work Plan (E.C. Jordan Co., 1990a).

During the field program, NYSDEC requested that three additional observation wells (MW-90-12 through MW-90-14) be installed to further define the hydrogeology of the site. The locations of the wells installed during this investigation, in addition to the existing site wells [groundwater monitoring well (GMW)-1 through GMW-6], and the wells, drains, and manholes associated with the leachate collection system, are shown in Figure 4-1.

John Mathes, Inc. (Mathes) of Jobstown, New Jersey, provided one all-terrain mounted drill rig and one skid mounted drill rig for the installation of the wells which took place between October 10, 1990 and November 29, 1990. The borings associated with the observation wells were advanced using 4.25-inch inside diameter (ID) hollow stem augers. The pumping well borings were completed using 8.25-inch ID hollow stem augers.

Drilling was conducted using modified Level C personal protection equipment in accordance with the Site Health and Safety Plan (E.C. Jordan Co., 1990b). An ISD Dual Detector was used to detect methane and other combustible gases and a TIP photoionization detector (PID) was used to detect volatile organic compounds throughout the drilling program. Drill cuttings and fluids were contained and

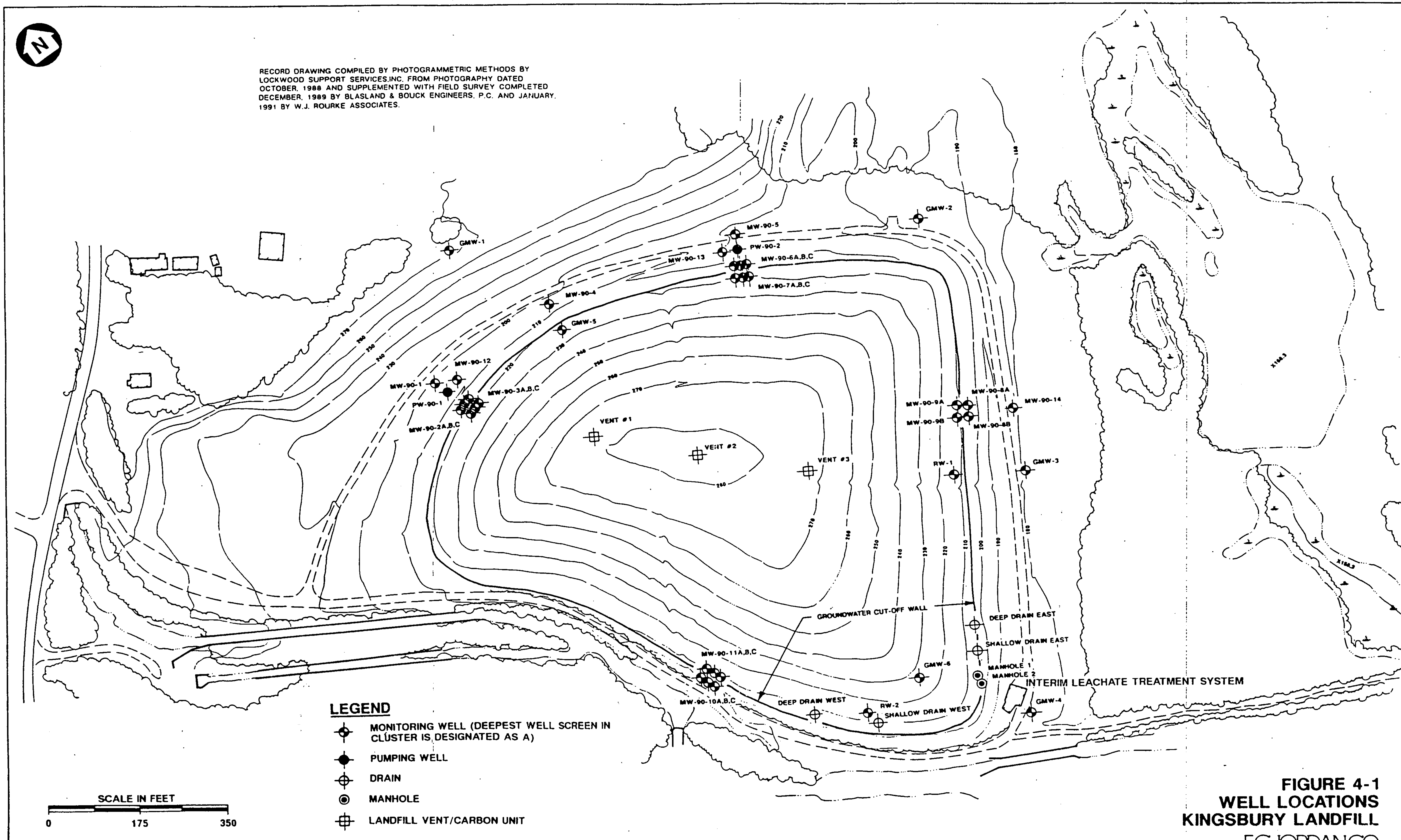

FIGURE 4-1
WELL LOCATIONS
KINGSBURY LANDFILL

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collected at the surface. Drill cuttings were placed in Department of Transportation (DOT)-approved 55-gallon drums. The drums were labelled and staged on pallets along the fenceline in the southern portion of the site. Any fluids generated during drilling were drummed, allowed to settle, and treated of at the ILTS.

Soil samples were obtained at depth using a 2-foot split-spoon sampler following the scheme detailed in Table 4-1. Soils were visually classified using the Unified Soil Classification System and logged by a field geologist; reference soil samples were stored in jars at the site. At the end of each day of drilling, the reference samples were screened in the field for the presence of volatiles using a PID. In addition, grain size analyses were performed on five of the samples taken from wells located outside the slurry wall that were screened in the sand aquifer. The grain size analyses are provided in Appendix A.

Monitoring well screens were installed at elevations based on visual examination of split-spoon samples. The "A" and "B" series observation wells were placed in the clay unit. The "A" series wells were installed approximately 15 feet into the lacustrine clay, and the "B" series wells were installed approximately 5 feet into the clay. Exceptions included wells MW-90-2A and MW-90-3A which were both installed within a till deposit encountered beneath the lacustrine clay unit, and well MW-90-6B, which was located within a sand unit bounded above and below by the lacustrine clay. The "C" series observation wells, and the remaining observation wells including the two pumping wells were screened in the sand unit immediately above the clay formation.

The wells were constructed of 2-inch ID 0.01-inch machine-slotted Schedule 40 polyvinyl chloride (PVC) and typically installed on a 6-inch to 1-foot bed of filter sand placed in the bottom of the boring. The "A" and "B" series wells were completed with one foot well screens. The "C" series wells, including wells MW-90-1, MW-90-4, and MW-90-5 were completed with 5-foot well screens; wells MW-90-12 and MW9-13 were constructed with 15-foot well screens.

The pumping test wells (PW-90-1, PW-90-2) were constructed of 6-inch I.D. Schedule 80 PVC with 0.07-inch machine slotted screen. The screen length varied from 15 to 10 feet for wells PW-90-1 and PW-90-2, respectively. Filter sand was backfilled to 2-feet above the top of the well screen, and a 2- to 3- foot bentonite pellet or grout seal was placed on the top of the sandpack. The specifications of the filter sand are

TABLE 4-1
GEOLOGIC SAMPLING SCHEME
KINGSBURY LANDFILL

WELL(S)	SCREENED MATERIAL	SCREENED LENGTH (FT)	GEOLOGIC SAMPLING SCHEME
Observation Wells: "A" Series	Clay	1	1) Collect 2-foot split-spoon samples every 5 feet until clay is encountered 2) Sample continuously thereafter for a minimum of 15 feet
Observation Wells: "B" Series	Clay	1	1) Collect one 2-foot split-spoon sample over the screened interval
Observation Wells: "C" Series	Sand	5	1) Sample continuously over the screened interval
Observation Wells: MW-90-1,4,5,14	Sand	5	1) Collect one 2-foot split-spoon sample every 5 feet 2) Sample continuously over the screened interval
Observation Wells: MW-90-12,13	Sand	15	1) Sample continuously over the screened interval
Pumping Wells: PW-90-1,2	Sand	15	1) Collect one 2-foot split-spoon sample every 5 feet 2) Sample continuously over the screened interval

shown in Table 4-2. The boring was then backfilled to the surface using a cement-bentonite grout (19:1 ratio) as the augers were being removed. After the grout seal had hardened over a 24-hour period, it was checked for settlement and additional grout was added as needed. The well was secured with a 5-foot steel protective casing and locking cap that was cemented in place around the well casing.

Drilling and sampling equipment were decontaminated by steam cleaning with potable water prior to arrival at the Kingsbury Site. The drilling rig, hollow stem augers, tools, and sampling equipment were also steam-cleaned with potable water between each boring. Well material was steam cleaned and wrapped in plastic to prevent cross contamination prior to well construction. Three samples of the potable water used in the steam-cleaning activities were analyzed for volatiles, semi-volatiles, and PCBs; the results are included in Appendix B. All steam-cleaning was completed at the on-site decontamination pad. The water that was collected in the decontamination pad was pumped to, and treated at, the ILTS. The solids remaining on the decontamination pad after dewatering, including the plastic liner used for the decontamination pad were drummed, labelled, and staged along with the drums containing the drill cuttings.

Each well was developed by the drilling subcontractor for the purpose of enhancing the well's hydraulic connection with the formation. The airlift method was used for the wells screened in the sand and bailing was used for those wells screened in the clay. Development in the sand wells continued until the turbidity of the water from the recovered well was 50 Nephelometric Units or less. Wells screened in the clay were considered developed when the above criteria was met, or upon removal of five (5) well volumes of groundwater. The development water from the observation and pumping wells was containerized and transported to the ILTS for treatment. The wells were allowed to equilibrate for two weeks following development before any groundwater sampling or aquifer testing was performed.

4.2 GROUNDWATER SAMPLING

Groundwater samples were collected from 19 monitoring wells screened in the sand formation and submitted for chemical analysis to determine the groundwater chemical quality both inside and outside the groundwater cutoff wall. The wells chosen in consultation with NYSDEC included:

TABLE 4-2
SAND FILTER PACK SPECIFICATIONS
KINGSBURY LANDFILL

			SAND FILTER PACK SPECIFICATIONS	
WELL DIAMETER (INCHES)	SLOT SIZE (INCHES)	SCREENED MATERIAL	70% CUMULATIVE RETAINED SIZE (INCHES)	UNIFORMITY COEFFICIENT
2	0.010	Clay	0.015	<1.8
2	0.010	Sand	0.030	<1.8
6	0.020	Sand	0.060	<1.8

NOTE: Specifications were based on grain size analyses from previous subsurface explorations performed by O'Brien and Gere.

- existing monitoring wells GMW-1 through GMW-6
- MW-90-2C, 3A, 3C, 4, 6C, 7C, 9B, 10C, 12, 13, 14
- PW-90-1, PW-90-2

The original plan called for sampling MW-90-1, MW-90-5, and MW-90-11C in addition to those listed above. However, due to frozen groundwater conditions in these three wells, sampling was not conducted.

The samples were collected by Clean Harbors, Inc. (CHI) from February 5, 1990 to February 20, 1990 and analyzed by the CHI Laboratory. The results of the analyses are shown in Appendix B. The water quality data were reviewed by Jordan to determine the potential for capturing contaminated groundwater during the aquifer pumping tests.

4.3 SURVEY

Observation and pumping wells, in addition to existing monitoring wells, relief wells, drains, and manholes were surveyed to the nearest 0.01 foot vertically, and 0.1 foot horizontally. All wells locations were tied to the National Geodetic Vertical Datum of 1929 and the New York State Plane Coordinates. Appendix C presents the location and elevation data completed by W.J. Rourke, Associates of South Glens Falls, New York, in January 1991.

4.4 GROUNDWATER LEVEL AND PAN LYSIMETER MEASUREMENTS

Groundwater level measurements of the existing groundwater monitoring wells (GMW-1 through GMW-6), relief wells, drains and manholes were obtained on a weekly basis using an electronic water level meter beginning in 1989. The collection of weekly groundwater elevation data for selected wells in the vicinity of and including the two pumping wells was initiated in March 1991. The data collected as of May, 1991 are included in Appendix D.

Monitoring of the four existing pan lysimeters (Figure 2-2) installed at the interface of the refuse and the compacted soil cap has been conducted since 1990 by CHI. The quantity of liquid that enters the pans was measured monthly and precipitation data from the Glens Falls Weather Station was also obtained. The information was used to determine the effectiveness of the cap. The raw data from the pan lysimeter measurements and calculations of the percent effectiveness of the cap as are shown in Appendix D.

4.5 Hydraulic Conductivity Testing

In situ hydraulic conductivity testing of each of the newly installed wells was performed to determine values of horizontal hydraulic conductivity (K_h) of soils immediately surrounding each well. Rising head tests were conducted on each of the sand wells by depressing the groundwater table within the well using pressurized breathing air. The depressed water table was maintained until the aquifer equilibrated. The pressure was instantaneously released and the water level response was continuously recorded by a pressure transducer and computerized data logger (manufactured by IN-SITU, Inc.).

Rising and falling head tests were performed on several of the wells screened in the clay or till by placing a solid "slug" in the well, allowing the water to equilibrate to within 5% of static level, and rapidly removing the slug. Water level measurements were recorded using a pressure transducer and data logger.

A third method of in-situ hydraulic conductivity testing was used for the remaining clay wells. Groundwater was removed from the well using a bailer; the recovery of the water to static conditions was monitored manually using an electronic water level meter.

The data from the in-situ hydraulic conductivity tests were analyzed using the computer program AQTESOLV (Geraghty and Miller, Inc., 1989). Data from wells under unconfined conditions were analyzed using the Bouwer and Rice method, and wells under confined conditions were analyzed using the Cooper and Jacob method. Data associated with aquifer testing and analysis are included in Appendix D.

4.6 PRELIMINARY GROUNDWATER FLOW MODEL

4.6.1 Objectives

A preliminary groundwater flow model was developed for screening purposes for the northern portion of the Kingsbury Landfill in order to: (1) assess whether the proposed aquifer pumping tests at PW-90-1 and PW-90-2 could be performed concurrently; and (2) establish a range of probable pumping rates to be utilized during the step drawdown tests.

4.6.2 Code Selection

Selection of a groundwater model numerical code was based on several considerations: the code must have the ability to include all significant hydrogeologic influences and boundary conditions, be well-accepted and documented, and be readily available for use by others (in the public domain). It was concluded that the U.S. Geological Survey (USGS) Modular Three-Dimensional Finite Difference Groundwater Flow Model Code (McDonald and Harbaugh, 1989) would be the most appropriate flow model to evaluate groundwater conditions and satisfy the above criteria. The USGS Modular Three-Dimensional Finite Difference Groundwater Flow Model Code (MODFLOW) is a finite difference model that provides the essential features needed to meet the preliminary modeling objectives.

4.6.3 Conceptual Model, Grid Configuration, and Model Inputs

Since the model was intended to be used for screening purposes, the conceptualization of the aquifer system was greatly simplified in order to quickly supply responses to the objectives stated in Section 4.6.1. The aquifer was modeled as a water table system of uniform thickness, with the only water inputs being recharge by precipitation and groundwater flux from upgradient, and the only discharges being through flux boundaries downgradient, and, during simulation, by pumping stresses. The slurry wall and underlying clay were taken as impermeable, no-flow boundaries. Under the expected conditions of the pumping tests, surface flow in the diversion ditch was not interpreted to have any significant effects, and therefore, was not included in the model. The model area, grid, and boundary conditions are shown on Figure 4-2.

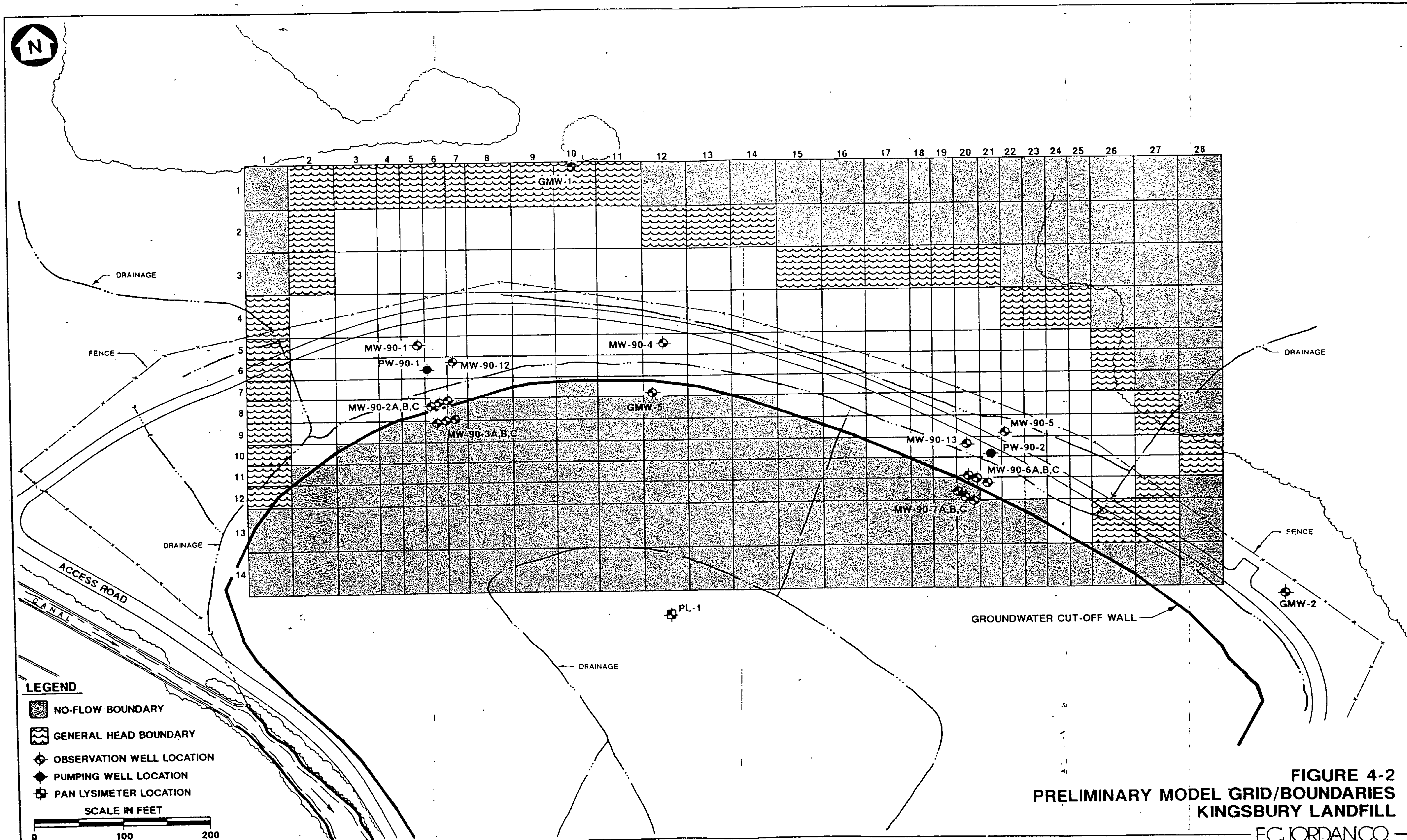

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FIGURE 4-2
PRELIMINARY MODEL GRID/BOUNDARIES
KINGSBURY LANDFILL

E.C. JORDAN CO.

The grid configuration for the preliminary groundwater flow model consists of 1 layer, 14 rows, and 28 columns. A total of 392 nodes were used to simulate groundwater flow conditions over an area of 550,000 ft^2, or approximately 12.6 acres. Of these, only 256 were active nodes, with the remainder representing areas not contributing water to the pump test area. The grid is block centered, which meaning that the groundwater head value calculated for each block is calculated at a point (node) located in the center of the block. Grid blocks vary in size ranging from 50 feet by 50 feet to 25 feet by 25 feet. Boundary conditions incorporated in the model include no-flow and general head (flux type) boundaries. The no-flow boundaries were used to represent the slurry wall and groundwater stream lines at the edges of the model, and the general head boundaries were used to allow groundwater flux into or out of the modeled area. The flux was estimated based on available data for site hydraulic conductivity and gradient, and the average thickness of the aquifer. The model general head boundary simulates a flux that is proportional to the hydraulic conductivity, the cross sectional area, and the difference between the calculated head at the model boundary and an assumed constant head at some point exterior to the model.

An average top of clay elevation of 180 feet was input for the aquifer bottom elevation. Recharge to the system was applied through a uniform recharge rate of 9 inches per year (.00205 ft/day). While this uniform recharge is important to the equilibrium calibration, this small amount of water over the short duration of the pumping test makes the model insensitive to reasonable variations in the recharge rate for purposes of determining test conditions.

4.6.4 Calibration

A steady-state flow model was developed using the data collected during the 1990 field efforts, as well as historical data. The hydraulic conductivities (K_h) used in model calibration were those obtained from the in situ hydraulic conductivity tests. A value of $2.2x10^{-2}$ cm/sec was input for the PW-90-1 area, and a value of $1.1x10^{-3}$ cm/sec was input for the PW-90-2 area. These values represent the geometric mean of the calculated K_h values. The preliminary groundwater flow model was calibrated to the groundwater elevations measured in monitoring wells throughout the modeled area. When the match was considered adequate, the calibrated steady-state model was used to perform transient simulations to evaluate various pumping test scenarios.

