

**Groundwater Flow
And
Solute Transport Modeling
At
A State Superfund Site
Northeastern St. Louis County, Minnesota**

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ABSTRACT

This study identifies hydrogeologic and contaminant transport parameters in a glacial till environment. The study site was a mixed solid waste land disposal site from 1972 to 1990.

A shallow, unconfined aquifer overlies Lower Precambrian bedrock. Saturated thickness varies from 5 to 20 feet, and the average linear groundwater velocity is 12 to 15 feet per year. The primary downgradient receptor is the Bear Island River.

A finite-difference numerical flow model, calibrated to observed water table elevations, provided hydraulic head and aquifer parameter data for a solute transport model. Dispersion coefficients were determined by calibration of the transport model to observed chloride concentrations. The model was used to predict future plume configurations for benzene, methylene chloride, and tetrahydrofuran, using retardation factors computed from measured aquifer material organic carbon content. Retardation factor for chromium was determined by model calibration to observed concentration.

This study found that the chloride and tetrahydrofuran plumes will reach the Bear Island River by year 2010. Benzene, methylene chloride, and chromium plumes are considerably more restricted in areal extent. None of these contaminants is expected to create a significant detrimental effect on river water quality.

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INTRODUCTION

General

Historically, interest in groundwater has focused more on quantity than quality. The importance of groundwater as a natural resource has been recognized for generations, but it is only in the past ten to 15 years that the severity of the groundwater contamination problem has been identified. Groundwater contamination is relatively widespread, and it has become a key environmental issue. Groundwater contamination by toxic or hazardous chemicals has been reported in every state in the United States (Moore and Jones, 1987). Contaminants typically detected include chloride, nitrite, trace metals, organics and pesticides. The list continues to grow as new data become available at both the federal and state levels. For example, in 1986 the Minnesota Department of Health issued a list of Recommended Allowable Limits for 73 contaminants in drinking water; the revised 1991 list includes 196 contaminants.

Concern over contaminated groundwater is not limited to drinking water supplies. Since rivers and lakes are discharge areas for groundwater, the impact of contaminated groundwater on surface water resources is also of vital interest.

Purpose of Study

Collection and evaluation of reliable field data are a necessity, as attempts are made to better understand the mechanisms governing groundwater flow and contaminant transport (Anderson, 1979, 1983; Hamilton, 1982).

Landfills are identified as a possible principal source of pollutants resulting in groundwater contamination. In this study, field investigations were conducted at a mixed municipal landfill, which is on the Minnesota Superfund List and, in 1988, was placed on the Environmental Protection Agency (EPA) list of federal lands at which possible groundwater contamination exists.

The primary goals of this study are to:

- Identify hydrogeologic parameters in glacial sediments.
- Identify contaminant transport parameters in drift-covered terrain.
- Establish numerical flow model calibrated to site-specific data.
- Establish contaminant transport model for selected inorganics, organics and trace metals.
- Determine the probable impact on downstream surface water resources, utilizing the flow and contaminant transport models as predictive tools.

Study Site

The study site is located in northeastern St. Louis County, Minnesota (Figure 1). The site is in Section 5, Township 61 North, Range 21 West, adjacent to County Highway 21, approximately midway between the cities of Ely and Babbitt (Figure 2). The study area is located near the Bear Island River as it flows from Bear Island Lake to One Pine Lake.

A mixed municipal landfill is located on the U.S. Government-owned site. The landfill was originally developed in 1972 and operated by a private company. The Northwoods Landfill Authority was formed in 1978 to oversee the landfill operation, under a Special Use permit issued by the U.S. Forest Service. The landfill was permitted by the Minnesota Pollution Control Agency (MPCA). The St. Louis County Comprehensive Solid Waste Management Plan (Evavold, 1988) estimates that 7,000 tons (17,500 cubic yards in-place) of waste were placed in the landfill annually over the past several years. The landfill was closed as a solid waste disposal facility in May 1990.

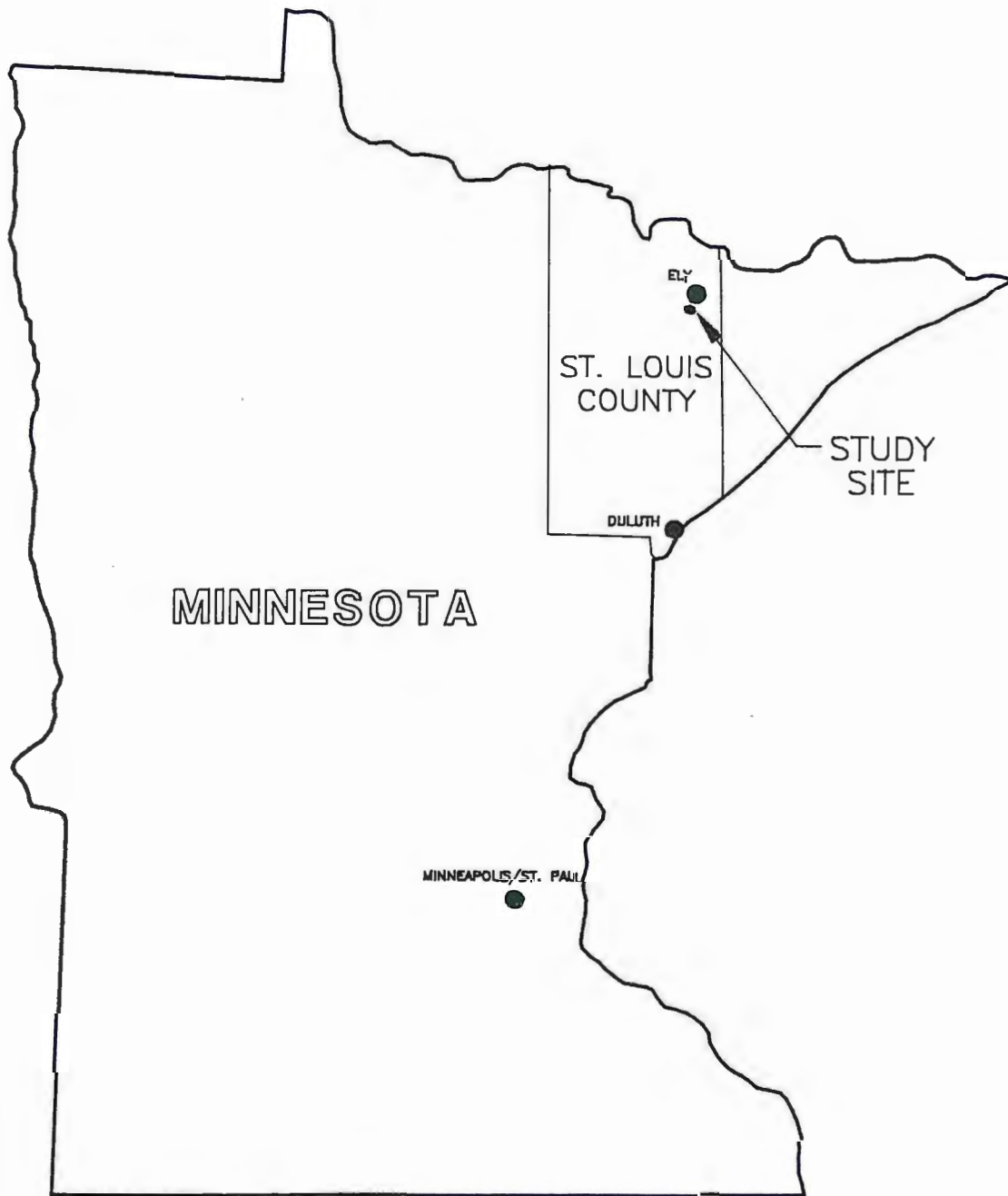


Figure 1. Study site location map.

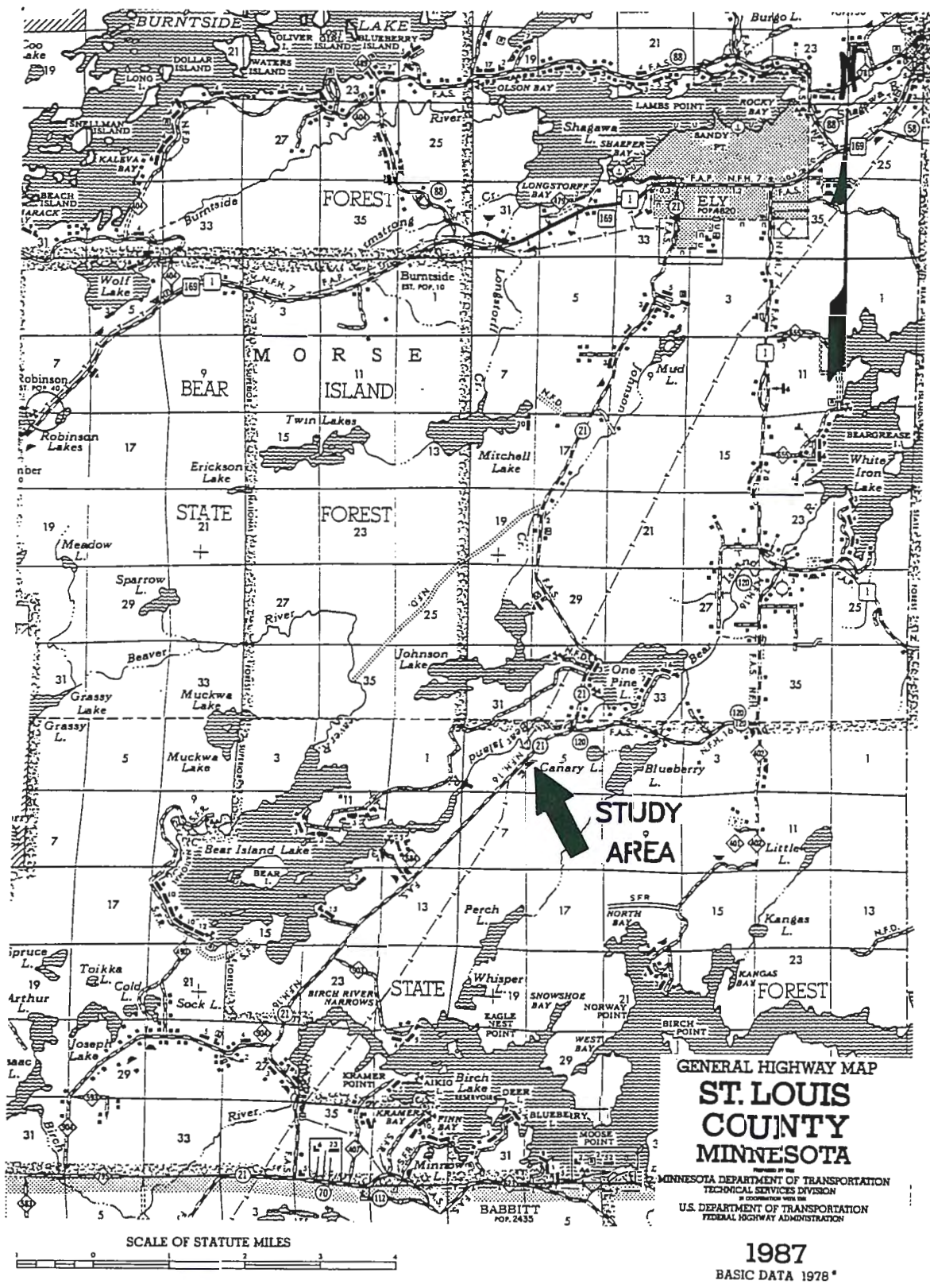


Figure 2. Study site area map.

Previous Investigations

Previous site-specific investigations to identify subsurface conditions include a limited number of shallow auger soil borings in 1972; and four borings, in which two-inch diameter monitoring wells were installed, in 1981. The four 1981 borings encountered bedrock at depths ranging from 9.5 feet to 23 feet. Each boring was also cored ten feet into bedrock. Three additional monitoring wells were installed in backhoe pits in 1981, at depths ranging from five feet to 10.5 feet below the ground surface.

The author has participated in previous studies to evaluate hydrogeologic and water quality conditions at the landfill. These include an existing condition study (Evavold and Madson, 1984), a leachate and water quality study (Evavold and Madson, 1985), and a hydrogeologic and site assessment study (Evavold and Reichhoff, 1988). The 1985 study recommended the installation of three additional monitoring wells, which were installed in 1986. An engineering report and project plans were prepared under the author's direction for a proposed vertical expansion of the landfill (Evavold and Reichhoff, 1988). The engineering report includes leachate generation computations.

Considerable information regarding area bedrock and glacial geology is found in *Geology of Minnesota: A Centennial Volume* (Sims and Morey, editors, 1972). Characteristics of regional glacial drift were compiled and described by Winter, et al (1973). Information on area water resources has been prepared by Cotter, et al (1965) and Ericson, et al (1976).

Field Methods

Geotechnical exploration and installation of five additional monitoring wells were conducted in March 1988, as part of this study. Soil samples were collected, using a two-inch (outside diameter) split-barrel sampler, driven into the soil by a 140-pound weight falling 30 inches [American Society of Testing Materials (ASTM): D1586]. Samples were visually and manually classified in the field, in accordance with ASTM: D2488.

Field hydraulic conductivity tests were conducted in April 1988 at the new wells and at four previously existing wells. The field procedure involved bailing the wells and measuring the recovery rate. Measurements were also made for field pH, specific conductance, temperature and static water level at each well, and samples were collected at each well for laboratory chloride analysis.

In early June 1988, groundwater samples were collected at nine monitoring wells for laboratory analysis for chloride, volatile organic compounds, lead, chromium, cadmium and mercury. Metals samples, except mercury, were filtered in the field through a 0.45 micron filter. Field measurements were made at each well for pH, specific conductance, temperature and static water level. Follow-up groundwater sampling for volatile organics, at three wells immediately downgradient of the landfill, was conducted in July 1988. Additional samples were collected at selected locations for chloride analysis, and static groundwater levels were measured at all wells at the site.

Field pH, specific conductance and water level measurements were made at all wells at the study site in early September 1988. Groundwater samples were collected for laboratory analysis for chloride at each well, for selected metals analysis at six wells, and for volatile organics at four wells. A leachate sample was also collected for volatile organic and selected metals analyses.

Laboratory Methods

All groundwater analyses, except for mercury, were made at commercial laboratories, in accordance with EPA or other standard methods. The minimum detection limit desired for mercury (nanograms/liter) was lower than attainable by established procedures used in commercial laboratories. However, the EPA Water Quality Laboratory in Duluth has established a procedure for testing at these levels, and the mercury samples were analyzed at the EPA facility.

Soil classification, in accordance with the United Soil Classification System (ASTM: D2487) was verified in the laboratory. Grain size distribution was

determined by U.S. Standard Sieve analysis and by hydrometer methods on the minus 200 (≤ 0.074 mm) sieve material. Laboratory hydraulic conductivity was determined for two remolded samples, using the Falling Head Permeability Test Method.

TOPOGRAPHY

Area topography shows the influence of Pleistocene glaciation. Differential erosion of the bedrock occurred during glaciation in areas of relatively soft rock and along joints and fractures. This is evidenced by the relatively rapid changes in elevation, the southwest-northeast bedrock dominated ridges, and the numerous lakes which formed in the eroded areas shown in Figure 3. As the glacier stagnated, eroded materials were deposited in low areas creating the hummocky terrain.

Ground elevations range from about 1500 feet (mean sea level) northeast of the study site to less than 1409 feet beneath One Pine Lake and Blueberry Lake. The Bear Island River, lying north-northeast of the study site, flows from Bear Island Lake (elevation 1418) to One Pine Lake (elevation 1409). Surface drainage from the study site discharges into the Bear Island River.

Figure 4 is a topographic map of the landfill site, depicting the ground surface as of November 1988. Since landfilling of mixed municipal solid waste began in 1972, the site has been filled to approximately elevation 1480, a maximum fill thickness of 30 to 40 feet.

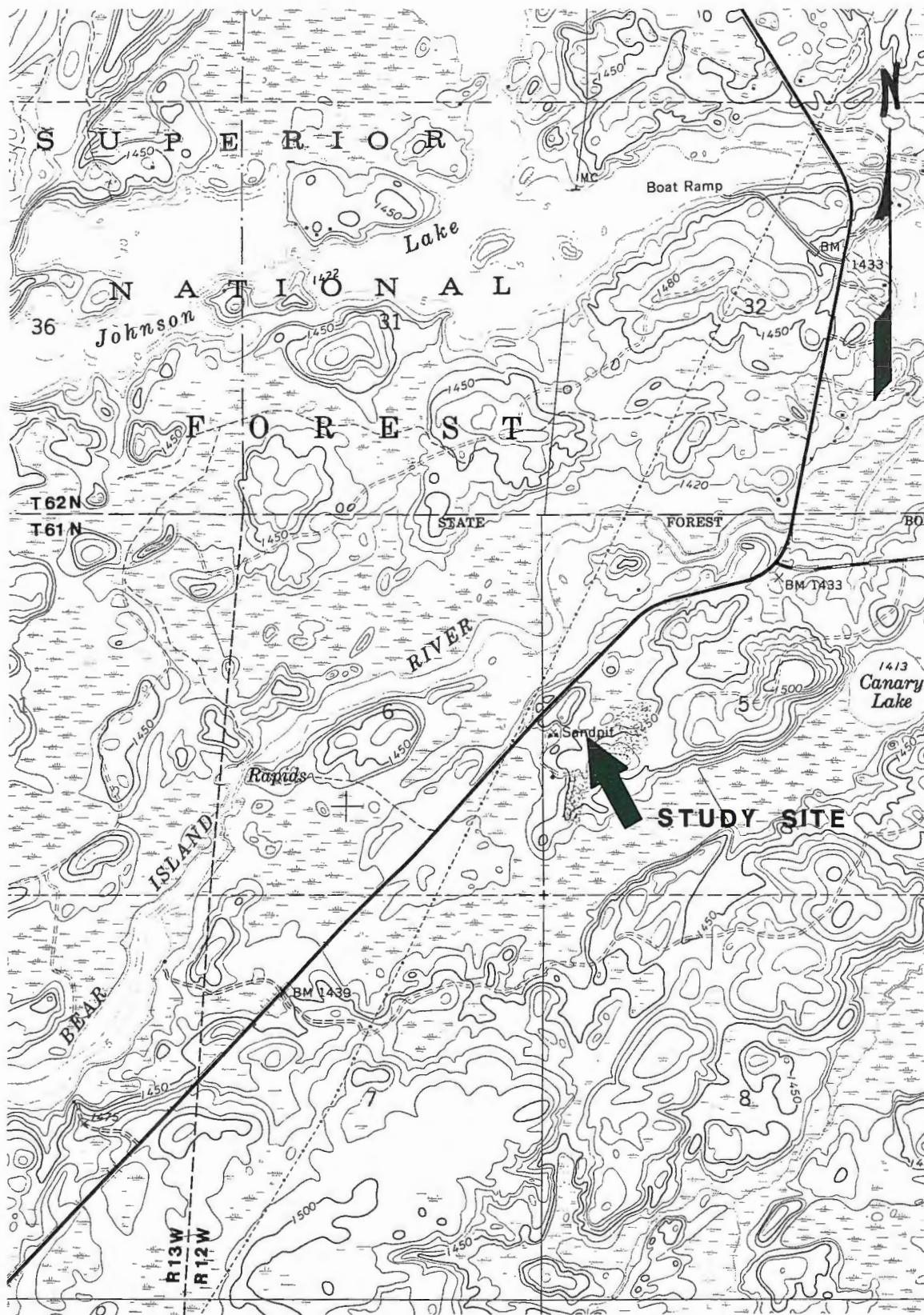


Figure 3. Area topographic map.

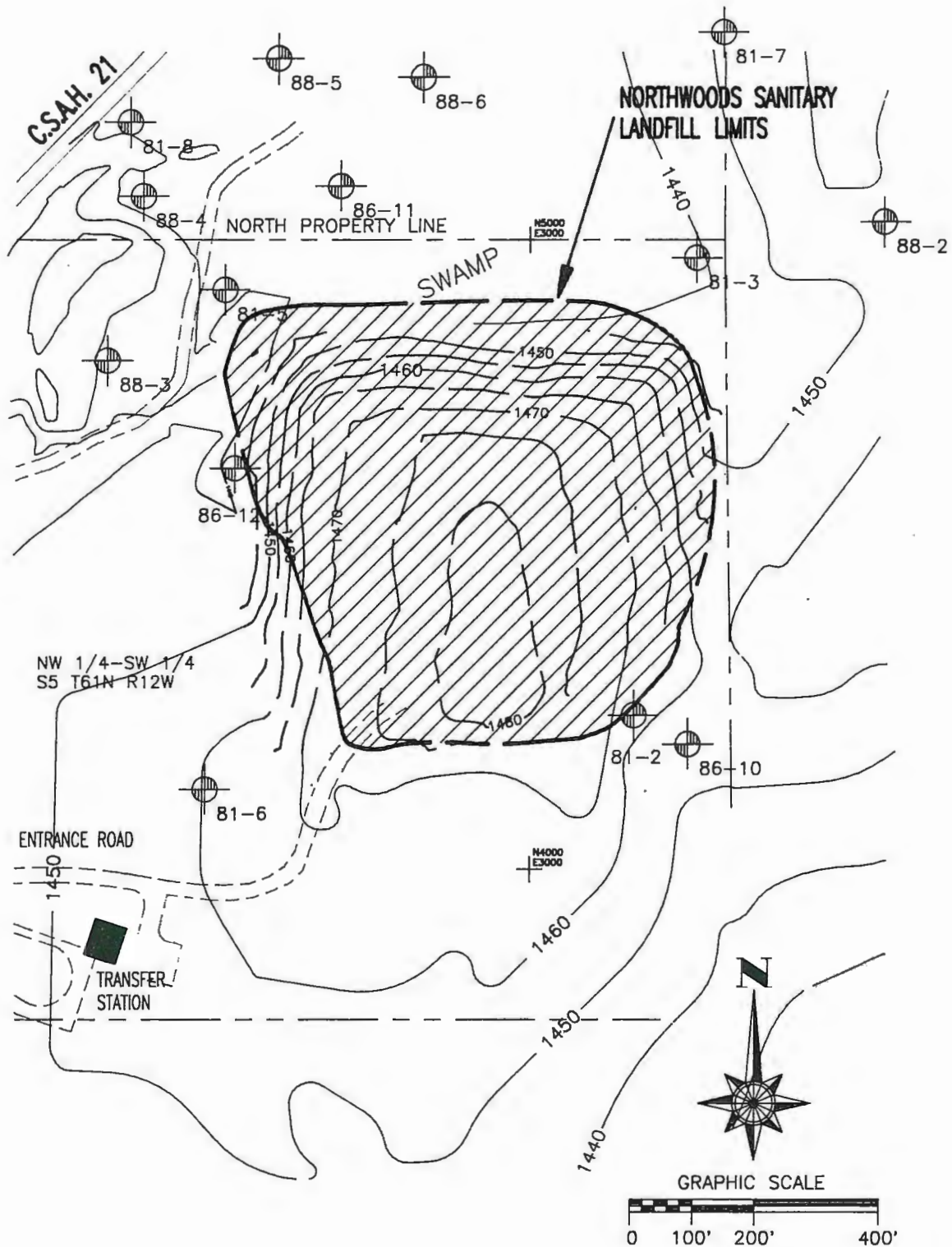


Figure 4. Study site topographic map.

GEOLOGY

Bedrock Geology

The Northwoods Landfill Service Area is part of the Vermilion Geologic District. This district is one of several large complexes of metamorphosed igneous and sedimentary rocks in Minnesota and Canada known as "Greenstone Belts." These greenstone belts generally consist of northeast trending bands of volcanic and sedimentary rocks enclosed by granite batholiths. They represent a complex history of volcanic activity, folding, faulting, intrusion and metamorphism which occurred during the Algoman Orogeny, approximately 2,700 million years ago (Sims and Morey, 1972).

The oldest formation in the Northwoods area is the Ely Greenstone, consisting of a thick sequence of submarine basaltic lava flows and volcanic debris containing lenses of iron formation. The sequence generally dips steeply to the north, but in the Ely area it is tightly folded into an east-plunging syncline, which has been displaced eastward by faulting.

Two large intrusive igneous complexes border the greenstone belt in the service area. The Burntside Lake fault marks the southern edge of the Vermilion Granitic Complex. The Giants Range Batholith continues northeastward from the Mesabi Iron Range to just south of Ely. The Giants Range Granite consists of medium-grained, pink granites and medium-grained gray monzonites. Where the younger granitic rocks contact the Ely Greenstones, they have thermally altered the greenschist to amphibolite. The granites are less resistant to erosion than the greenstones, and form the basins of White Iron Lake and Bear Island Lake, and a portion of Birch Lake. All of the formations in the area have a distinct northeasterly trend.

The study site is underlain by the Lower Precambrian Giants Range Granite as shown in Figure 5 (Ojakangas and Matsch, 1982; Sims and Viswanathan, 1972). The site lies approximately three-fourths of a mile west of the Camp

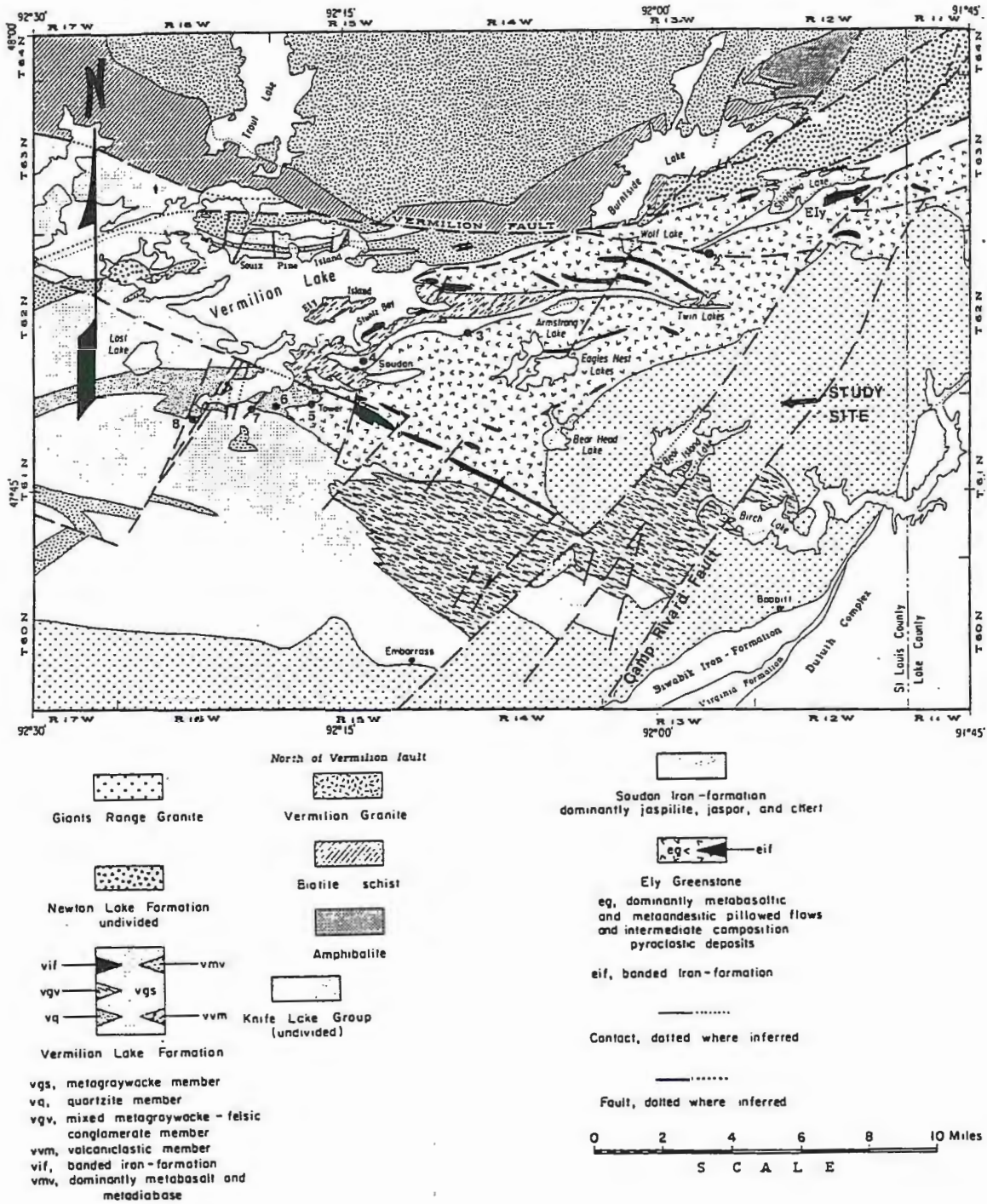


Figure 5. Bedrock Geology (after Ojakangas and Matsch, 1982).

Rivard fault. Bedrock cores taken at the site in 1981 exhibited a medium-grained, granitoid texture with some lineation and foliation (Evavold and Madson, 1984). This is consistent with Giants Range granite.

Regional bedrock topography (Olsen and Mossler, 1982) indicates the landfill site is in a bedrock trough, trending northeast-southwest, with elevations of 1500 or greater to the southeast and northwest. Site specific data indicates the bedrock immediately beneath the landfill slopes in a north-northwesterly direction from a bedrock ridge, trending in a northeast-southwest direction, located near the southeast edge of the landfill. Bedrock elevation along the ridge at the south edge of the landfill is 1445 to 1450 feet (mean sea level). At the north edge of the landfill, the bedrock surface elevation varies from 1410 to 1417 feet. The bedrock surface continues to decrease in elevation toward Bear Island River. Bedrock surface elevation between the river and the landfill is estimated to be 1385 to 1400 feet.

Glacial Geology

Several advances of the Rainy Lobe occurred during the Wisconsin Glaciation (Wright, 1972). Between approximately 30,000 and 15,000 years ago, the Rainy Lobe advanced from the northeast, concurrent with an advance of the St. Croix phase of the Superior Lobe. Subsequently, during the Automba Phase, a weaker advance of the Rainy Lobe formed the Vermilion Moraine (Figure 6). The Vermilion Moraine is the most prominent glacial feature in the study area. It forms the southwest shoreline of Bear Island Lake, as well as Eagles Nest Lake to the northwest and Birch Lake to the southwest. The Wahlsten Moraine (Figure 7) extends in an east-west direction, intersecting the Vermilion Moraine near Bear Island Lake. Both of these moraines have associated outwash plains.

Much of the surrounding area contains thin and discontinuous glacial deposits. In general, the advancing glacier caused erosion rather than deposition; therefore, deposits occur primarily in moraine areas. As shown in Figure 7, several ice-contact or disintegration moraines formed by stagnant ice

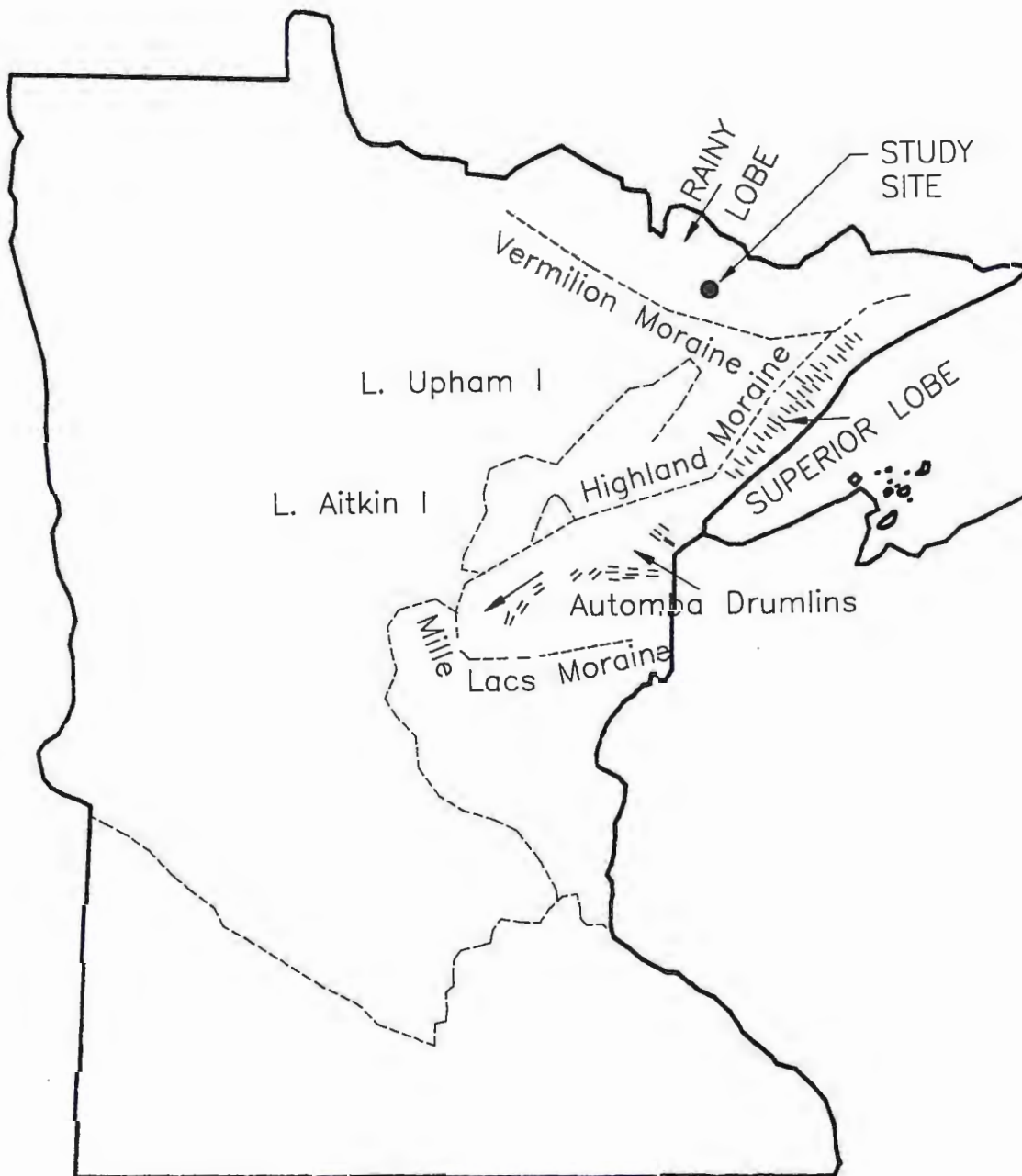


Figure 6. Glaciation, Automba Phase Superior and Rainy Lobes (after Wright, 1972). ■ Page 14

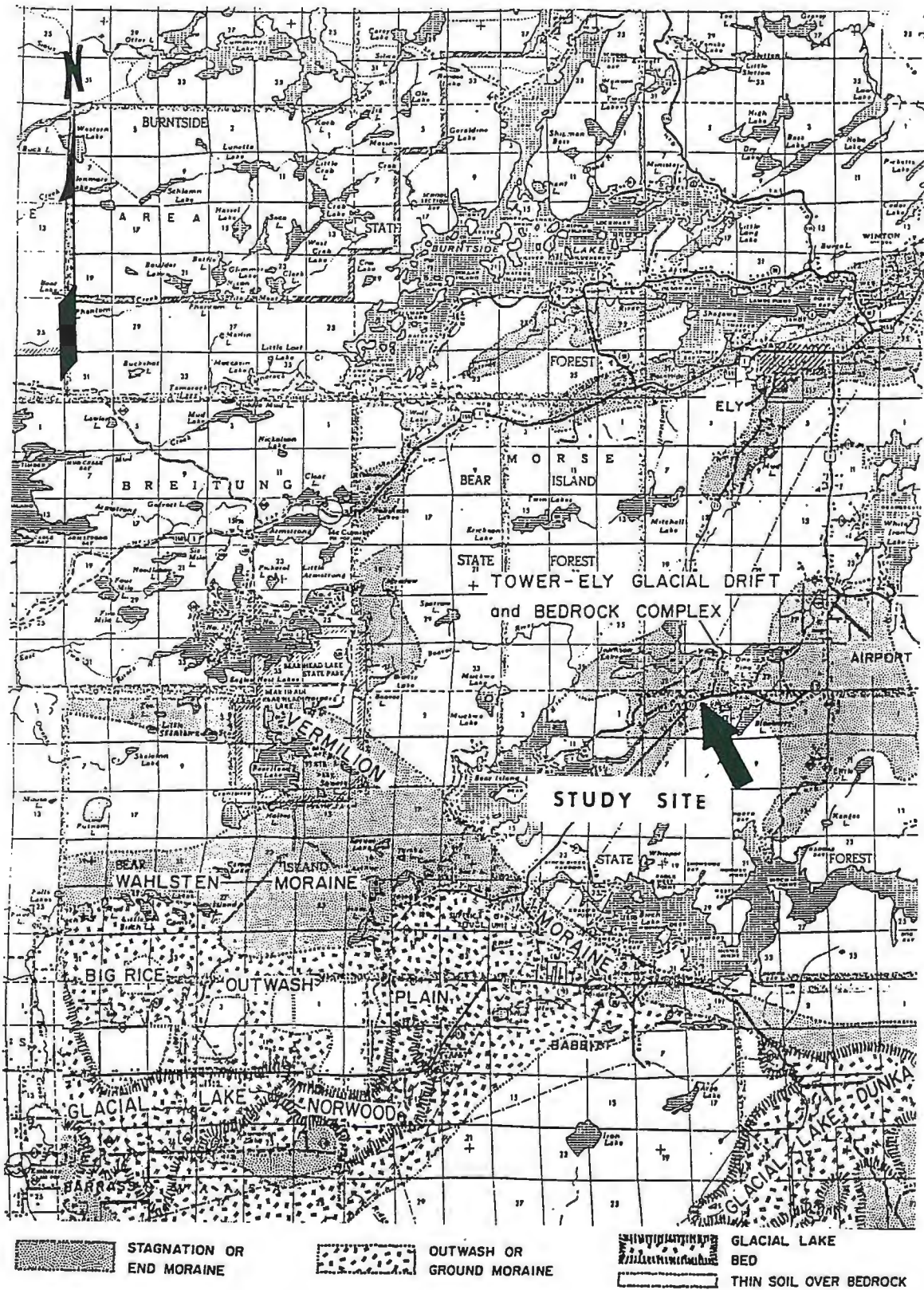


Figure 7. Glacial geologic features (after RREM 1984).

are located north and east of the Vermilion Moraine. These moraines are not as prominent as the end moraine, but exhibit the same "glacial karst" topography, characterized by randomly oriented kames and kettles, relatively high local relief and many lakes. End moraines are generally poorly sorted, composed mainly of glacial till. The disintegration moraines display a higher degree of sorting in localized areas, with rapid changes in soil materials and sorting common over short distances. The outwash plains display a much higher degree of sorting, due to the action of flowing water.

Two glacial lake plains occur south and southwest of the study area. The largest of these is Lake Norwood, in the center of the Big Rice Outwash plain, and Lake Dunka, east of Babbitt (Figure 7). The Lake Norwood sediments consist primarily of sand from Rainy Lobe meltwater (Winter, et al, 1973), with sand and gravel deposits found along the lake limits. The Glacial Lake Dunka plain is covered with red sand and gravels from Superior Lobe meltwater, identified as a North Shore assemblage (Lehr and Hobbs, 1992). Lehr and Hobbs also suggest that subsequent to the recession of the Rainy Lobe from the Vermilion Moraine, meltwater from the Superior Lobe flowed into Lake Norwood, depositing sand and gravel as a delta.

Glacial erosion tends to exaggerate the pre-existing topography. Points of weakness, such as faults, fractures and differences in lithology, are all exploited by the actively eroding glacier. Porous and fine-grained rocks are less resistant to erosion than massive coarse-grained rocks. Depressions have been scoured along the Waasa and Camp Rivard faults to form Bear Island River, Bear Island Lake and White Iron Lake.

Material eroded by the glacier is sometimes entrained and carried along as the glacier advances. Very little mixing transverse to flow occurs; therefore, materials contained in the till can usually be attributed to an upglacier source. Various lobes of the Laurentide ice sheets contain characteristic tills. Tills of the Rainy Lobe contain large amounts of crystalline material derived from the Precambrian Shield.

Distance of transport has an effect on the distribution of grain sizes in glacial tills. Crushing and abrasion are the normal processes in transforming rock to till. Exceptions to this exist if the source rock is chemically vulnerable, such as a calcareous or pre-weathered type. Most tills consist of two components: clasts or rock fragments and the till matrix consisting of fine mineral fragments. Over distance, the crushing action reduces the coarse fraction to a characteristic grain size. A till containing a large percentage of coarser materials is considered immature and relatively close to its source rock. The amount of transport needed to reduce a source rock to its minimal grain size is dependent on lithology. For example, dolomite takes less distance than granite (Sugden and John, 1979). Dreimanis and Vagners (1971) observed that the till matrix nodes are restricted to certain particle-size grades which they termed "terminal grades." In an investigation of Wisconsin-age tills in Ontario, they found that the terminal grade is determined not only by the resistance of a mineral to crushing and abrasion, but also by the size and shape of the mineral in the rocks.

Description of the coarse nature of glacial deposits in the study area (Winter, et. al., 1973; University of Minnesota, 1981) suggests that these are relatively immature till units, since they contain a relatively low percentage of silt and clay sized mineral fragments. The Vermilion Moraine till fits this description, as it is generally brown colored, coarse-textured and bouldery, containing gabbro, granite and greenstone from the Precambrian Shield.

The study site is located within a stagnation moraine (Figure 7). Soil borings taken in glacial deposits at the site confirm the presence of a coarse-textured till with cobbles and boulders. Boring locations are shown in Figure 8 and boring logs are summarized in Figure 9. Using the Unified Soil Classification System, the till is generally dense to very dense silty sand (SM) and poorly graded sand (SP), with some sandy silt (ML), for that portion passing the No. 40 sieve (4.8 mm). The percent passing the No. 200 sieve (.074 mm size) varies from less than ten percent to 47 percent. Most of the minus 200 sieve material is of silt size rather than clay. It appears that cobbles and boulders are found at any

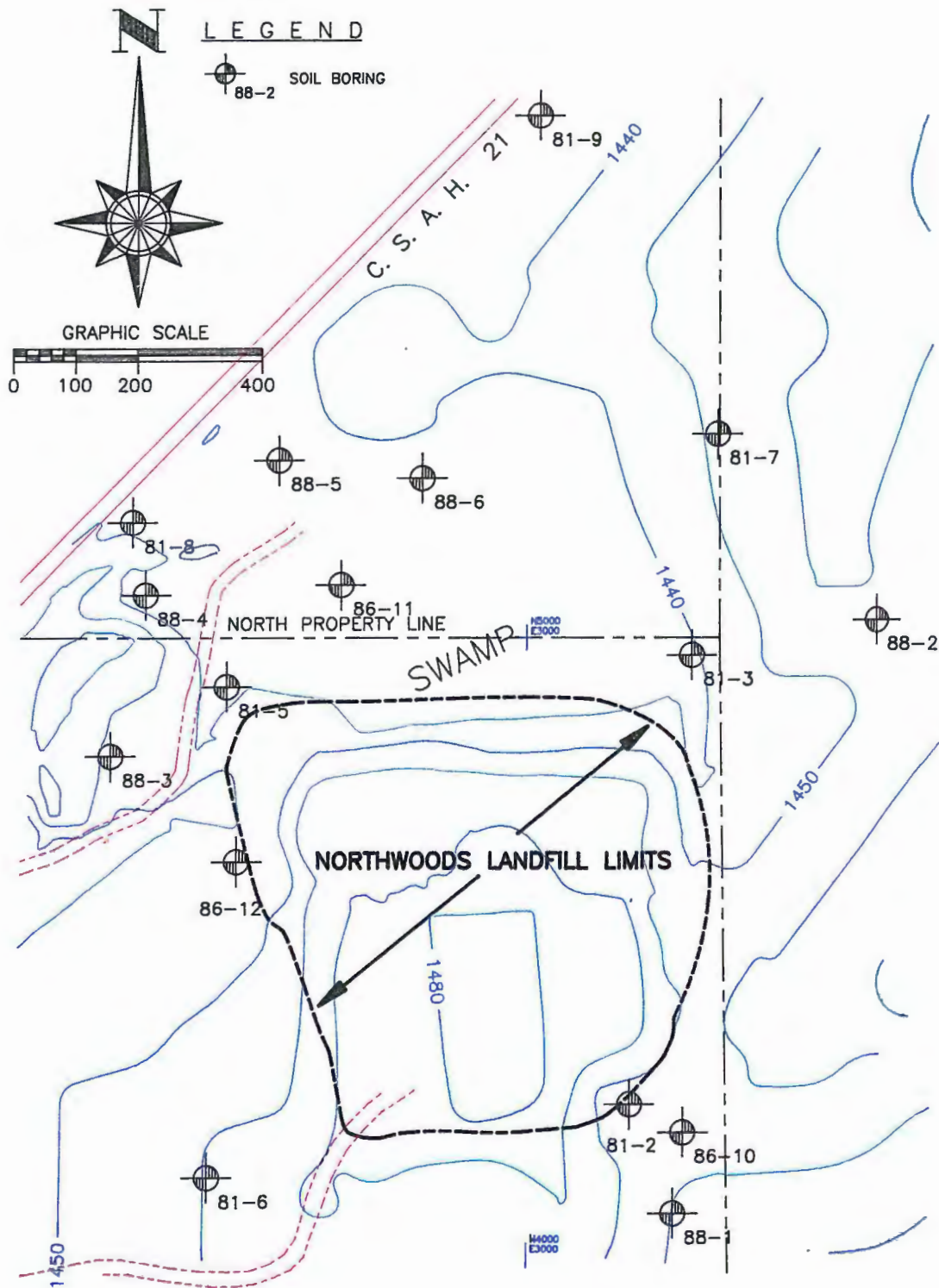


Figure 8. Soil boring location map.

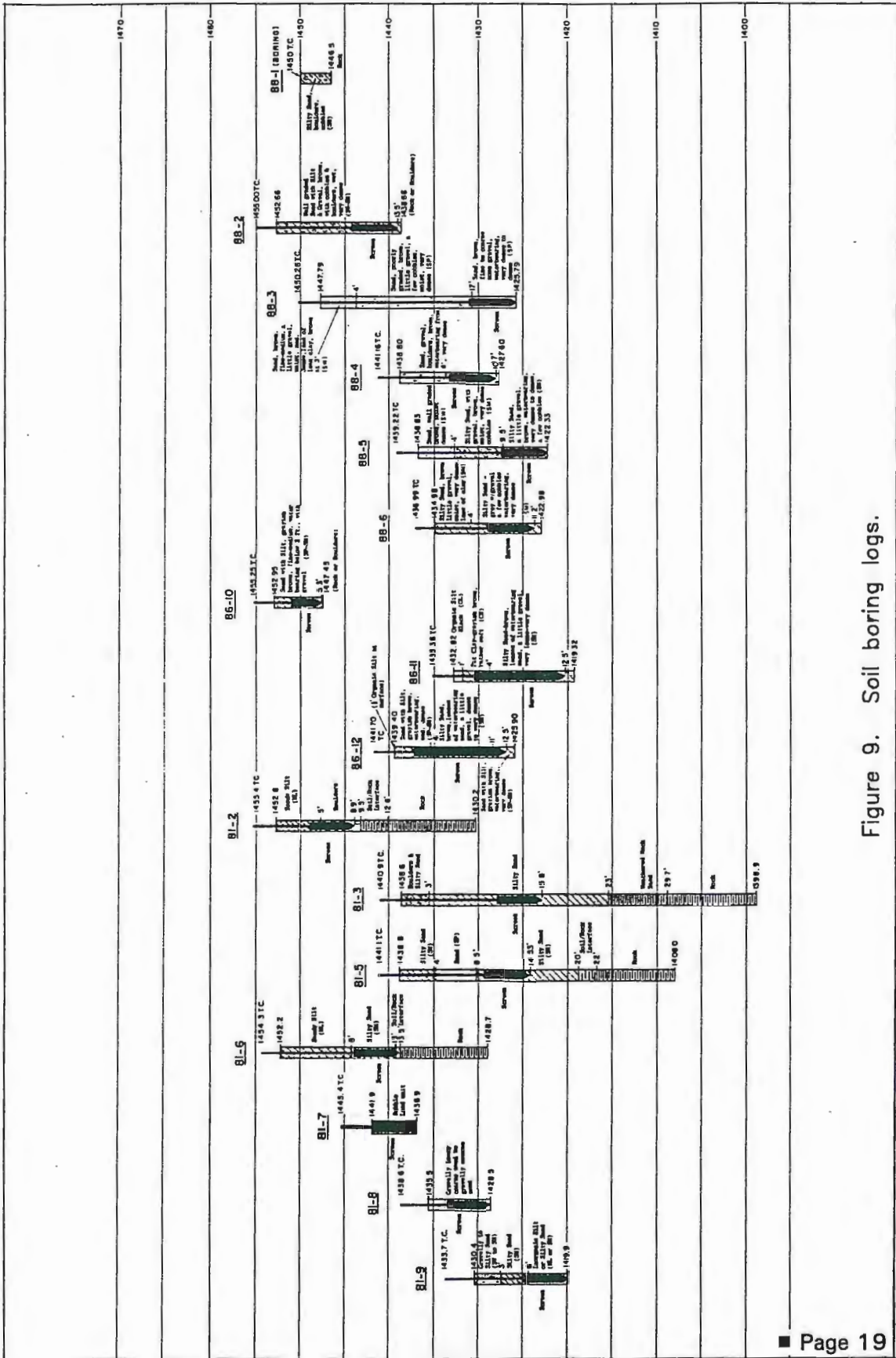


Figure 9. Soil boring logs.

depth, generally varying in size from three inches to twelve inches, although boulders in excess of one cubic yard in volume have been excavated at the site. Depth of glacial till overlying the bedrock varies from approximately five feet, immediately south of the landfill limits, to more than 20 feet at the north edge of the landfill.

HYDRAULIC CONDUCTIVITY

Hydraulic conductivity represents a measure of the rate at which a liquid will move through a permeable medium. It is a function of the grain-size distribution of the porous medium and the specific weight and dynamic viscosity of the liquid. Hydraulic conductivity is the constant of proportionality (K) in Darcy's Law:

$$Q = KA \left[\frac{dh}{dl} \right]$$

where Q is discharge (L^3/t).
K is hydraulic conductivity (L/t).
A is flow area (L^2).
dh/dl is hydraulic gradient (dimensionless).

Typical units for hydraulic conductivity are centimeters per second (cm/sec) and feet per day (feet/day).

Hydraulic Conductivity Estimates

Because hydraulic conductivity is related to grain-size distribution, an approximation of conductivity values can be determined using a representative grain-size diameter. A simple empirical relationship (Freeze and Cherry, 1979) relates hydraulic conductivity to the effective grain size as follows:

$$K = A (D_{10})^2$$

where K is hydraulic conductivity (L/t).
 D_{10} is effective grain size (L), defined as the grain-size diameter at which 10 percent of the soil particles are finer.
A is coefficient with a value of 1.0 for K in cm/sec and D_{10} in millimeters for well-sorted sand; for poorly sorted, very fine sand, $A = 0.4$.

This relationship was developed for uniform sands; however, it is useful in providing an approximation of hydraulic conductivity values for other materials.

In this study, grain-size distribution data were collected for a silty sand sample (boring number 88-5), medium sand (88-3) and well-graded, gravelly sand with silt (88-2). Grain-size distribution data were also collected for silty sand and silty sand with gravel samples in a previous site study conducted by the author. A measure of the degree of sorting in a sediment can be obtained from the uniformity coefficient, which is the ratio of the grain-size at which 60 percent of the sediment is finer to the grain-size at which 10 percent of the sediment is finer ($C_u = D_{60}/D_{10}$). A uniformity coefficient less than four indicates a well sorted material and a value greater than six is indicative of poorly sorted sediment. The uniformity coefficient for six samples from the study site ranges from four to 128. All samples, except possibly boring 88-3, are poorly sorted. Table 1 contains a summary of the sand and silt-clay content of the six samples and estimated hydraulic conductivity, based on effective grain size, D_{10} . The arithmetic mean hydraulic conductivity of the six samples is 3.2×10^{-3} cm/sec. The mean value for the four silty sand samples is 2.3×10^{-4} cm/sec. The estimated hydraulic conductivity of the six samples varies from 3.6×10^{-5} to 1.6×10^{-2} cm/sec, more than 2.5 orders of magnitude. This is frequently a typical range within a given hydrogeologic unit. The arithmetic mean tends to weight more heavily the more permeable values, and Fetter (1988) suggests that the geometric mean may provide a better representation of the average hydraulic conductivity of a sediment. The geometric mean hydraulic conductivity of the six samples is 5.7×10^{-4} cm/sec, and the geometric mean value for the four silty sand samples is 1.7×10^{-4} cm/sec.