4.6.5 Aquifer Pumping Test Scenarios

The calibrated steady-state model was used as a basis for the transient model to simulate concurrent aquifer pumping tests at PW-90-1 and PW-90-2. The initial heads for the transient runs were those calculated by the steady-state model. The simulated pumping test consisted of three stress periods. The first stress period simulated PW-90-1 pumping for one day, the second simulated PW-90-1 and PW-90-2 pumping for two days, and the third simulated PW-90-2 pumping for one day. The transient simulations were performed with selected pumping rates at the two production wells. Arbitrary pumping rates of 30 gpm and 60 gpm were selected for PW-90-1, while pumping rates of 5 gpm and 15 gpm were chosen for PW-90-2. The higher pumping rates for PW-90-1 reflect the consistently higher hydraulic conductivity values obtained for the PW-90-1 area compared to the PW-90-2 area, based on the in situ hydraulic conductivity tests. Hydraulic conductivity values were also varied in the pumping test simulations. Log mean K values were initially used with the maximum pumping rates. Simulations were also run using the lowest and highest hydraulic conductivities at the maximum withdrawal rates. The minimum and maximum hydraulic conductivity values calculated for the PW-90-1 area were 1.1×10^{-2} cm/sec and 4.4×10^{-2} cm/sec, respectively. The minimum and maximum hydraulic conductivity values calculated for the PW-90-2 area were 1.2×10^{-4} cm/sec and 6.6×10^{-3} cm/sec, respectively. An effective porosity of 0.25 was used throughout the transient simulations. Effects of vertical anisotropy were not considered in the model.

4.6.6 Results

The simulation utilizing the maximum pumping rates and log mean hydraulic conductivities indicated no probably interference between the two pumping tests. However, when the maximum hydraulic conductivities and pumping rates were input into the model, an area of inferred overlap of approximately 0.5 feet of drawdown was observed between MW-90-4 and PW-90-2.

The simulation drawdown overlap of 0.5 feet between MW-90-4 and PW-90-2 indicated that some interference between the two pumping tests could result if they were performed concurrently under actual conditions of maximum estimated hydraulic conductivity values and selected pumping rates. Although actual aquifer K values are typically higher than those determined from single point hydraulic conductivity tests, no significant interference in interpreting the results of each pump test was anticipated between the tests even at the higher rates. It was also recognized that variation in the clay surface or aquifer thickness could impact the amount of drawdown observed during the pumping tests; however, a detailed knowledge of the variability of the top of clay in the vicinity of the pumping tests was not available and could not be considered. At the request of the NYSDEC, the two pumping tests were performed successively rather than concurrently to avoid possible interference.

Based on the results of the simulated pumping test scenarios, the following pumping ranges were established for use in the step drawdown tests:

- PW-90-1; 15, 30, 45, 60 gpm
- PW-90-2; 2, 4, 6, 8 gpm

4.7 AQUIFER PUMPING TEST

Pumping tests were conducted at two test well arrays (PW-90-1 and PW-90-2) for the purpose of determining the values of transmissivity (T), and storativity (S); K_h values were calculated by dividing T values by the aquifer thickness at each site. In addition, the tests were conducted to evaluate the degree of hydraulic connection, if any, across the groundwater cut-off wall or clay floor of the landfill.

Submersible pumps were installed in wells PW-90-1 and PW-90-2, and drop pipes were used to facilitate accurate water level measurements in the pumping wells. Purge water from well PW-90-1 was discharged to the landfill drainage ditch at a point approximately 200 feet southwest of the well and eventually drained into the Feedertow Canal. Water from well PW-90-2 was discharged to a drainage swale at a point approximately 200 feet northeast of the well. The swale drained to the east-southeast into a wooded area. Discharge water quality is discussed in Section 5.2.4.

The discharge rate from each pumping well was regulated using a gate valve. The discharge rate was measured using an in-line rate and totalizing flow meter. The totalizing meter, which had an analog display, and the flow meter, which had a digital display, were used to measure the cumulative flow rate and the instantaneous flow rate, respectively.

The PW-90-1 test array, shown on Figure 4-2, consisted of wells PW-90-1, MW-90-1, 2A, 2B, 2C, 3A, 3B, 3C and 12. The PW-90-2 test array was comprised of wells PW-90-2, MW-90-5, 6A, 6B, 6C, 7A, 7B, 7C and 13 (Figure 4-2). With the exception of MW-90-7A, the water levels in each of these wells were monitored throughout the testing periods with pressure transducers; the data was recorded digitally by an electronic data logger. Water level measurements in MW-90-7A were collected manually by Jordan using an electronic water-level indicator. Additionally, MW-90-4 and GMW-5 were also monitored throughout drawdown and recovery tests with pressure transducers and a separate data logger.

4.7.1 Trend and Barometric Data Adjustments

On April 10, 1991, five days before the start of the pumping tests, Jordan installed pressure transducers and data loggers at three wells to monitor background water-level conditions. Pressure transducers were installed in wells MW-90-14, 2C and 3C to measure the changing water levels inside and outside the containment system resulting from variations in barometric pressure. A barometric pressure probe was also connected to a data logger at well MW-90-14. Background monitoring at well MW-90-14, which was not influenced by pumping at either test array, continued at 15 minute intervals throughout the duration of the pumping and recovery tests.

Water-level measurements collected prior to pumping at well MW-90-2C, MW-90-3C and MW-90-14 were analyzed for barometric efficiency (BE). The barometric pressure data collected over the same period was converted into units of hydraulic head. The change in head (i.e., expressed as feet of water), assumed to result from the change in barometric pressure, was compared to the recorded water-level fluctuation in each background well over the same period. The BE of the well was determined by dividing the change in hydraulic head by the converted barometric hydraulic head change during the same time interval.

4.7.2 Step-Discharge Tests

Variable rate (step) tests were conducted at both test well arrays (PW-90-1 & PW-90-2) prior to conducting the constant rate tests. The tests were conducted to determine the optimum pumping rate for the constant rate tests. Step test rates were selected based on the analyses of slug-test data collected from the existing wells, as well as preliminary numerical computer modelling results.

On April 16, 1991, step tests were conducted at well PW-90-2. The well was pumped at rates of 1.71 and 1.96 gallons per minute (gpm) for approximately 120 minutes. A third test was performed at a rate of 4.70 gpm for 64 minutes, however because of excessive drawdown in the pumping well, the test had to be terminated. Step testing of well PW-90-1 commenced one day later on April 17. Three tests were performed at rates of 19.3, 47.2, and 74.4 gpm. Each test lasted for approximately 120 minutes.

4.7.3 Constant-Discharge Tests

Prior to the start of both constant rate tests, the gate valves on the discharge line of the pumping wells were adjusted to the maximum feasible flow rate. Minimal adjustments to the gate valves were required periodically throughout the tests to maintain a relatively consistent flow rate (less than 5% variation). In addition to the digitally recorded water levels, periodic manual measurements of the wells in the test array were made by Jordan personnel throughout both tests.

The constant-rate test of well PW-90-2 was initiated on April 17. Because equilibrium drawdown was achieved earlier than anticipated, the duration of the test was limited to 3180 minutes (53 hours). Over this period, 7,980 gallons were removed from the aquifer, at an average discharge rate of 2.5 gpm. The test was stopped on April 20, and the recovery test was initiated.

One hour after the recovery test began at PW-90-2, the constant rate test at PW-90-1 was initiated. The test was conducted for 4320 minutes (72 hours) at an average discharge rate of 74.4 gpm. The totalizing meter indicated that 321,421 gallons of water were removed from the aquifer over the 72-hour testing period.

4.7.4 Groundwater Sampling and Analysis

Samples of the discharged water were collected on a daily basis by Jordan personnel during the course of both constant rate tests. The samples were collected to monitor possible contaminant migration induced by pumping. The samples were collected directly from the end of the discharge hose using clean sampling bottles. The samples were collected for metals (Fe, CN), semivolatile, and PCB analyses. The samples that were to be analyzed for metals were preserved with nitric acid (pH <2). All of the samples were stored and shipped in an ice-packed cooler. The samples were shipped via overnight delivery to the ABB Environmental Analytical Laboratory for analysis.

4.7.5 Data Reduction and Analysis

All of the water level and barometric data that was collected on the data loggers was downloaded onto a computer while onsite. The information transfer was completed using Hermit DM, a communications package designed by In-Situ, Inc. for use with In-Situ's hermit data loggers. A periodic check of the transfer process was conducted by manually downloading a portion of the data stored on the data logger and comparing it to the output of the electronic transfer. The data were organized into approximately 138 data files during the downloading process.

The data were analyzed using Theis-type analytical models. Most of the analyses were conducted using AQTESOLV (Geraghty & Miller, Inc., 1989) computer software, although some of the data required manual curve matching to complete the analysis. The analytical models that were selected accommodated both confined and unconfined conditions, and included recognition of such factors as delayed drainage, aquitard permeability, and boundary conditions.

5.0 RESULTS OF INVESTIGATION

Presented below are the results of the hydrogeologic field investigation conducted at the Kingsbury Landfill site by E.C. Jordan Co. from October, 1990 to April, 1991.

5.1 SITE SPECIFIC GEOLOGY

The geologic mechanisms controlling the deposition of glacial sediments within the vicinity of the Kingsbury Landfill have been described previously by O'Brien & Gere (1983) and Cushman (1953). The grain size, color, texture and stratification of soil samples that were collected by Jordan during the installation of the monitoring wells were consistent with the description and interpretation of soil samples that were collected during a previous investigation by O'Brien & Gere (1982). The descriptive logs of soil samples collected by Jordan are provided in Appendix A.

The geologic environment surrounding the landfill consists of glacial delta sediments comprised of a poorly sorted mixture of sand, silt and clay. Underlying the delta sediments are glacial lacustrine sediments consisting of alternating thin layers of clay and silty fine sand. Glacial till was encountered below the lacustrine sediments at the base of two borings (MW-90-2A and MW-90-3A) that are located in the northeast portion of the site.

Grain size (sieve) analyses were conducted on several soil samples of delta sediments that were collected in well borings PW-90-1, PW-90-2, MW-90-2C, MW-90-6C, MW-90-10C and MW-90-12. The analyses, which are included in Appendix B, show that the grain size is coarsest in the northwest portion of the site. The grain size analyses of soils collected from wells PW-90-1, MW-90-2C and MW-90-12 indicate a size distribution of 50 to 60 percent medium sand and 25 to 50 percent fine sand. The remaining portion of the samples consist of coarse sand or silt.

The grain size distribution that was measured in the samples collected from wells PW-90-2 and MW-90-6C indicates an overall fining in the sediment size. The distribution of grain sizes varied from 0 to 15 percent medium sand, 50 to 85 percent fine sand and 10 to 45 percent silt size material or smaller. Grain size analyses of soils from well MW-90-10C yielded similar results. The grain size distribution varied from 70 to 80 percent fine sand and 20 to 30 percent silt size particles or smaller.

The delta deposits identified at the Kingsbury Landfill resulted when the glacial Hudson River entered glacial Lake Albany. According to Cushman (1953), the grain size of the deltaic sediments usually becomes coarser in the direction of the delta head. The grain size distribution observed at the Kingsbury Landfill is consistent with the findings of Cushman. The coarsest sediments are located in the northwest portion of the site, which is the direction of the delta head and Hudson River. Cushman also reported that along the outer margin of the delta, the deposits may gradually grade into or interfinger with lacustrine sediments--this condition has also been identified at the site.

5.2 SITE SPECIFIC HYDROGEOLOGY

The results presented in this section identify the groundwater flow conditions and hydraulic parameters at the Kingsbury Landfill site.

5.2.1 Groundwater Flow Direction

Analyses of water level measurements were conducted to evaluate the horizontal and vertical movement of groundwater under natural conditions. The results of these analyses are provided below.

5.2.1.1 Horizontal Flow Under Static Conditions. A potentiometric surface map was constructed using the elevation of the water table that was measured in sand wells inside and outside the slurry wall. Water-level measurements collected on April 26, 1991 were used to construct the map shown on Figure 5-1.

The map indicates that groundwater outside of the slurry wall is flowing in a southeast direction. The hydraulic gradient along the north and south side of the landfill varies significantly. The gradual increase in hydraulic gradient from west to east along the north side of the landfill is a manifestation of the decrease in aquifer hydraulic conductivity that was measured during aquifer stress tests in that area. Although the hydraulic conductivity of the aquifer along the south side of the landfill is also low, the subdued hydraulic gradient reflects the release of hydrostatic pressure resulting when the groundwater discharges into the Feedertow canal.

The potentiometric surface inside the slurry wall system indicates that groundwater is also flowing in a southeast direction. The permeability of the sand aquifer within the containment system is similar to that of the surrounding system; therefore, the small hydraulic gradient inside the containment system indicates that the rate of flow is much slower than groundwater outside of the containment system.

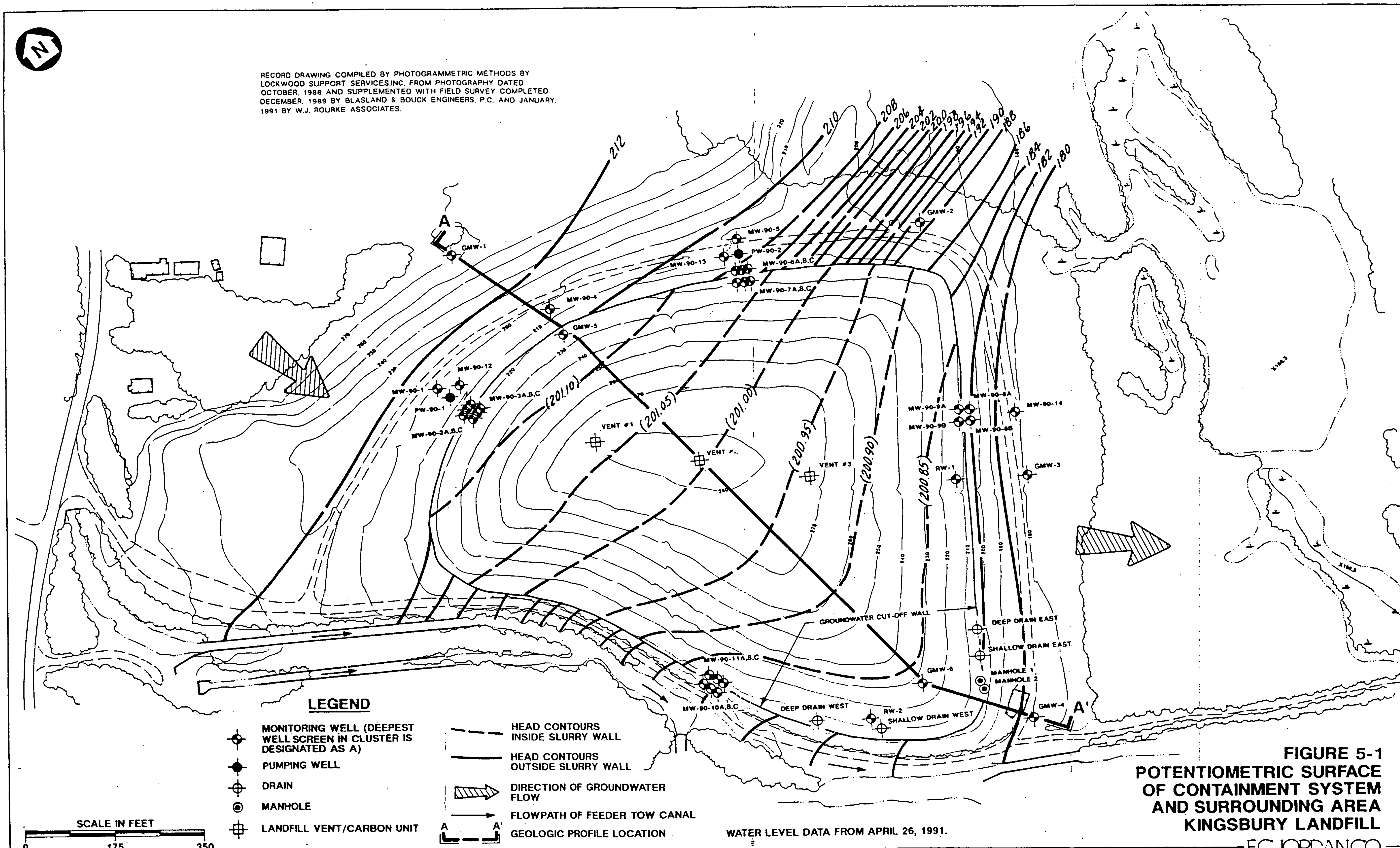

FIGURE 5-1
POTENTIOMETRIC SURFACE OF CONTAINMENT SYSTEM AND SURROUNDING AREA KINGSBURY LANDFILL

ECJORDANCO.

Comparing the difference in hydraulic head in wells inside and outside of the containment system illustrates the relative effectiveness of the slurry wall as a hydraulic barrier at the points of measurement (Figure 5-2). A hydraulic head difference of approximately nine-feet exists between wells MW-90-2C and MW-90-3C on the upgradient side. The hydraulic head is greater in well MW-90-2C indicating that the potential for flow is into the containment system. In contrast, the hydraulic head on the downgradient side of the landfill is approximately twenty-feet greater in well GMW-6 compared to well GMW-4, indicating a potential for flow out of the containment system.

5.2.1.2 Vertical Flow Under Static Conditions. Analysis of vertical head relationships was performed on nested well clusters located on both sides of the slurry wall. Comparisons of head elevations from twelve rounds of measurements, made from March to May 1991, show consistent head differentials between the shallow aquifer wells and the deeper aquitard wells.

On the upgradient (northwest) side of the landfill, hydraulic head on the outside of the slurry wall declines with depth in wells MW-90-2C, MW-90-2B and MW-90-2A; the hydraulic head in wells MW-90-3C, MW-90-3B and MW-90-3A--the corresponding nested well cluster located on the inside of the slurry wall--increases with depth. The relationship between the vertical hydraulic head differentials in the sand and aquitard wells on both sides of the slurry wall, combined with a driving force resulting from a head differential of 9 feet across the slurry wall, suggests that groundwater is flowing under or through the slurry wall at a point below the deeper aquitard wells along the northeast portion of the landfill.

On the north side of the landfill, the vertical hydraulic head differentials observed at the MW-90-6 and MW-90-7 nested well clusters indicate that the potential for groundwater flowing into the landfill is significantly reduced. The elevation of the heads in wells MW-90-6C, MW-90-6B and MW-90-6A indicate that groundwater from deeper in the aquitard, as well as groundwater within the shallow aquifer, is flowing toward a permeable zone located in the upper portion of the aquitard in which well MW-90-6B is installed. Inside the slurry wall a consistent decrease in head with depth is observed in wells MW-90-7C, MW-90-7B and MW-90-7A. Although there is approximately 6 feet of head difference across the slurry wall in the sand wells, hydraulic communication across or under the wall at this location is not evident from the vertical head relationships.

The vertical head relationships in downgradient nested well clusters MW-90-8A/MW-90-8B and MW-90-9A/MW-90-9B, in addition to wells MW-90-10A/MW-90-10B/

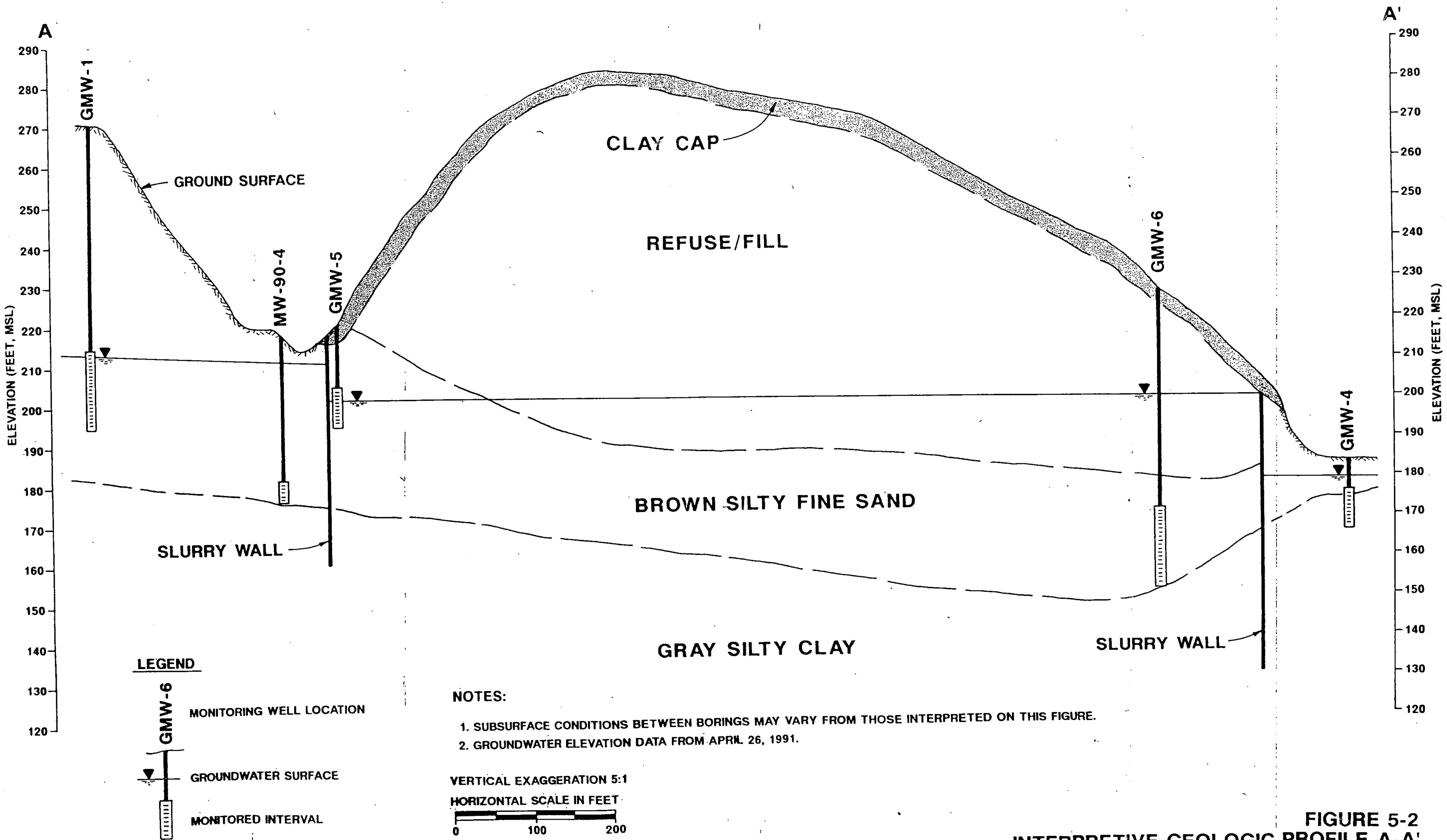

FIGURE 5-2
INTERPRETIVE GEOLOGIC PROFILE A-A'
KINGSBURY LANDFILL

ECJORDANCO.