TABLE 1
ESTIMATED HYDRAULIC CONDUCTIVITY
BASED ON EFFECTIVE GRAIN SIZE, D_{10}

Location	Sample Depth (feet)	Unified Soil Classification	Percent Sand (4.8-0.074 mm)	Percent Silt & Clay (<0.074 mm)	Effective(1) Grain-size (D_{10} , mm)	Hydraulic(2) Conductivity (cm/sec)
Monitoring Well:						
86-10	1-5	SM, silty sand with gravel	48	17	0.028	3.1×10^{-4}
86-11	4-12	SM, silty sand	53	47	0.0095	3.6×10^{-5}
86-12	4-12	SM, silty sand with gravel	51	31	0.024	2.3×10^{-4}
88-2	12-13.5	SW-SM, well graded gravelly sand with silt	56	10	0.075	2.2×10^{-3}
88-3	14.5-16	SP, poorly graded medium sand	86	3	0.2	1.6×10^{-2}
88-5	4.5-6	SM, silty sand with gravel	56	14	0.03	3.6×10^{-4}

(1) Grain-size diameter at which 10 percent of the soil particles are finer.

(2) Using coefficient $A = 0.4$ in formula: $K = A(D_{10})^2$

Field Hydraulic Conductivity

In situ hydraulic conductivity tests were performed at several of the monitoring wells at the site. Monitoring well locations are shown in Figure 10. All of the monitoring wells are two inches in diameter. The wells installed in 1988, as part of this research project, are constructed of stainless steel with five-foot long stainless steel screens. Wells 86-11 and 86-12 are constructed with ten-foot long screens and well 86-10 has a two-foot screen. All other well screens are five feet long.

The Hvorslev (1951) Method for determining hydraulic conductivity was used to analyze the collected field data at all wells. The Bouwer and Rice (1976) procedure was also used, for comparison purposes. Data and graphs for both methods are included in Appendix A.

In conducting field tests on small diameter wells, a volume of water is removed (bail test) or added (slug test) to the well. The slug test can also be conducted by inserting a slug, such as a capped pipe, into the well, which is equivalent to adding a volume of water to the well equal to the slug volume. Immediately upon completing bailing or slugging, the water level in the well is measured to determine the distance the water surface is below or above the static level. Water level measurements are collected at timed intervals as the water level returns to equilibrium conditions.

The Hvorslev theory is that inflow or outflow at the well is proportional to the hydraulic conductivity and the unrecovered head. A plot of recovery data should show an exponential decline in recovery rate with time. Defining H_0 as the maximum drawdown or unrecovered head at time zero and H as the residual drawdown or head at time t , a plot of the ratio H/H_0 (logarithm scale) versus time (linear scale) should approximate a straight line. The basic time lag (T_0), as defined by Hvorslev (1951), is the time required for equalization of head at the original inflow rate. T_0 is determined from the plot as time (t) at which

H/H_0 equals 0.37 ($\ln H/H_0 = -1$). Hydraulic conductivity is calculated as follows:

$$K = \frac{r_c^2 \ln(L/R)}{2LT_0}$$

where K is hydraulic conductivity (L/t).
 r_c is radius of well casing (L).
 L is length of well screen (L).
 R is radius of well screen (L).
 T_0 is basic time lag (t).
 ln is natural logarithm.

It is noted that the Hvorslev Method was originally used to analyze data from a point piezometer, where the casing opening or screen is fully submerged in the saturated aquifer. In this study, the effective screen length used in the Hvorslev Method is that portion of the screen that is within the saturated zone of the aquifer.

Bouwer and Rice (1976) present a procedure for determining hydraulic conductivity for a well in an unconfined aquifer, after removing a slug of water from the well. Hydraulic conductivity is computed using the following equation:

$$K = \frac{r_c^2 \ln \left[\frac{R_e}{r_w} \right]}{2L} \left[\frac{1}{t} \right] \ln \left[\frac{y_0}{y_t} \right]$$

where K is hydraulic conductivity (L/t).
 r_c is radius of well casing (L).
 r_w is radius distance from center of well to undisturbed aquifer (L).
 R_e is effective radial distance over which residual drawdown (head) is dissipated (L).
 L is saturated screen length (L).
 y_0 is maximum drawdown or head (L).
 y_t is residual drawdown or head at time t (L).

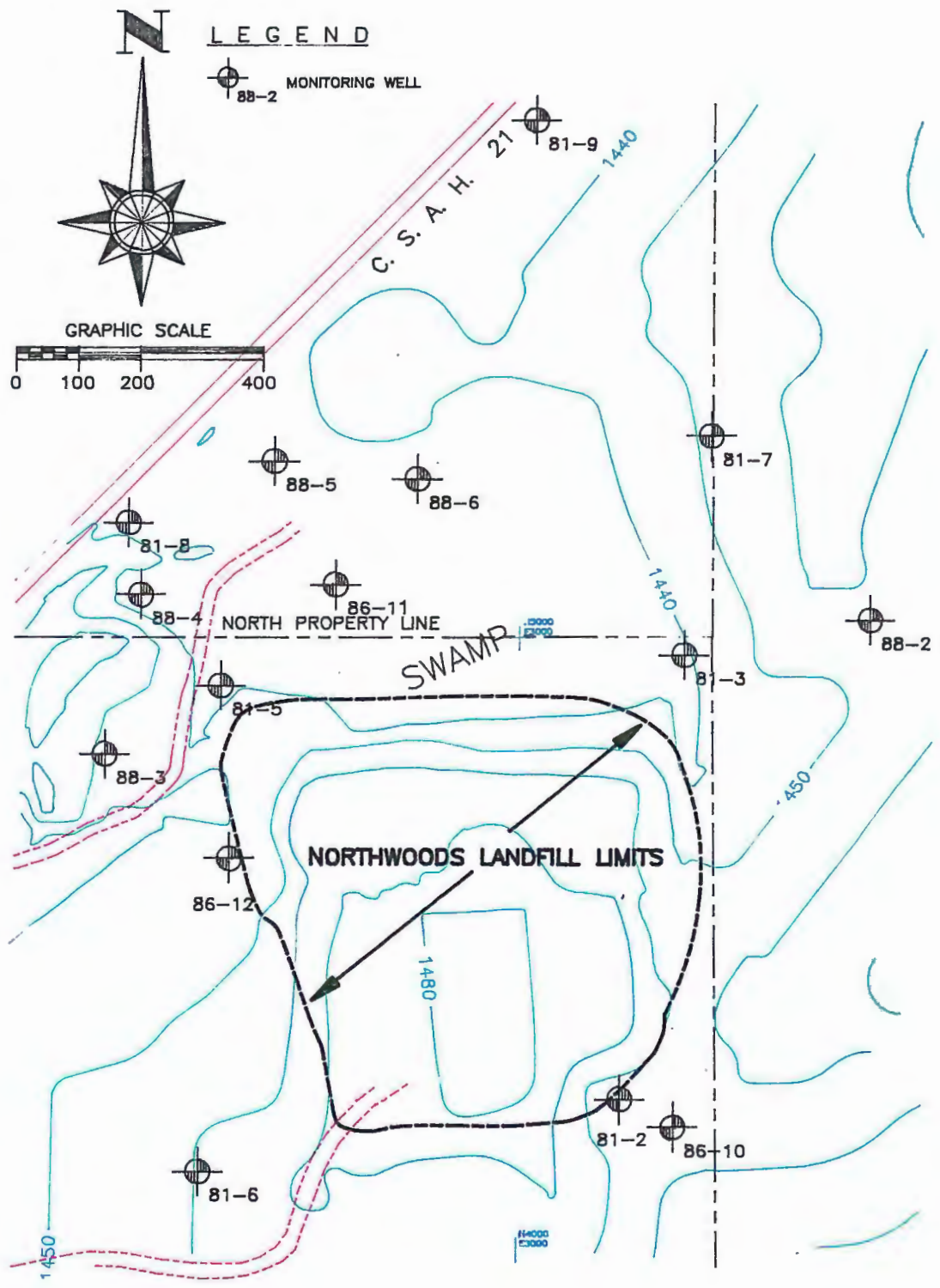


Figure 10. Monitoring well location map. ■ Page 26

The reader is referred to Bouwer and Rice (1976) for discussion on determination of the effective radial distance, R_e . In the Bouwer and Rice procedure, a plot of residual drawdown (logarithmic scale) versus time (linear scale) is made and a best-fit straight line of the data yield y_t and t values for determining:

$$\left(\frac{1}{t} \right) \ln \left(\frac{y_o}{y_t} \right)$$

Table 2 contains a classification of the material in which each well screen is located, and the results of field hydraulic conductivity test analyses. Note that the largest hydraulic conductivity is at well 86-11, which is screened in silty sand with lenses of sand, and a little gravel, with in situ compaction identified as very loose to very dense.

The range of field hydraulic conductivity values and mean values are summarized in Table 3. The overall range is about 1.7 orders of magnitude. This is not unexpected, as the literature indicates that within the same hydrogeologic unit, hydraulic conductivity commonly varies by more than two orders of magnitude (Fetter, 1988). The author has found similar results at other sites in northeastern Minnesota (Evavold, 1985). The Hvorslev and Bouwer and Rice analysis procedures give comparable results. Both the overall geometric mean and arithmetic mean are near mid-range of expected values of 10^{-5} to 10^{-3} cm/sec (Fetter, 1988). It is interesting to note that mean field hydraulic conductivity test results and mean values computed using the effective grain-size for the silty sand samples correlate reasonably well. Arithmetic mean is 1.7×10^{-4} cm/sec (field), compared to 2.3×10^{-4} cm/sec (empirical), and geometric mean is 8.1×10^{-5} cm/sec (field) versus 1.7×10^{-4} cm/sec (empirical).

Additional hydraulic conductivity data were obtained from a pump test conducted in 1990 at a water supply well drilled for a solid waste transfer station. The well is located about 600 feet southwest of the southwest edge of the landfill in a fine to medium sand with some gravel and boulders. Drawdown data

evaluation for a pumping rate of eight to ten gallons per minute indicated hydraulic conductivity values of 7.3×10^{-4} cm/sec to 8.7×10^{-4} cm/sec.

Laboratory Hydraulic Conductivity

Laboratory hydraulic conductivity for a remolded silty sand sample from well 88-5 is 3.6×10^{-5} cm/sec. This compares well with the computed field values of 1.7×10^{-5} and 1.2×10^{-5} cm/sec (Table 2). Laboratory hydraulic conductivity for a remolded sample of poorly graded sand from well 88-3 was determined to be 3.6×10^{-3} cm/sec.

**TABLE 2
FIELD HYDRAULIC CONDUCTIVITY**

<u>LOCATION</u> Monitoring	<u>MATERIAL</u> <u>CLASSIFICATION</u> ⁽¹⁾ <u>AT SCREENED</u> <u>INTERVAL</u>	<u>HYDRAULIC CONDUCTIVITY</u>			
		<u>Hvorslev Method</u>		<u>Bouwer and Rice Method</u>	
<u>Well:</u>		<u>cm/sec</u>	<u>ft/day</u>	<u>cm/sec</u>	<u>ft/day</u>
81-3	Silty sand (SM)	1.3x10 ⁻⁴	0.37	9.5x10 ⁻⁵	0.27
81-5	Silty sand (SM)	2.6x10 ⁻⁴	0.74	2.6x10 ⁻⁴	0.74
81-6	Silty sand (SM)	2.8x10 ⁻⁵	0.08	1.7x10 ⁻⁵	0.05
86-11	Silty sand (SM), very loose to very dense	6.3x10 ⁻⁴	1.79	4.8x10 ⁻⁴	1.36
86-12	Silty sand (SM), dense to very dense	3.7x10 ⁻⁵	0.10	2.8x10 ⁻⁵	0.08
88-4	Sand (SP), gravel, boulders, very dense	3.8x10 ⁻⁴	1.08	2.1x10 ⁻⁴	0.60
88-5	Silty sand (SM), very dense to dense	1.7x10 ⁻⁵	0.05	1.2x10 ⁻⁵	0.03
88-6	Silty sand (SM), with gravel, very dense	4.6x10 ⁻⁵	0.13	3.3x10 ⁻⁵	0.09

⁽¹⁾ Unified Soil Classification, ASTM Designation D2487

**TABLE 3
FIELD HYDRAULIC CONDUCTIVITY SUMMARY**

	<u>HVORSLEV METHOD</u>		<u>BOUWER AND RICE METHOD</u>	
	<u>cm/sec</u>	<u>ft/day</u>	<u>cm/sec</u>	<u>ft/day</u>
Range	6.3x10 ⁻⁴ to 1.7x10 ⁻⁵	1.79-0.05	4.8x10 ⁻⁴ to 1.2x10 ⁻⁵	1.36-0.03
Arithmetic Mean	1.9x10 ⁻⁴	0.54	1.4x10 ⁻⁴	0.40
Geometric Mean	9.5x10 ⁻⁵	0.27	6.8x10 ⁻⁵	0.19
Arithmetic Mean (both methods)	1.7 × 10 ⁻⁴ cm/sec (0.48 ft/day)			
Geometric Mean (both methods)	8.1 × 10 ⁻⁵ cm/sec (0.23 ft/day)			

GROUNDWATER FLOW

Direction

Table 4 contains water table elevations measured at the site monitoring wells. Elevations for 1983 through 1987, 1989, and 1990 are from measurements made during quarterly water quality sampling. Elevations in 1988 include measurements made specifically as part of this research. An examination of the elevation data indicates a groundwater divide corresponding to the location of the bedrock high at or near the south edge of the landfill. This results in groundwater flow to the north-northwest under the landfill as shown on the site groundwater surface contour map, Figure 11, prepared from measurements taken in September 1988. The area groundwater surface contour map, Figure 12, shows the northeast-southwest trending groundwater divide, with the water north of the divide flowing toward Bear Island River. The probable maximum limits of influence of the landfill on the Bear Island River, from the groundwater perspective, are also identified in Figure 12.

Gradient

Typical average water table gradients vary from 2.7 percent beneath the landfill to 1.2 percent at the north edge of the landfill, based on September 1988 measurements.

Velocity

Although groundwater velocity can vary considerably in glacial moraine material, it is useful to identify average groundwater velocity in the primary flow direction. Average linear (seepage) groundwater velocity can be determined using the following formula:

$$V_s = \frac{K}{n} \left[\frac{dh}{dl} \right]$$

where V_s is average linear (seepage) velocity (L/t).
K is hydraulic conductivity (L/t).
dh/dl is gradient expressed as a decimal.
n is porosity expressed as a ratio (estimated as 0.25).

Beneath the landfill, the average linear velocity is computed to be 1.16×10^{-5} cm/sec (12 feet/year), based on a gradient of 2.7 percent and an average hydraulic conductivity of 1.1×10^{-4} cm/sec, using field values for wells 81-3, 81-5, 81-6, and 86-12. From the north edge of the landfill past well 86-11, the average linear velocity is 1.44×10^{-5} cm/sec (14.9 feet/year), using an average hydraulic conductivity of 3.0×10^{-4} cm/sec for wells 86-11, 88-4 and 88-6.

TABLE 4
NORTHWOODS LANDFILL WATER TABLE ELEVATIONS

<u>DATE</u>	<u>81-2</u>	<u>81-3</u>	<u>81-5</u>	<u>81-6</u>	<u>81-7</u>	<u>81-8</u>	<u>81-9</u>	<u>86-10</u>	<u>86-11</u>	<u>86-12</u>	<u>88-2</u>	<u>88-3</u>	<u>88-4</u>	<u>88-5</u>	<u>88-6</u>
07/05/83	52.15	38.57	39.93	46.30	43.57	36.68									
10/18/83	55.10	39.15	36.35	44.80	44.30	35.30									
04/18/84	53.80	37.20	37.40	41.70	40.60	33.20	25.50								
07/10/84	53.48	37.69	34.10	41.05	40.73	33.10	24.95								
04/16/85	53.07	36.73	32.77	41.23	31.85	23.62									
07/11/85	52.73	35.98	34.10	41.51	40.11	33.43	24.87								
05/14/86	54.15	36.17	34.68	42.88	40.58	33.27	25.70	51.58	33.69	38.40					
07/10/86	53.10	36.23	35.93	40.50	40.15	32.77	24.95	51.08	33.03	36.53					
10/13/86	53.50	36.00	34.23	38.88	41.73	33.06	25.41	51.45		37.40					
04/30/87		36.67	33.17	41.54	40.59	32.50	24.92	52.01	32.68	37.01					
07/14/87		36.29	33.54		40.47	32.62	24.35	51.31	32.31	36.17					
10/15/87		36.22	33.19		38.88	32.24	24.93	51.65	32.16	36.26					
04/16/88			33.50						31.86	38.50	50.12	32.21	32.26	28.12	32.55
05/12/88		36.64	33.47		40.28			52.40	32.30	38.00	51.79				
06/04/88		35.95	33.23			32.32			32.36	35.86	50.00	33.06	32.70	28.95	31.08
07/07/88	51.35	35.52	32.72	40.74		32.07	24.06	49.68	31.49	34.70	48.14	32.56	32.81	28.53	30.24
07/21/88	50.56	35.72	32.60		39.73				31.02	34.64	47.44				
09/03/88	54.40	36.66	34.76	43.60	40.60	33.55	25.81	52.62	32.62	38.63	52.20	34.76	34.73	31.19	32.70
10/14/88		36.06	34.19		40.27			52.19	32.30	36.68	50.64				
05/03/89	54.12	36.55	34.80	42.75	40.56			52.78	32.42		52.21	35.96	34.67	31.28	32.91
07/12/89	53.41	35.83	34.21	41.01				52.23	33.17	36.13	49.91	34.13	34.17	30.00	31.23
05/11/90	53.80	36.30	34.54	43.36				52.64	32.49	38.11	51.86		35.02	30.74	

Note: Elevations are in feet above 1400 feet mean sea level

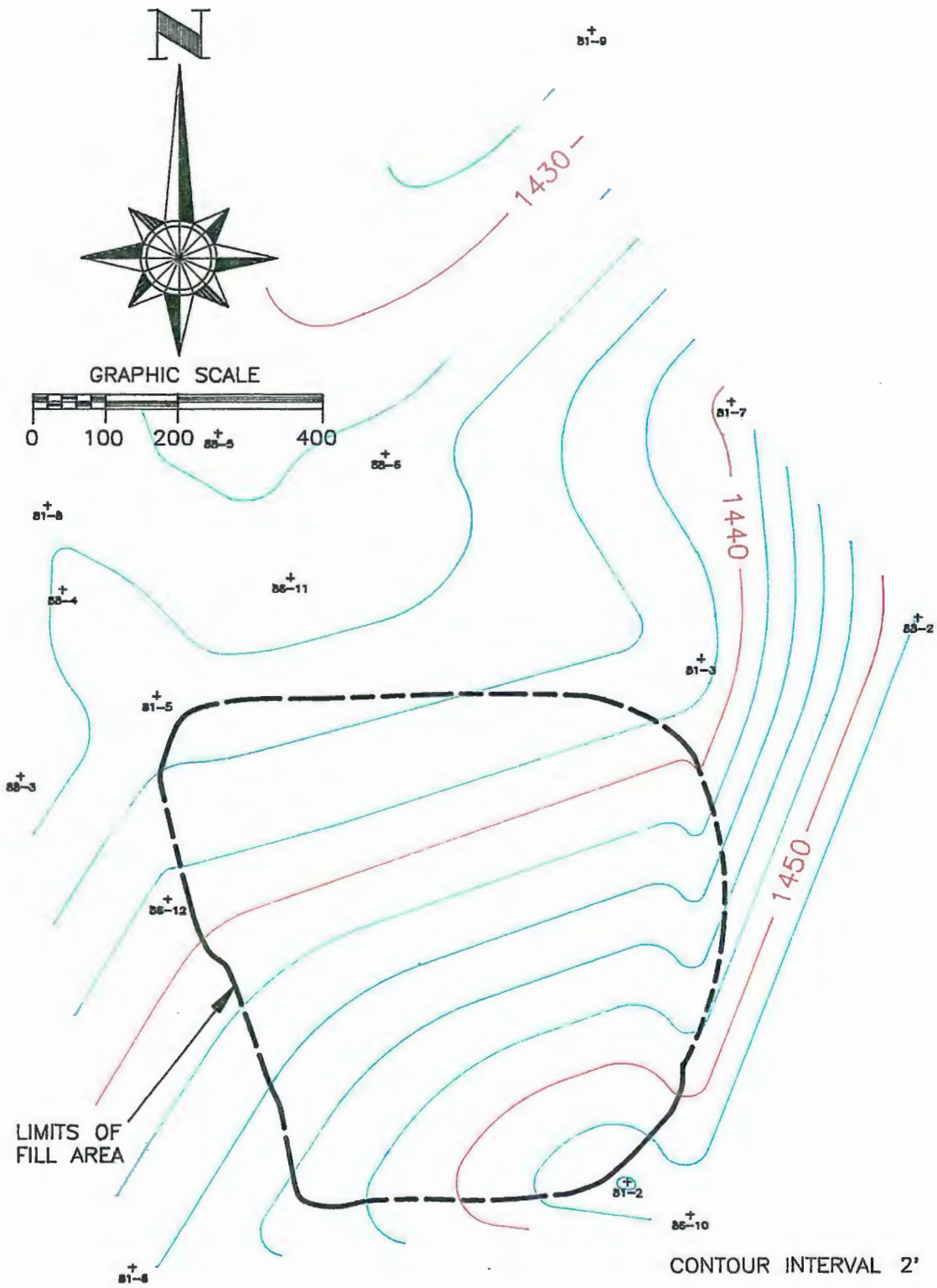


Figure 11. Site groundwater surface elevation contours, September, 1988.

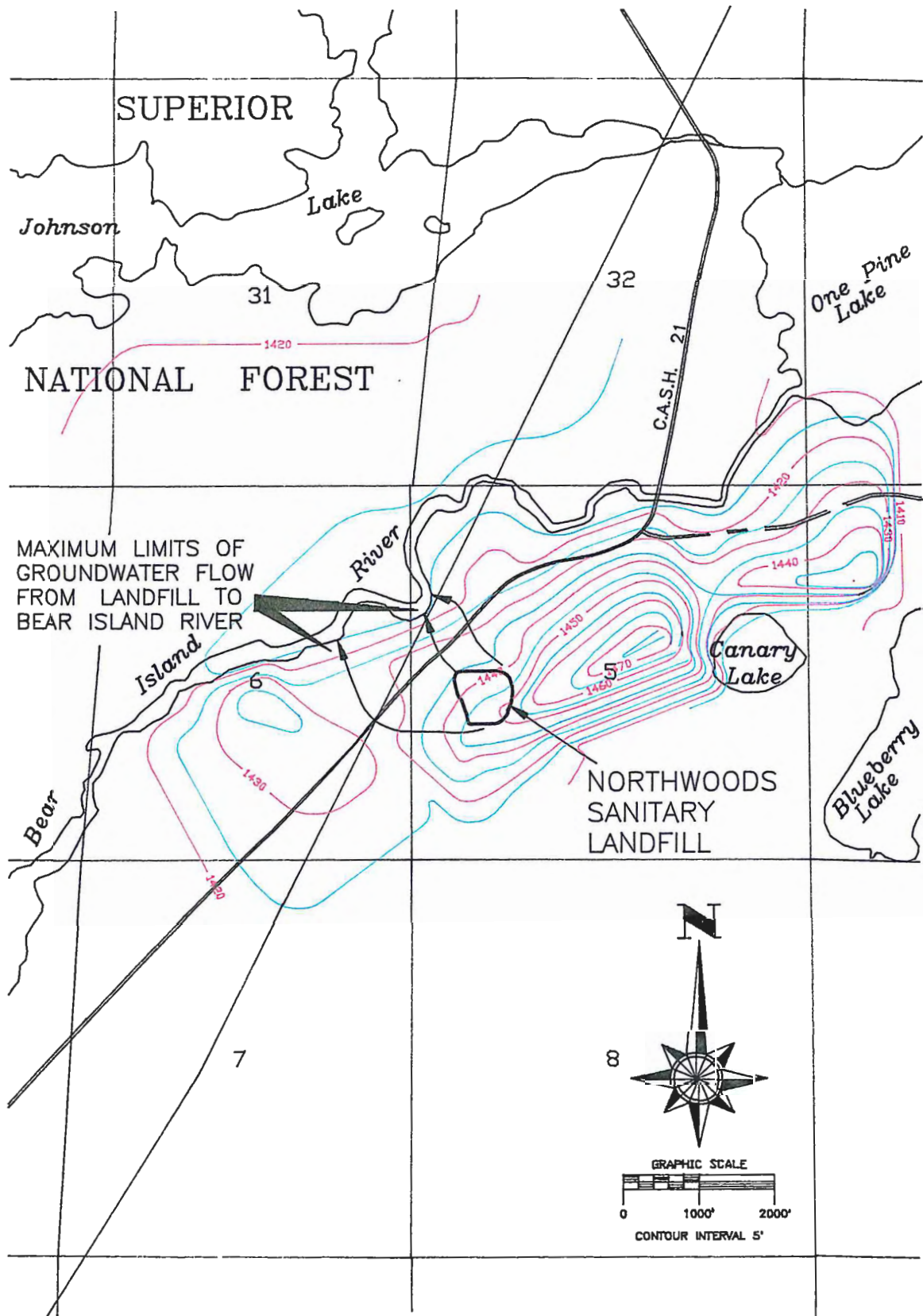


Figure 12. Area groundwater surface elevation contours.

GROUNDWATER QUALITY

General

Groundwater samples have been collected and analyzed for selected inorganic constituents since 1981. Since 1986, after the completion of wells 86-10, 86-11 and 86-12, metals and volatile organics have been included in the sampling program. As a component of this research project, additional sampling and analysis was conducted at the new wells installed in 1988, and at selected existing monitoring locations.

Landfill Leachate

Water that percolates from the solid waste cell is called leachate. Leachate commonly contains significant concentrations of inorganic and organic compounds, which cause groundwater contamination. Table 5 shows concentrations of chemicals detected in a leachate sample collected from a seep at the north edge of the landfill, as part of this research at the Northwoods Landfill. Also shown in Table 5 are:

- typical ranges for a number of constituents detected in mixed municipal solid waste leachate;
- Minnesota Department of Health (MDH) Recommended Allowable Limits (RALs) for drinking water; and
- the Minnesota Pollution Control Agency (MPCA) Intervention Limits for landfill sites.

Leachate concentrations in the Northwoods sample are generally in the lower half of the typical range and, for most constituents, in the lower quartile. It is noted that concentrations of contaminants in the sample collected at the seep may vary from concentrations found elsewhere in the landfill.

Fifteen of fifty volatile organic compounds for which analysis was made were detected in the leachate sample. Of these fifteen organics, nine are assigned an Intervention Limit under current MPCA landfill rules. Leachate concentration

exceeds the Intervention Limits for acetone, benzene, methyl ethyl ketone, toluene, 1,2-dichloropropane and dichloromethane (methylene chloride). Five of the organics detected in the leachate sample are at concentrations which exceed the MDH Recommended Allowable Limits for Drinking Water. These compounds include acetone, benzene, methylene chloride, methyl ethyl ketone and tetrahydrofuran.