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MW-90-10C and MW-90-11A/MW-90-11B/MW-90-11C indicate that groundwater is discharging beneath or through the slurry wall. The nested well clusters located inside the slurry wall show that head is decreasing with depth, which indicates a potential for downward flow. Conversely, nested wells on the outside of the slurry wall show that head increases with depth indicating upward flow. A head difference of approximately 20 feet across the slurry walls along the down gradient side of the landfill is the driving force for potential leakage from the containment system.

5.2.2 Hydraulic Conductivity Testing Results

The data collected during the hydraulic conductivity (K_h) testing of the wells installed in the aquifer (delta deposits) and aquitard (lacustrine/till deposits) indicate unconfined to semiconfined conditions across the site. Therefore, the Bouwer and Rice (1976) model was selected for determining K_h from the rising head test data. The modeled data plots can be found in Appendix D.

The results presented on Table 5-1 are organized according to well location and stratigraphy. The wells were divided into four groups based on their proximity to each other. The average hydraulic conductivity was determined for the aquifer and aquitard for each group of wells.

Table 5-1 shows that the average hydraulic conductivity decreases in a downgradient direction from northwest to southeast across the site within the aquifer and aquitard. These results are consistent with the conceptual geologic model of the site presented previously in Section 5.1. The average hydraulic conductivity of the aquifer wells range from 3.2×10^{-3} centimeters per second (cm/sec) to 2.9×10^{-2} cm/sec. Aquitard wells yielded significantly lower values of hydraulic conductivity; values range from 7.3×10^{-5} cm/sec to 3.8×10^{-7} cm/sec.

5.2.3 Groundwater Quality During Pumping Test

The parameter levels remained relatively constant throughout the pumping test. None of the parameters that were analyzed for exceeded the concentration limit set by the NYSDEC for discharge to the Feedertow canal. The groundwater sampling results are included in Appendix B.

5.2.4 Step-Discharge Test Results

The data were analyzed using the Birsoy and Summers (1980) method. Data plots and calculations for wells PW-90-1 and PW-90-2 can be found in Appendix E. The

TABLE 5-1
IN SITU HYDRAULIC CONDUCTIVITY TESTS
KINGSBURY LANDFILL

WELL NO.	HYDRAULIC CONDUCTIVITY SAND WELLS (cm/sec)	HYDRAULIC CONDUCTIVITY CLAY WELLS (cm/sec)
PW-90-1	4.4×10^{-2}	
MW-90-1	1.1×10^{-2}	
MW-90-2A		3.9×10^{-6}
MW-90-2B		5.6×10^{-5}
MW-90-2C	2.6×10^{-2}	
MW-90-3A		2.0×10^{-4}
MW-90-3B		7.9×10^{-7}
MW-90-3C	2.1×10^{-2}	
MW-90-4	4.4×10^{-4}	
PW-90-2	4.6×10^{-4}	
MW-90-5	1.9×10^{-3}	
MW-90-13	6.6×10^{-3}	
MW-90-6A		4.8×10^{-7}
MW-90-6B		8.4×10^{-6}
MW-90-6C	5.5×10^{-3}	
MW-90-7A		4.1×10^{-7}
MW-90-7B		1.3×10^{-5}
MW-90-7C	1.2×10^{-4}	
MW-90-14	7.1×10^{-5}	
MW-90-8A		1.8×10^{-5}
MW-90-8B		5.0×10^{-7}
MW-90-9A		1.8×10^{-7}
MW-90-10A		3.4×10^{-7}
MW-90-10B		5.4×10^{-7}
MW-90-10C	1.6×10^{-3}	
MW-90-11A		2.0×10^{-7}
MW-90-11B		4.3×10^{-7}
MW-90-11C	4.0×10^{-3}	

results indicated that the maximum feasible pumping rate for wells PW-90-1 and 2 over a 72-hour period were approximately 75 and 2.5 gpm, respectively.

5.2.5 Barometric Efficiency Adjustments

Figures 5-3 through 5-5 illustrate how varying the percent of BE correction affects the plot of the background water levels in wells MW-90-14 and MW-90-3C. These figures indicate that the BE of wells MW-90-14 and MW-90-2C is equal to or less than 10 percent, and the BE of well MW-90-3C is between 10 and 25 percent.

The landfill cap is believed to provide a relatively effective hydraulic seal. However, the cap is vented; the response to barometric pressure fluctuations may vary from well to well depending on their proximity to the vents. A BE of 25 percent was used to adjust the water levels in wells inside the containment system. This value reflects the efficiency expected for a leaky (semiconfined) aquifer system. Prior to the analyses, the raw water-level data collected outside the containment system during the pumping tests were adjusted to reflect a BE of 10 percent.

5.2.6 Constant-Discharge Test Results

Pumping tests, conducted in wells PW-90-1 and PW-90-2 at the Kingsbury Landfill, were analyzed to derive values of transmissivity (T) and storativity (S), and to determine the degree of hydraulic connection, if any, across the groundwater cut-off wall or clay floor of the landfill. Vertical leakage occurred during the second test, and data plot analysis permitted calculation of vertical hydraulic conductivity of the aquitard (K_v').

The analyses presented below represents the best fit of the data to the most appropriate radial flow models. Plots of the data showing time versus drawdown and/or relative head for each well in the two pumping test arrays can be found in Appendix E.

5.2.6.1 PW-90-1 Pumping Test. The analysis of data collected during the pumping test at well PW-90-1 was complicated by a heavy continuous rain that occurred during the first day of pumping. The rain significantly affected the water levels in the wells, and corrections for recharge caused by the rain were not possible to determine. Even though the drawdown data beyond 720 minutes of pumping were disregarded upon evaluating drawdown plots, the early data were sufficient for a reliable calculation of T and S. The rainfall runoff concentrated in the drainage ditch, which is located in proximity to the pumping well. Therefore, the water levels in the

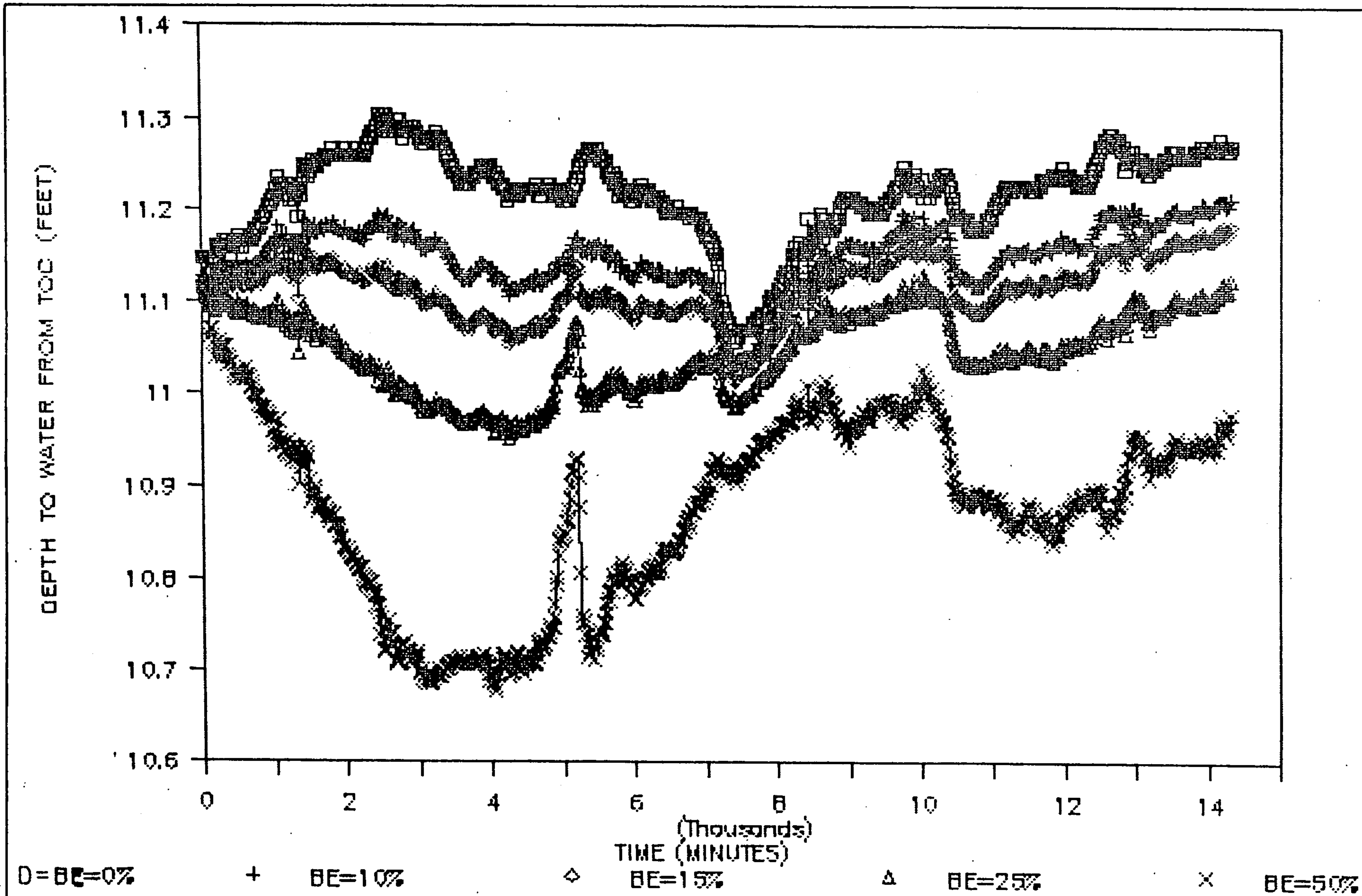

FIGURE 5-3
WELL MW-90-14:
CORRECTIONS FOR BAROMETRIC EFFICIENCY
KINGSBURY LANDFILL

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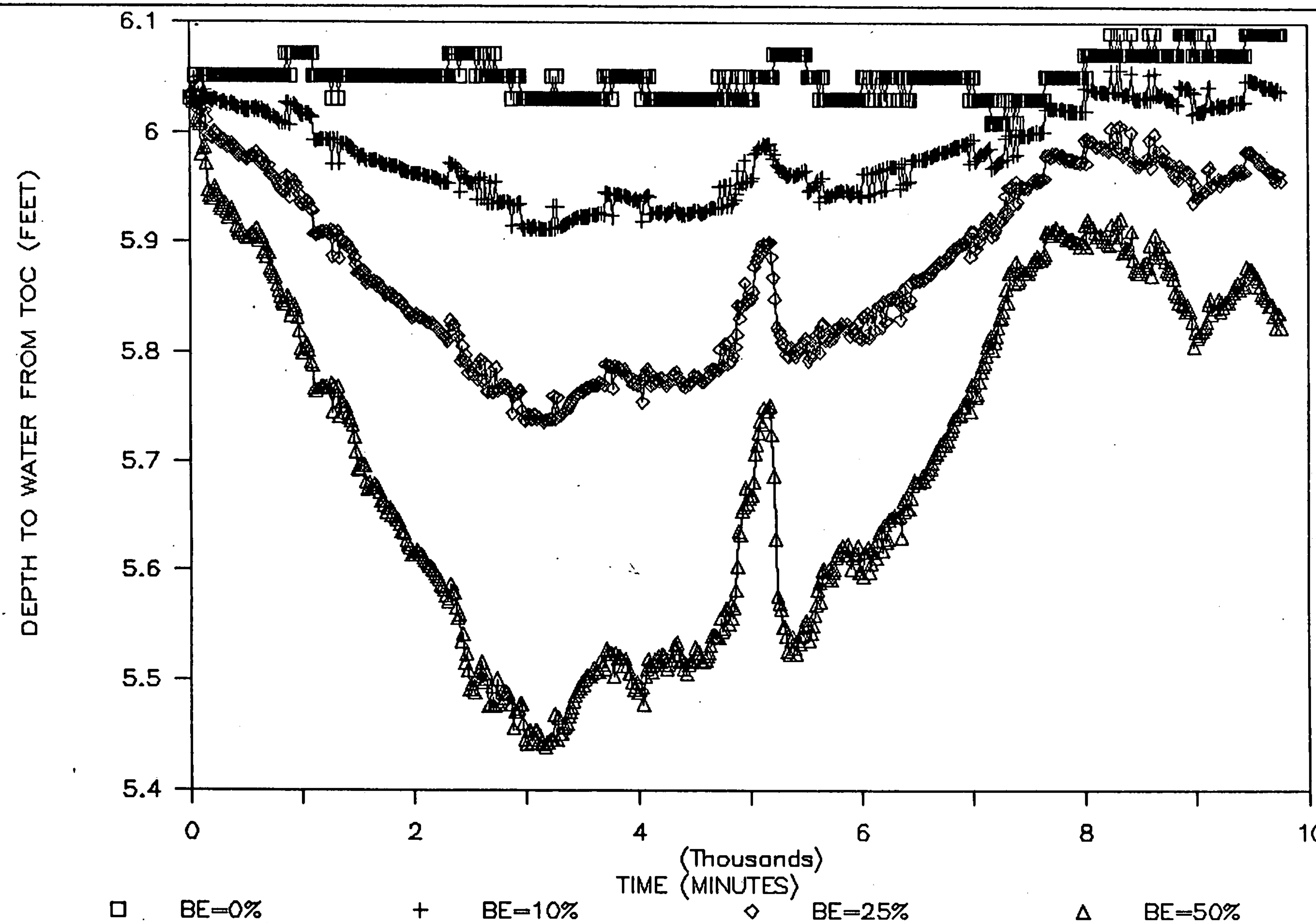

FIGURE 5-4
WELL MW-90-2C:
CORRECTIONS FOR BAROMETRIC EFFICIENCY
KINGSBURY LANDFILL

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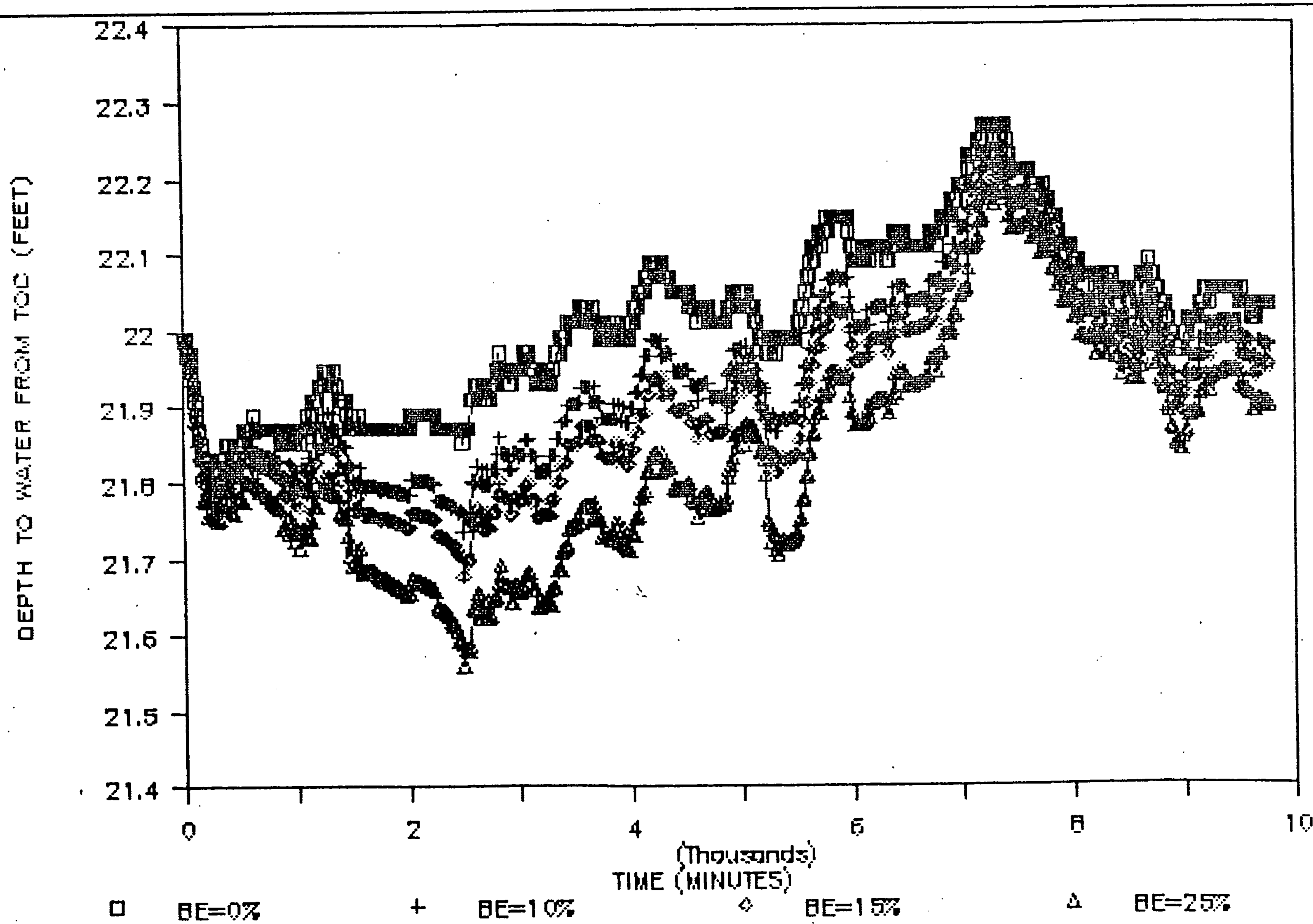

FIGURE 5-5
WELL MW-90-3C:
CORRECTIONS FOR BAROMETRIC EFFICIENCY
KINGSBURY LANDFILL

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background well (MW-90-14) could not be used to quantify the effects of surface water infiltration through the drainage ditch.

The duration of the test was 4320 minutes at a sustained rate of approximately 74 gpm. As a result of the increased discharge rate and much higher aquifer transmissivity as compared to the PW-90-2 test, barrier boundary effects were a dominant feature in the data plots of the PW-90-1 test array. However, the rate of water removal from the aquifer was sufficiently large that the nearby landfill drainage ditch was dewatered several hundred feet in both directions. The drainage ditch remained dewatered even throughout the heavy rain. Rapid recharge from the rainfall stopped the increase in drawdown for approximately one half day, but this delay in attaining the target drawdown at the outside of the wall is not considered critical.

Well MW-90-1

As shown on Figure 5-6, the Theis curve matches the observed data for the first 30 minutes, after which time a barrier boundary is evident. The Theis solution yielded $T = 1.03 \times 10^4$ ft^2/day and $S = 1.0 \times 10^{-3}$. The distance to the barrier boundary was evaluated using Stallman type curves. The theoretical image well was calculated to be 448 feet from the pumping well; therefore, an impermeable type barrier may be located at a distance of approximately 223 feet. This value obviously does not agree with the location of the slurry wall. Two plausible explanations exist: (1) recharge from the ditch has distorted the barrier boundary manifestation on the drawdown plot, thereby precluding an accurate determination of the distance to the boundary, and/or (2) the boundary is geologic in nature, that is, a significant reduction in transmissivity was sensed by the spreading drawdown cone. The latter hypothesis implies that the recharge from the stream completely masked the effects of the slurry wall.

Well MW-90-12

The data follows the Theis curve for approximately 4 minutes. Negative type deviations (i.e., increasing rates of drawdown) occur at approximately 5 and 288 minutes. These deviations, illustrated on Figure 5-7, are probably affected by the initial and final stages of ditch dewatering. The latter deviation is thought to represent the full boundary effect of the slurry wall. The Theis fit to the early data gave a $T = 1.79 \times 10^4$ ft^2/day and $S = 6.0 \times 10^{-4}$.