Leachate Indicators

Key leachate indicators include chloride, specific conductance and pH. Chloride, since it is relatively nonreactive, is a useful indicator of the extent of the leachate plume and of hydrodynamic dispersion. Table 6 contains chloride concentrations from 1988 sampling. Monitoring wells 88-2 and 81-7 are considered upgradient wells, or monitoring points where groundwater is unaffected by the landfill. The low chloride concentrations (0.6—12.5 mg/l) can be considered background values.

The immediate downgradient monitoring wells 81-5 and 86-11 have chloride concentrations generally in the 200 and 300 mg/l range, indicating the presence of a leachate plume. Chloride concentrations at the wells located farther downgradient (88-3, 88-4, 88-5 and 88-6) show slightly elevated concentrations, identifying the leading edge of the dispersive plume. Monitoring wells 81-8 and 81-9 were constructed in backhoe-dug pits and are no longer considered reliable monitoring points. Chloride concentrations at these two locations are also probably influenced by deicing operations on Highway 21.

TABLE 5
NORTHWOODS LEACHATE ANALYSIS
 Concentrations in mg/l, except VOC ($\mu\text{g/l}$), pH (std units)
 and Specific Conductance ($\mu\text{mhos/cm}$)

<u>Parameter</u>	<u>Northwoods Leachate</u> ⁽¹⁾	<u>Range Reported in Literature</u> ⁽²⁾	<u>RAL</u> ⁽³⁾	<u>Intervention Limit</u> ⁽⁴⁾
Total Alkalinity (CaCO ₃)	2380	0-20,850	NA	NA
Ammonia-nitrogen	220	0-1250	NA	NA
Cadmium	0.01	ND-1.16	0.004	0.00125
Chloride	403	5-4350	NA	NA
Chromium	0.13	ND-22.5	0.1	0.03
COD	13,100	0-89,520	NA	NA
Iron	260	0.2-42,000	NA	NA
Lead	< 0.03	ND-6.6	0.02	0.005
Mercury	< 0.0004	ND-0.16	0.001	0.00075
Nitrate-nitrogen	0.15	0-10.29	10	2500
pH	5.5	1.5-9.5	NA	NA
Specific Conductance	7800	2810-16,800	NA	NA
Sulfate	90	0-84,000	NA	NA
<u>Volatile Organics</u>				
Acetone	1800	NA	700	175
Benzene	78	ND-19	10	3
1,1-Dichloroethane	47	510-6300	70	NA
1,2-Dichloroethylene (cis and trans)	12	96-2200	170	17
1,2-Dichloropropane	3.9	ND-54	5	1.5
Ethyl Benzene	110	100-250	700	170
Ethyl Ether	250	NA	1000	NA
Methyl Ethyl Ketone	9800	NA	300	43
Methyl Isobutyl Ketone	110	NA	300	NA
Methylene Chloride (Dichloromethane)	530	106-20,000	50	12
Tetrahydrofuran	250	NA	100	NA
Toluene	600	280-1600	1000	500
1,1,2-Trichloroethylene	3.6	NA	30	7.8
1,1,2-Trichlorotrifluoromethane	10	NA	2000	NA
Xylenes (Total)	16	NA	10,000	110

⁽¹⁾ September 1988 sample.

⁽²⁾ McGinley and Kmet, 1984.

⁽³⁾ Minnesota Department of Health Recommended Allowable Limits for Drinking Water, 1991.

⁽⁴⁾ Minnesota Pollution Control Agency Intervention Limits at Landfill Compliance Boundary, 1988.

NA — No standard established, or no data.

ND — Not detected

TABLE 6
CHLORIDE CONCENTRATION, mg/l
(1988)

Monitoring Well	SAMPLING DATE					
	<u>4/6/88</u>	<u>5/12/88</u>	<u>6/4-5/88</u>	<u>7/7/88</u>	<u>7/21/88</u>	<u>9/3/88</u>
81-2				35		56
81-3	222	81	33		144	52
81-5	379	356	306		312	257
81-6				2		4
81-7		2			0.6	2
86-10		12.5		25		39
86-11	65	220	257		247.5	232
86-12	18	12.5	3		312	58
88-2		12.5			8.6	2
88-3	33		23			9
88-4	10		16			11
88-5	21		17			13
88-6	80		51			3

Figure 13 contains the estimated chloride plume limits, based on samples collected September 3, 1988. Due to hydrodynamic dispersion, some of the groundwater travels faster than the average linear groundwater velocity. This should be true also for a nonreactive such as chloride. This accounts for the shape of the plume, elongated parallel to the primary groundwater flow direction.

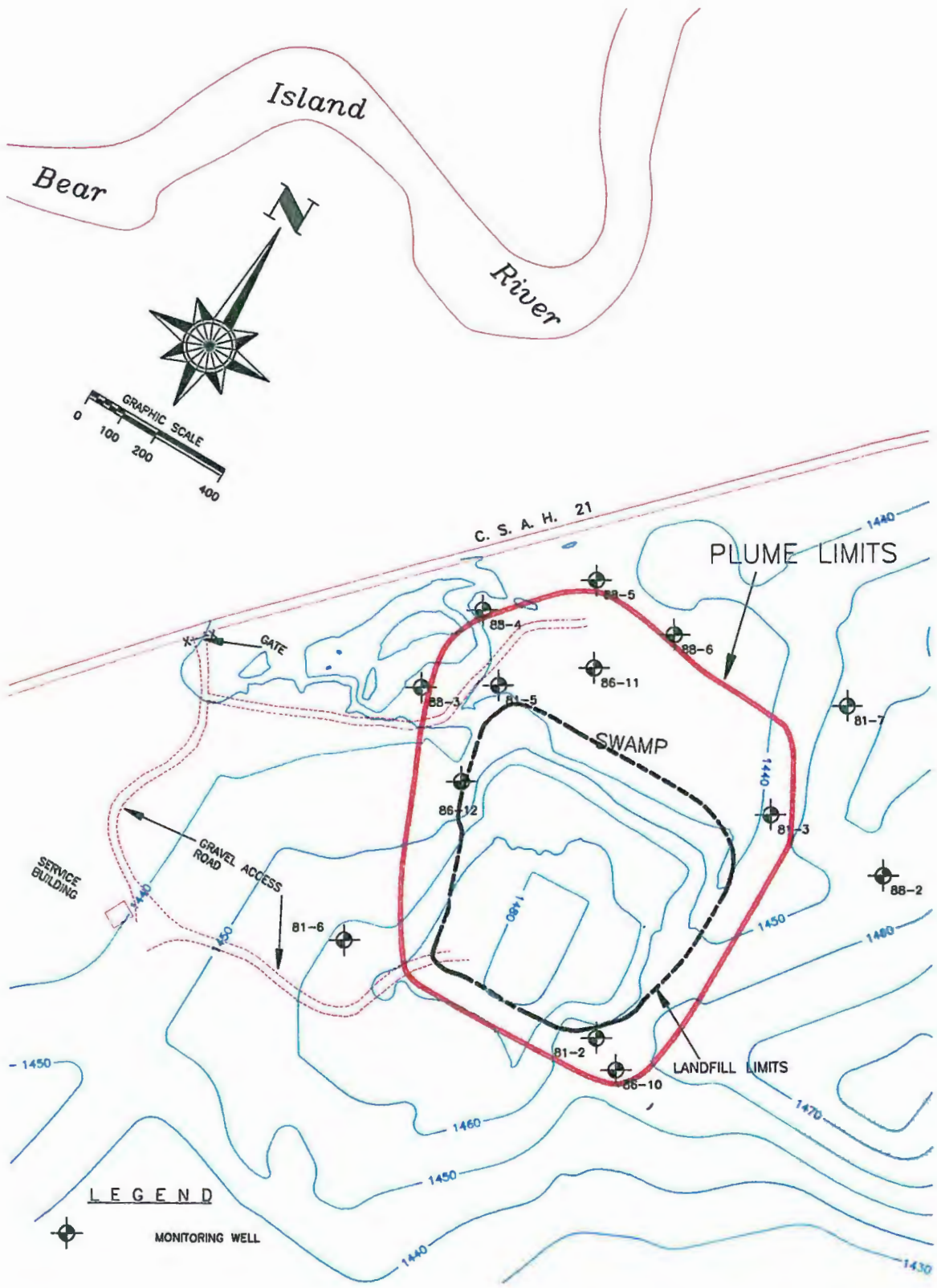


Figure 13. Chloride plume limits (20mg/l), September, 1988

Table 7 contains values for specific conductance in the groundwater during 1988 sampling. As with chloride concentrations, specific conductance is considerably higher (1700-4000 umhos/cm) in the immediate downgradient wells 81-5 and 86-11 than background values (25-150 umhos/cm) found at wells 81-7 and 88-2. Figure 14 is a map of the estimated leachate plume, based on specific conductance. There is general agreement between the specific conductance and chloride plumes.

Field pH values are shown in Table 8. In general, groundwater containing leachate will be slightly acidic. Background pH (well 88-2) is 7.0 to 7.7, whereas four of five values at downgradient well 86-11 were below 7.0, ranging from 6.4 to 6.8. Lower pH values at some locations in September 1988 are attributed to more than thirteen inches of precipitation in August. Due to extremely dry conditions prior to August 1988, a significant percentage of that precipitation recharged the groundwater.

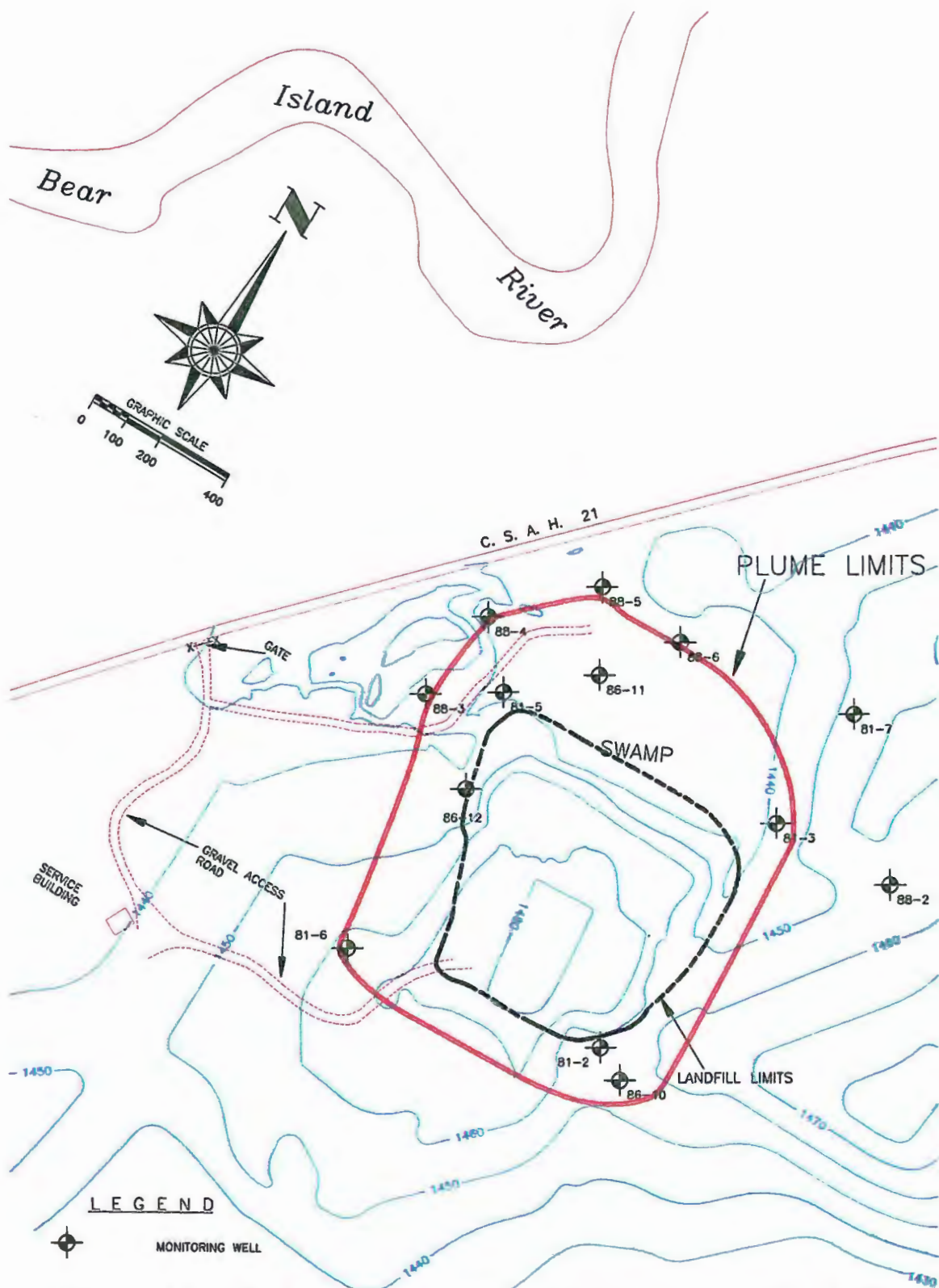


Figure 14. Estimated leachate plume, September, 1988
 (Based on specific conductance)

TABLE 7
SPECIFIC CONDUCTANCE, umhos/cm
(1988)

Monitoring Well	SAMPLING DATE				
	<u>4/16/88</u>	<u>5/12/88</u>	<u>6/4-5/88</u>	<u>7/21/88</u>	<u>9/3/88</u>
81-2					1850
81-3		342	510	1100	1000
81-5	4100	1900	3050	2380	3150
81-6					380
81-7		25		25	54
86-10		610			820
86-11	1700	2100	4200	2380	4100
86-12		288	1300	2950	1300
88-2		88	145	135	150
88-3	440		440		130
88-4	200		227		280
88-5			265		245
88-6	970		1400		100

**TABLE 8
FIELD pH
(1988)**

Monitoring Well	SAMPLING DATE				
	<u>4/16/88</u>	<u>5/12/88</u>	<u>6/4-5/88</u>	<u>7/21/88</u>	<u>9/3/88</u>
81-2					6.3
81-3		7.75	7.1	6.5	6.5
81-5	6.9	7.5	7.0	6.8	6.4
81-6					7.7
81-7		8.3		6.6	7.6
86-10		7.75			6.6
86-11	6.8	7.3	6.7	6.7	6.4
86-12		7.45	6.8	6.75	5.9
88-2			7.7	7.0	7.6
88-3	7.4		6.7		7.7
88-4	7.95		7.05		7.1
88-5			6.55		6.45
88-6	6.9		6.6		5.9

Metals

Intervention limits for arsenic, cadmium, chromium, copper, lead and mercury have been established for landfill sites by the Minnesota Pollution Control Agency (Minnesota Rules, 1988). Table 9 contains groundwater concentrations for these metals in 1988. The relatively high concentrations at well 88-2 during May and June 1988 sampling is attributed to inadequate well development. Samples taken in July, after additional well development, are considered representative of groundwater conditions.

The lead concentration of 120 $\mu\text{g/l}$ at well 81-3 in June 1988 appears to be an anomaly in the sampling or analysis. In subsequent analysis, the concentration was below the minimum detection limit of 2.0 $\mu\text{g/l}$.

At the onset of this research project, MPCA representatives expressed concern that the landfill may be a significant contributor of mercury to the Bear Island River system. The EPA Laboratory in Duluth has refined the analysis procedure such that mercury concentrations can be analyzed in the nanograms/liter range. The mercury analyses shown in Table 9 for June 1988, were analyzed at the EPA Laboratory; concentrations ranged from 8 to 62 ng/l. The higher concentration occurred at the new wells and is attributed to excess sediment in the samples, as these analyses were made on non-filtered samples. It is estimated that one-half to two-thirds of the mercury concentration in the new wells in the June 1988, samples is associated with fine soil particles in the samples. Nevertheless, mercury concentrations at all monitoring points were at least one order of magnitude lower than the MPCA Intervention Limit for the samples analyzed by the EPA for this study. As a part of the on-going regulatory sampling at the site, a mercury concentration of 1.6 $\mu\text{g/l}$ was identified in a sample from well 86-12 in July 1988. Subsequent sampling in July 1989, showed the concentration to be below the minimum detection limit of 0.4 $\mu\text{g/l}$ used by the commercial laboratory.

TABLE 9
METALS ANALYSIS, $\mu\text{g/l}$
(1988)

<u>METAL</u>	<u>SAMPLE DATE</u>	<u>INTER-VENTION LIMIT⁽¹⁾</u>	<u>RAL⁽²⁾</u>	<u>Monitoring Well</u>										
				<u>81-3</u>	<u>81-5</u>	<u>81-7</u>	<u>86-11</u>	<u>86-12</u>	<u>88-2</u>	<u>88-3</u>	<u>88-4</u>	<u>88-5</u>	<u>88-6</u>	
Arsenic	5/12/88	12.5	0.2						< 2					
	7/21/88			< 2	< 2	< 2	3.2	< 2	< 2					
Cadmium	5/12/88	1.25	4.0	0.76						1.4				
	6/4-5/88			< 0.2	< 0.1		< 0.1	< 0.1	2.6	0.16	0.16	0.35	0.14	
	7/21/88				< 0.2	< 0.2	< 0.2	< 0.2	< 0.2					
	9/3/88									0.40	0.40	1.0	0.70	
Chromium	5/12/88	30	100.0						7.6					
	6/4-5/88			6.1	8.4		4.1	3.9	15	1.0	1.0	0.67	2.6	
	7/21/88			5.7	3.7	1.4		9.2	4.2					
	9/3/88									2.8	6.2	2.4	2.6	
Copper	5/12/88	325	1000.0						350					
	7/21/88			10	10	< 10	10	20	30					
Lead	5/12/88	5	20.0						17					
	6/4-5/88			120	< 1.0		< 1.0	< 1.0	52	< 1.0	< 1.0	< 1.0	< 1.0	
	7/21/88			< 2.0	< 2.0	< 2.0	< 2.0	< 2.0	< 2.0					
Mercury	5/12/88	0.75	1.0						< 0.4					
	6/4-5/88 ⁽³⁾			0.03	0.011		0.014	0.008	0.031	0.057	0.018	0.062	0.021	
	7/21/88			0.5	0.5	< 0.4	< 0.4	1.6	0.7					

⁽¹⁾ Minnesota Pollution Control Agency Intervention Limit at Landfill Compliance Boundary.

⁽²⁾ Minnesota Department of Health Recommended Allowable Limit for Drinking Water, 1991.

⁽³⁾ Total mercury

Concentrations of the six heavy metals discussed herein are below Minimum Detection Limits (MDLs) and/or MPCA Intervention Limits at all groundwater sampling locations at the site, except for the July 1988, mercury concentration at well 86-12. Metals concentrations are also below the Department of Health RALs for drinking water, except arsenic. Note that the minimum detection limit for arsenic exceeds the RAL, as established in 1991.

Volatile Organics

Groundwater samples for volatile organic analyses were collected on June 4-5 and July 7, 1988. In addition, samples were collected on July 21, 1988, as part of the on-going regulatory monitoring at the site. Each sample was analyzed for fifty organic compounds. Table 10 contains a summary of seventeen organic compounds detected at one or more sampling locations in 1988. Intervention Limits have been established for eleven of these compounds. The Intervention Limits and Recommended Allowable Limits for drinking water as established by the Minnesota Department of Health are also shown in Table 10.

In 1988, MPCA Intervention Limits were exceeded for acetone, benzene, methyl ethyl ketone, methylene chloride, toluene and vinyl chloride at one or more sampling locations. Benzene concentrations also exceeded the RAL of ten $\mu\text{g/l}$ at three wells: 81-5, 86-11 and 86-12. Methyl ethyl ketone exceeded the RAL of 300 $\mu\text{g/l}$ at well 86-11. The RAL for acetone (700 $\mu\text{g/l}$) was exceeded in the July 7, 1988, sample from well 86-11. The RAL of 1000 $\mu\text{g/l}$ for toluene was exceeded at wells 86-11 and 86-12. Vinyl chloride concentrations exceed the RAL of 0.10 $\mu\text{g/l}$ at wells 81-5, 86-11 and 86-12. Since the minimum detection limit for vinyl chloride is 1.0 $\mu\text{g/l}$, the areal limits over which this contaminant exceeds the RAL cannot be readily determined from the available data.

TABLE 10
VOLATILE ORGANICS ANALYSIS, $\mu\text{g/l}$
(1988)

<u>Compound</u>	<u>SAMPLE DATE</u>	<u>INTER-VENTION LIMIT⁽¹⁾</u>	<u>RAL⁽²⁾</u>	<u>Monitoring Well</u>									
				<u>81-3</u>	<u>81-5</u>	<u>81-7</u>	<u>86-11</u>	<u>86-12</u>	<u>88-2</u>	<u>88-3</u>	<u>88-4</u>	<u>88-5</u>	<u>88-6</u>
Acetone	6/4/88	175	700	< 10	31	—	96	49	< 10	< 10	< 10	< 10	< 10
	7/7/88			< 10	30		1600	57					
	7/21/88			< 10	< 10	< 10	68	73					
Benzene	6/4/88	3	10	< 1	8.4		20	5	< 1	< 1	< 1	< 1	< 1
	7/7/88			< 1	14		20	13					
	7/21/88			< 1	< 1	< 1	< 1	19	< 1				
Chloroethane	6/4/88	NA	NA	< 1	2.3		7.9	< 1	< 1	< 1	< 1	< 1	< 1
	7/7/88			< 1	9.8		13	3.6					
	7/21/88			< 1	< 1	< 1	< 1	< 1	< 1				
Chloromethane	6/4/88	NA	NA	< 5	< 5		< 5	< 5	< 5	< 5	< 5	< 5	< 5
	7/7/88			< 5	40		51	23					
	7/21/88			< 5	< 5	< 5	< 5	< 5	< 5				
1,4-Dichlorobenzene	6/4/88	18.8	10	< 0.2	1.4		< 0.2	< 0.2	< 0.2	< 0.2	< 0.2	< 0.2	< 0.2
	7/7/88			< 0.2	< 0.2		< 0.2	1.2					
	7/21/88			< 0.2	< 0.2	< 0.2	< 0.2	< 0.2	< 0.2				
1,1-Dichloroethane	6/4/88	NA	70	< 1	< 1		1.8	< 1	< 1	< 1	< 1	< 1	< 1
	7/7/88			< 1	< 1		1.8	< 1					
	7/21/88			< 1	< 1	< 1	< 1	< 1	< 1				

TABLE 10, CONTINUED
VOLATILE ORGANIC ANALYSIS, $\mu\text{g/l}$

<u>Compound</u>	<u>SAMPLE DATE</u>	<u>INTER-VENTION LIMIT⁽¹⁾</u>	<u>RAL⁽²⁾</u>	<u>Monitoring Well</u>									
				<u>81-3</u>	<u>81-5</u>	<u>81-7</u>	<u>86-11</u>	<u>86-12</u>	<u>88-2</u>	<u>88-3</u>	<u>88-4</u>	<u>88-5</u>	<u>88-6</u>
1,2-Dichloroethylene (cis & trans)	6/4/88	17	170	< 0.2	2.8		1.9	1.7	< 0.2	< 0.2	< 0.2	< 0.2	< 0.2
	7/7/88				7.9		2.8	2.7					
	7/21/88			< 0.2	< 0.2	< 0.2	< 0.2	< 0.2	< 0.2				
Ethyl Benzene	6/4/88	170	700	< 1	5.5		12	4.8	< 1	< 1	< 1	< 1	< 1
	7/7/88				9.6		11	16					
	7/21/88			< 1	< 1	< 1	< 1	22	< 1				
Ethyl Ether	6/4/88	NA	1000	26	< 400		< 370	190	< 1	14	11	13	< 1
	7/7/88				460		220	540					
	7/21/88			71	250	< 1	14	610	< 1				
Methylene Chloride (Dichloromethane)	6/4/88	12	50	< 5	< 5		9.4	< 5	< 5	< 5	< 5	< 5	< 5
	7/7/88				< 5		18	< 5					
	7/21/88			< 5	< 5	< 5	< 5	< 5	< 5				
Methyl Ethyl Ketone	6/4/88	43	300	< 5	< 5		< 2300	16	< 5	< 5	< 5	< 5	< 5
	7/7/88				< 5		2400	< 5					
	7/21/88			< 5	< 5	< 5	89	< 5	< 5				
Methyl Isobutyl Ketone	6/4/88	NA	300	< 5	< 5		91	28	< 5	< 5	< 5	< 5	< 5
	7/7/88				< 5		100	140					
	7/21/88			< 5	< 5	< 5	< 5	320	< 5				

TABLE 10, CONTINUED
VOLATILE ORGANIC ANALYSIS, $\mu\text{g/l}$

<u>Compound</u>	<u>SAMPLE DATE</u>	<u>INTER-VENTION LIMIT⁽¹⁾</u>	<u>RAL⁽²⁾</u>	<u>Monitoring Well</u>									
				<u>81-3</u>	<u>81-5</u>	<u>81-7</u>	<u>86-11</u>	<u>86-12</u>	<u>88-2</u>	<u>88-3</u>	<u>88-4</u>	<u>88-5</u>	<u>88-6</u>
Tetrahydrofuran	6/4/88	NA	100	< 5	59		66	< 5	< 5	< 5	< 5	< 5	< 5
	7/7/88				70		54	52					
	7/21/88			< 5	< 5	< 5	< 5	> 5	< 5				
Toluene	6/4/88	500	1000	3	4.8		>4600	>1200	< 1	2.8	< 1	3.2	< 1
	7/7/88				6.6		3400	4300					
	7/21/88			< 1	< 1	< 1	230	>2000	< 1				
1,1,2-Tri-chloroethylene	6/4/88	7.8	30	< 0.2	< 0.2		1.9	< 0.2	< 0.2	< 0.2	< 0.2	< 0.2	< 0.2
	7/7/88				0.7		2.5	< 0.2					
	7/21/88			< 0.2	< 0.2	< 0.2	< 0.2	< 0.2	< 0.2				
Vinyl Chloride	6/4/88	0.037	0.10	< 1					< 1	< 1	< 1	< 1	< 1
	7/7/88				1.5		5.0	1.3					
	7/21/88			< 1	< 1	< 1	< 1	< 1	< 1				
Xylenes (total)	6/4/88	110	10,000	< 1	11		27	11	< 1	< 1	< 1	< 1	< 1
	7/7/88				17		23	42					
	7/21/88			< 1	< 1	< 1	< 1	32	< 1				

(1) Minnesota Pollution Control Agency Intervention Limit at Landfill Compliance Boundary.

(2) Minnesota Department of Health Recommended Allowable Limit for Drinking Water, 1991.

NA — No standard established.

GROUNDWATER FLOW MODEL

Mathematical Model

Mercer and Faust (1981) identify a mathematical model as "a set of equations which, subject to certain assumptions, describes the physical processes active in the aquifer." A primary application of groundwater flow modeling is history matching to approximate the behavior of the aquifer and to predict the value of the unknown or dependent variable: hydraulic head.