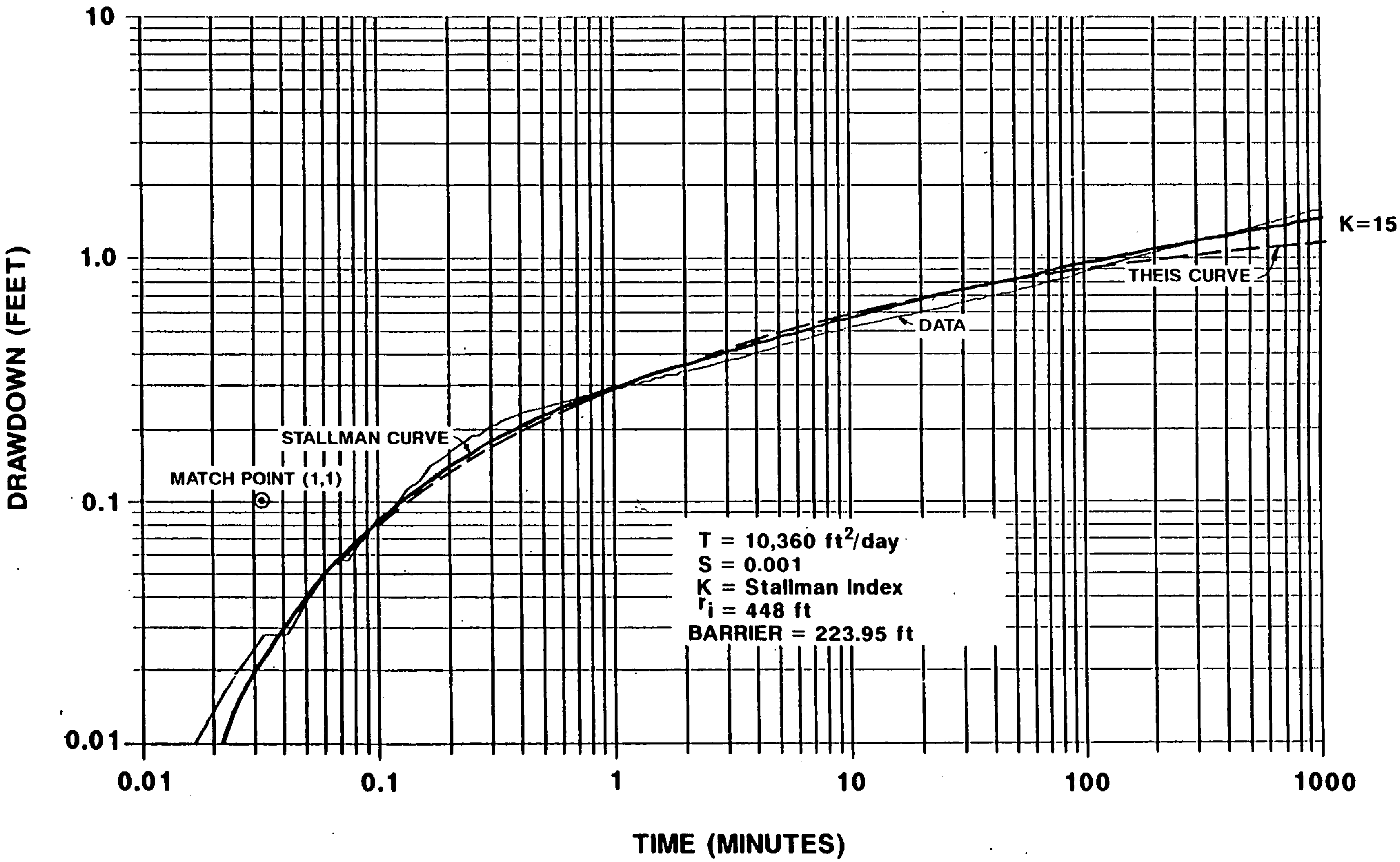

FIGURE 5-6
WELL MW-90-1:
BARRIER BOUNDARY ANALYSIS
KINGSBURY LANDFILL

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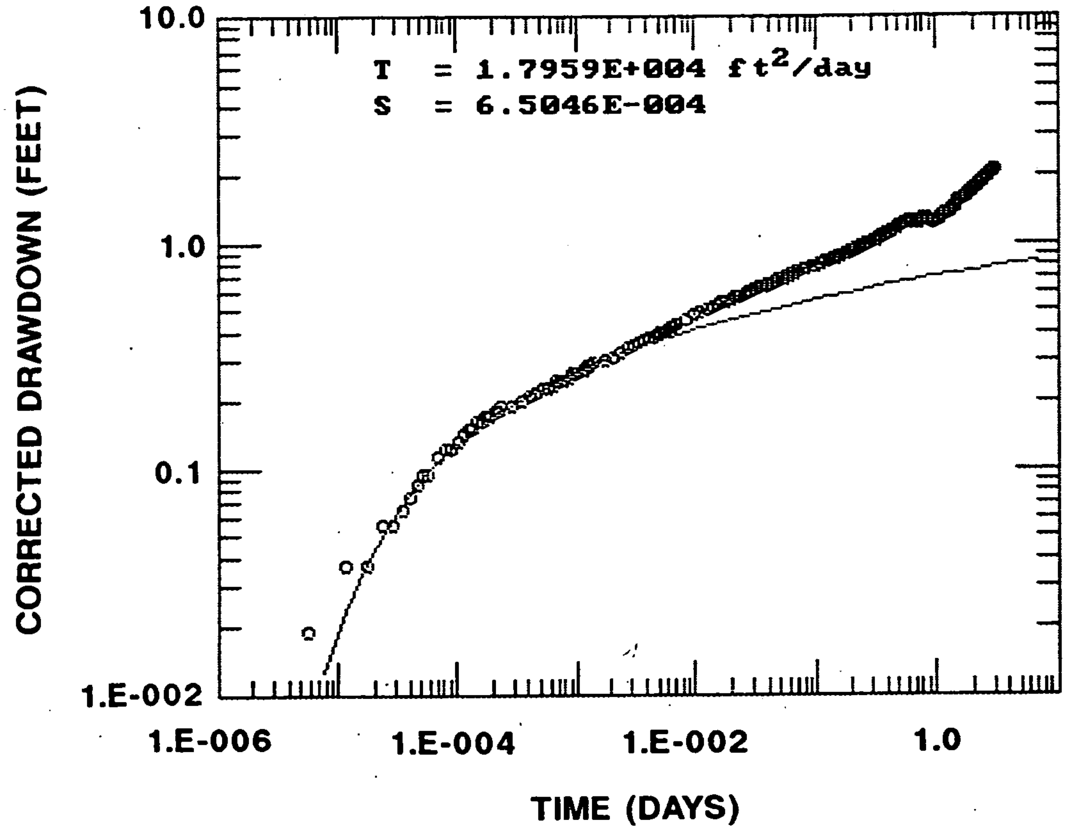

FIGURE 5-7
WELL MW-90-12:
THEIS PLOT AND ANALYSIS
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Well MW-90-2C

The Theis curve fits the data satisfactorily to 120 minutes, after which a barrier boundary effect is evident. Figure 5-8 illustrate the Theis solution to the early drawdown data produced a T = 1.08×10^4 ft^2/day and S = 9.0×10^{-4}.

Wells MW-90-2A and MW-90-2B

These wells, which were installed in typical aquitard material, responded to the pumping at well PW-90-1; approximately 1.8 feet of drawdown was observed in both wells. The response time however, was delayed compared to that observed in the aquifer well MW-90-2C. The response delay is a manifestation of the low hydraulic conductivity of the aquitard.

The shape of the time versus drawdown curve from well MW-90-2B was similar to that of the Theis type curve as indicated by Figure 5-9, however; the early data plotted to the left of the type curve, which is typical of a well that is partial penetrating an aquifer. The boring log and well construction diagram indicates that the well was installed five feet into silty clay. However, the observed drawdown indicates that the aquifer/aquitard contact is not hydraulically abrupt. The slug-test results from wells MW-90-2B and MW-90-2A indicate a decline in horizontal hydraulic conductivity with depth. It is more likely that the reduction in aquitard permeability is transitional.

The response observed in well MW-90-2A, as shown on Figure 5-10, is indicative of a leaky system. The early time-drawdown data (time = 14 to 720 minutes) plot to the right of the Theis-type curve indicating vertical leakage through the aquitard. The vertical hydraulic conductivity of the aquitard (K_v') was analyzed using the Neuman and Witherspoon (1972) method for determining aquifer parameters of leaky multiple aquifer systems. The analytical method uses the ratio of the drawdown in the aquitard to that measured in the aquifer at the same time and the same radial distance from the pumping well.

The ratio of the time-drawdown data collected from wells MW-90-2C and MW-90-2A was considered for the Neuman and with a spoon analysis. In order to complete the analysis, the aquitard thickness must be known or assumed. Because it appears that well MW-90-2B is installed in a transition zone with respect to aquifer permeability, the top of the effective aquitard is assumed to begin at the bottom of well MW-90-2B (i.e., 47 feet below ground surface).

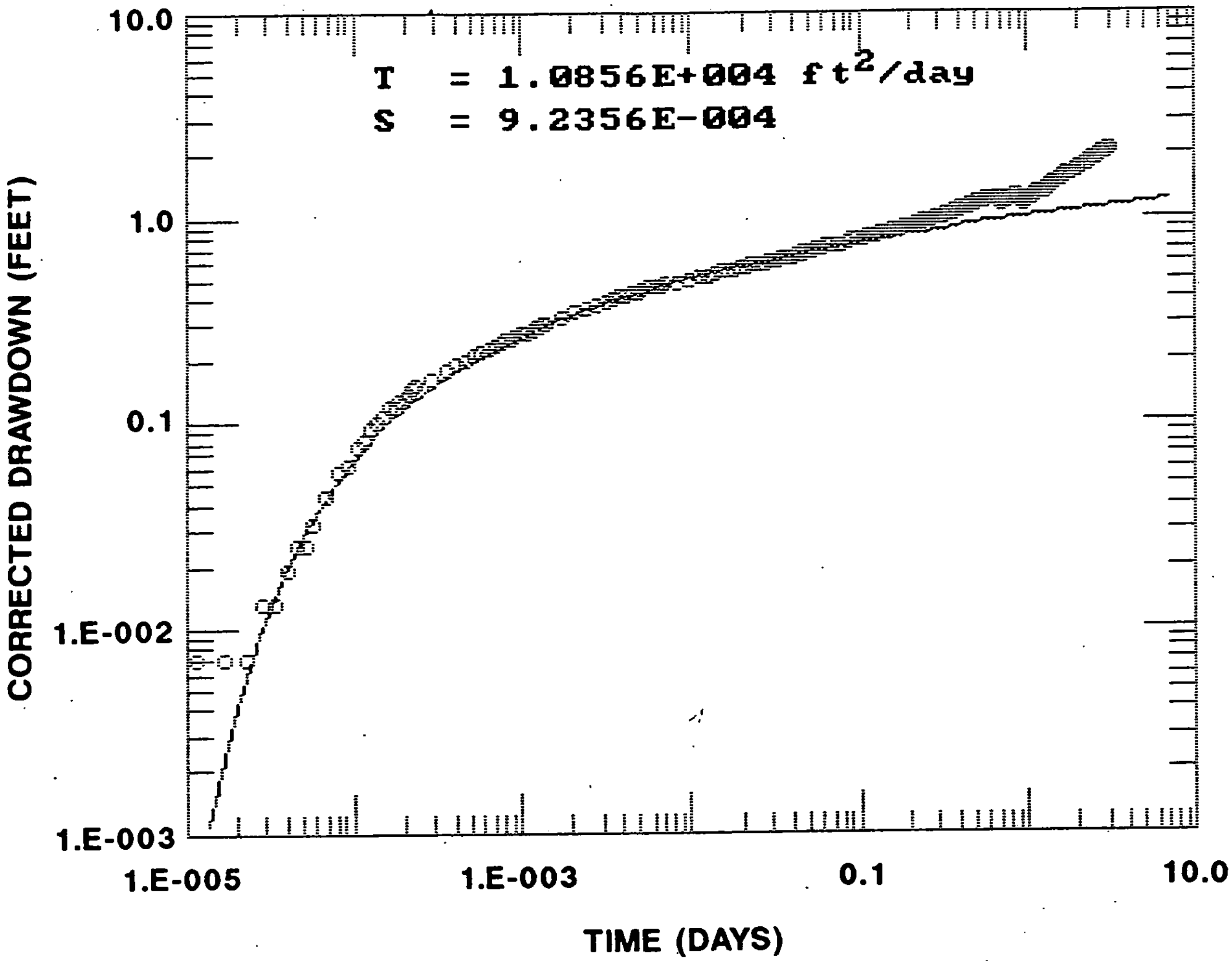

FIGURE 5-8
WELL MW-90-2C:
THEIS PLOT AND ANALYSIS
KINGSBURY LANDFILL

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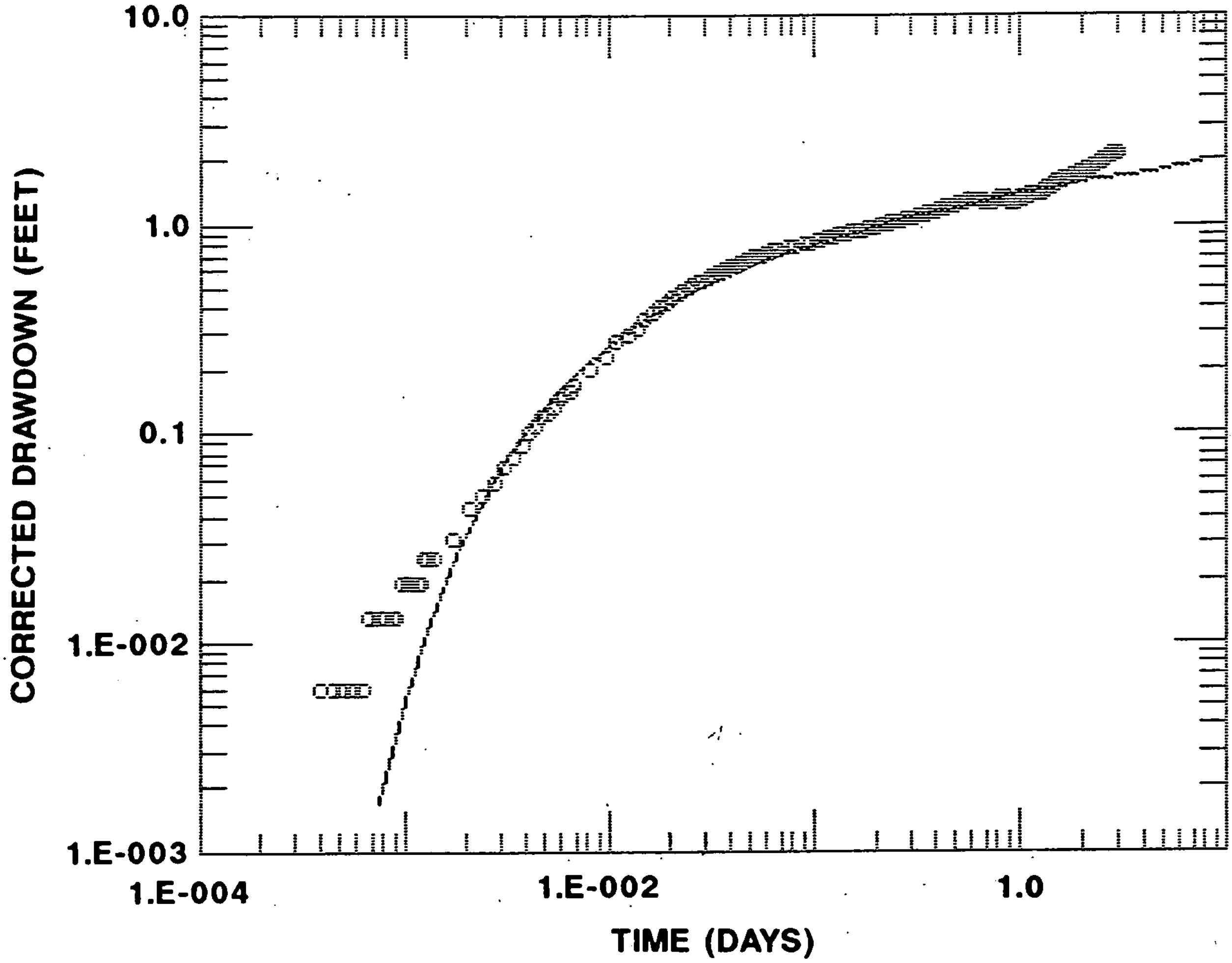

FIGURE 5-9
WELL MW-90-2B:
THEIS PLOT
KINGSBURY LANDFILL

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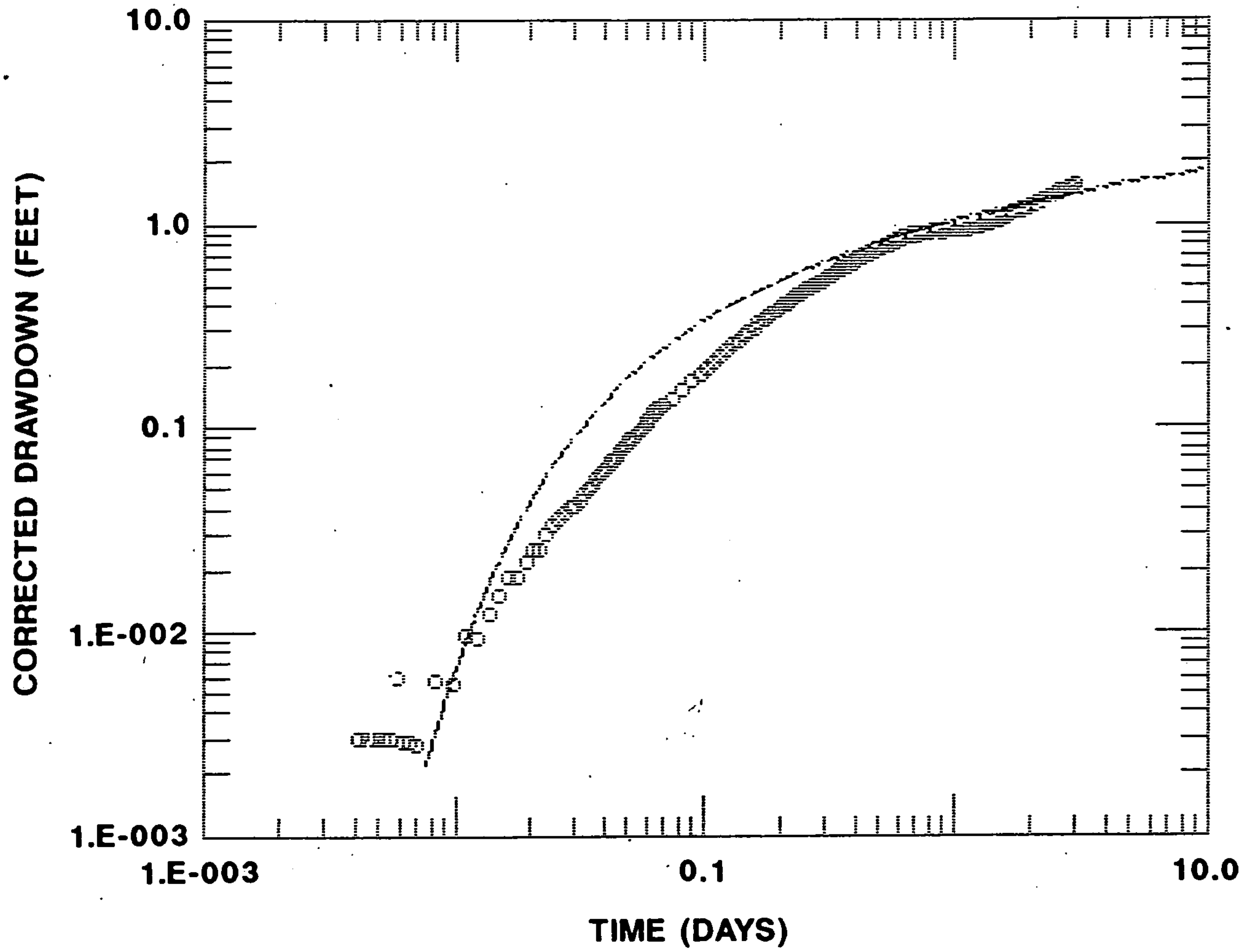

FIGURE 5-10
WELL MW-90-2A:
WELL MW-90-2A THEIS PLOT
KINGSBURY LANDFILL

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The results of the analyses indicate that K_v' is approximately 4.0 x 10^{-4} ft/day (1.41 x 10^{-7} cm/sec). This value is within one order of magnitude of slug test results for well MW-90-2A. Although slug tests are designed to measure the horizontal permeability of the aquifer, Freeze and Cherry (1979) suggest that the ratio of horizontal to vertical hydraulic conductivity in nature is generally less than 3:1. Therefore, K_v' determined by the ratio method appears reasonable.

Well MW-90-4

Figure 5-11 shows approximately one foot of drawdown was observed at well MW-90-4, which is located approximately 272 feet northeast of well PW-90-1. Only the very early data (time < 288 minutes) could be matched to a Theis curve; therefore, the results should be considered a rough approximate of the true aquifer parameters. The Theis match of the early data yields T = 4258 ft^2/day and S = 0.027. The value of transmissivity for well MW-90-4 is approximately midway between the average values obtained from analysis of the two pumping tests. It is likely that the well is installed within the transition zone between geologic environments.

After approximately 290 minutes, the data indicates that a recharge boundary was encountered. It is likely that the positive deviation from the Theis curve is the result of vertical leakage of stream flow from the ditch. At approximately 580 minutes, the hydraulic effects of the slurry wall are apparent in the data plot. The rapidly declining water level, resulting from the spreading drawdown cone coming into contact with the slurry wall barrier boundary, is offset by the rain fall recharge at 720 minutes into the test.

The Cooper and Jacob (1946) straight-line method for analysis was also attempted. However, a key criterion for applying the analytical method was not met (i.e., u < 0.01); therefore, the results of the Cooper and Jacob analyses are not reliable, and are not given in this report.

PW-90-1 Test Summary

The early time-drawdown data (time <720 minutes) collected during the pumping test at well PW-90-1 revealed that the slurry wall acted as a barrier boundary as expected, resulting in observed drawdowns that were greater than anticipated by standard Theis analysis.

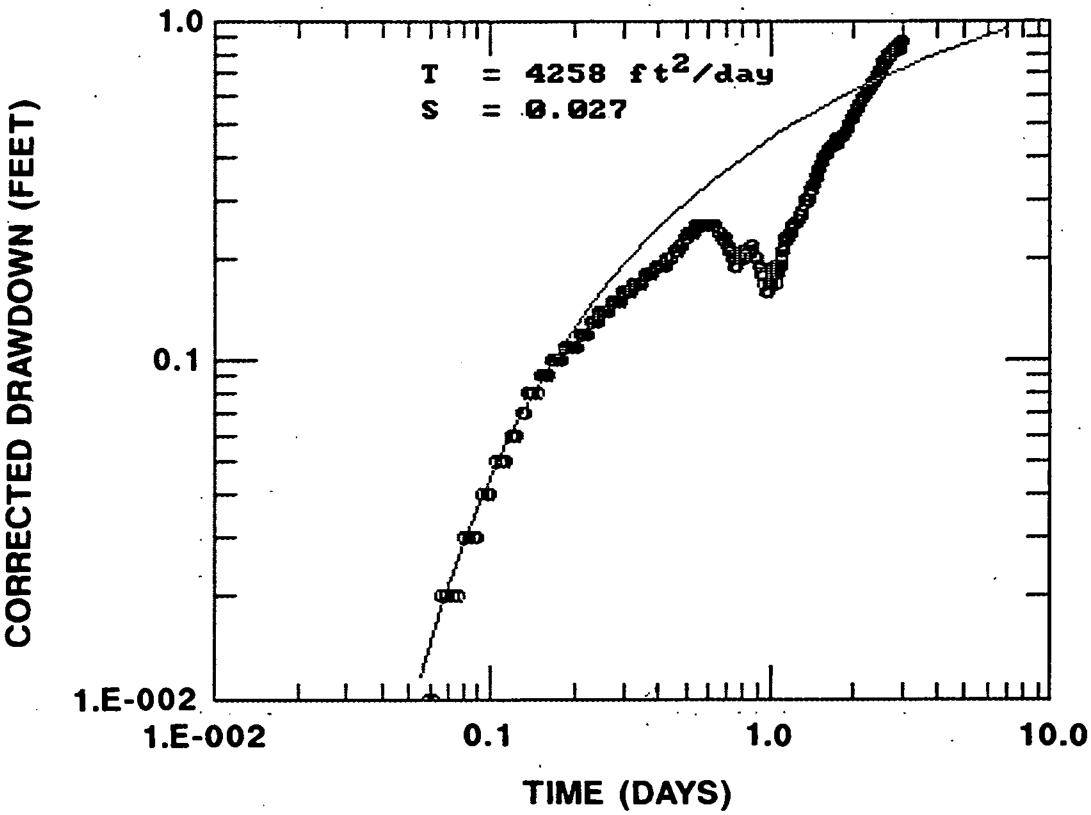

FIGURE 5-11
WELL MW-90-4:
THEIS PLOT AND ANALYSIS
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Aquifer transmissivity was found to be very similar between wells MW-90-1 and MW-90-2C, but transmissivity calculated at well MW-90-12 was greater by almost a factor of two. It is not unusual to observe variations of this order in pumping test data. It is suspected that the observed variation in aquifer transmissivity is most likely attributable to the natural heterogeneities in the aquifer matrix. The value of aquifer transmissivity calculated for well MW-90-4 may not be truly representative of the aquifer matrix. However, the relative value of transmissivity is consistent with the decline in transmissivity from west to east inferred from the analyses of data from the two pumping tests.

A summary and arithmetic mean of the PW-90-1 test array hydraulic parameter values, excluding MW-90-4, are provided below:

Well	T (ft^2/day)	S
MW-90-1	$1.03x10^4$	$1.0x10^{-3}$
MW-90-2C	$1.08x10^4$	$9.0x10^{-4}$
MW-90-12	$1.79x10^4$	$6.0x10^{-4}$
Arithmetic Mean	**$1.30x10^4$**	**$8.3x10^{-4}$**

5.2.6.2 PW-90-2 Pumping Test. The 3180 minute test was conducted at a continuous rate of approximately 2.5 gpm. For the most part, the recharge effects of the ditch flow either balanced or dominated the effects of the slurry wall for the PW-90-2 test array.

<u>Well MW-90-5</u>

The data were fit to the Walton (1962) leaky aquifer model that assumes storage in the aquitard is not significant, as illustrated on Figure 5-12. The analytical model considers an aquifer which is bounded on top by a confining bed of low permeability. The confining layer in turn, is bounded on top by an unconfined aquifer. The effects of storativity in the confining bed are considered negligible in the model.

The first 12 minutes of the data from well MW-90-5 fall to the left of the Theis curve. Deviations to the left of the type curve could result from stratification in the aquifer, partial penetration and/or barrier boundaries. Assuming it was the latter, the analysis was performed with data that was recorded after 12 minutes. Theis analysis yielded the following values: T = 91.6 ft^2/day, S = 2.6 x 10^{-3}, and K_v' of the overlying aquitard = 0.129 ft/day.

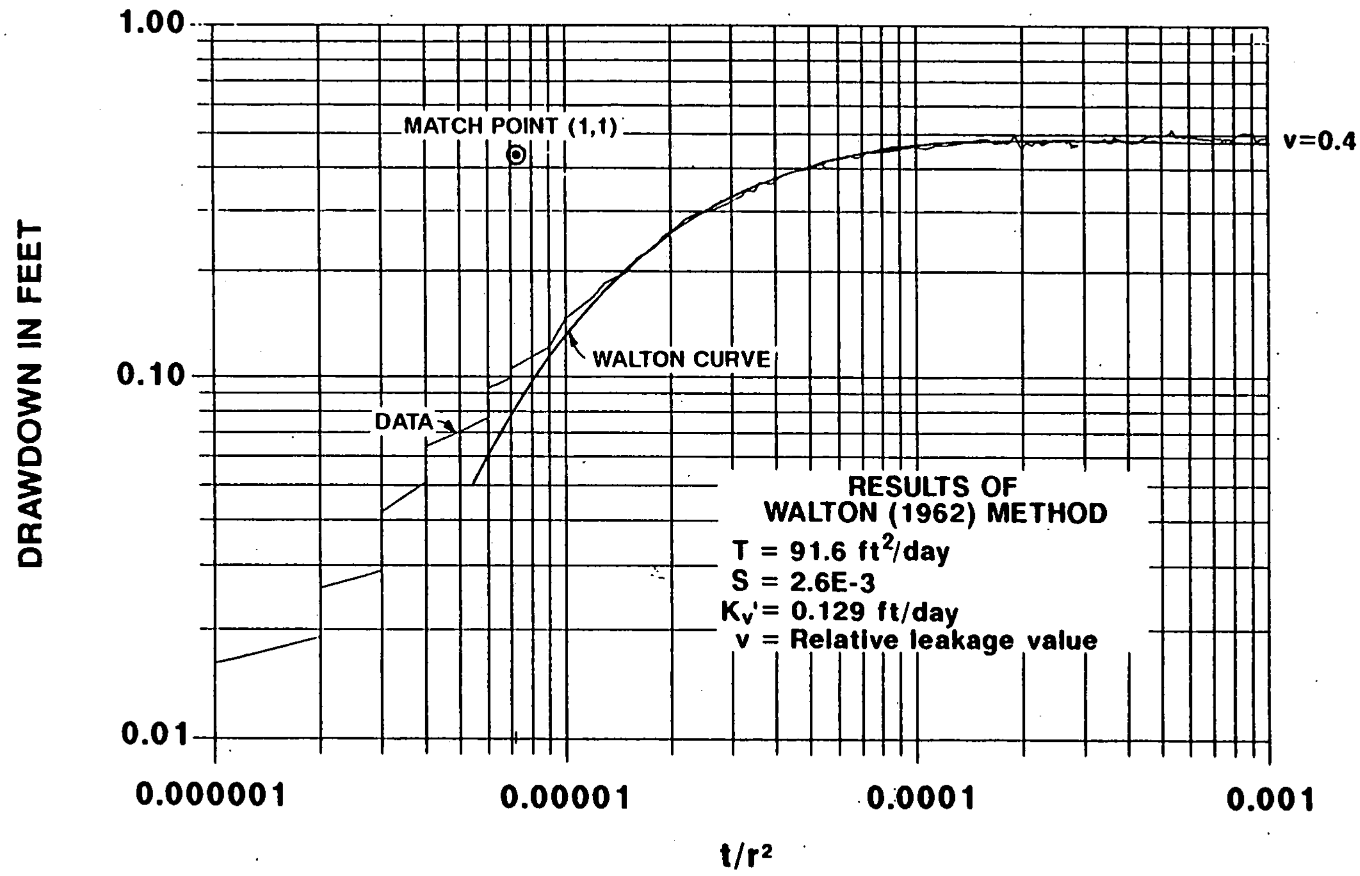

FIGURE 5-12
WELL MW-90-5:
LEAKY AQUIFER ANALYSIS
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Wells MW-90-6C and MW-90-13

Analyses of both data plots give practically the same values of T, S and K_v'. It should be noted that the early time data (< 12 min.) for well MW-90-13, displayed on Figure 5-13, plotted very similarly (to the left of the Theis curve) to the plot of well MW-90-5 data. However, well MW-90-13 is fully penetrating (whereas MW-90-5 is not), and the radial distance from the pumping well is virtually the same for both MW-90-5 and MW-90-13 wells. Therefore, the deviation of the early time data in both wells is probably the result of aquifer stratification. The analysis for wells MW-90-6C and MW-90-13 was conducted using the Walton (1962) leaky aquifer model (with no aquitard storage). The analysis of well MW-90-6C, shown on Figure 5-14, yielded T = 27.8 ft^2/day, S = 3.1 x 10^{-4}, and K_v' = 0.017 ft/day. Similarly, well MW-90-13 produced T = 31.1 ft^2/day, S = 4.3 x 10^{-4}, and K_v' = 0.025 ft/day.

Wells MW-90-6A and MW-90-6B

At the onset of the constant-rate test, both of these aquitard wells were still recovering from the very recent step-discharge tests. The recovery continued through the early portion of the constant rate test. Figure 5-15 shows that the water level in MW-90-6B continued to rise until approximately 60 minutes into the constant-rate test. The water level in deeper well MW-90-6A, shown on Figure 5-16, continued to rise until approximately 1800 minutes into the constant rate test. Because the water levels were not at static conditions at the start of the constant-rate test, the results of any data analyses would be unreliable, and therefore, were not conducted. However, qualitative analyses of the significantly longer observed lag in response time suggests that the aquitard in the vicinity of the PW-90-2 test array has a lower permeability compared to the aquitard in the vicinity of the PW-90-1 test array.

Well MW-90-4

The cone of influence from pumping well PW-90-2 did not reach well MW-90-4 during the pumping test. Well MW-90-4 is located approximately 392 feet from the pumping well (PW-90-2).

PW-90-2 Test Summary

The tests results from well PW-90-2, are interpreted as indicating that the surface water in the drainage ditch, which is located between the pumping well and slurry wall, acts as a recharge boundary.

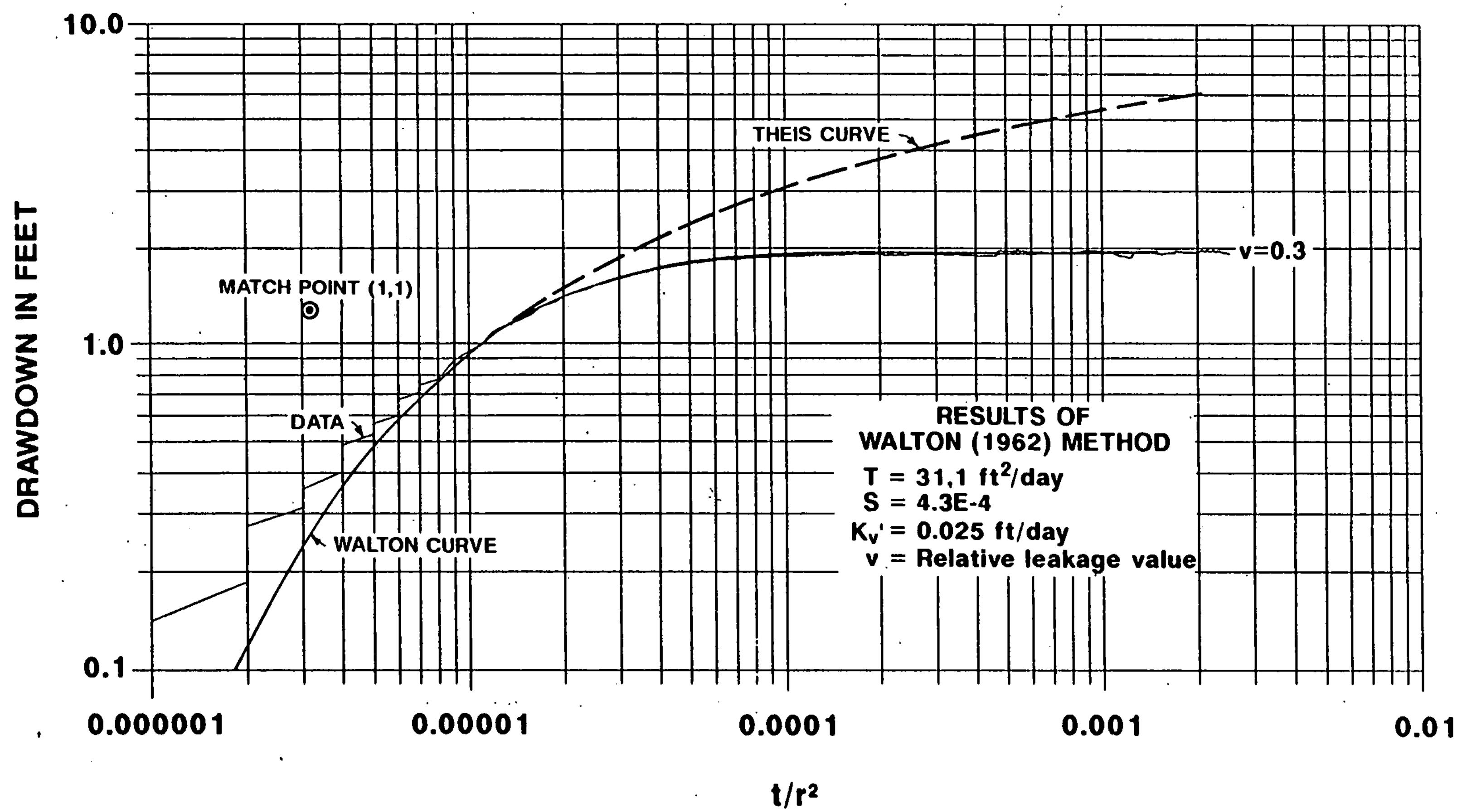

FIGURE 5-13
WELL MW-90-13:
LEAKY AQUIFER ANALYSIS
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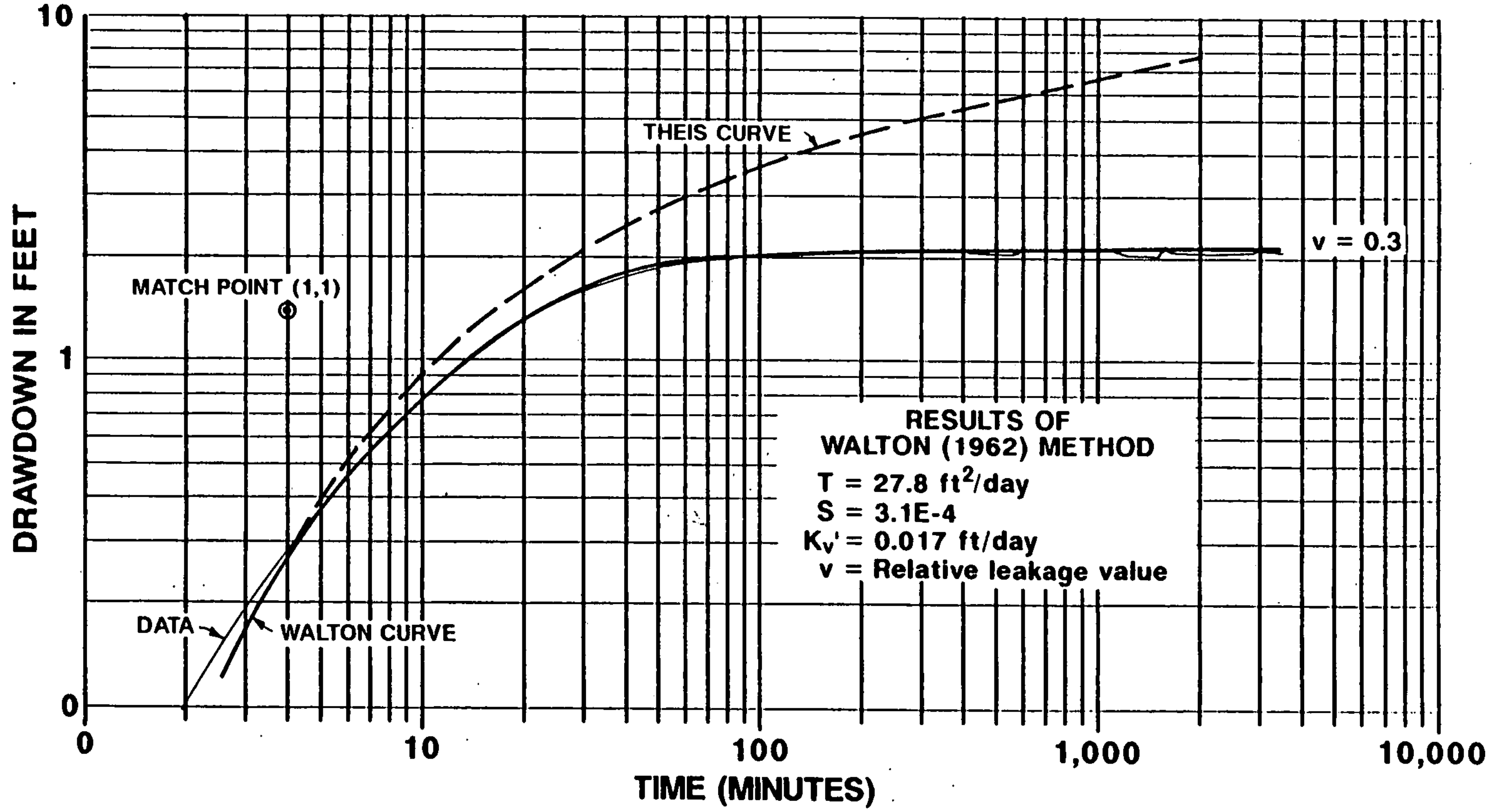