The first step in mathematical modeling is to understand how the system operates. This understanding requires the application of concepts and laws such as the storage concept for mass balance and Darcy's Law for momentum balance. The physical behavior is then translated into mathematical expressions, which for groundwater flow are partial differential equations. Figure

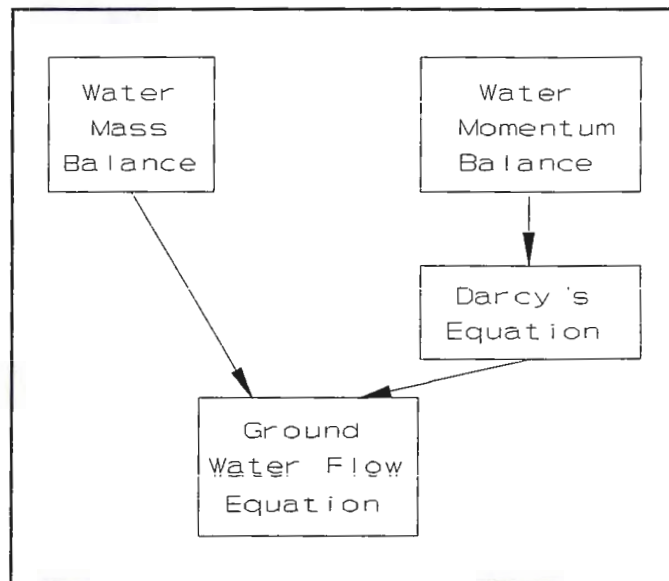


Figure 15, Major components of flow equation (Mercer and Faust, 1981).

15 contains the major components of the groundwater flow equation. These equations, together with boundary and initial conditions, are used to simulate flow processes within the aquifer.

Flow Equations

The following differential equation (McDonald and Harbaugh, 1984) describes nonequilibrium, three-dimensional flow of constant density groundwater through a heterogeneous, anisotropic porous medium.

$$\frac{\partial}{\partial x} \left[K_x \frac{\partial h}{\partial x} \right] + \frac{\partial}{\partial y} \left[K_y \frac{\partial h}{\partial y} \right] + \frac{\partial}{\partial z} \left[K_z \frac{\partial h}{\partial z} \right] - W = S_s \frac{\partial h}{\partial t}$$

where x, y, z are cartesian coordinates.

K_x, K_y, K_z represent hydraulic conductivity assumed to be collinear with the coordinate system (L/t).

h is hydraulic head (L).

W is sink or source, i.e., volume of water injected or removed per unit aquifer volume per unit time (t^{-1}).

S_s is specific storage (L^{-1}).

t is time (t).

The first three terms in the equation represent differences in flow rates into and out of a given aquifer volume. W represents the rate of water removed from or added to the aquifer control volume through a sink or source. The rate of change of water stored in the aquifer volume is represented by the right-hand side of the equation.

In many groundwater flow situations, the vertical variation in aquifer properties and hydraulic head is relatively small; therefore, there are a wide range of flow conditions which can be modeled in two dimensions, x and y . The two-dimensional flow equation for a confined aquifer is obtained by integrating the general flow equation over the aquifer thickness, resulting in the following linear partial differential equation (Mercer and Faust, 1981):

$$\frac{\partial}{\partial x} \left[T_x \frac{\partial h}{\partial x} \right] + \frac{\partial}{\partial y} \left[T_y \frac{\partial h}{\partial y} \right] - W = S \frac{\partial h}{\partial t}$$

where T_x , T_y are transmissivity values aligned with the coordinate axes (L^2/t); equals hydraulic conductivity (K) multiplied by the aquifer thickness (b).
 S is the storage coefficient, or storativity (dimensionless); equals specific storage (S_s) multiplied by the aquifer thickness (b).

Transmissivity is a function of hydraulic head in unconfined or water table aquifers. The two-dimensional, unconfined flow equation (Mercer and Faust, 1981) is:

$$\frac{\partial}{\partial x} \left[K_x b \frac{\partial h}{\partial x} \right] + \frac{\partial}{\partial y} \left[K_y b \frac{\partial h}{\partial y} \right] - W = S_y \frac{\partial h}{\partial t}$$

where b is the saturated thickness (L).
 S_y is aquifer specific yield (dimensionless).

For steady-state flow conditions, there is no change in head with time, i.e., $\frac{\partial h}{\partial t} = \text{zero}$. If the aquifer can also be considered to be isotropic, $T_x = T_y$, the steady-state, two-dimensional flow equation becomes:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} - \frac{W}{T} = 0$$

Boundary and Initial Conditions

In addition to flow equations, boundary and initial conditions must be provided to mathematically simulate a specific physical process. Both boundary and initial conditions are required to solve nonequilibrium flow, whereas only boundary conditions are necessary for steady-state flow. Boundary conditions for groundwater flow are generally one of the following types:

Specified Value

Values of head are specified at the boundary. This represents a recharge boundary, such as a lake, where there is essentially no drawdown or change in water level, regardless of the stresses occurring in the aquifer.

Specified Flux

The rate of flow is specified and related to the normal derivative, i.e., change in head perpendicular to the boundary. A special case is a barrier boundary with no flow across the boundary.

Value-Dependent Flux

The rate of flow is dependent on both the change in head perpendicular to the boundary and the value of the head at the boundary.

Initial conditions are the values assigned to the dependent variable (head) in the aquifer within the model boundaries.

Numerical Model

Mathematical flow models include analytical and numerical models. An analytical solution is possible if an algebraic equation can be written and solved using calculus. Generally, an analytical solution requires unrealistic assumptions and a numerical solution is often needed to solve the mathematical model. In the numerical solution, discrete equations in time and space are established to approximate the continuous partial differential equations. The area of interest is subdivided into discrete elements, with an equation for each element and time step. The finite-difference method (FDM) and the finite-element method (FEM) are numerical techniques most often used in groundwater applications (Mercer and Faust, 1981; Wang and Anderson, 1982). The finite-difference method was used in this study. The primary components of FDM model development are shown in Figure 16.

In FDM modeling, a grid with dimensions Δx and Δy is superimposed on the aquifer. The Δx and Δy dimensions are chosen such that the discrete model reasonably represents the continuous aquifer system. Common types of grids are block-centered and point- or mesh-centered. Figure 17 shows these grid configurations and the node points representing the locations at which values of the dependent variable (head in flow applications) are obtained. The cell associated with each node represents a prism of the porous medium, within which hydraulic properties are considered constant.

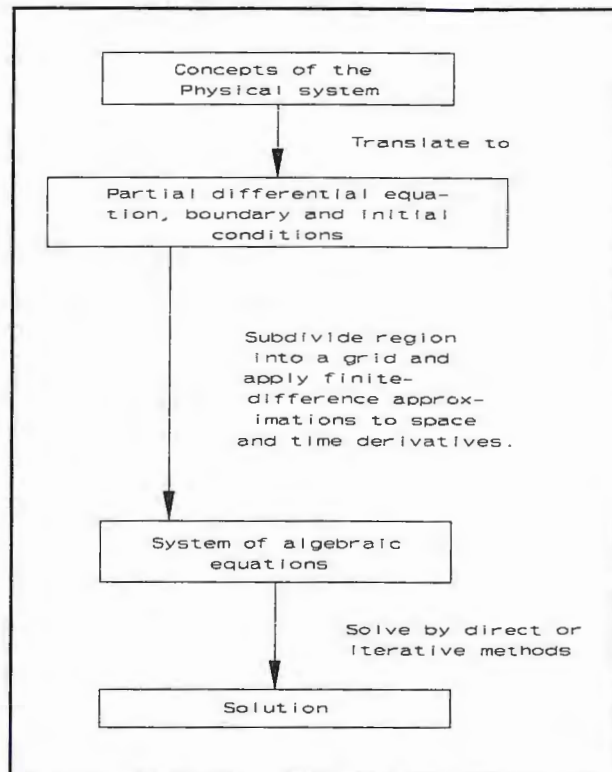
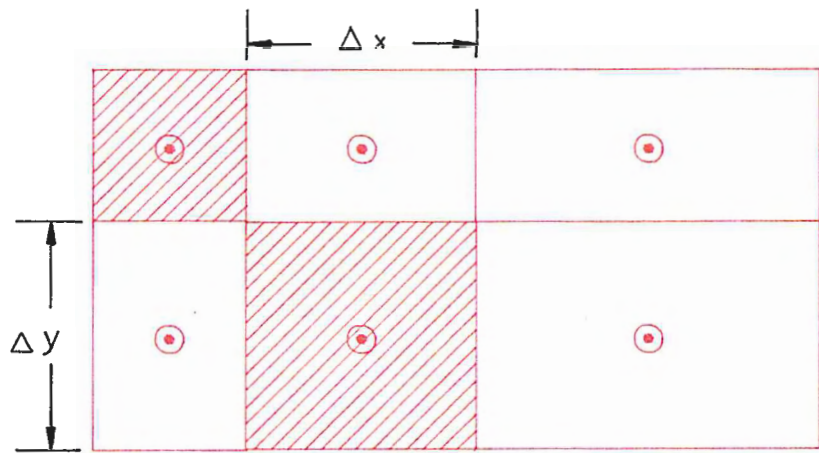


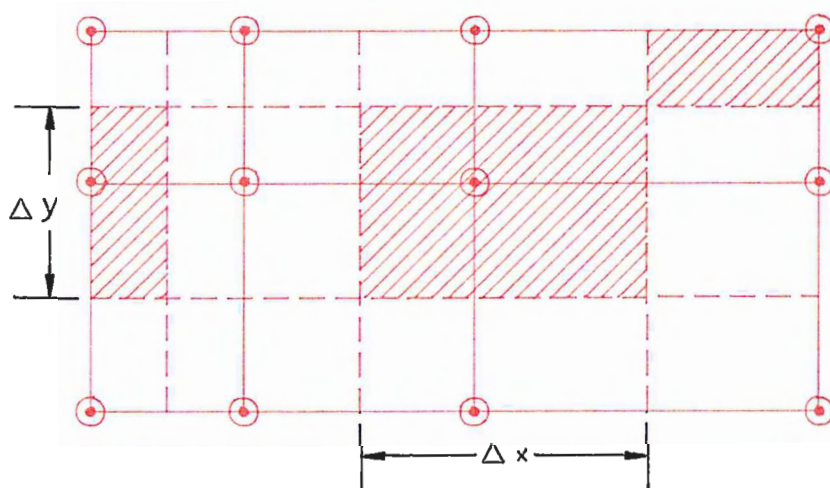
Figure 16, Finite-difference model (Mercer and Faust, 1981).

The computer software used for groundwater flow modeling in this study is a finite-difference model developed by McDonald and Harbaugh (1984) commonly referred to as MODFLOW. MODFLOW uses a block-centered grid. Figure 18 contains a typical discretization of a three-dimensional aquifer system, showing the I, J, K, coordinate system used in MODFLOW. Relating to a cartesian coordinate system, points along a row (I) are parallel to the x axis, points along a column (J) are parallel to the y axis, and points along the vertical (K) are parallel to the z axis.

Considering continuity or water balance and Darcy's Law, a finite-difference equivalent of the partial differential flow equation can be developed. As an example, let us examine the flow regime for an interior nodal cell and the four surrounding cells in a confined aquifer as shown in Figure 19.



Block-Centered Grid System

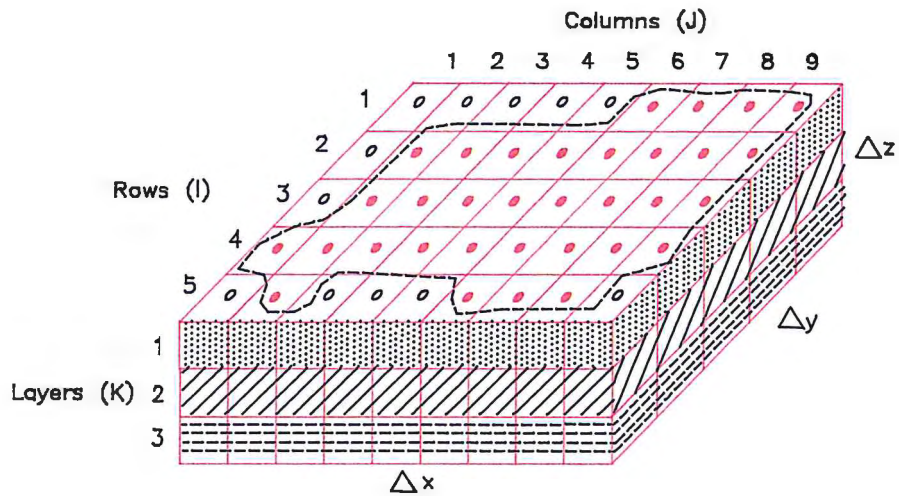


Point-Centered Grid System

Explanation	
●	Nodes
- - -	Grid Lines
—	Cell Boundaries for Point Centered Formulation
▨	Cells Associated With Selected Nodes

Figure 17. Block-centered and point-centered finite-difference grids.

(after McDonald and Harbaugh, 1984).



Explanation

---- Aquifer Boundary

● Active Cell

○ Inactive Cell

Δx Dimension of Cell Along the Row Direction.
Subscript (J) Indicates the Number of the Column

Δy Dimension of Cell Along the Column Direction.
Subscript (I) Indicates the Number of the Row

Δz Dimension of the Cell Along the Vertical Direction.
Subscript (K) Indicates the Number of the Layer

Figure 18. Discretized aquifer system (after McDonald and Harbaugh, 1984).

To maintain continuity, net rate of flow into cell 5 must equal the time rate of change of storage within the cell. This can be expressed as (Freeze and Cherry, 1979):

$$Q_{15} + Q_{25} + Q_{35} + Q_{45} = S_{s5} \Delta x \Delta y b \frac{\partial h_5}{\partial t}$$

where Q_{15} , Q_{25} , Q_{35} , and Q_{45} represent the flow from each surrounding cell to cell 5.

S_{s5} is specific storage of cell 5.

x , y are grid dimensions.

b is aquifer thickness.

$\partial h_5 / \partial t$ is time rate of change of head in cell 5.

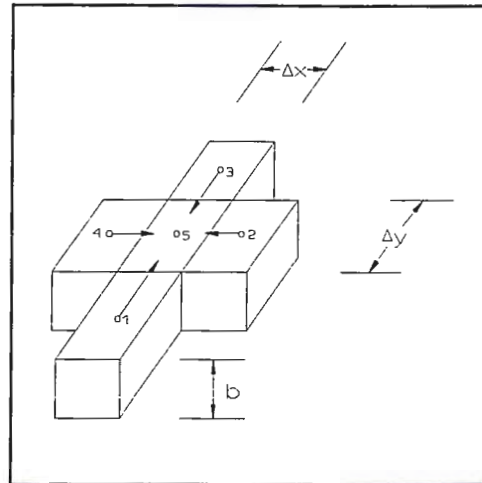


Figure 19, Water balance, FDM (Freeze and Cherry, 1979).

From Darcy's law:

$$Q_{15} = T_{15} \Delta x \left[\frac{\partial h}{\partial y} \right]_{15}$$

where T_{15} is transmissivity between nodes 1 and 5.

The hydraulic head derivative can be approximated as:

$$\left[\frac{\partial h}{\partial y} \right]_{15} \approx \frac{h_1 - h_5}{\Delta y}$$

The flow equation from node 1 to node 5 becomes:

$$Q_{15} = T_{15} \Delta x_5 \left[\frac{h_1 - h_5}{\Delta y} \right]$$

Similar expressions may be written for flow from nodes 2, 3, and 4 to node 5. If we consider the aquifer to be homogeneous and isotropic, transmissivity (T) and specific storage (S_s) are constant. Assuming a uniform grid, i.e. $\Delta x = \Delta y$, and noting that storativity(S)= $S_s b$, substitution of the Darcy flow equation into the continuity equation yields:

$$h_1 + h_2 + h_3 + h_4 - 4h_5 = \frac{S}{T} \Delta x^2 \frac{\partial h_5}{\partial t}$$

The time derivative on the right side of the equation may be approximated as

$$\frac{\partial h_5}{\partial t} \approx \frac{h_5^m - h_5^{m-1}}{\Delta t}$$

where m represents the end of the current time step.
 Δt is the time step from time t_{m-1} to t_m .

Substituting this expression for the time derivative, and using the general i, j coordinate system, the finite-difference equation for node i, j is:

$$h_{i,j-1}^m + h_{i+1,j}^m + h_{i,j+1}^m + h_{i,j-1}^m - 4h_{i,j}^m = \frac{S}{T} \frac{\Delta x^2}{\Delta t} (h_{i,j}^m - h_{i,j}^{m-1})$$

A similar equation is written for each variable-head cell in the system, and since there are "n" equations in "n" unknowns, the equations are solved simultaneously. MODFLOW solves the finite-difference equation using an iterative method. The iterative solution begins with an initial trial solution, which is used to calculate an interim solution. The interim solution becomes the new trial solution, and the computational procedure is repeated. During each repetition (iteration), heads at all nodes are computed. Iteration continues until the difference between the interim-head value and the trial-head value at all nodes is less than a "closure criterion" established by the modeler. When

this level of closure has been obtained for a given time step, an acceptable approximation to the solution of the set of equations has been obtained. The Strongly Implicit Procedure used in this study for solving the simultaneous equations is described in detail in McDonald and Harbaugh (1984).

Study Site Flow Model

Figure 20 shows the finite-difference, 35-column × 30-row grid used in discretizing the study site. The grid orientation was selected to approximate the general groundwater flow direction parallel to the grid columns. Grid dimensions are 50 feet by 50 feet immediately downgradient of the landfill where a finer grid was desirable to more accurately simulate the continuous aquifer system. The grid dimensions increase to 100 feet by 100 feet at the east and west extremities. The overall grid dimensions are 2300 feet by 2550 feet, covering an area of 135 acres.

Bedrock at the site is relatively impervious, compared to the overlying sediments; therefore, the flow model established is a two-dimensional model. Steady-state flow conditions were used with a one-year computational period. The output from this steady-state flow model provides the initial head conditions for transient flow simulation, i.e., determining the effect of removing water from the aquifer by wells.

Boundary conditions consist of both barrier (no-flow) and constant-head boundaries. The aquifer base (bedrock surface) is considered a barrier boundary and the south, east, and west grid limits, i.e., columns one and 35, and row 30, are barrier boundaries. Row 30 was located at the approximate location of a bedrock ridge, which is considered to be a local groundwater divide. Figure 21 contains a contour plot and a three-dimensional perspective of the aquifer bottom used in the groundwater model. Bedrock elevations are known at selected locations near the landfill area; at other locations, bedrock elevations are inferred. The Bear Island River serves as a groundwater discharge area and was considered to be a constant-head boundary in the flow model.

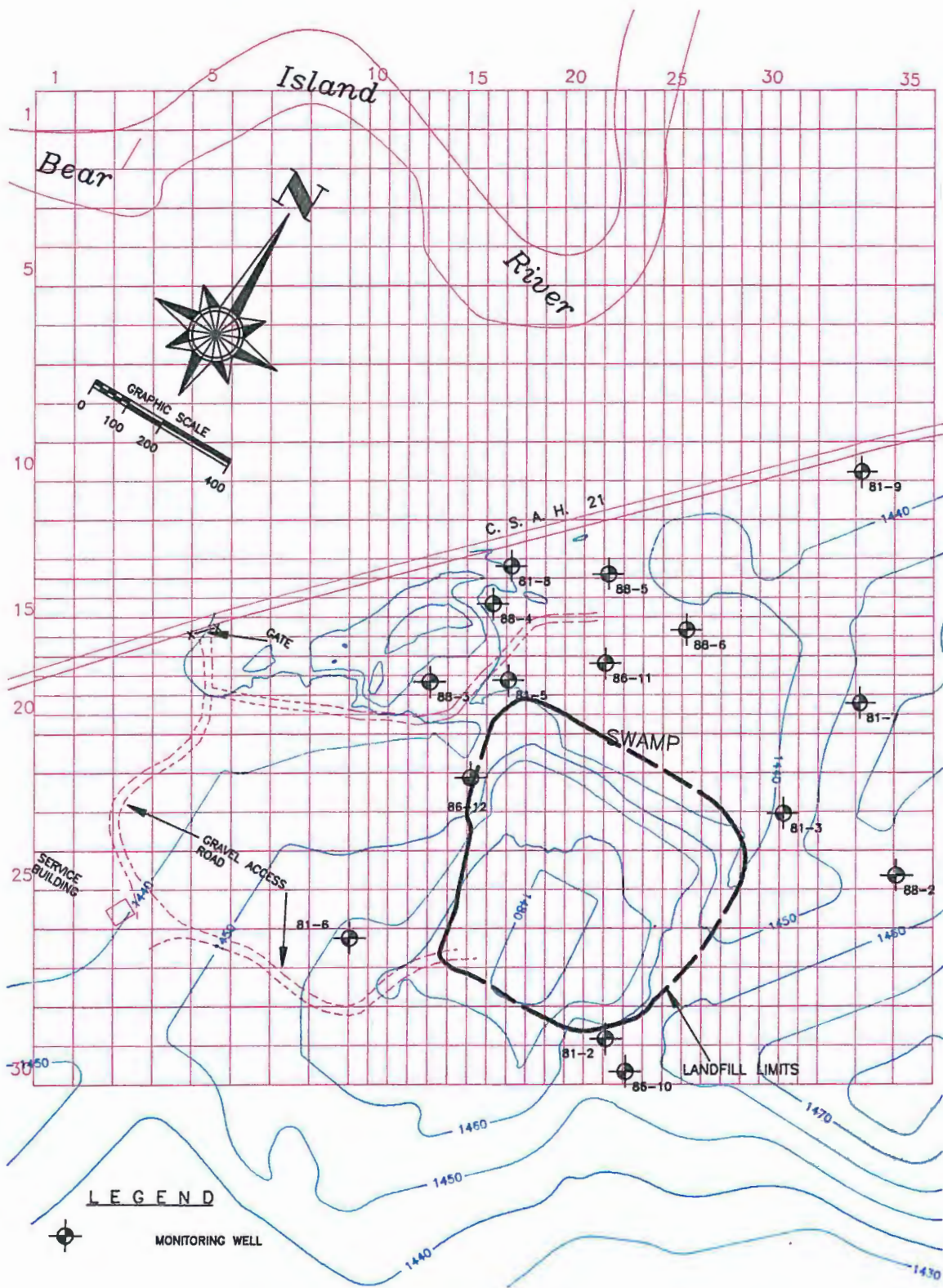


Figure 20. Study site finite-difference grid.

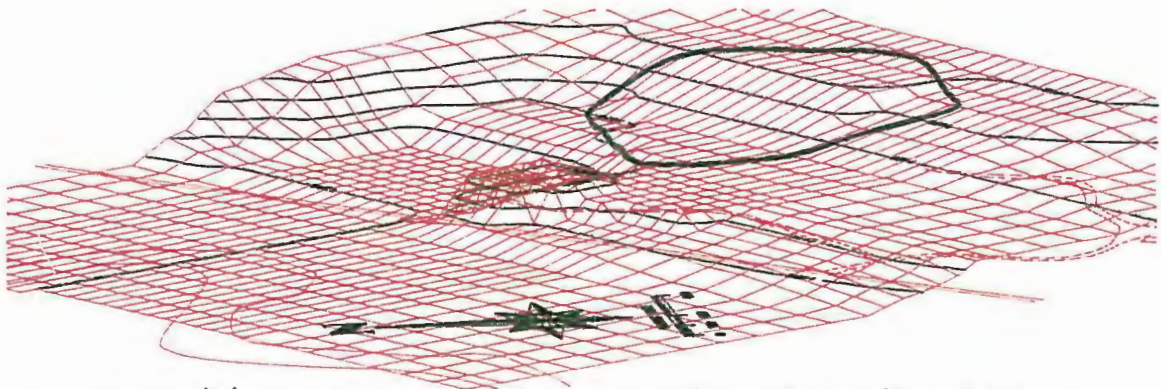
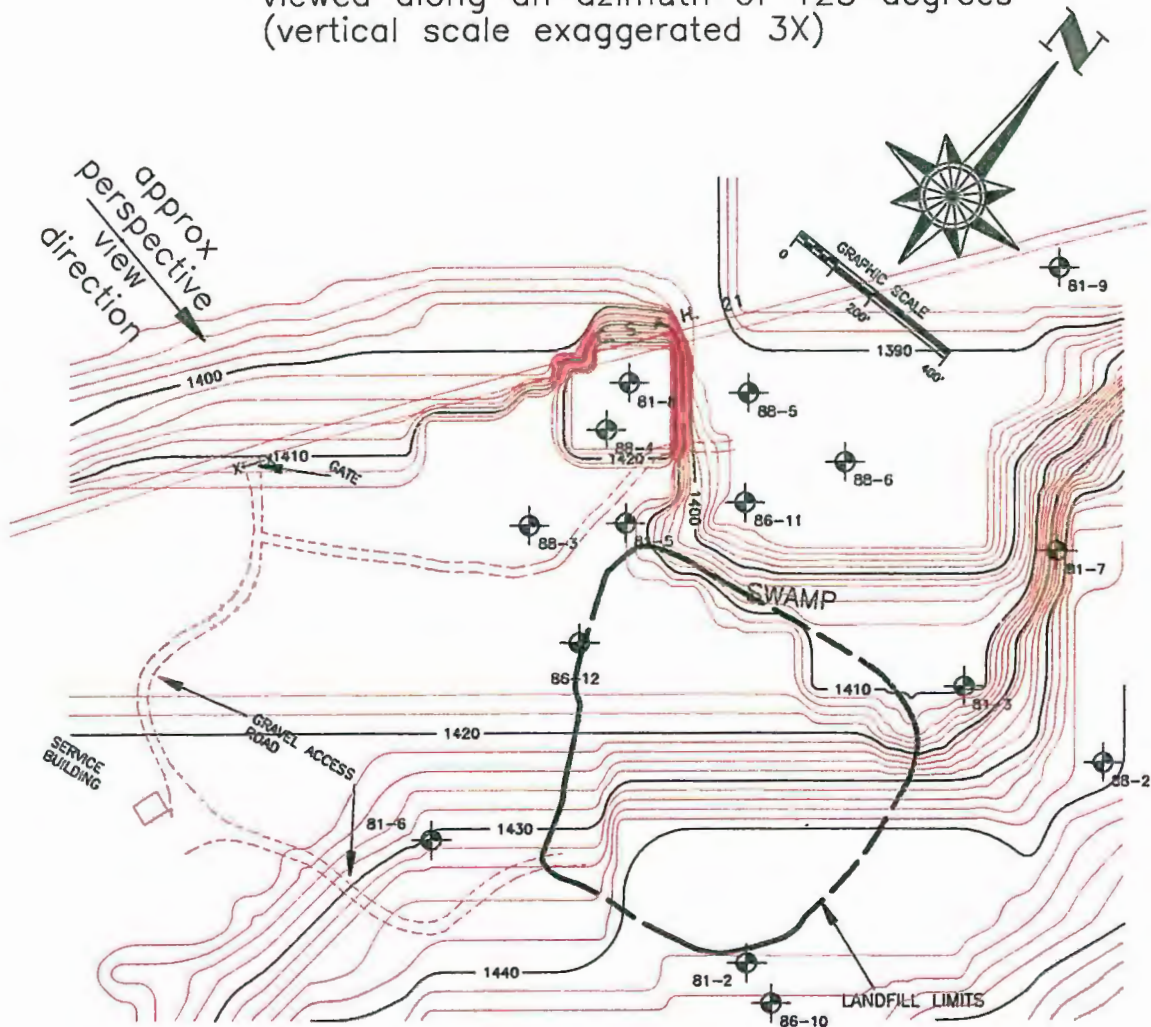


Figure 21(b) 3-dimensional perspective of aquifer bottom; viewed along an azimuth of 125 degrees (vertical scale exaggerated 3X)



LEGEND


 MONITORING WELL

Figure 21(a) Contour plot of aquifer bottom in flow model

Rate of flux into the river from the aquifer is dependent on the head in the aquifer and the river water surface elevation. A constant river water elevation of 1414.0 was used in the model.