FIGURE 5-14
WELL MW-90-6C:
LEAKY AQUIFER ANALYSIS
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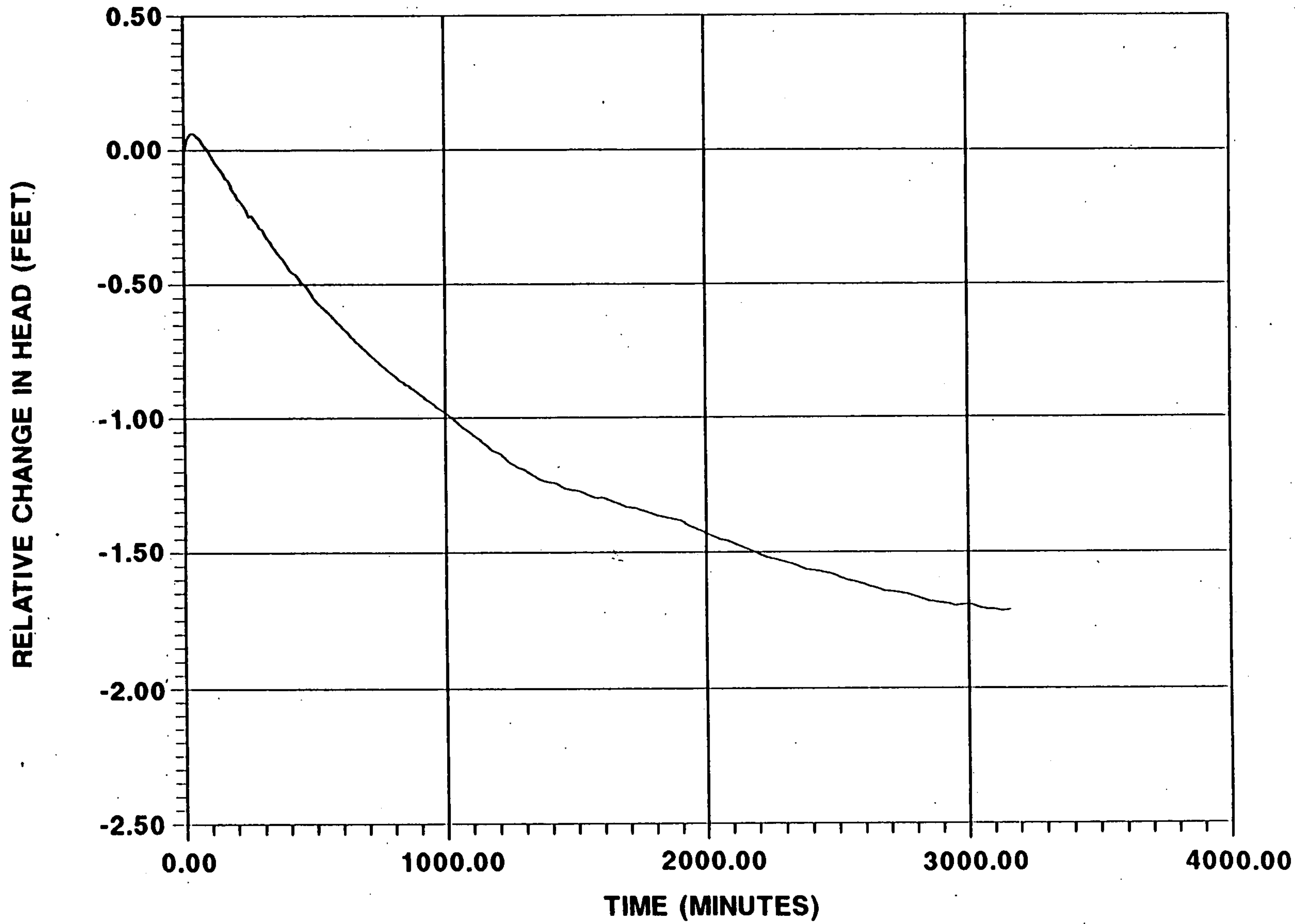

FIGURE 5-15
EFFECT OF PUMPING ON MW-90-6B
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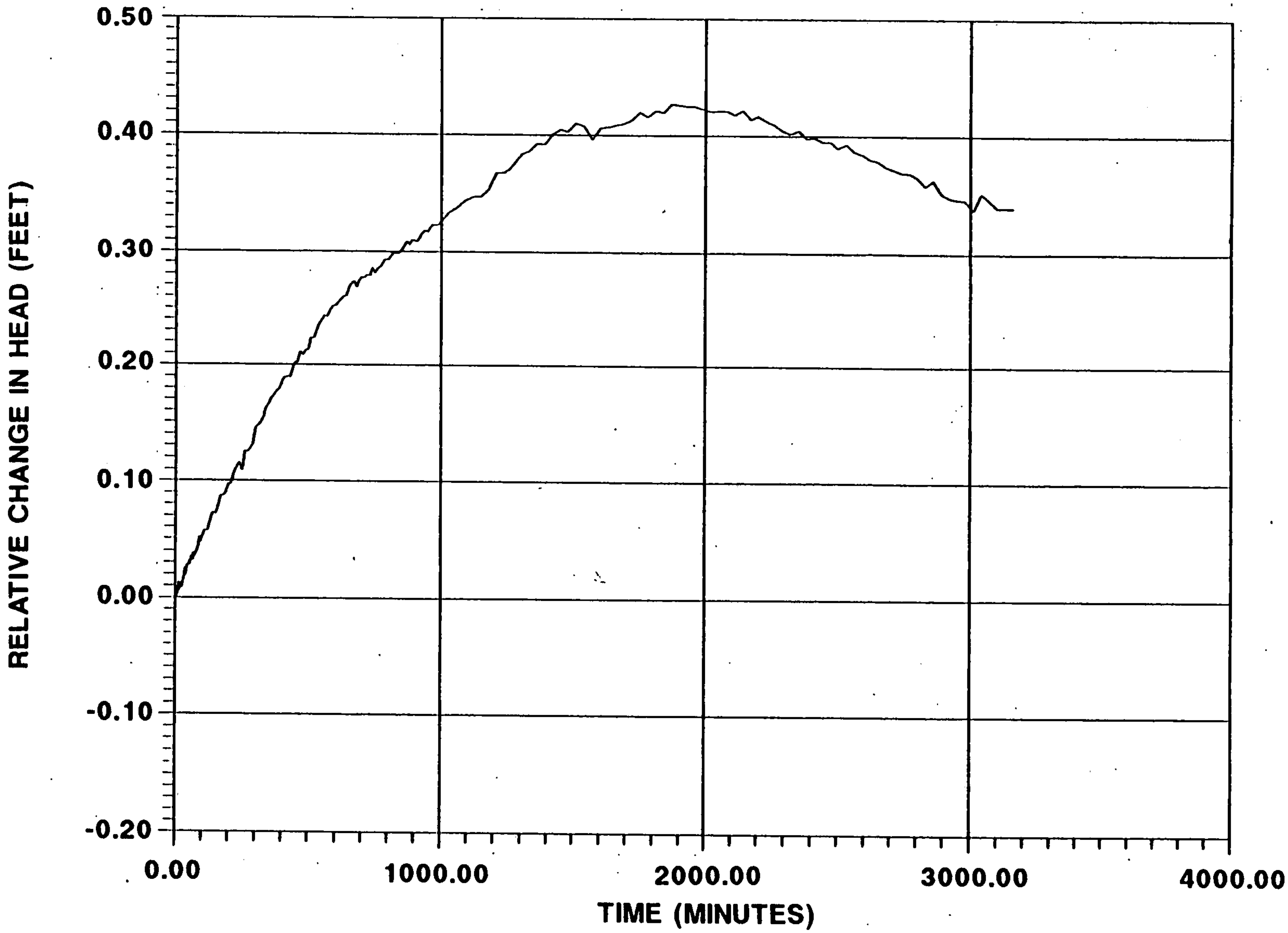

FIGURE 5-16
EFFECT OF PUMPING ON MW-90-6A
KINGSBURY LANDFILL

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Transmissivity apparently is greater at well MW-90-5 compared to wells MW-90-6C and MW-90-13 (a factor of three indicated). This may be the result of a coarsening of the aquifer matrix towards well MW-90-5. A summary and arithmetic mean of the hydraulic parameter values of the three wells are:

Well	T (ft^2/day)	S	K_v' (ft/day)
MW-90-5	91.6	2.6×10^{-3}	0.129
MW-90-6C	27.8	3.1×10^{-4}	0.017
MW-90-13	31.1	4.3×10^{-4}	0.025
Arithmetic Mean	**50.2**	**1.1×10^{-3}**	**0.057**

5.2.6.3 Response of Wells Inside Slurry Wall. Drawdown was not detected in any well located inside the slurry wall during the course of the pumping tests. In particular, the analysis focused on the water-level record (corrected for barometric effects) for sand aquifer wells MW-90-3C and MW-90-7C. At the start of the test, the head was greater on the outside of the slurry wall by 9.2 and 6.5 feet at PW-90-1 and PW-90-2 test arrays, respectively. These head differences indicate any groundwater flux through the slurry wall moves into the landfill area. Neither pumping test sufficiently stressed the aquifer on the outside of the slurry wall to reverse the direction of gradient (i.e., inside to outside), although approximately 2 feet of drawdown was created at the outside of the wall as the test plan specified. The maintenance of a reduced head differential at both test sites over most of the test periods produced no evidence of drawdown in wells inside the slurry wall.

5.2.6.4 Conclusions from Pumping Tests. Both pumping tests produced data from which reliable values of T, K_v', and S were derived. These parameters required quantification for use as inputs in a groundwater flow model, discussed in Section 6.0.

A second goal for the pumping tests was to create hydraulic stress at the outside of the cut-off wall, and subsequently determine if the stress (drawdown) migrated across or under the wall. The target stress of two feet of drawdown at the outside of the wall was achieved by both pumping tests, and was sustained between one to two days. Continuous water level readings in monitoring wells inside the wall indicated no drawdown occurred. The entire wall was not tested, however, due to the inherent focusing of drawdown at the tested locations and clustering of monitoring wells within these areas.

The test durations were considered long enough to interpret these results as demonstrating no gross movement of groundwater across or under the cut-off wall. In this hydrogeologic setting, pumping tests of any duration could not be expected to detect seepage at low rates which might occur with K_v' values of 10^{-1} to 10^{-2} ft/day, under the existing head differential.

6.0 GROUNDWATER FLOW MODEL

6.1 OBJECTIVES

A numerical groundwater flow model was developed to aid in evaluating the groundwater flow system at the Kingsbury Landfill. The primary objectives of the groundwater modeling effort were to: (1) develop a calibrated groundwater flow model based on pre-slurry wall conditions; and (2) utilize this calibrated model to aid in evaluating the cause of the post-slurry wall high groundwater level within the landfill. The flow model, modified as needed, will be used in Task 3 as a tool for the evaluation of remedial alternatives for the effective long term control of the groundwater level within the landfill.

The groundwater modeling effort involved the following components:

- model conceptualization
- selection of model code
- calibration of model to observed conditions
- evaluation of cause of elevated groundwater within the landfill.

6.2 CONCEPTUAL MODEL

A conceptual flow model was developed for the Kingsbury Landfill based on data collected during the 1990 field efforts as well as available historic data. The overburden hydrogeology was conceptualized as a three layer system, with groundwater flowing primarily within an upper silty sand unit and a lower till unit separated by a low permeability silty clay layer. The upper silty sand unit is interpreted to be a semi-confined aquifer based on the aquifer pump test data (see Section 5.2.5.2). No significant horizontal flow is assumed to occur in the silty clay layer, and the silty sand and till units are believed to be largely hydrogeologically isolated. Shallow groundwater flowing within the modeled area is interpreted to discharge along the southern model boundary, and to the wetlands and man-made pond along the southeastern boundary of the model. Recharge to the modeled area occurs through uniform infiltration of precipitation and through groundwater inflow along the northern and western boundaries of the model.

6.3 CODE SELECTION

Selection of a groundwater model numerical code was based on the criteria outlined in Section 4.6.2 under Preliminary Groundwater Flow Model. The USGS code MODFLOW was again determined to be the most appropriate flow model to evaluate groundwater conditions at the Kingsbury Landfill based on the selection criteria. MODFLOW is a finite difference model which provides the necessary features to fulfill the groundwater modeling objectives:

- capable of three dimensional representation
- capable of representing a variety of boundary conditions
- ability to express variability in thickness of aquifer/aquitard units
- ability to simulate effects of wells, drains, and surface water bodies

6.4 DATA COLLECTION

Data used in the groundwater flow model include information obtained through Jordan's investigation at the Kingsbury Landfill and previous investigations (O'Brien and Gere, 1982 and 1983; and Blasland and Bouck, 1990).

The transmissivity of the silty sand aquifer has been estimated to be 13,000 ft^2/day in the PW-90-1 area and 50 ft^2/day in the PW-90-2 area based on the aquifer pumping tests performed by Jordan. The hydraulic conductivities of the till and silty clay units have been estimated to be 7.7×10^{-2} ft/day (2.7×10^{-5} cm/sec) and 3.2×10^{-3} ft/day (1.1×10^{-6} cm/sec), respectively, based on in situ rising and falling head slug tests performed by Jordan. These values represent the log median of the calculated K values. Recharge to the site due to infiltration of precipitation was estimated to be 9 in/yr (O'Brien and Gere, 1982).

6.5 PREPROCESSOR DESCRIPTION

The data preprocessor used to structure the data set for input to the model was PREPRO3FLO: A Preprocessor for the USGS Modular Groundwater Flow Model (GeoTrans, 1988). PREPRO3FLO is a user interactive program designed to create data sets accurately and quickly for use with MODFLOW. Input data files were checked by visual inspection of the model's data input echo, by evaluating the model's response to specified stresses, and by reviewing the model's mass balance outputs.

6.6 MODEL DESCRIPTION

MODFLOW allows the user to simulate two-dimensional and three-dimensional water table or confined aquifer systems as well as several boundary conditions with options that include: wells, recharge, rivers, drains, evapotranspiration, and general head boundaries. For site-wide calibration, the shallow groundwater was assumed to be hydrogeologically isolated from the till aquifer, and no significant flow was assumed in the silty clay. These assumptions lead to a two-dimensional (single layer) representation for the shallow aquifer. In later simulations, some flux into the landfill was simulated as leakage through the clay base (some connection between sand and till units). These later simulations were based on a 2-layer, three-dimensional representation of the hydrogeologic system in the vicinity of the landfill.

6.7 ASSIGNMENT OF MODEL PARAMETERS

The finite difference grid developed for this model is composed of 37 rows and 40 columns with a variable grid spacing ranging from 50 feet to 100 feet. A one-layer model was used to simulate flow in the silty sand unit for calibration purposes. A second layer was added to the model during the evaluation of high groundwater level within the slurry wall to represent the till unit, and a vertical leakage factor was input to represent flow between the two layers through the silty clay unit. Figure 6-1 (in rear map pockets) depict the modeled area, grid network, and established boundary conditions for layer 1 (sand) for this phase of the modeling.

Boundary Conditions: The boundary conditions for layer 1 consist of no-flow conditions along the northeastern and southwestern boundaries, constant-head cells on the southeastern boundary, and general-head boundary (GHB) or specified flux conditions on the north, northwestern, and southeastern boundaries of the model area (see Figure 6-1). The no-flow boundaries correspond to conveniently selected groundwater streamlines that limit the area of the model. The constant-head cells were used to represent the man-made pond and wetland that have been interpreted as groundwater discharge zones. The constant head cells have specified hydraulic heads that do not vary during the model simulation. However, the cells may either contribute or receive water from the active portion of the model as necessary to satisfy the overall mass balance of the model. Based on available topographic data, the constant head cells representing the man-made pond were given head values of 168 feet, and the constant-head cells representing the wetland were given head values of 158 feet. The general-head boundaries (GHB) were used to allow flux into or out of the model. By using general-head boundaries, the model did not have to be excessively large in order to coincide with natural hydrogeologic boundaries, such as groundwater divides. The flow rate into or out of the model along a GHB is based on Darcy's Law. The flow rate is proportional to the assigned hydraulic conductivity of the model nodes; the difference in head between the active cells along the GHB and an assumed external constant head condition. Inclusion of a drain package to represent possible groundwater discharge to the canal did not improve the match to observed groundwater heads, and so was not included in further simulations.

Transmissivity: The silty sand aquifer was modeled with a T of 13,000 ft^2/day in the PW-90-1 area and along the northern portion of the model. The transmissivity for the remaining modeled area was initially input as 50 ft^2/day. The lower T value has to be increased during calibration, however, to 1,000 ft^2/day in order to obtain reasonable head matches with probable uniform (average) recharge rates and to obtain an adequate match with estimated flux rates through the model area. The value of T used in the model represents an increase by a factor of 1.5 when compared to slug test hydraulic conductivities.

Recharge: Recharge due to infiltration of precipitation was applied to the modeled area through the recharge package. A uniform constant rate of 9 in/yr (0.00205 ft/day) was applied to the model to represent average pre-closure conditions over the site.

6.8 MODEL CALIBRATION

To simulate the semi-confined aquifer flow system, a steady-state flow model was first calibrated to reflect pre-closure groundwater flow conditions. The process involved defining and applying the modeled hydraulic parameters of the aquifer materials to match, as closely as possible, observed field conditions and create a reasonable model of flow with respect to direction, gradient, and overall mass balance. With the general site hydrogeologic conditions established for pre-closure conditions, boundary conditions in the model could then be modified to simulate post-closure conditions.

The initial data set for the model was input using PREPRO3FLO and then used in a trial run for data checking and observation of general model response. The resulting output from this trial run was checked against the input data for data entry or data format errors. The model was calibrated through a series of runs by establishing boundary conditions and varying the global transmissivity and recharge rate. The calibration process involved comparing the simulated output, computer-generated potentiometric surface contour maps produced by Golden Software's SURFER© program, with the pre-closure groundwater elevation values for the silty sand unit (O'Brien and Gere, 1983). These pre-closure groundwater elevation values for December 22, 1982 were used to create a groundwater contour map for the silty sand unit (Figure 6-2). The initial simulation was run with two zones of transmissivity of 13,000 ft^2/day and 50 ft^2/day based on pumping test data and observations of interpreted head contours, and a recharge rate of 9 in/yr.

The resulting potentiometric surface was higher than the observed pre-closure contours. The elevated heads were caused by too much water entering the model combined with insufficient aquifer permeability to transmit the groundwater through the model area. The northern GHB was modified to reduce the amount of groundwater entering the model, and the lower transmissivity zone was also modified by increasing it by factors of 2, 5, 10, and 20, with a final calibration T value of 1,000 ft^2/day. Note that the determination of T in this area during the pumping tests was subject to more difficulties, and was probably less reliable than the 13,000 ft^2/day T value determined by the pumping test at PW-90-1. The recharge rate was cut to 6 in/yr with a negligible effect on the water level contours. The model was not very sensitive to reasonable variations in the recharge rate since, at 9 in/yr, recharge represented only 12% of the total water balance. Therefore, recharge was restored to the original 9 in/yr in subsequent simulations and for final calibration in accordance with reported recharge rates.

The final calibrated model results from RUN21bOUT are shown in Figure 6-3 in the form of potentiometric surface contours for the silty sand unit. The December 22, 1982 water table elevations for selected wells across the model area were compared with the calibrated model results as follows:

Well #	Observed head value (ft)	Simulated head value(ft)	Difference(ft)
K28	200.1	203.9	+ 3.8
K31	178.9	180.0	+ 1.1
K13	184.9	184.7	- 0.2
KF2	201.3	206.4	+ 5.1
K14A	202.8	208.2	+ 5.4

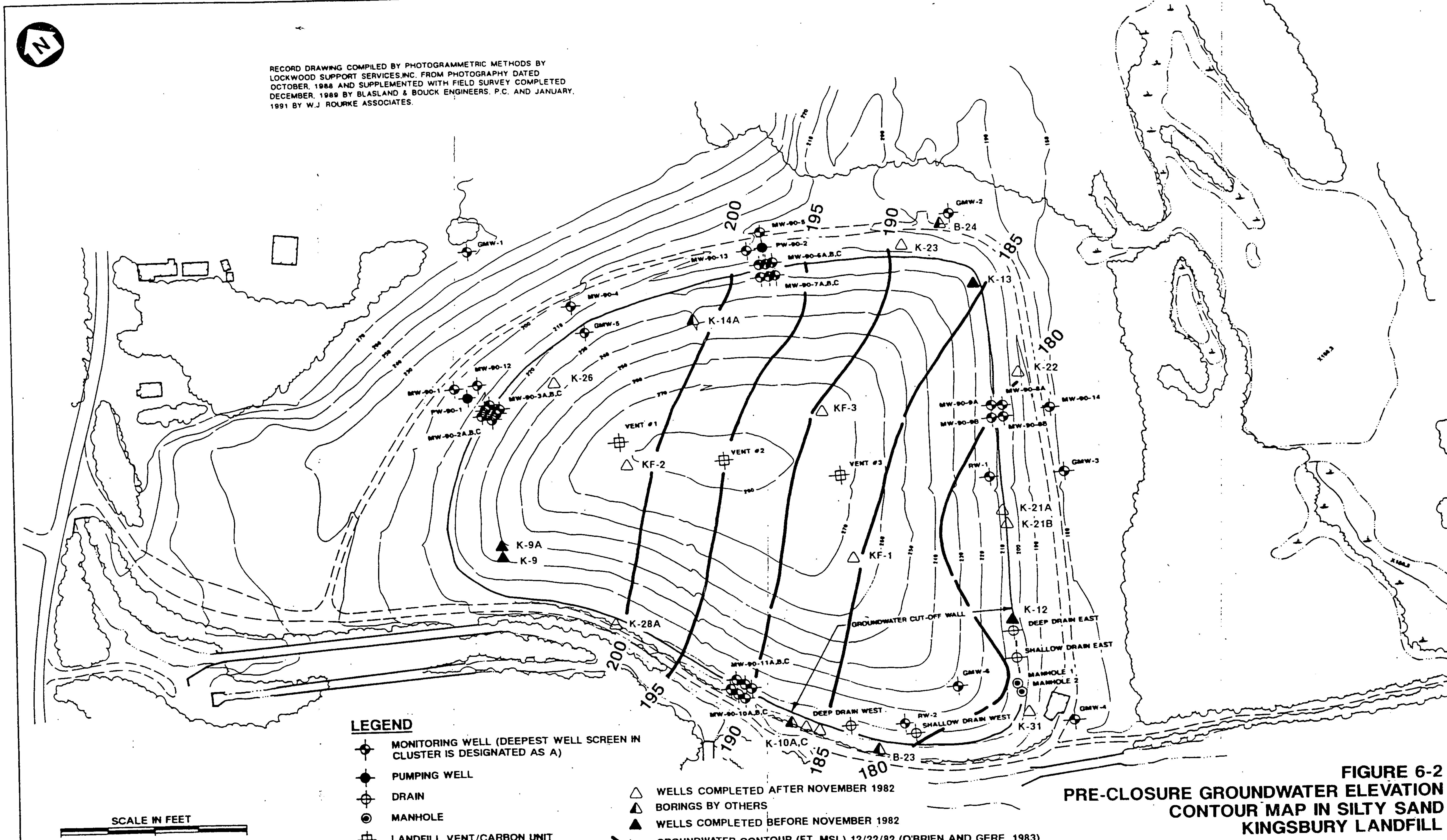

FIGURE 6-2
PRE-CLOSURE GROUNDWATER ELEVATION CONTOUR MAP IN SILTY SAND
KINGSBURY LANDFILL

ECJORDANCO.

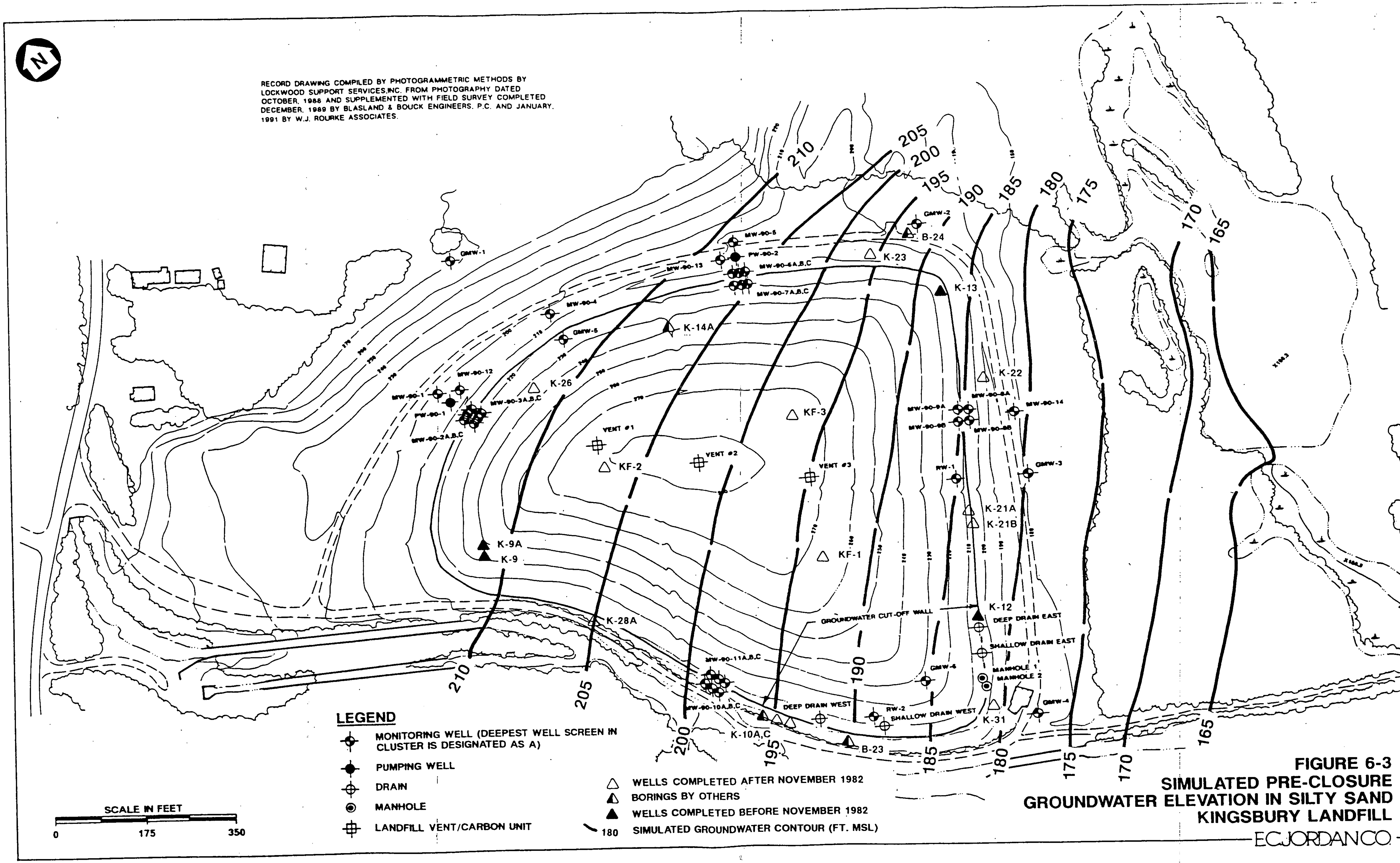

FIGURE 6-3
SIMULATED PRE-CLOSURE GROUNDWATER ELEVATION IN SILTY SAND
KINGSBURY LANDFILL

Wells KF2 and K14A are located in the northern portion of the landfill where groundwater elevation data appears to be inconsistent. Wells K14 and K14A are located in close proximity to one another and are both screened in the silty sand, but the two wells have a head difference of 5 feet. Well K14A, the lower value, was used to create Figure 6-2 as it was more consistent with groundwater elevations in other nearby wells. The other representative wells match within a few feet between the observed and simulated head values. A hand calculation of water flowing through the site versus the model mass balance was within 10% (Appendix F). The estimated flow through the site was nearly four times that estimated by Blasland and Bouck, primarily because of the greater transmissivity of the aquifer, based on the pumping test at PW-90-1. The simulated average gradient was 0.032 versus an observed of 0.029. The flow directions, observed and simulated, matched over most of the modeled area. The overall simulated gradient, flow direction, and mass balance, therefore, adequately match observed conditions and the model was considered adequately calibrated for further use in the evaluation.

6.9 Evaluation of Cause of Elevated Groundwater Within the Landfill

The calibrated preclosure steady-state model had to be modified to reflect the slurry wall and cap construction to evaluate the cause of the high groundwater within the landfill. The modified boundary conditions are shown in Figure 6-4 (in rear map pockets). The slurry wall was simulated in the model using low transmissivity values for the nodes corresponding to the wall, and the cap was represented by reducing recharge rate to the model over the capped area. A correction factor was applied to the horizontal K value for the nodes representing the wall, obtained through the insitu rising and falling head slug tests, to account for the actual thickness of the slurry wall compared to the size of the model grid block used to represent the wall. A transmissivity (2.3 ft^2/day) for model elements representing the slurry wall was obtained by using the adjusted K value and an assumed saturated aquifer thickness of 35 feet. To determine the recharge rate over the landfill area, the pan lysimeter data was used to estimate infiltration through the cap (Appendix D). A recharge rate of 0.22 in/yr (0.00005 ft/day) was input for the landfill area.

The numerical-solution scheme became unstable and failed to converge on a solution when the modifications were attempted. This is not an uncommon occurrence with MODFLOW when the steady-state solution option is applied to a complex set of boundary and hydrogeologic conditions. To overcome the convergence difficulties, the steady-state model was modified to run on a transient basis with a 10-year simulation period--long enough for equilibrium (i.e., steady-state) conditions to become established within the model. This was verified by checking the storage values in the model mass balance output. A storativity (S) value of 9.6×10^{-4} was used in these transient simulations. The S value represents a log median of the values obtained during the aquifer pumping tests. However, since equilibrium head values were being sought, the value of S input into the model is not important as long as it is low enough to produce equilibrium during the simulation period.

A general match was obtained between the simulated and observed values at observation wells outside the landfill. To achieve the match, the northern and western GHB conditions were modified to account for observed changes in the flux boundaries as a result of the slurry wall. The southwestern no-flow boundary was also moved further from the landfill as a consequence of the barrier to flow to allow more area for water to flow around the landfill and out of the system. It was anticipated that these changes to the calibrated model would be necessary to simulate the impact of the slurry wall, cap construction and the resulting decrease in the cross-sectional area of the groundwater flow system. Insufficient data exists to provide accurate estimates of this displacement in the flow boundary, but it was moved only far enough to provide an acceptable mass balance in the model. The overall mass balance was maintained, with only a decrease of about 10% in total model flow between the two representative models (pre-closure and post-closure).

With the model modified to approximate post-closure conditions, a series of simulations and sensitivity runs were made to assess possible alternatives for explaining the high water levels within the landfill. The alternatives examined were:

- Leakage through the landfill base
- Infiltration through the cap
- Leakage through the slurry wall
- Existence of a mound within the landfill

The existence of a seep within the slurry wall area presented simulation problems. A drain package with an invert elevation equal to the top of the slurry wall could be used to estimate seepage flow. However, in order to use the drain package, a series of wells along the seepage area outside of the slurry wall had to be simulated to reintroduce the water removed by the drain package, thereby completing the simulation of seepage over the slurry wall. No attempt was made to represent the drain system that was later installed to mitigate the seep, as non-pumping long-term equilibrium conditions were considered the appropriate scenario for the modeling.

Descriptions of all model runs, and selected Surfer© contoured plots of simulations appear in Appendix F.

Landfill Base: When the simulated head values outside the landfill adequately matched the observed head values, layer 2 was added to the model to determine the possible impact of leakage through the silty clay unit and its relationship to the elevated groundwater within the landfill.

The boundary conditions for layer 2 consist of inactive or no-flow model node for the entire grid with the exception of the area underlying the landfill, which was represented with constant head nodes (see Figure 6-5, in rear map pockets). The constant head nodes were assigned head values to establish a gradient representative of observed conditions. Though the exact hydraulic gradient in the till is unknown, it is assumed to be the same as in the upper aquifer a reasonable assumption based on the observed head differences in the two till wells. Constant-head nodes under the landfill area were provided with head values that varied from 206 feet, at the northern upgradient edge of the landfill, to 186 feet at the southern downgradient edge of the landfill. A vertical conductance term was introduced in the model to represent flow through the silty clay layer between layer 1 and layer 2 over the landfill area. A vertical K value of 3.2×10^{-4} ft/day (1.1×10^{-7} cm/sec) and a clay thickness of 40 feet were used to calculate the vertical conductance. With a uniform K in the clay base, sensitivity analysis indicated a variable clay thickness known to exist from 10 feet in the north to 70 feet in the south, had no significant effect over the assumption of a uniform clay thickness. The addition of the second layer had a negligible effect on the simulation head distribution results even with the following variations:

- The vertical conductance term was uniformly increased by a factor of five.

- In a 200 square-foot section of the base, the vertical conductance term was increased by two orders of magnitude and the constant head values in the till were increased by 5 feet to provide a greater upward driving head.

Infiltration Through the Cap: The pan lysimeter data shown in Appendix D indicates that no significant infiltration occurs through the cap. Runs were made increasing the infiltration from 0.2 in/yr.to 2 in/yr with no significant increase in heads within the landfill (less than 0.5 foot change).

Landfill Slurry Wall: To determine if a relationship existed between the slurry wall and the elevated groundwater within the landfill, three upgradient slurry wall nodes along the upper wall, a 200-foot length, were given T values of 100 ft^2/day--about 40 times the calculated T. This would represent a fairly large expanse of slurry wall to be at less than design hydraulic conductivity. The wells representing the cascading seep at the southern end of the landfill were provided with new flow rates to equal the amount of water collected by the drains representing the seep outflow. This water represents the combined recharge into the cap area and groundwater influx through the leaky (more permeable) slurry wall nodes, but may also represent a poor keying of the wall into the clay in the north, or significantly more underflow in the north than in the south.

In TRUN32OUT, the model simulation run with a leaky slurry wall, the simulated groundwater levels with the landfill closely matched those observed in the monitoring wells and relief wells. However, in an area immediately downgradient of the landfill, the simulated heads were still too low compared to the observed values. To determine the quantity of water necessary to input into the model to match the observed heads, four injection wells were placed along the downgradient perimeter of the slurry wall. Flow rates of 1, 5, 10, and 15 gpm for each well were simulated. The results from the 15 gpm per well simulation, TRUN33cOUT, yielded the best match with simulated head values downgradient of the slurry wall, within three feet of the interpreted observed values.

Existence of a Mound: The total water pumped out from within the slurry wall in 1990 was about 1.5 million gallons. Given a porosity of 0.3 and the area within the slurry wall of about 17 acres, approximately one foot of stored water within this area would provide the 1.5 million gallons. Since piezometers and wells do not exist within the central portion of the landfill, it is not presently possible to directly observe whether or not a mound of stored water exists within the landfill as Blasland and Bouck suggest it might.

In order to evaluate potential mounding within the landfill area, recharge was input as 18 in/yr to represent increased potential of recharge during operation. Due to the relatively high transmissivity of the sand layer, no significant mounding occurred as a result of this stress in the model. Note, however, that if the water retention capacity of the wastes themselves was high, water might be stored in the wastes and lead to subsequent drainage. One might expect this drainage rate to rapidly decrease over time. As no data was available for the hydraulic parameters of the waste material, no attempt was made to include the waste layer in the model.

6.10 SUMMARY

A two-dimensional numerical groundwater flow model of the Kingsbury Landfill was developed and calibrated to pre-closure conditions. The model results matched well with interpreted observed pre-closure conditions with respect to flow directions, groundwater flux, and gradients. The two-dimensional steady-state calibrated model was then modified to include the containment system and a till layer, and run to evaluate the cause of the elevated groundwater within the slurry wall. In summary, the results of the simulation and sensitivity runs were:

- Increased K in the clay base and driving head were not significant factors in creating higher groundwater levels within the landfill slurry wall. That is, upflow through the clay base from the till did not appear to be a likely mechanism.

- Increased recharge of up to 2 in/yr (much higher than observed and estimated) over the entire landfill area did not significantly increase water levels within the landfill. Such higher infiltration rates appear unlikely based on pan lysimeter data, and would not be sufficient by themselves to provide the 1.5 million gallon per year pumpage apparently needed to maintain level control to eliminate the seep.

- Increasing the horizontal K of the simulated slurry wall in the north was the only condition which significantly increased water levels within the landfill to where a seep condition with a flow rate similar to the pumping rates was created. While simulated as a leaky portion of the slurry wall along the north side, this flux could also occur if the wall were not keyed properly along this side, or if a significant sand seam passes beneath the wall and daylights within the upper slurry wall area. The present density of explorations along the northern slurry wall does not provide sufficient information to select from among these alternatives.

  Observed downward and upward gradients associated with well clusters next to the slurry wall suggest flow beneath the wall. The need to supply water to areas downgradient of the slurry wall in the model in order to match observed heads also suggests a significant flow in this area unaccounted for in the scenario of a tight slurry wall construction or in the current understanding of the site hydrogeology.

- Insufficient information exists to completely evaluate the possibility of a mound within the waste material acting as a short-term storage of water interior to the slurry wall. If the waste is freely draining, a model run suggested that the sand layer would be an effective drain and no significant mounding would occur, even with high recharge rates during operation. If the waste is not freely draining, then a mound significantly large enough might exist which could provide the amounts of water generated during the pumping of the seep drains.

7.0 EVALUATIONS

7.1 EVALUATION OF CAUSE OF HIGH GROUNDWATER IN LANDFILL

The calibrated groundwater flow model was used to aid in the evaluation of high groundwater levels within the landfill. Four scenarios were evaluated during the modeling effort. The four alternatives examined were:

- Leakage through the base
- Infiltration through the cap
- Leakage through the slurry wall
- Existence of a mound within the landfill

Simulation of the first two alternatives produced no significant increase in water levels in the landfill with reasonable increases in the vertical conductance and recharge, respectively. Neither of these alternatives appear to be a likely source for high water levels within the landfill. Leakage through the slurry wall was simulated by assigning an increased transmissivity to a 200-foot section of the slurry wall in the northern portion of the landfill. This could also be interpreted as flow beneath the wall, a poorly keyed wall, or a sand seam daylighting through the clay base and into the landfill interior. The simulation was successful in matching the elevated water levels observed within the slurry wall and the approximate volumes of water removed in pumping the seep drains. Immediately downgradient of the slurry wall, however, simulated heads were approximately 7 feet lower than observed heads. This suggests a significant amount of flow in this area is unaccounted for in the scenario of a tight slurry wall construction, and relatively high K in the sand layer. Downward and then upward gradients at the nested piezometers inside and outside the wall suggests a potential for groundwater flow beneath the wall. The mounding alternative could not be completely evaluated due to insufficient information about the waste and its water content and water levels in the central region of the landfill. However, a model run with 18 in/yr of recharge demonstrated the sand layer would be an effective drain and no significant mounding would occur if the waste is freely draining. If water is tightly bound in the wastes and water is slowly draining, this exfiltrating water could cause elevated groundwater levels in the landfill which would diminish over time.

Based on the model results and interpretations from the hydrogeologic data, the most likely cause of the elevated groundwater levels within the landfill is a leaky or poorly-keyed slurry wall, or a sand seam beneath the wall which discharges within the upper slurry wall area. For remediation purposes, determination of the precise mechanism(s) of leakage through the upgradient slurry wall is not critical. The 10 to 20-foot head differences across the slurry wall indicates that the slurry wall is relatively effective, and decreasing groundwater levels observed inside the landfill during ILTS operation indicates that net leakage through the upgradient slurry wall, if present, is occurring at a low rate (on the order of one to two gpm).

7.2 Evaluation of Seasonal Water Table Fluctuations in Landfill

Water level data between January 1989 and June 1991 was plotted as a function of time for GMW-5 and GMW-6 to evaluate the seasonal water table fluctuations in the landfill. The water level data for GMW-1 was also plotted to determine background seasonal water table fluctuations upgradient of the landfill containment system (Figure 7-1). Included on this figure is the delineation of the effective dates of pumping from the ILTS. The available water level data for GMW-1 beginning at May 1989 is not continuous through June 1991. Figure 7-1 also presents inches of rainfall as a function of time to aid in evaluating seasonal fluctuations in the water table. The effects of the ILTS operations must also be considered when determining the seasonal water table fluctuations in the landfill. GMW-5 and GMW-6 are located at approximately 1100 feet and 120 feet from Manhole #2, respectively, which is the leachate collection point. Therefore, the effects of the ILTS operations are expected to be more evident at GMW-6 than GMW-5.

A definite seasonal trend in the water level data for GMW-1 is not evident. However, a correlation between precipitation and water level data at GMW-1 is evident. A large 8.4-inch rainfall event occurred in August 1990, and produced an observed increase of approximately 0.4 feet. The amount of available long-term precipitation appears to have a more substantial influence on the water table trend at GMW-1. An overall trend of increasing water levels is demonstrated throughout the period.

Water levels at GMW-5 began at 200.65 feet in January 1989 with fluctuations up to 1.5 feet through June 1990, when it began a relatively steady decline to 201.2 feet in June 1991. During the 1990 ILTS operation, GMW-5 water levels dropped approximately 0.5 to 0.8 feet and continued to decline another 0.15 feet during ILTS shutdown from December 1990 to April 1991. During the same period of time,

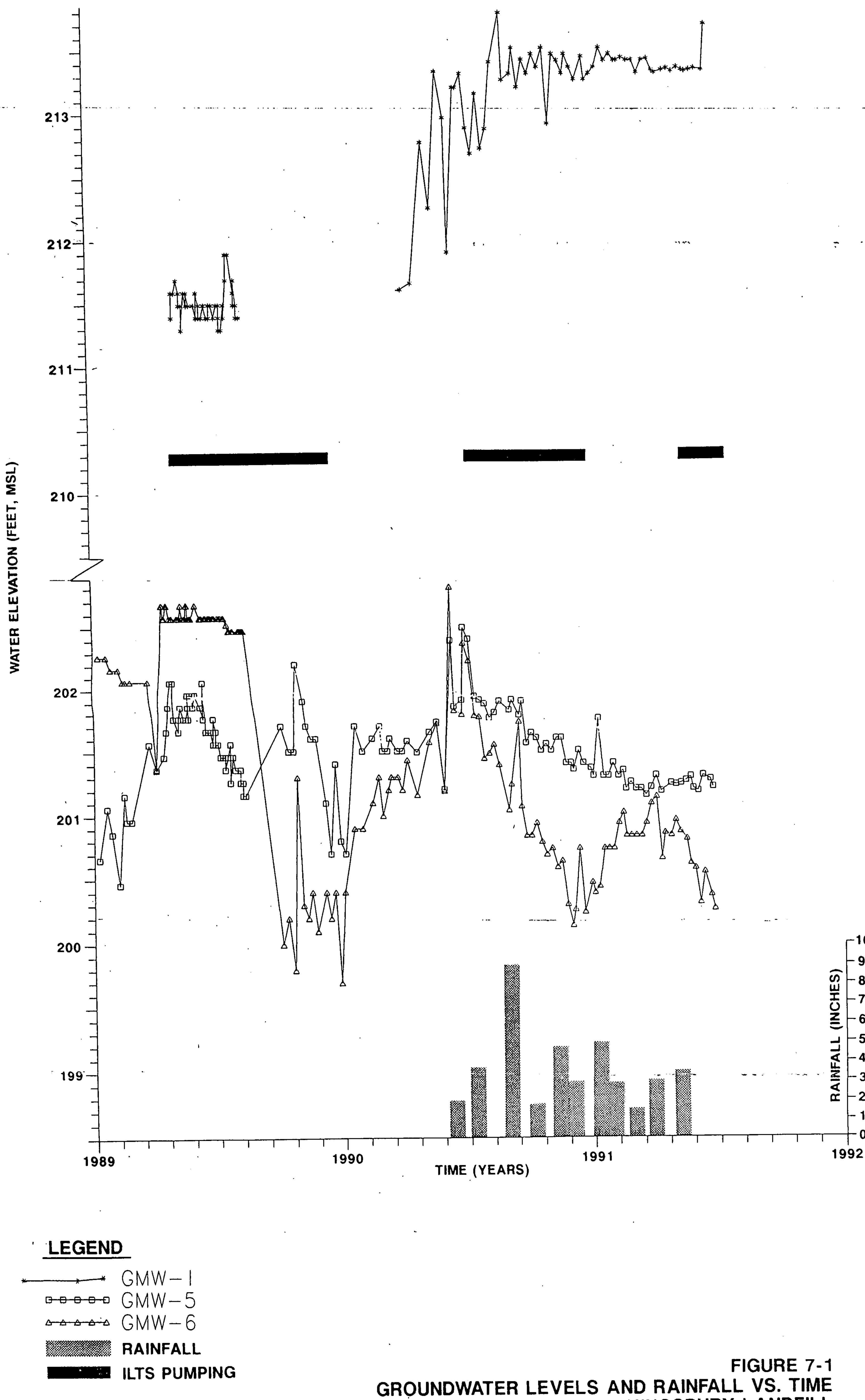

FIGURE 7-1
GROUNDWATER LEVELS AND RAINFALL VS. TIME
KINGSBURY LANDFILL

E.C. JORDAN CO.