Calibration

The flow model was calibrated to water surface elevations, as measured in September 1988. A 0.5-foot "closure criterion" was used. Parameters that may vary during the calibration process include hydraulic conductivity and recharge. Recharge will undoubtedly vary somewhat throughout the modeled area, due to variations in vegetation and topography. Seasonal and annual variations in recharge also occur due to moisture conditions. For purposes of this study, however, recharge has been considered constant throughout the modeled area, while hydraulic conductivity was varied to arrive at a calibrated "fit" with measured aquifer head.

Aquifer recharge occurs primarily from snowmelt and spring rain, as summer precipitation is normally lost through evaporation. Areal recharge can be estimated by determining the difference between maximum water table elevation and the water table elevation with no recharge, as extrapolated from the declining limb of a well hydrograph. This difference is considered to be the rise in water surface due to recharge and is multiplied by the aquifer specific yield to obtain a value for recharge. Although monthly water surface elevations are not available for the wells at the study site, interpretation of available data provide the following spring or early summer recharge values based on an estimated specific yield of 0.20.

Well 81-5	(April 1, 1988)	1.67 inches
Well 86-11	(June 1988)	1.4 inches
Well 88-3	(June 1988)	2.3 inches
Well 88-4	(June 1988)	1.6 inches

A recharge value of 1.5 inches per year was used in establishing the study site model. This translates to a recharge of 40,700 gallons per acre per year.

Over fifty computer calibration runs were made. Table 11 contains measured and calibrated aquifer head at 15 locations. The difference between calibrated and measured values (September 1988) is equal to or less than one foot at all locations. Calibrated values also compare favorably with more recent head measurements shown in Table 4. In the final calibration run, 19 iterations were made before the head closure criterion was met. A good balance was obtained between input and output volumes. For the one year computation period, system input (recharge) is 688,110 cubic feet (5.15 million gallons) compared to system output (discharge into Bear Island River) of 680,290 cubic feet (5.09 million gallons), a difference of 1.14 percent.

Figure 22 contains a contour plot of the groundwater surface, as computed by the calibrated flow model. There is reasonable correlation of the flow gradient beneath the landfill and immediately downgradient of the landfill as computed from the model data, compared to the gradients determined from the limited number of field measurement locations. Computed flow gradients are 2.9 percent and 1.1 percent, beneath the landfill and immediately downgradient, respectively, compared to 2.7 percent and 1.2 percent, as discussed in the Groundwater Flow section.

Flow model calibration results in the hydraulic conductivity distribution shown in Figure 23. Calibrated values range from 3.05×10^{-5} cm/sec (0.09 ft/day) to 1.28×10^{-3} cm/sec (3.63 ft/day). All of the values fall within the range found in the field tests conducted at the monitoring wells, except for the maximum calibrated conductivity, which occurs over less than four percent of the modeled area. The 1.6 order of magnitude range in calibrated hydraulic conductivity is similar to the range determined from field test data.

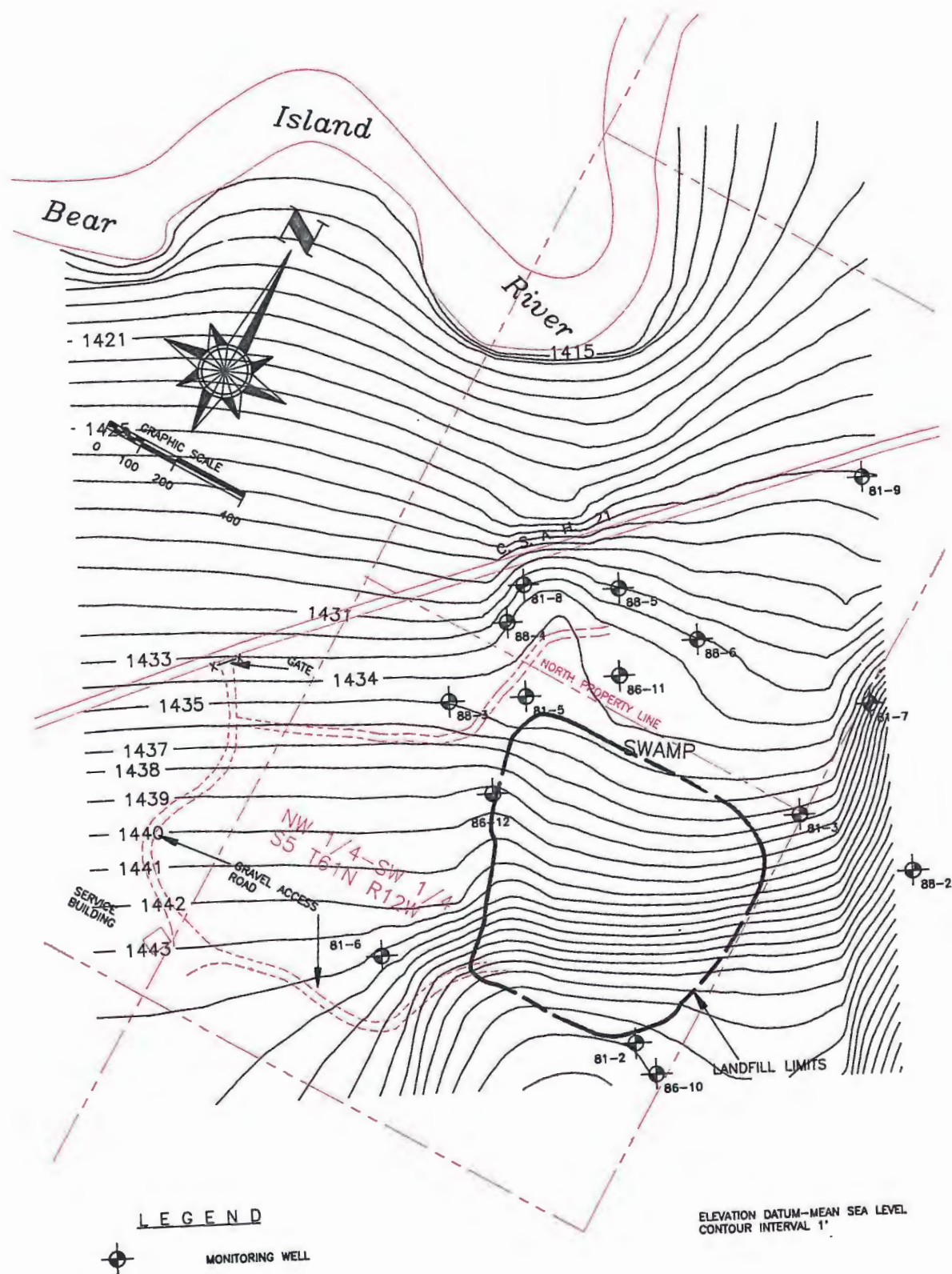
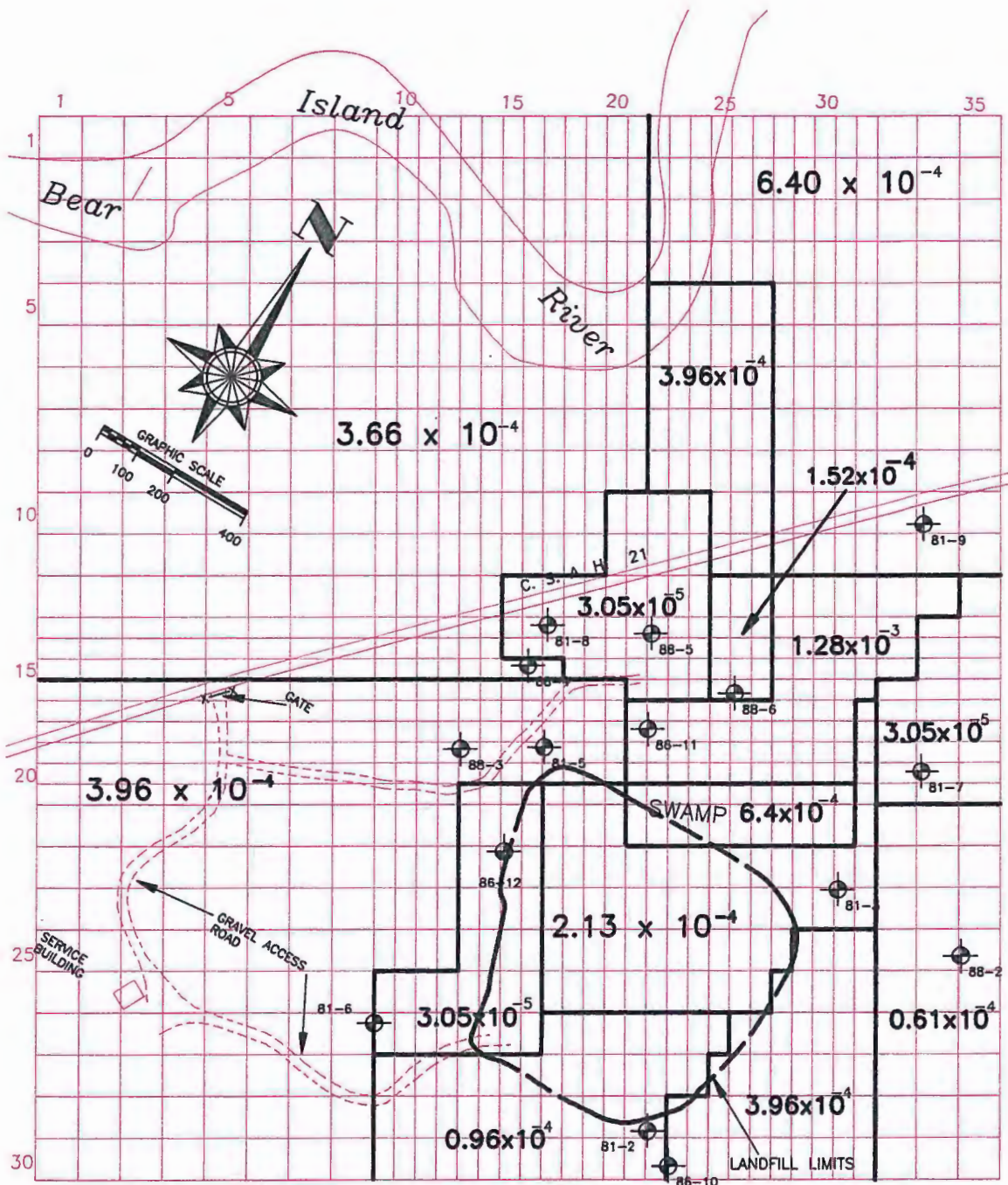


Figure 22. Contour plot of computed groundwater surface elevation — Calibrated flow model.



LEGEND

 MONITORING WELL

Figure 23. Calibrated hydraulic conductivity values, cm/sec.

The calibrated hydraulic conductivity and recharge values are not necessarily unique. Other combinations of these parameters are possible. However, based on data collected, the calibrated values are reasonable and the calibrated model is a good approximation of the aquifer system at the study site.

Sensitivity Analysis

Additional analyses were made to determine the effect on the aquifer flow model of changes in input parameters. Specifically, hydraulic conductivity was changed, while maintaining constant recharge; then recharge was varied, with hydraulic conductivity maintained at calibrated values.

Due to the relatively shallow depth to groundwater at this site and the limited aquifer thickness, the range in hydraulic conductivity at a given location is limited.

Computed hydraulic head exceeds the ground surface elevation at several locations when hydraulic conductivity is reduced below 95 percent of calibrated values, at the constant recharge rate of 1.5 inches per year used in the model calibration. Similarly, an increase in hydraulic conductivity of more than 150 percent of calibrated values causes portions of the aquifer to become completely unsaturated. Therefore, for a recharge rate of 1.5 inches per year, the hydraulic conductivity at a given location in the modeled area appears to be limited to a range of 95 percent to 150 percent of calibrated values.

At 95 percent of calibrated conductivity, the average increase in computed water surface at the 15 measuring locations is 0.9 feet. The average decrease in the computed groundwater surface at 150 percent of calibrated conductivity is 6.9 feet. Due to changes in flow gradients, the average groundwater flow time from the landfill to the Bear Island River is estimated to reduce by about ten percent.

Analyses to evaluate the effect of different recharge values indicate that, at the calibrated hydraulic conductivity values, recharge must be maintained at 1.125 inches per year or more to prohibit the aquifer from becoming unsaturated. Likewise, recharge greater than 1.65 inches per year causes the computed groundwater surface to exceed ground surface elevations. Therefore, the recharge is limited to a range of 75 percent to 110 percent of the value used in the calibrated model.

The average decrease in aquifer head at the measuring locations is 3.6 feet at the lower recharge rate, and average groundwater travel time from the landfill to the river increases by about 24 percent. The average increase in head at the higher recharge rate is 1.5 feet, and average travel time to the river increases about eight percent.

**TABLE 11
MEASURED AND CALIBRATED HEAD
MEAN SEA LEVEL DATUM**

Location Monitoring Well	Measured Head, feet (9/3/88)	Calibrated Head (feet)
81-2	1454.4	1453.5
81-3	1436.7	1437.4
81-5	1434.8	1434.6
81-6	1443.6	1443.9
81-7	1440.6	1441.3
81-8	1433.6	1433.1
81-9	1425.8	1425.7
86-10	1452.6	1453.5
86-11	1433.6	1433.5
86-12	1438.6	1438.7
88-2	1452.2	1452.4
88-3	1434.8	1435.0
88-4	1434.7	1433.7
88-5	1431.2	1430.8
88-6	1432.7	1432.0

SOLUTE TRANSPORT MODEL

Solute Transport

A solute transport equation must be solved simultaneously with the groundwater flow equation to describe the transport of a dissolved chemical species in groundwater. Primary components of the solute transport equation are shown in Figure 24.

The conservation of mass for an elemental volume within the flow regime can be described as follows:

(net rate of change of solute mass) =

(solute flux out of element) -

(solute flux into element) \pm

(solute mass loss or gain due to reactions and/or sources).

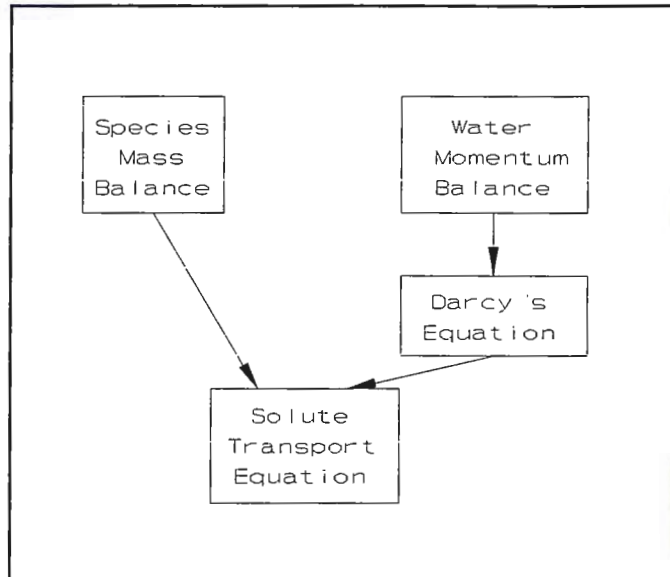


Figure 24, Components of solute transport equation (Mercer and Faust, 1981).

Advection/Hydrodynamic Dispersion

Advection and hydrodynamic dispersion are the physical processes that govern solute flux into and out of the elemental volume. Advection (in aqueous phase transport) is transport of a solute along groundwater flow streamlines at the average groundwater velocity. Under typical aquifer flow conditions, advection is the dominant transport process; therefore, an understanding of groundwater flow patterns is a key in interpreting and predicting transport of contaminants.

Hydrodynamic dispersion includes hydraulic mixing due to "local" variations in groundwater velocity and diffusion. The longitudinal coefficient of hydrodynamic dispersion is expressed as:

$$D_L = \alpha_L V_s + D^*$$

where α_L is dispersivity, a property of the porous medium (L).
 V_s is average groundwater velocity (L/t).
 D^* is the diffusion coefficient (L^2/t).

Since the apparent diffusion coefficient in a porous medium is 10^{-8} to 10^{-9} $cm^2/second$, diffusion is not an important transport mechanism under most groundwater flow conditions. It would be significant only in a low permeable medium, such as clay.

Transverse dispersion also occurs in solute transport, with the transverse dispersion coefficient expressed as:

$$D_T = \alpha_T V_s$$

where α_T equals the transverse dispersivity.

Generally, longitudinal dispersivity is much larger than transverse dispersivity. In homogeneous medium, transverse dispersivity may be similar in value to the diffusion coefficient.

Dispersivity is scale-dependent, although in relatively homogeneous medium, longitudinal dispersivity appears to approach a constant value at 50 to 100 meters from the source. Field dispersivity data is limited. Fetter (1988) identifies longitudinal dispersivity values for alluvial sediments ranging from 39 to 200 feet (12 to 61 meters) and transverse dispersivity varying from 13 to 98 feet (4 to 30 meters). One study on glacial deposits, identified longitudinal and transverse dispersivity equal to 69 feet (21 meters) and 13 feet (4 meters), respectively. Fetter (1988) suggests that, as a rule of thumb, longitudinal dispersivity can be estimated as one-tenth of the flow distance from the source to the location of interest.

At a tracer test site in a shallow sand aquifer in Canada, which included more than 5000 sampling points (Domenico and Schwartz, 1990), longitudinal dispersivity approached a constant value of about 1.4 feet (0.43 meters) at approximately 300 feet (91 meters) from the source. Transverse dispersivity was found to remain relatively constant at a value of about 0.13 feet (0.04 meters). The ratio of horizontal to vertical dispersivity at the site is approximately 11.

One form of the one-dimensional advection-dispersion equation, assuming the X-direction is parallel to flow, is:

$$D_x \frac{\partial^2 C}{\partial x^2} - V_s \frac{\partial C}{\partial x} \pm C_s Q = \frac{\partial C}{\partial t}$$

where D_x is longitudinal dispersion coefficient (L^2/t).
 V_s is average linear groundwater velocity (L/t).
 C is concentration in the groundwater (M/L^3).
 C_s is source/sink concentration (M/L^3).
 Q is source or sink flux per unit volume (t^{-1}).

A two-dimensional advection-dispersion equation (Freeze and Cherry, 1979) for saturated heterogeneous isotropic medium, exclusive of a sink or source term, can be written as:

$$\frac{\partial}{\partial s_L} \left[D_L \frac{\partial C}{\partial s_L} \right] + \frac{\partial}{\partial s_T} \left[D_T \frac{\partial C}{\partial s_T} \right] - \frac{\partial}{\partial s_L} (V_s C) = \frac{\partial C}{\partial t}$$

where s_L and s_T are directions parallel and normal to the groundwater flowlines, respectively.

Reactions

Changes in solute mass within the groundwater system can be the result of chemical and biochemical reactions. Of the several reactions that may occur, surface reactions are probably the most important chemical processes affecting solute transport, particularly for many organics and metals.

Adsorptive capacity of sediments can be determined experimentally. The weight of the chemical species adsorbed per unit weight of soil (C^*) is determined and plotted versus the equilibrium concentration (C) of the aqueous solution. Data for many solutes in geologic medium fit a straight line on a logarithmic plot. This relationship can be expressed as:

$$\log C^* = b \text{ Log } C + \log K_d$$

where b is the slope of the data plot
 K_d is the intercept of the data plot axis

The above equation can be written in the following form, known as the Freundlich isotherm:

$$C^* = K_d C^b$$

If the slope (b) equals 1, the isotherm is linear and the plot of C^* versus C will approximate a straight line on an arithmetic plot. The above equation can then be written as:

$$K_d = \frac{dC^*}{dC}$$

where K_d is the distribution coefficient representing the partitioning between liquid and solids.

Linear isotherms are used in most mathematic solute transport models at the present time. It is fortunate that many contaminants in groundwater meet the requirements of fast reversible adsorption and a linear isotherm, such that the distribution coefficient can adequately describe the partitioning.

Hydrophobic organics tend to adsorb onto organic carbon in the porous medium. The distribution coefficient for these organic species can be defined as:

$$K_d = K_{oc}f_{oc}$$

where K_{oc} is the partition coefficient between organic carbon and water.

f_{oc} is the weight fraction organic carbon in the porous medium.

Values for K_{oc} can be found in the literature or can be computed from octanol-water partition coefficients (K_{ow}).

Organic carbon content at the study site was determined for two sediment samples collected from each of four locations as identified in Table 12. All four locations are downgradient of the landfill. Organic carbon content ranged from 1.02 to 4.44 percent by weight, with a mean value of 2.46 percent.

TABLE 12
ORGANIC CARBON CONTENT
Percent by Weight

<u>Sample Location</u>	<u>Organic Carbon</u>
MW81-5	1.11 1.02
MW86-11	2.69 3.53
MW86-12	2.46 1.83
MW88-6	4.44 2.57
Mean	2.46

The effect of surface reactions is to retard the movement of the solute relative to the movement of groundwater. The retardation of the solute front is described by the Retardation Factor (R), which is related to the distribution coefficient as follows:

$$R = 1 + \frac{\rho_b K_d}{n} = \frac{V_s}{V_c}$$

where ρ_b is the bulk density of the porous medium.

n is porosity of the medium.

V_s is average linear groundwater velocity.

V_c is the velocity of the contaminant at a relative concentration (C/Co) equal to 0.5; Co is the source concentration.

For reactive constituents, the one-dimensional transport equation becomes:

$$\frac{D_x}{R} \frac{\partial^2 C}{\partial x^2} - \frac{V_s}{R} \frac{\partial C}{\partial x} = \frac{\partial C}{\partial t}$$

The effect of applying the retardation factor is to decrease the transport parameters: dispersion coefficient (D_x) and mean velocity (V_s).

Modeling Software

Computer software used for transport modeling in this study is a finite-difference model called InterTrans, developed by Voorhees (1985). InterTrans is a particle transport model that utilizes the output from a companion groundwater flow model—InterSat—for the advective parameters. Implicit in the InterTrans model are fine discretizations: X, Y, Z, time (t) and solute mass (M). This computer code is based on the theory that the concentration distribution of dissolved chemical constituents, in the saturated zone of an aquifer, can be represented by the distribution of a finite number of discrete particles. Each particle is assigned a specific mass representing a fraction of the total mass.

The modeling software utilizes the random-walk technique in determining particle movement by dispersion. This technique is based on the concept that dispersion in porous media is a random process tending to a normal distribution (Prickett, et.al., 1981). The standard deviation for longitudinal dispersion is

$$(2\alpha_L V_s \Delta t)^{1/2}$$

where α_L is longitudinal dispersivity (L).
 V_s is average linear velocity (L/t).
 Δt is the computational time increment (t).

For transverse dispersion the standard deviation is:

$$(2\alpha_T V_s \Delta t)^{1/2}$$

where α_T is transverse dispersivity (L).

The random dispersion movement, in a plus or minus direction, is determined by multiplying the standard deviation by a random number, generally limited to a range of -6 to +6. This random dispersive movement is added to the advective component to determine the new particle locations at the end of each computational time increment.

Nonreactive Constituent Transport

The chloride anion was selected as the indicator of present and future leachate plume configuration. A common constituent of solid waste, chloride also can provide a general indication of the inorganic strength of leachate. Because of its relative physical and biological nonreactivity, chloride is the leachate parameter of choice for evaluating dispersivity of the porous medium.

Transport Parameters

Transport simulation parameters include longitudinal and transverse dispersivity, particle mass, source loading and porosity. Hydraulic head, aquifer top and bottom elevations and cell dimensions are obtained from the calibrated flow model output. These data are used to compute cell-to-cell groundwater

velocities for the determination of the advective and dispersive components of contaminant particle movement.

Model Simulation

The incremental simulation time period used in this study is one year. The total simulation time from initiation of landfilling operations to collection of field data for this research in 1988 is 16 years.

Transport simulation was conducted as a three-step procedure based on estimated historical source loading conditions. It is estimated that approximately 55 percent of the ten-acre landfill area was contributing to leachate production during the period from 1972-1977. The remaining 45 percent of the landfill area was considered to be added as a leachate source from 1978-1981. From 1982 through 1988, the entire ten-acre fill site was considered as a leachate source.

Model Calibration

Several trial-and-error simulations were made to obtain a satisfactory match between observed and simulated chloride concentrations. An average chloride source loading of ten pounds per day was established for the ten-acre site, and a unit mass of 20 pounds per particle was used. There were 2920 particles in the system at the end of the 16-year calibration period.

Dispersivity values were varied until an acceptable calibration was achieved. Particular attention was given to downgradient monitoring locations 81-5 and 86-11. Data from wells 81-8 and 81-9 were not used. Calibrated longitudinal and transverse dispersivity is 60 feet (18.3 meters) and 6 feet (1.8 meters), respectively.

The simulated chloride plume for 1988 is shown in Figure 25. The downgradient plume configuration varies somewhat from the plume limits estimated from the limited number of monitoring locations (Figure 13).