GMW-6 water levels dropped approximately 1.5 feet then rebounded 0.5 feet. This indicates that the average water level inside the slurry wall was lowered about 0.6 to 1.0 feet from June 1990 to April 1991, which includes post-pumping equilibration of the drawdown around the GMW-6 drainage/pumping area. Compared to the estimated 0.9-foot water level drop expected from the 1.52 million gallons pumped out the landfill by the ILTS during 1990, the net leakage into the landfill containment system is estimated to be less than 30% of the average 1990 ILTS pumping rate of 6 gpm. This estimation is supported by the observation that water levels at GMW-5 declined during the after ILTS pumping, which indicates that potential leakage along the northern portion of the slurry wall near GMW-5 occurred at a rate less than the ILTS pumping rate.

After the ILTS startup in April 1991, water levels in GMW-5 and -6 appear to be declining similar to the 1990 trend. If this trend continues throughout 1991, the average landfill water level is expected to decrease at least one foot during 1991 ILTS operations. Monitoring water levels for an extended pumping shutdown period after the planned December 1991 ILTS shutdown, would provide further indication of the magnitude of groundwater leakage into the landfill.

7.3 Evaluation of Optimum Groundwater Equilibrium Level in Landfill

The strategy for selecting an appropriate optimum equilibrium level of water within the landfill depends on the actual mechanisms controlling flow into and out of the slurry wall containment area. The probable contributing factors as discussed in Section 6.10 and summarized in Section 7.1 are:

- leakage from storage within the landfill wastes (prior mounding of groundwater);
- leakage under or through weak points in the northerly portion of the slurry wall, including the possibility of continuous sand stringers through the clay linking the outer and inner sand layer; and
- net gain within the landfill due to underflows or higher K of the slurry wall than desired (this may occur in limited areas of the wall construction).

Of course, some or all of these factors could be acting together to some unknown degree. However, it is best to consider what the effects of each separate factor may have in determining a strategy.

If the primary mechanism for high levels presently observed within the landfill is leakage from water stored within the wastes, not much is to be gained by reducing the equilibrium head much lower than that needed to prohibit the reoccurrence of the seep. The stored water should drain out of the waste at a decreasing rate (and this may be a means of determining if this could be the primary mechanism). Drainage could be removed by pumping at rates not necessarily higher than those currently employed. Estimates of the total water remaining in the system could be made based on the apparent drainage rates over time. The other alternative is to pump out quantities of water from the sand layer to provide a buffer capacity for future drainage to occupy without causing seepage overflow. After drainage is complete, the landfill, if uniformly tight around its periphery, should seek an equilibrium level somewhere around the 195-foot elevation.

If the mechanism for observed high levels is a weakness or underflow in the northerly reaches of the slurry wall, or a sand stringer which connects the outer and inner hydrogeologic systems, it may be necessary to dewater, e.g., through diversion ditches, the overburden upgradient of the slurry wall to drop the level below that in the landfill. This and other remedial alternatives will be examined in Task 3. However, if this is, indeed, the mechanism, then the level in the landfill should be kept as high as possible (yet prevent the seep); otherwise, the head differential between outside and inside would increase, as would the flux through seepage. Therefore, the amount of water pumped out of the landfill would increase proportionately to the selected lower equilibrium level, probably at rates somewhat higher than presently experienced (i.e., 6 to 12 gpm).

If the high water level within the landfill is caused by a positive net flux into the landfill, but basically due to active flow through/under the slurry wall, both to the north and south, then it would be necessary to reduce heads upgradient of the slurry wall and reduce heads within the landfill as much as possible to prevent further leachate migration downgradient of the landfill. Recall that a simulation of a tight (low flux) slurry wall in the southern section of the landfill produced simulated heads much lower than those observed downgradient of the slurry wall. It was necessary in the model to introduce about 60 gpm into this area to bring the heads up to the observed levels. Part of this deficit may be due to actual hydraulic conductivities in the area being lower than modeled, but it also suggests some underflow or leakage through the wall. The present available data do not permit a further resolution of the cause or possible flux rates involved at this time. To reduce flux downgradient of the landfill would then require reduction of the head within the landfill to decrease the differential head through the wall. However, if the maintenance of heads downgradient of the wall is dependent on the flux through/under the wall, then it may be necessary to nearly dewater the interior of the slurry wall to prevent any flux downgradient. Pumping rates to achieve this dewatering would be dependent on the timeframe selected to achieve the desired scale of dewatering, and the capacity of whatever treatment systems are selected to process the water removed. Water in storage within the sand (with an assumed specific yield of 0.25) beneath the wastes is approximately 1.3 million gallons per foot based on an approximate landfill area of 17 acres.

BE	barometric efficiency
CHI	Clean Harbors, Inc.
cm/sec	centimeters per second
DOT	Department of Transportation
ft/day	feet per day
GE	General Electric Company
ghb	general-head boundary
gpd	gallons per day
gpm	gallons per minute
ID	inside diameter
ILTS	Interim Leachate Treatment System
K_h	horizontal hydraulic conductivity
K_v'	vertical hydraulic conductivity of aquitard
Mathes	John Mathes, Inc.
MODFLOW	Modular Three-Dimensional Finite Difference Groundwater Flow Model Code
MSL	Mean Sea Level
NYSDEC	New York State Department of Environmental Conservation
NYSDOH	New York State Department of Health
NYSDOT	New York State Department of Transportation
PCBs	polychlorinated biphenyls
PID	photoionization detector
PVC	polyvinyl chloride
S	storativity
SUNY	State University of New York
T	transmissivity
TLTS	Temporary Leachate Treatment System
USGS	U.S. Geological Survey

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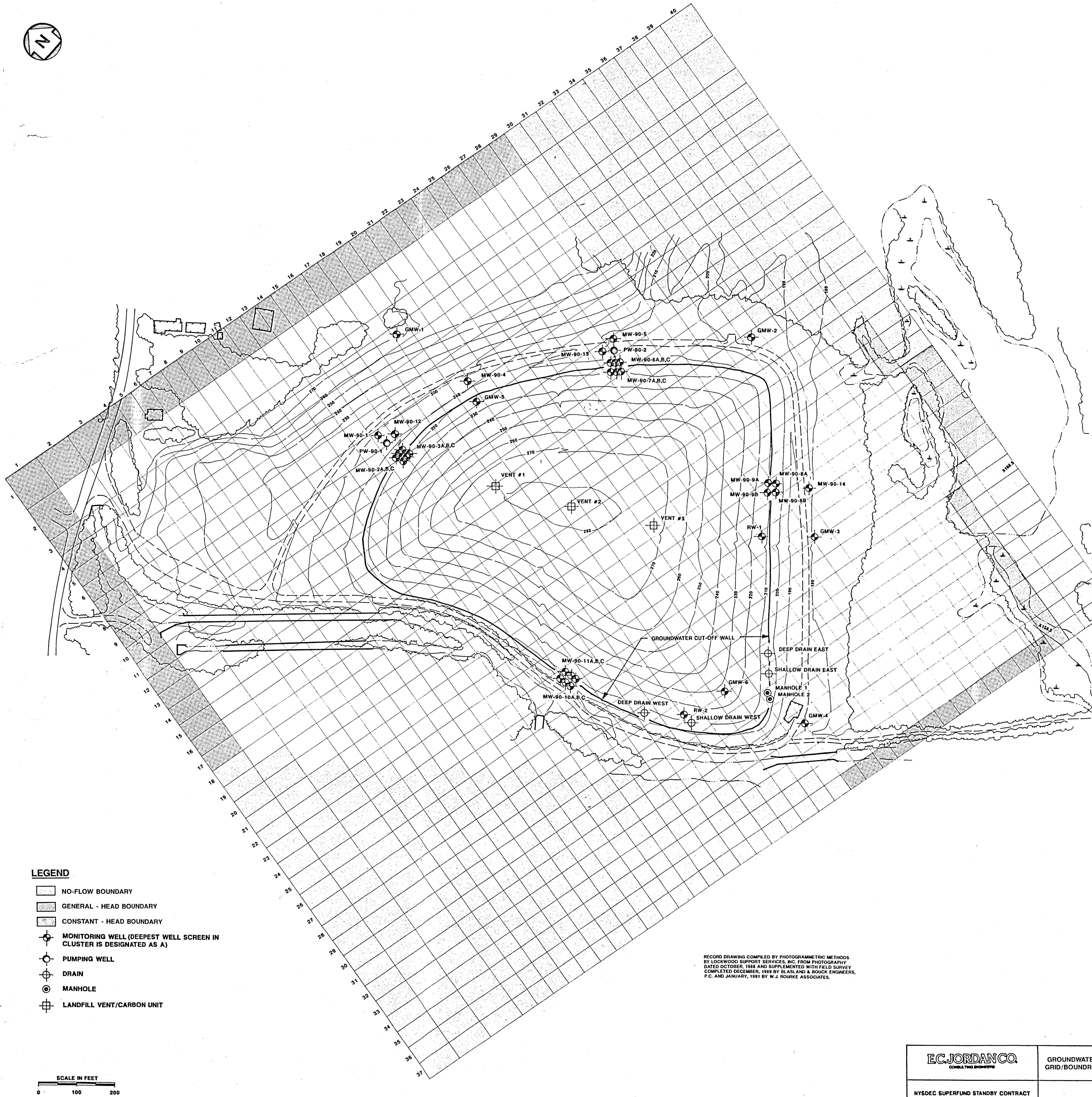
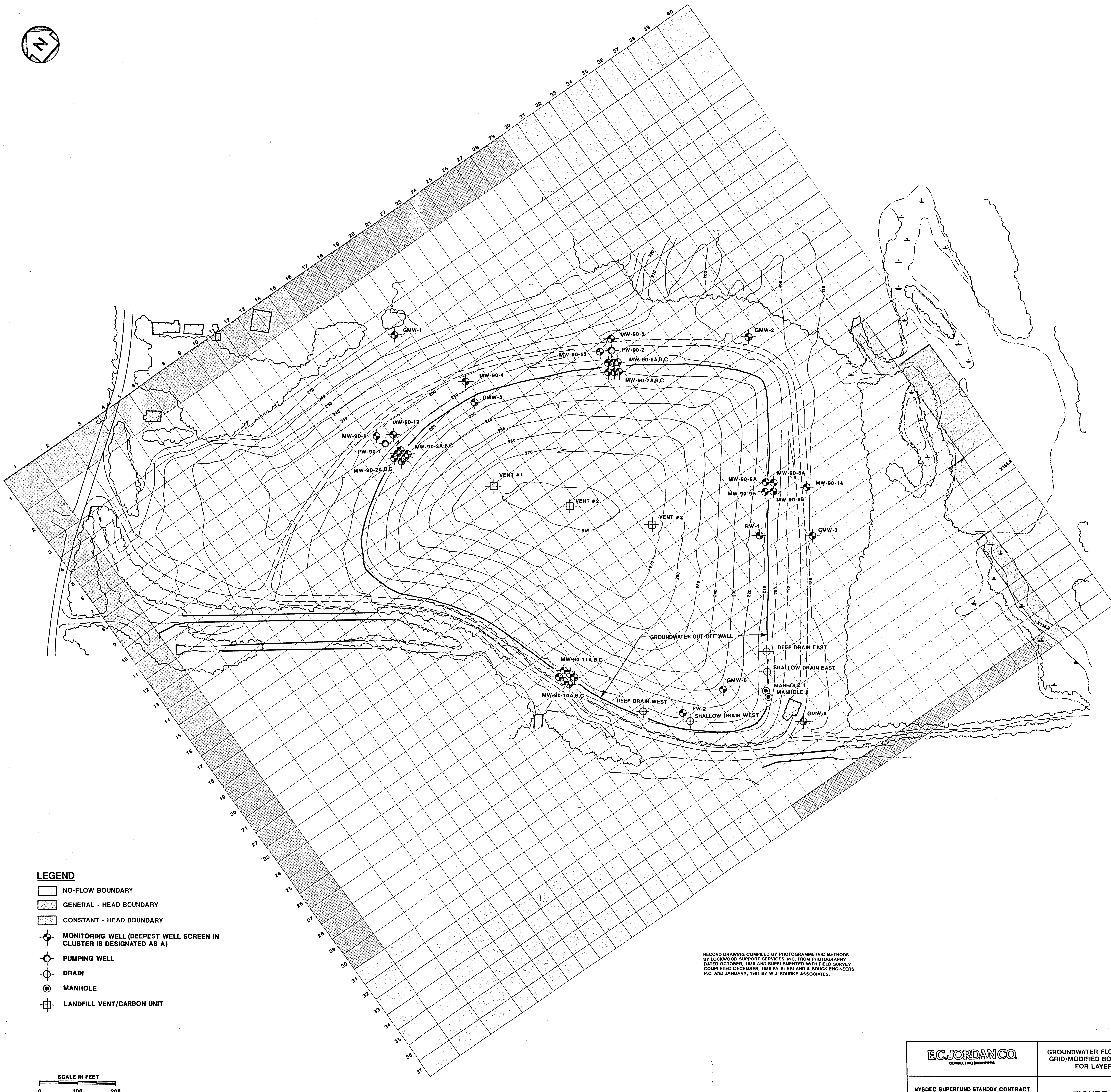


RECORD DRAWING COMPILED BY PHOTOGRAMMETRIC METHODS BY LOCKWOOD SUPPORT SERVICES, INC. FROM PHOTOGRAPHY DATED OCTOBER, 1988 AND SUPPLEMENTED WITH FIELD SURVEY COMPLETED DECEMBER, 1989 BY BLASLAND & BOUCK ENGINEERS, P.C. AND JANUARY, 1991 BY W.J. ROURKE ASSOCIATES.

E.C. JORDAN CO. CONSULTING ENGINEERS	GROUNDWATER FLOW MODEL GRID/BOUNDRIES FOR LAYER 1
NYSDEC SUPERFUND STANDBY CONTRACT KINGSBURY LANDFILL SITE	FIGURE 6-1



LEGEND

- NO-FLOW BOUNDARY
- GENERAL - HEAD BOUNDARY
- CONSTANT - HEAD BOUNDARY
- MONITORING WELL (DEEPEST WELL SCREEN IN CLUSTER IS DESIGNATED AS A)
- PUMPING WELL
- DRAIN
- MANHOLE
- LANDFILL VENT/CARBON UNIT

SCALE IN FEET
0 100 200

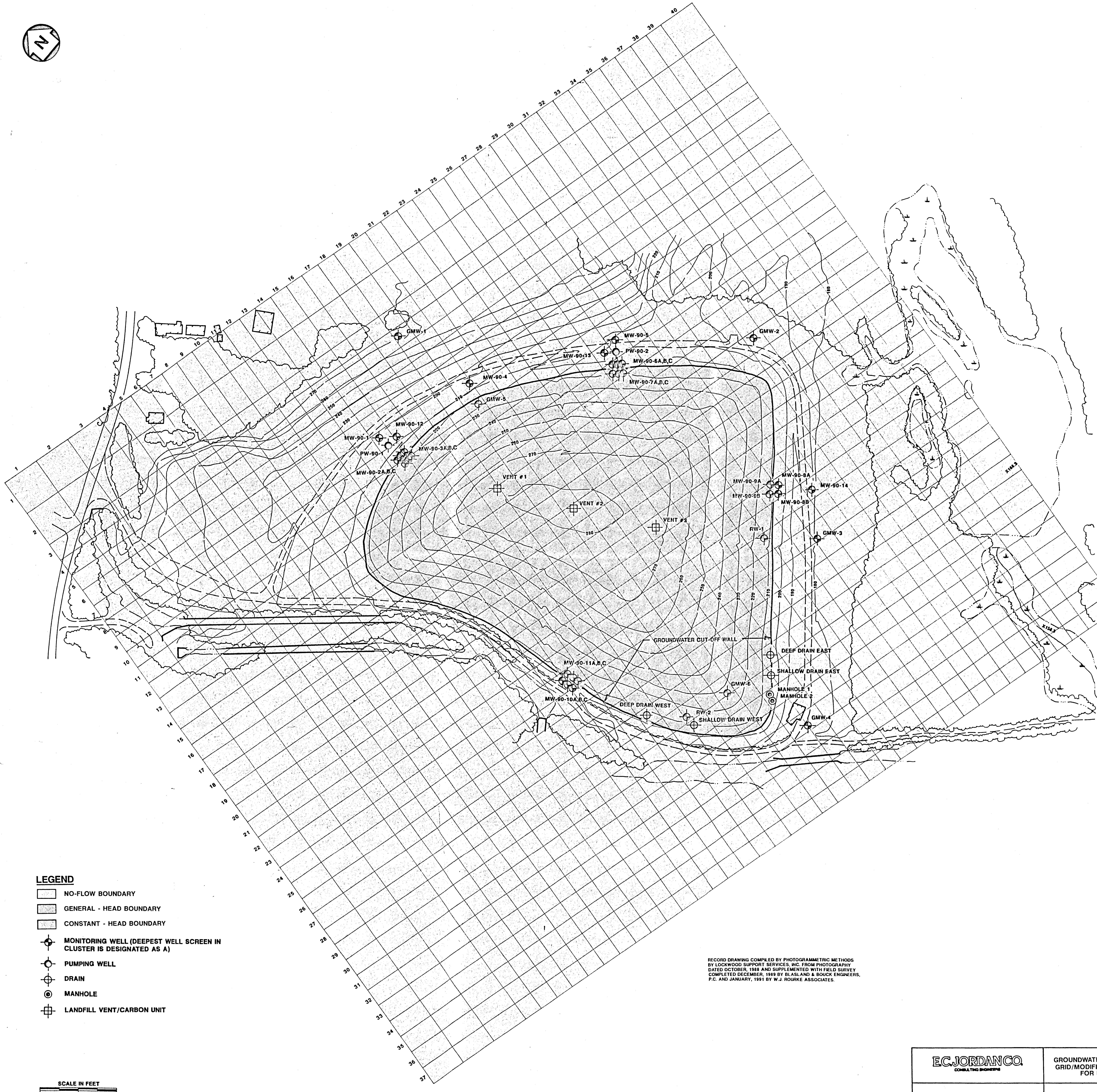
RECORD DRAWING COMPILED BY PHOTOGRAMMETRIC METHODS BY LOCKWOOD SUPPORT SERVICES, INC. FROM PHOTOGRAPHY DATED OCTOBER, 1988 AND SUPPLEMENTED WITH FIELD SURVEY COMPLETED DECEMBER, 1989 BY BLASLAND & BOUCK ENGINEERS, P.C. AND JANUARY, 1991 BY W.J. ROURKE ASSOCIATES.

E.C.JORDAN CO.
CONSULTING ENGINEERS

GROUNDWATER FLOW MODEL GRID/MODIFIED BOUNDARIES FOR LAYER 1

NYSDEC SUPERFUND STANDBY CONTRACT
KINGSBURY LANDFILL SITE

FIGURE 6-4



RECORD DRAWING COMPILED BY PHOTOGRAMMETRIC METHODS BY LOCKWOOD SUPPORT SERVICES, INC. FROM PHOTOGRAPHY DATED OCTOBER, 1988 AND SUPPLEMENTED WITH FIELD SURVEY COMPLETED DECEMBER, 1989 BY BLASLAND & BOUCK ENGINEERS, P.C. AND JANUARY, 1991 BY W.J. ROURKE ASSOCIATES.

E.C.JORDAN CO. CONSULTING ENGINEERS	GROUNDWATER FLOW MODEL GRID/MODIFIED BOUNDARIES FOR LAYER 2
NYSDEC SUPERFUND STANDBY CONTRACT KINGSBURY LANDFILL SITE	FIGURE 6-5

File on eDOCs X Yes ______ No

Site Name Kingsbury Landfill

Site No. 558008

County Washington

Town Kingsbury

Foilable X Yes ______ No

File Name 1991-12-01. Hydro Report Vol. 9

Scanned & eDOC ______