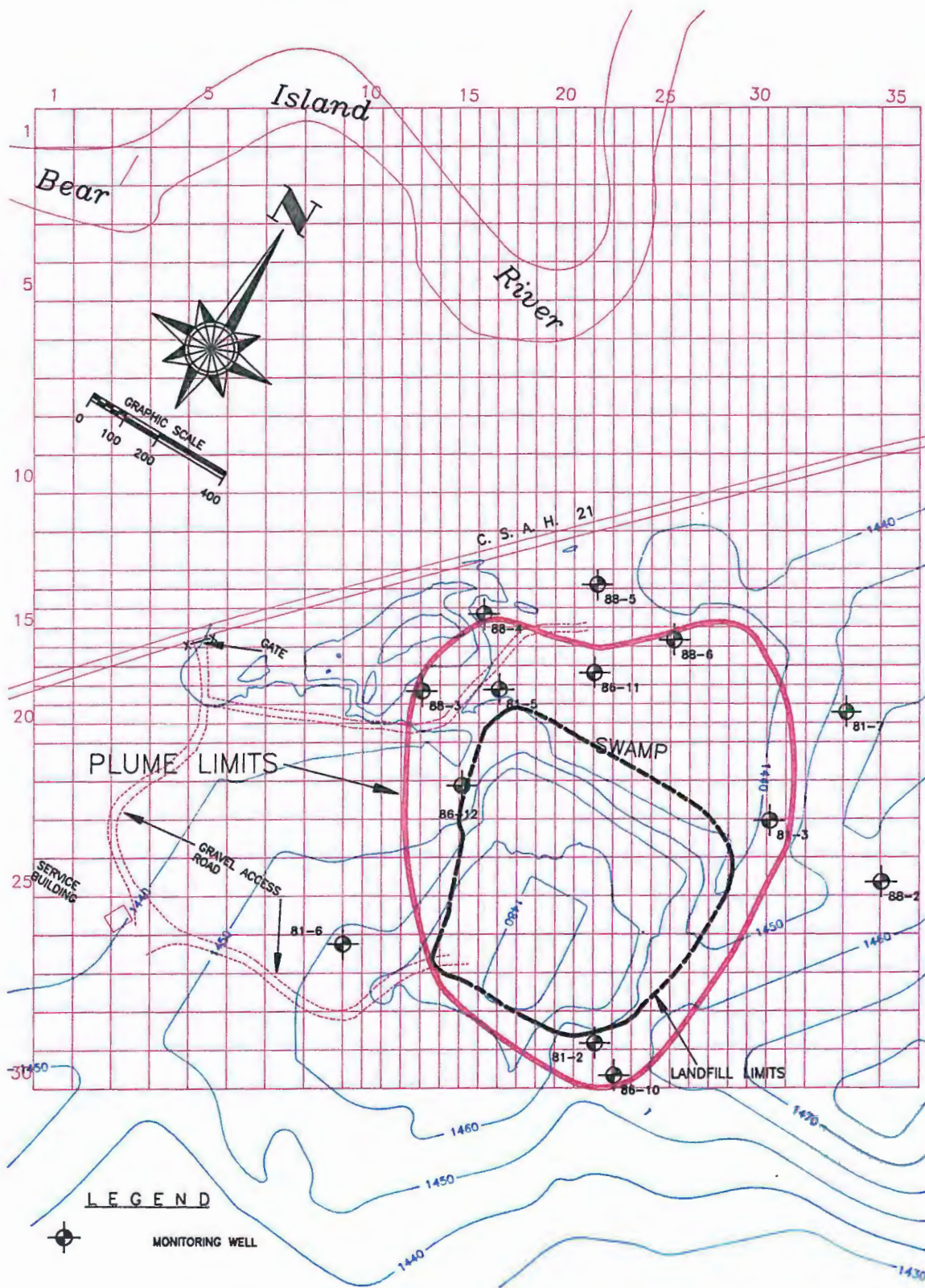


Figure 25. Modeled chloride plume - 1988 ■ Page 76

A portion of the simulated plume extends farther to the north, east of monitoring well 88-6. With no monitoring wells in that area, the plume could not be accurately estimated from available field data.

There is also an apparent plume extension to the northwest, in the area between monitoring wells 88-3 and 88-4:

Although the maximum measured chloride concentration at any monitoring well in 1988 was less than 400 mg/l, the maximum computed concentration in the calibrated model is 1200 mg/l. This concentration occurs approximately 100 feet north of the north edge of the landfill. Computed chloride concentrations at some model cells within the landfill are in the 500 to 900 mg/l range.

Sensitivity Analysis

Sensitivity analyses were made by varying source loading and dispersivity values. Increasing the source loading 50 percent—to 15 pounds per day—caused the chloride concentrations to be considerably higher than observed.

Longitudinal dispersivity was varied from 30 feet (9.1 meters) to 100 feet (30.48 meters), with the transverse dispersivity varied to maintain a longitudinal/transverse ratio of ten. The larger dispersivity values caused the plume to elongate sufficiently, such that computed downgradient concentrations did not acceptably simulate measured values. At a longitudinal dispersivity of 30 feet, the reverse was true. The chloride plume front did not extend sufficiently downgradient to simulate observed conditions. It is concluded that dispersivity values of 60 feet (18.3 meters) and 6 feet (1.8 meters), longitudinal and transverse respectively, are representative values for the glacial sediments at the site.

Predicted Plume Configuration

The landfill was capped with a clay barrier layer in 1990; therefore, in the predictive simulation, the source loading rate was reduced to approximately 40 percent of the loading used prior to capping, due to a projected reduction in leachate generation. To maintain the total number of particles within the maximum permitted in the computer software, the unit particle mass was increased to 40 pounds.

Using calibrated parameters, the predicted chloride (leachate) plume configuration for year 2010 is shown in Figure 26. The leachate plume will have reached the Bear Island River by year 2010, although chloride concentrations are expected to be well below the chronic standard of 230 mg/l for Minnesota Class 2 surface waters for fisheries and recreation.

As shown in Figure 26, the downgradient plume has a second front, which extends to within 250 feet of the river. The separation of the plume into two fronts is dictated by the bedrock topography, as shown in Figure 21.

Organic Compound Transport

Adsorption of organic chemicals onto organic matter in the porous medium is an important reaction affecting the movement of organics in an aquifer. The hydrophobic characteristic of certain organic molecules results in the partitioning of organic compounds between water and organic matter and diminishes the solubility of these organics in water.

To demonstrate the variation in organic compound transport at the study site, three organics detected in both the leachate sample and groundwater were evaluated. The three compounds consist of an aromatic hydrocarbon (benzene), a halogenated hydrocarbon (methylene chloride) and an organic solvent (tetrahydrofuran). These compounds exhibit a relatively wide range of solubility and affinity for sorption onto organic carbon. Using the mean organic carbon content (f_{oc}) for the study site sediments of 2.46 percent by weight, a

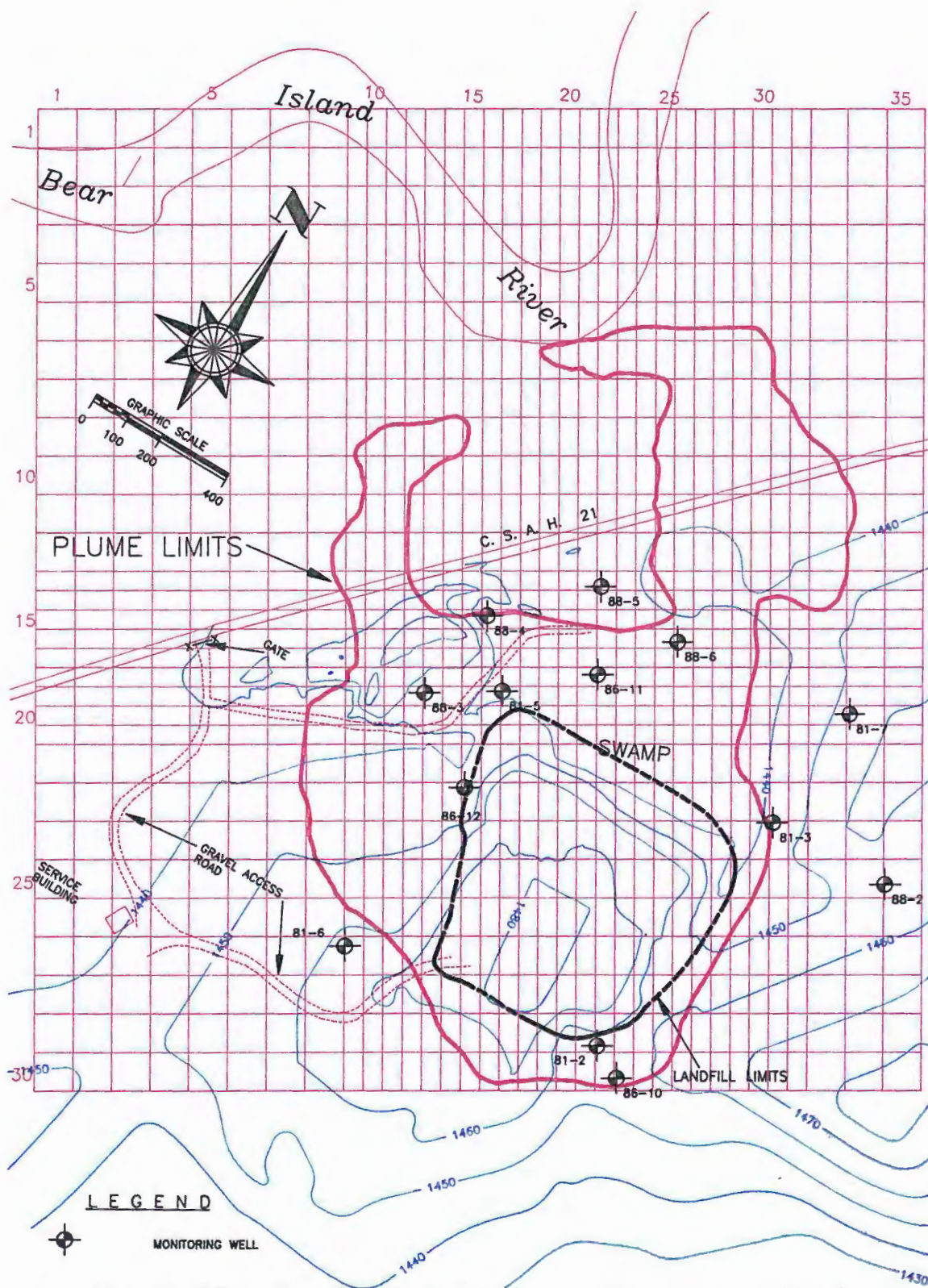


Figure 26. Predicted leachate plume – year 2010
 (Based on chloride transport)

soil bulk density of 1.75 grams/cm³, and porosity of 0.25, retardation factors were computed, as shown in Table 13.

**TABLE 13
RETARDATION FACTORS**

Compound	$K_{oc}^{(1)}$	$K_d^{(2)}$	Retardation Factor (R)
Benzene	97	2.4	18
Methylene Chloride (Dichloromethane)	25	0.6	5.2
Tetrahydrofuran	1	0.025	1.2

⁽¹⁾ organic carbon-water partition coefficient (Fetter, 1988).

⁽²⁾ distribution coefficient: $K_d = K_{oc}f_{oc}$.

Transport simulations were made for each of the three organics to calibrate to measured concentrations in the groundwater at downgradient monitoring wells, using the previously calibrated dispersivity values and computed retardation factors. Source loadings were varied to obtain a satisfactory match to observed data.

Benzene

A continuous source loading of 0.02 pounds per day was used for the period from 1972 to 1990. The source loading was reduced to 40 percent of the above value after 1990, reflecting reduced leachate generation after landfill closure. A unit particle mass of 0.1 pounds was used in the simulation.

Figure 27 contains the predicted benzene plume limits for year 2010. Note that benzene movement is significantly restricted in comparison to the predicted leachate (chloride) plume shown in Figure 26. The benzene predictive simulation contained a total of 1998 particles representing the contaminant mass. Maximum concentration within the plume is estimated to be approximately 550 µg/l. This occurs within the landfill limits.

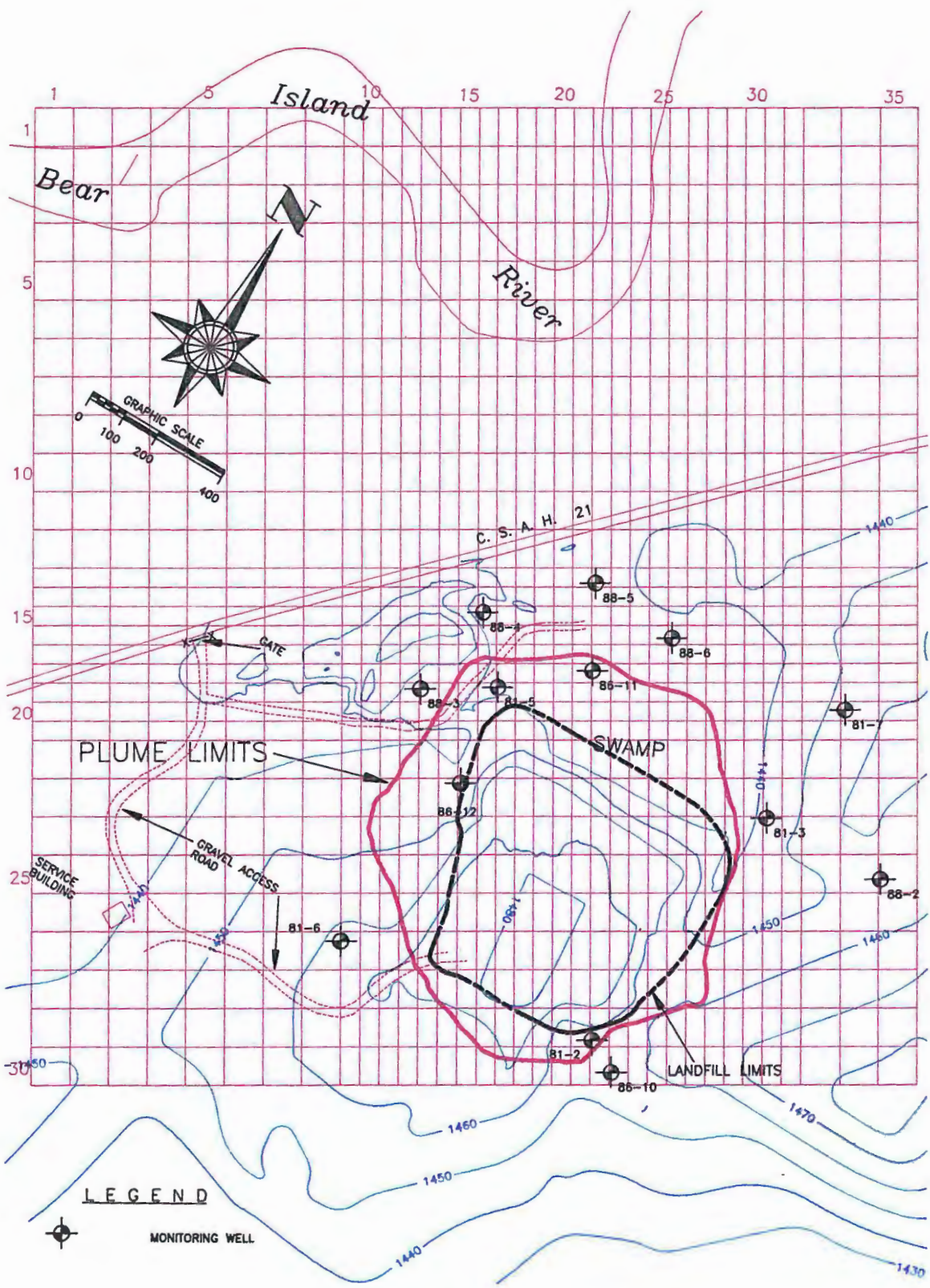


Figure 27. Predicted benzene plume – year 2010 ■ Page 81

In addition to the landfill Intervention Limit of $3 \mu\text{g/l}$ and the Recommended Allowable Limit for benzene in drinking water of $10 \mu\text{g/l}$, a chronic standard of $5.9 \mu\text{g/l}$ has been established for State of Minnesota Class 2 waters for fisheries and recreation. It appears highly unlikely that the benzene plume will reach the Bear Island River before the total benzene leaching potential of the landfill has been reached. It appears that this "refuse stabilization" may also occur before the concentration of benzene equals the Intervention Limit at the downgradient monitoring wells installed in 1988 as part of this research project.

Methylene Chloride

The calibrated source loading rate is 0.02 pounds per day for 1972-1990, with a rate of 0.008 pounds per day for the period from 1990-2010. Utilizing a unit particle mass of 0.075 pounds, 2405 discrete particles were required to simulate the movement of methylene chloride to year 2010. The predicted plume is shown in Figure 28. As expected, the limits of this plume extend farther from the source than the benzene plume. The plume begins to extend to the north and to the northwest, showing contaminant movement corridors similar to the chloride plume (Figures 25 and 26). The maximum computed concentration within the plume is $1800 \mu\text{g/l}$, occurring approximately 50 feet north of the north landfill limits.

The established chronic standard for methylene chloride in State of Minnesota Class 2 waters (fisheries) is $45 \mu\text{g/l}$, compared to an RAL of $50 \mu\text{g/l}$ and a landfill Intervention Limit of $12 \mu\text{g/l}$. The simulation analysis indicates that the Intervention Limit will be reached or exceeded at downgradient monitoring well 88-6 and possibly at well 88-3. It appears unlikely that any significant concentration of methylene chloride will reach the Bear Island River.

Tetrahydrofuran

Tetrahydrofuran represents those organics with little affinity toward sorption onto organic matter in the subsurface environment, as indicated by its

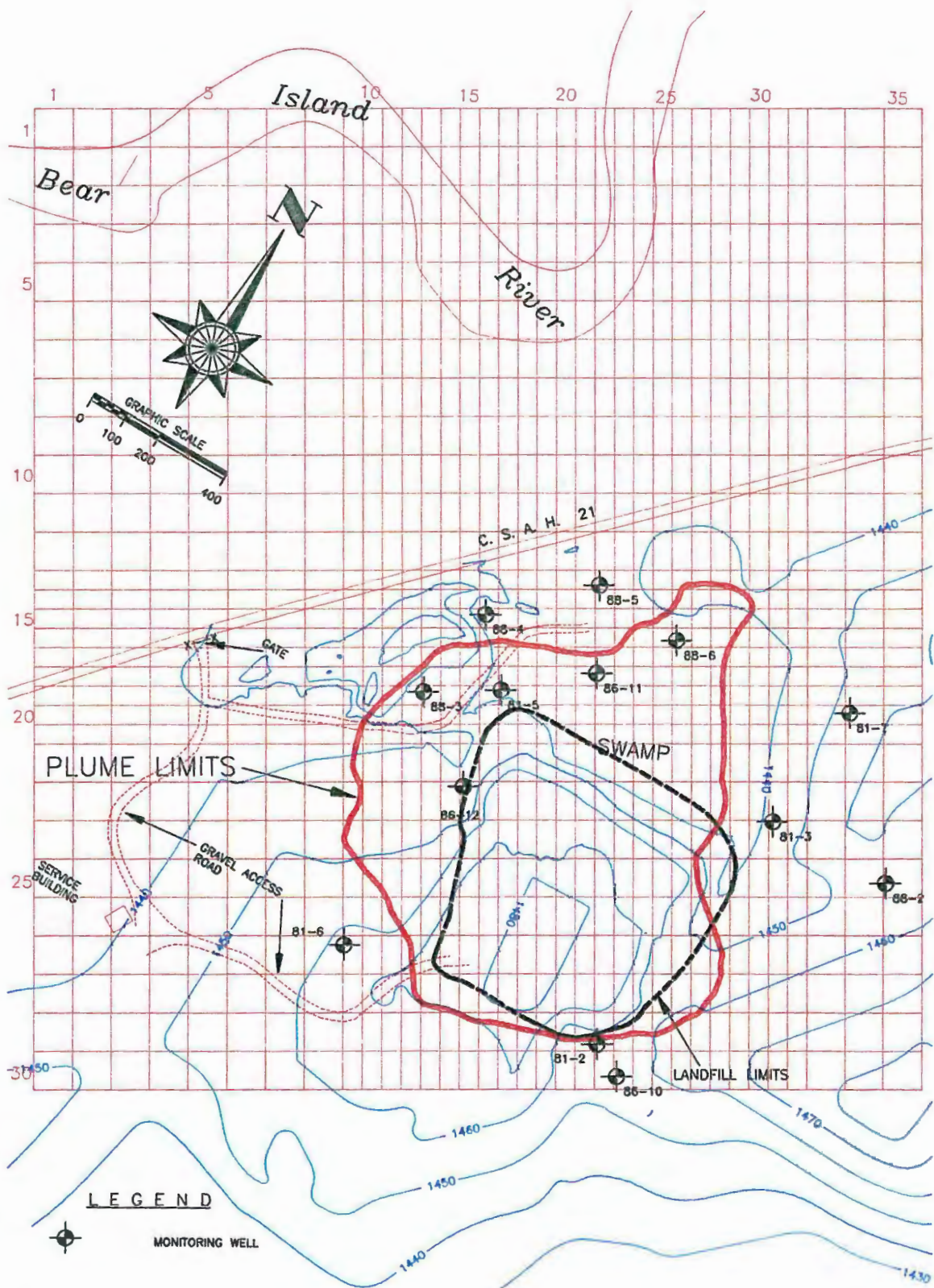


Figure 28. Predicted methylene chloride plume – year 2010 ■ Page 83

low retardation factor. As shown in Figure 29, it is expected that the plume will reach the Bear Island River by year 2010. The tetrahydrofuran plume is similar in shape and extent to the predicted chloride plume (Figure 26).

The source loading rate used in the simulation is 0.01 pounds per day for 1972-1990, decreasing to 0.004 pounds per day after 1990. Using a particle mass of 0.04 pounds, 2535 particles are used to simulate contaminant movement and compute tetrahydrofuran concentrations to year 2010. Maximum computed concentration within the predicted plume is 6600 $\mu\text{g/l}$, occurring approximately midway between the north edge of the landfill and well 86-11.

The Recommended Allowable Limit for tetrahydrofuran in drinking water is 100 $\mu\text{g/l}$; however, there is no established landfill Intervention Limit or fisheries chronic standard. Assuming that refuse stabilization with respect to tetrahydrofuran has not been achieved by year 2010, the predicted concentration at well 88-3 and well 88-6 is 100 $\mu\text{g/l}$ and 75 $\mu\text{g/l}$, respectively. Computed concentrations at downgradient wells 81-5 and 86-11 are 700 and 390 $\mu\text{g/l}$, respectively, in year 2010.

Metals Transport

Surface reactions also play an important role in metals contamination attenuation. Several heavy metals adsorb readily onto solid surfaces in the groundwater flow system or are attracted to clay minerals through cation exchange.

As previously presented, heavy metal concentrations are relatively low in the leachate (Table 5) and groundwater (Table 9) at the study site. Chromium was selected for transport modeling, as it was detected in downgradient monitoring wells, albeit at concentrations significantly below the landfill Intervention Limit.

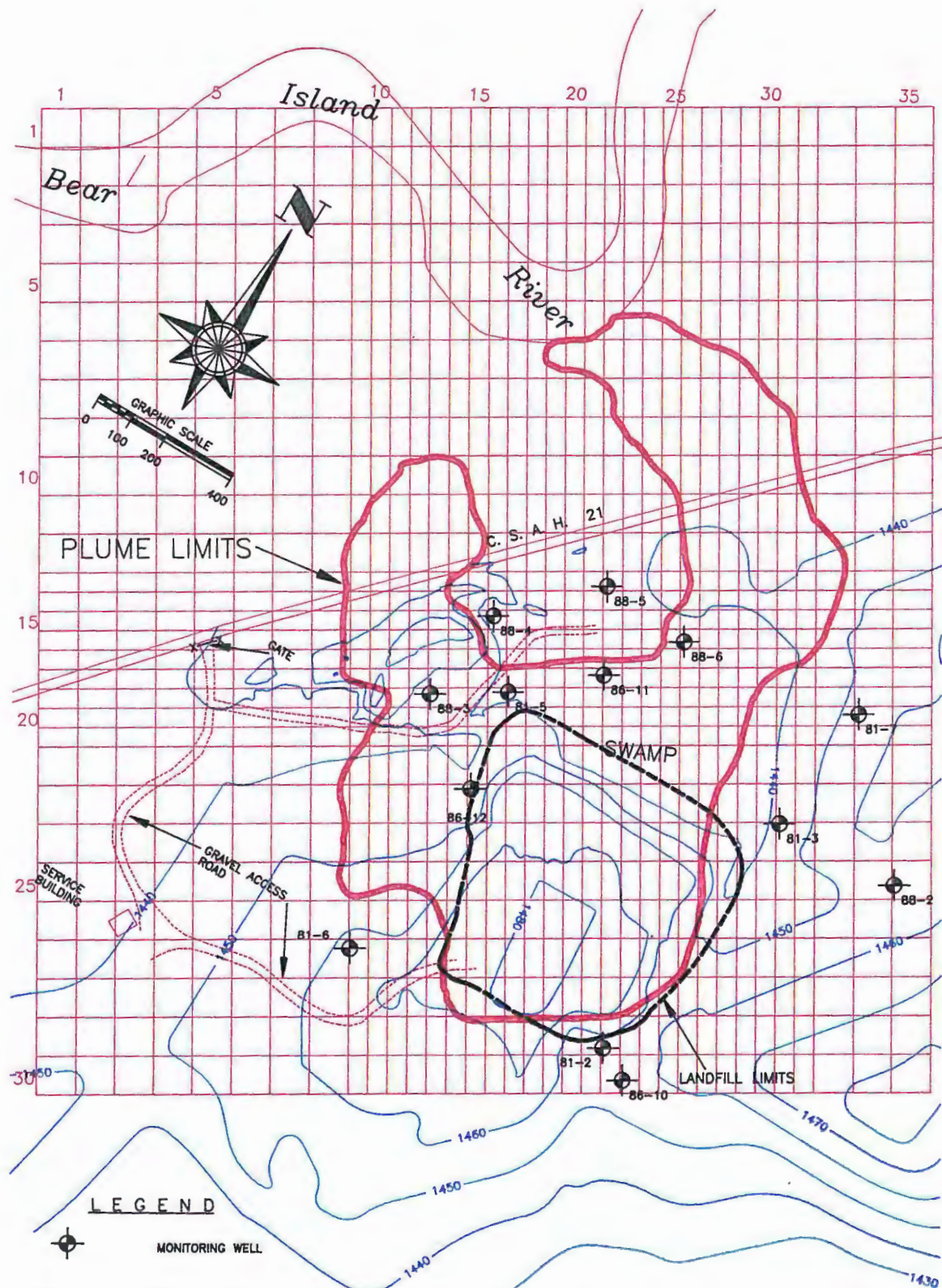


Figure 29. Predicted tetrahydrofuran plume — year 2010

Chromium

Concentration of chromium in the leachate sample was 130 $\mu\text{g/l}$. This compares to an RAL of 100 $\mu\text{g/l}$, an Intervention Limit of 30 $\mu\text{g/l}$, and a chronic standard for fisheries waters ranging from 117 to 365 $\mu\text{g/l}$, depending upon water hardness.

Based on the leachate concentration, a source loading of 0.004 pounds per day was used for 1972-1990, with a reduction to 40 percent of this loading subsequent to landfill closure. Using a discrete particle mass of 0.015 pounds, transport simulations were made to obtain a satisfactory match between observed and simulated concentrations. As a result of this calibration, a retardation factor equal to ten was determined to be appropriate for this site.

A total of 2172 particles were used to model the contamination movement to year 2010. Figure 30 contains the predicted plume limits in 2010. The chromium plume is relatively restricted with a computed maximum concentration of 156 $\mu\text{g/l}$ located approximately 100 to 150 feet south of well 86-11.

It is projected that the plume limits will not reach the downgradient 1988 monitoring wells by 2010. It is highly unlikely that any significant chromium will enter Bear Island River from the landfill source.

Refuse Stabilization

Solute transport analysis in this study was based on a constant loading rate to the groundwater flow system for each contaminant. It is expected, however, that at some point in time, the total leaching potential of each contaminant is reached, for a given refuse quantity. This "stabilized state" is probably the result of both continued leaching and refuse decomposition.

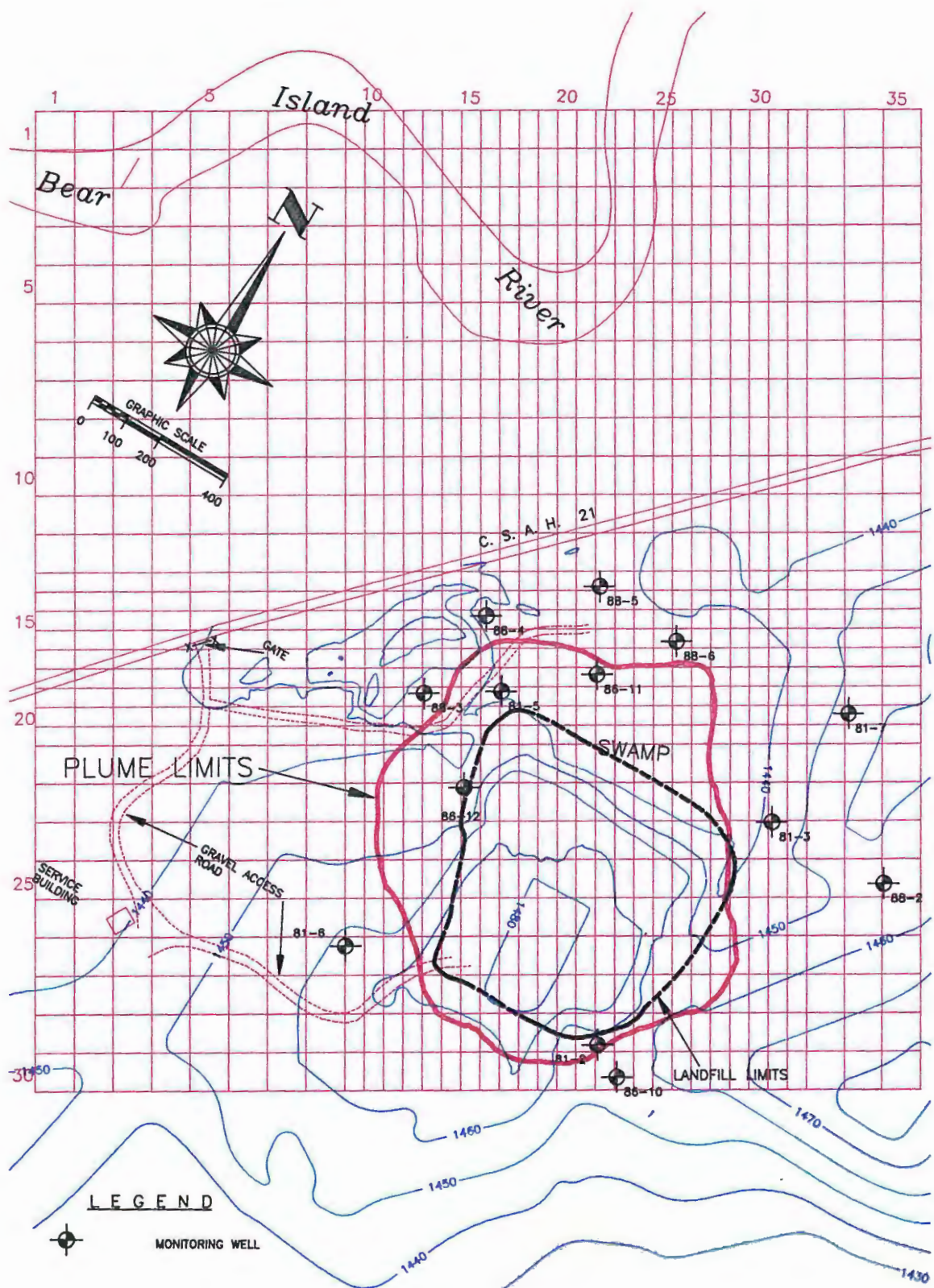


Figure 30. Predicted chromium plume – year 2010 ■ Page 87

McGinley and Kmet (1984) provide data estimating the refuse stabilization time frame. Two methods are used to establish time frame limits. The upper limit is defined based on the generation of three liters of leachate per kilogram of solid waste. A second method, identifying the lower limit, considers that stabilization is reached when a total of 95 inches of leachate has been generated, subsequent to saturation of the waste. Based on the refuse depth at the study site, the lower and upper stabilization time limits range from 30 to 90 years. McGinley and Kmet (1984) suggest that the first method probably overestimates the stabilization time requirement and the second method likely underestimates the time period.

The effect of refuse stabilization will be a subsequent reduction in contaminant concentrations in the groundwater, due to a limiting source and ongoing dispersion and reactions within the aquifer.

SUMMARY AND CONCLUSIONS

The study site was used as a mixed municipal solid waste land disposal facility from 1972 to 1990. The landfill was closed in May 1990, with final capping completed in September 1990.

The site is underlain by Lower Precambrian Giants Range Granite. The bedrock slopes north-northwest from a northeast-southwest trending bedrock ridge located near the southeast edge of the landfill. This site is located within the Vermilion Moraine of the Rainy Lobe. The depth of glacial till overlying the bedrock varies from approximately 5 feet to more than 25 feet. The till consists of dense to very dense silty sand and poorly graded sand, with some sandy silt. The relatively low percentage of silt and clay confirms that the till is relatively immature.

The aquifer at the site is shallow and unconfined, with a saturated thickness varying from less than 5 feet to slightly more than 20 feet. Based on field tests at eight locations at the site, the arithmetic mean hydraulic conductivity is 1.7×10^{-4} cm/sec (0.48 ft/day), and the geometric mean value is 8.1×10^{-5} cm/sec (0.23 ft/day). Average linear groundwater velocity, beneath and immediately downgradient of the landfill, is estimated to be 12 to 15 feet per year.

Analysis of a leachate seep sample identified contaminant concentrations in the lower one-half to one-quarter of typical concentration ranges found in landfill leachate.

Groundwater was sampled at ten monitoring wells for this study. Leachate indicators in the groundwater, chloride and specific conductance provide an indication of leachate plume limits. Heavy metal concentrations in the groundwater are less than established landfill Intervention Limits and/or less than the minimum detection limits.

Seventeen organic compounds were detected at one or more groundwater sampling locations. Minnesota Pollution Control Agency landfill Intervention Limits were exceeded at one or more sampling locations for acetone, benzene, methyl ethyl ketone, methylene chloride, toluene and vinyl chloride. Minnesota Department of Health Recommended Allowable Limits for drinking water were exceeded for benzene and vinyl chloride at the three most immediate downgradient monitoring wells 81-5, 86-11 and 86-12. Toluene concentrations exceeded its RAL at wells 86-11 and 86-12, and the RALs for acetone and methyl ethyl ketone were exceeded at well 86-11.

A finite-difference numerical model (MODFLOW) was used to model groundwater flow in this study. The study area, encompassing approximately 135 acres, was discretized into a rectangular grid system, consisting of 35 columns and 30 rows. Compared to the glacial till, the underlying bedrock is considered to be impervious; therefore, the groundwater flow system was modeled as a single layer. The Bear Island River, the primary downgradient receptor, serves as a groundwater discharge boundary and was simulated as a constant head boundary in the model. The bedrock ridge, approximated by row 30 in the model, was used as a no-flow barrier boundary, as were the other flow region extremities, columns 1 and 35.

A steady-state simulation was made to calibrate the flow model to observed hydraulic head data. A constant recharge of 1.5 inches per year (40,700 gallons/acre/year) was used, with hydraulic conductivity serving as the calibration variable. A satisfactory match of computed and observed hydraulic head was obtained, after more than 50 calibration runs. Calibrated hydraulic conductivity values range from 3.05×10^{-5} cm/sec (0.09 ft/day) to 1.28×10^{-3} cm/sec (3.63 ft/day).

Sensitivity analyses confirmed that recharge and hydraulic conductivity values used to establish the final calibrated flow model are appropriate for the study area. The calibrated values for these parameters are not necessarily

unique; however, they appear reasonable and provide a good correlation to observed hydraulic head data in the flow system.

The groundwater flow model established in this study provides the mechanism to numerically simulate the effect of changed conditions or stresses in the groundwater flow system. These changes may include, but are not limited to, aquifer pumping, excessive or deficient precipitation, and surface changes affecting recharge or infiltration rates.

Solute transport modeling was conducted, utilizing the finite-difference particle transport model, InterTrans. The previously calibrated flow model was transformed into an InterSat flow model, which provides hydraulic head and aquifer parameter data as input to the transport model. InterTrans uses the random-walk technique to determine the dispersive component of discrete particle movement.

The transport model was calibrated to observed chloride concentrations to obtain longitudinal and transverse dispersivity values for the site aquifer. Utilizing the calibrated dispersivity values and retardation factors based on site-specific organic carbon content, solute transport of three organic compound—benzene, methylene chloride, and tetrahydrofuran—was simulated to identify probable plume limits by year 2010. Transport of one metal—chromium—was also simulated, based on a retardation factor determined by calibration to observed concentrations.

Table 14 contains the transport-related parameters identified in this study.

TABLE 14
SITE-SPECIFIC TRANSPORT PARAMETERS

Dispersivity	
Longitudinal	60 feet (18.3 meters)
Transverse	6 feet (1.8 meters)
Organic Carbon (Mean)	2.46 percent by weight
Retardation Factors	
Benzene	18
Methylene Chloride	5.2
Tetrahydrofuran	1.2
Chromium	10

Solute transport modeling indicates that chloride and tetrahydrofuran plumes will reach the Bear Island River by year 2010. It is not expected that either of these contaminants will have a significant detrimental effect on river water quality related to aquatic life. The modeling also shows that the plumes split into two fronts.

The benzene, methylene chloride and chromium plumes are much more restricted in areal extent. It is considered unlikely that any of these three contaminants will create any significant environmental impact on the Bear Island River.

This study provides additional data base information on hydraulic conductivity and organic carbon content of glacial till, dispersivity values and retardation factors. The transport model provides a better indication of plume configuration than could be determined from a limited number of monitoring locations. This will assist in selecting proper additional monitoring well locations, if necessary, in the future. The transport model can also be used as a predictive tool and can be used to analyze the effect on contaminant transport of changes in groundwater flow conditions and contaminant source loading.

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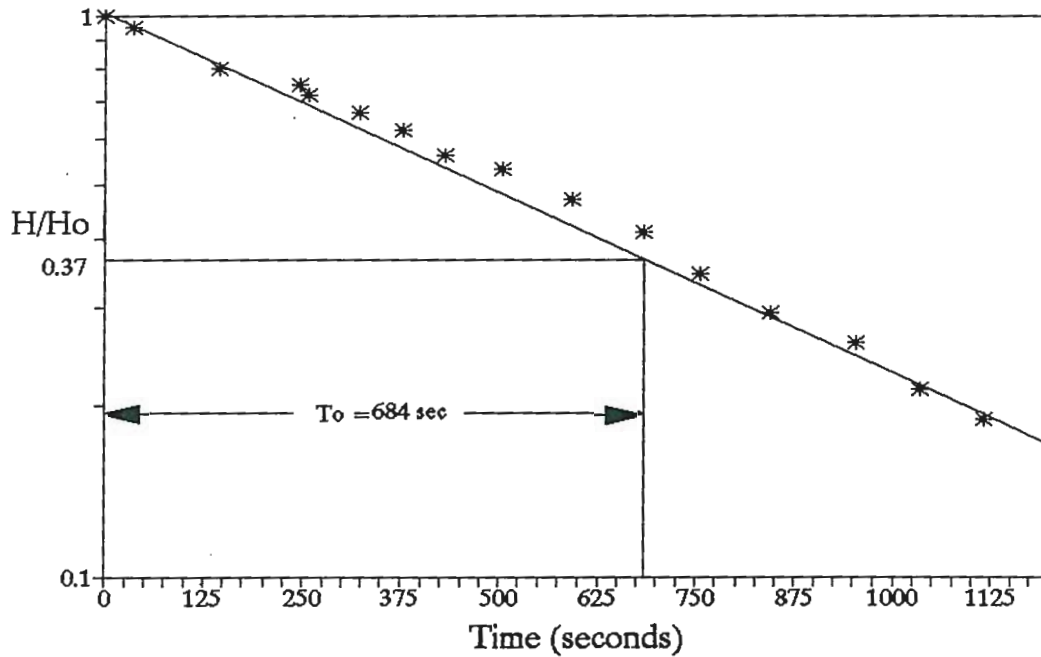
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APPENDIX A

FIELD HYDRAULIC CONDUCTIVITY

Head Ratio vs. Time - Hvorslev Method
WELL 81-3



$$K = \frac{r_c^2 \ln(L/R)}{2 L T_o} = \text{Hydraulic Conductivity}$$

$$T_o = 684 \text{ seconds} = \text{Basic Time Lag}$$

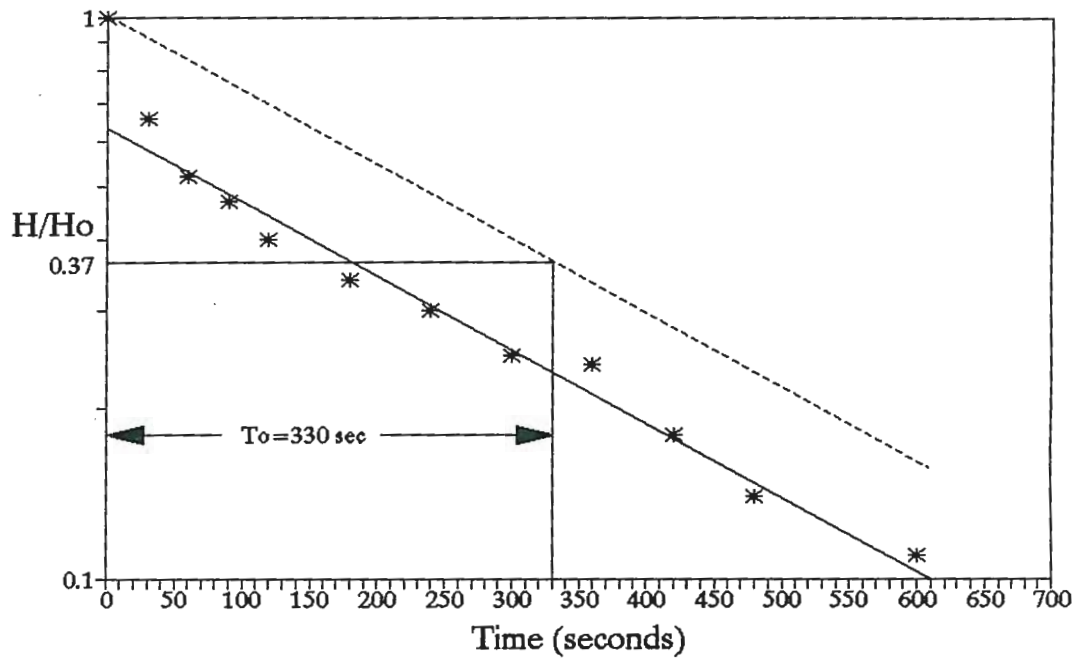
$$R = r_c = 2.54 \text{ cm} = \text{Casing/Screen Radius}$$

$$L = 152.4 \text{ cm} = \text{Screen Length}$$

$$L/R = 60$$

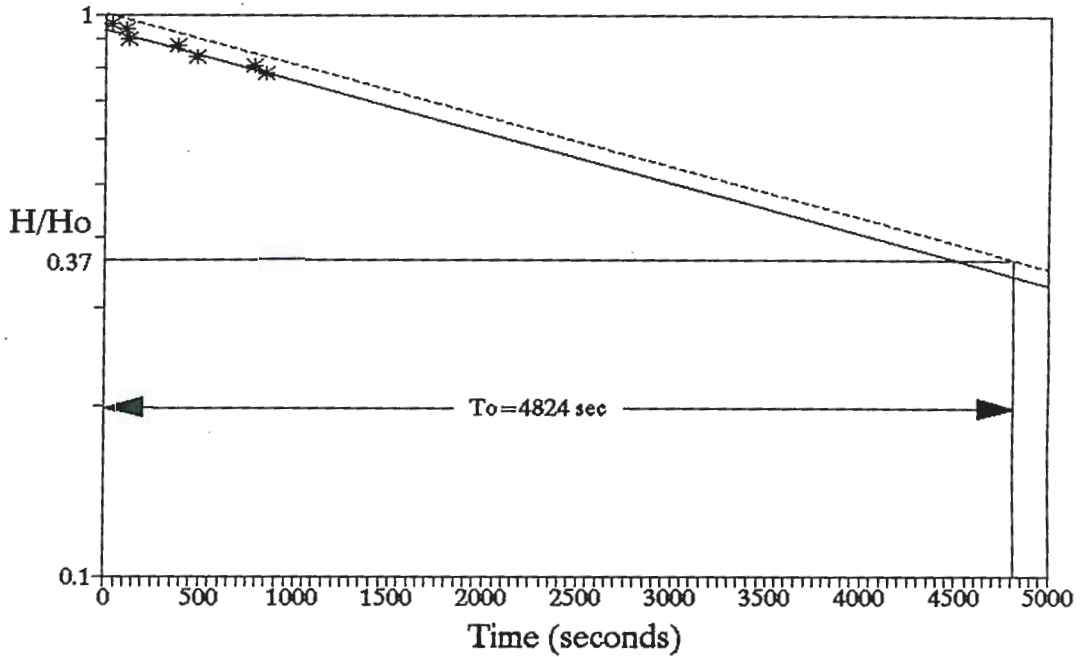
$$K = \frac{(2.54)^2 \ln(60)}{2(152.4)(684)}$$
$$= 1.3 \times 10^{-4} \text{ cm/sec}$$

Head Ratio vs. Time - Hvorslev Method
WELL 81-5



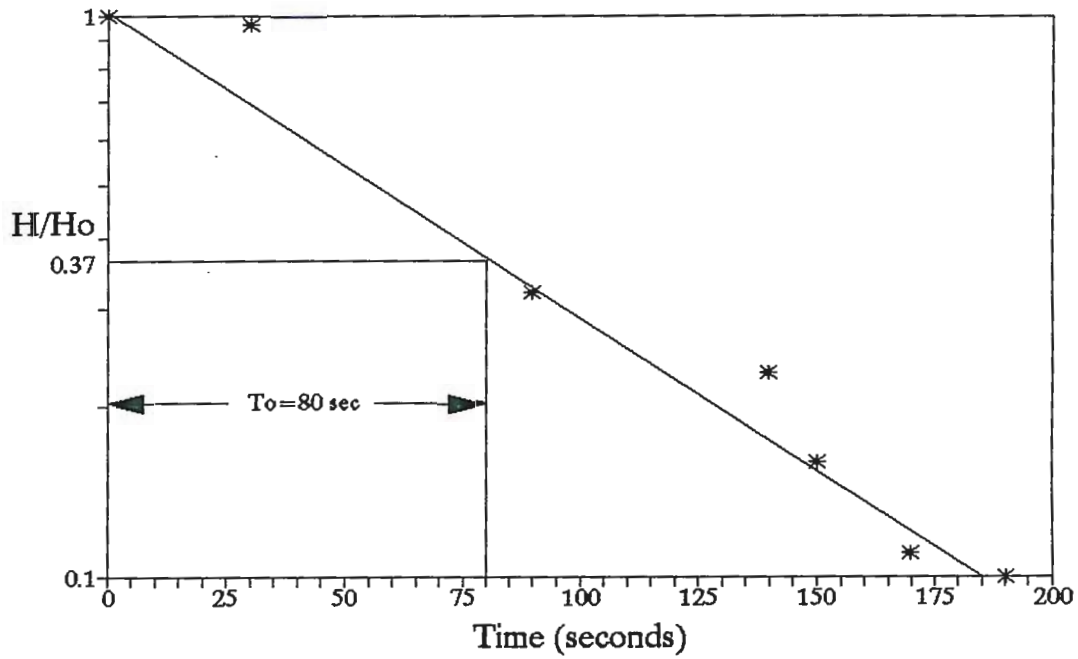
$$\begin{aligned}
 T_o &= 330 \text{ seconds} \\
 L &= 152.4 \text{ cm} \\
 L/R &= 60 \\
 K &= \frac{(2.54)^2 \ln(60)}{2 (152.4) (330)} \\
 &= 2.6 \times 10^{-4} \text{ cm/sec}
 \end{aligned}$$

Head Ratio vs. Time - Hvorslev Method
WELL 81-6



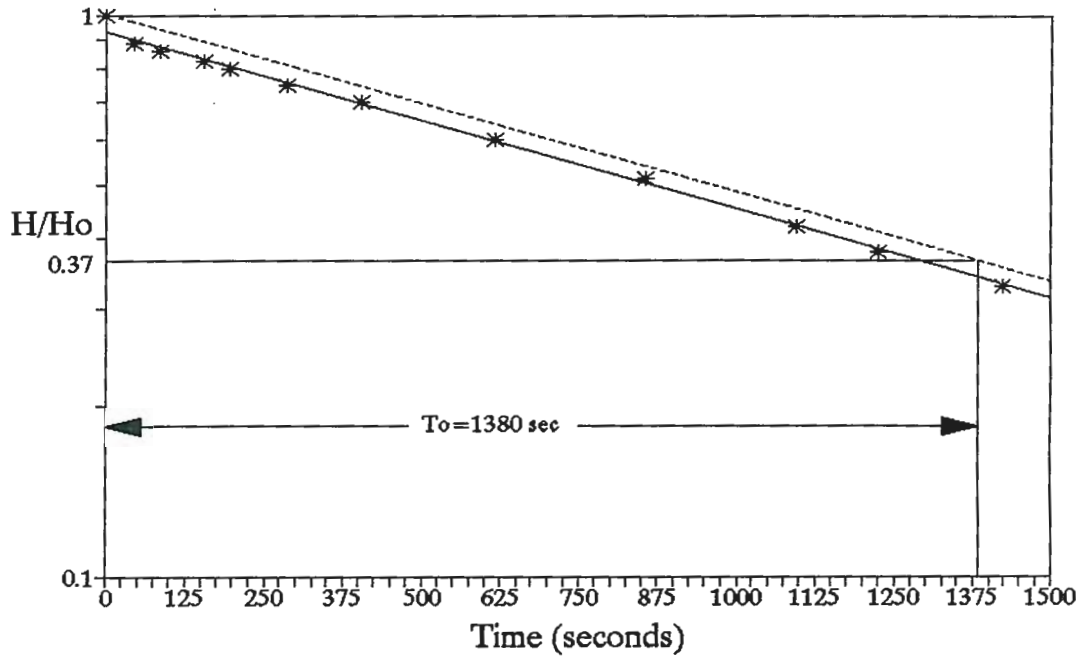
$$\begin{aligned}
 T_o &= 4824 \text{ seconds} \\
 L &= 2.8 \text{ feet} = 85.34 \text{ cm} \\
 L/R &= 33.6 \\
 K &= \frac{(2.54)^2 \ln(33.6)}{2 (85.34) (4824)} \\
 &= 2.8 \times 10^{-5} \text{ cm/sec}
 \end{aligned}$$

Head Ratio vs. Time - Hvorslev Method
WELL 86-11



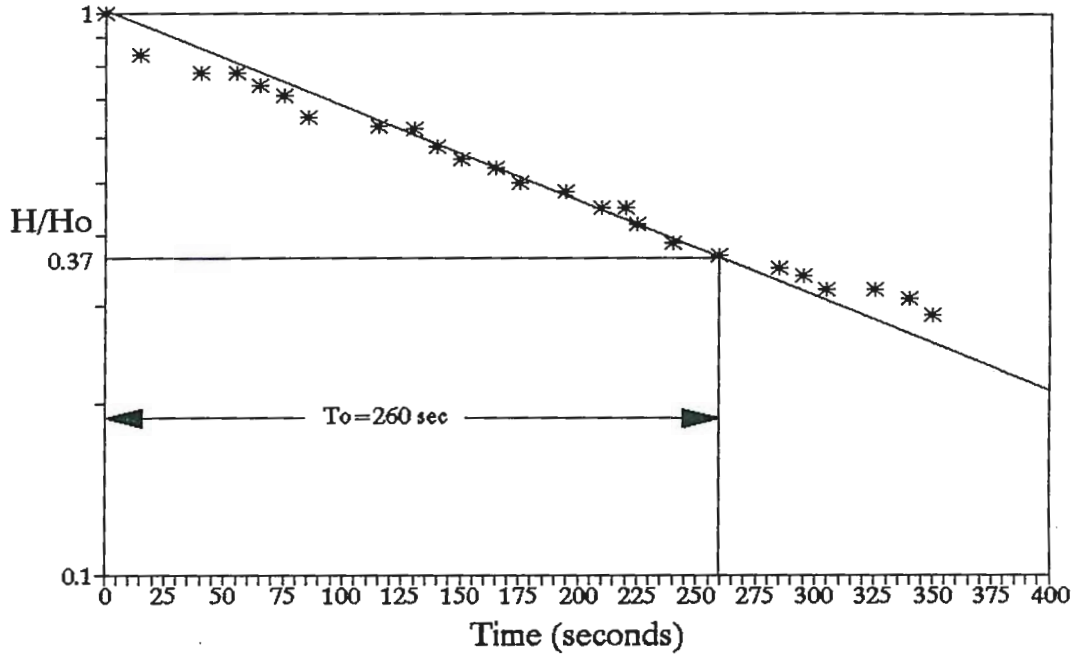
$$\begin{aligned}
 T_o &= 80 \text{ seconds} \\
 L &= 10 \text{ feet} = 304.8 \text{ cm} \\
 L/R &= 120 \\
 K &= \frac{(2.54)^2 \ln(120)}{2 (304.8) (80)} \\
 &= 6.3 \times 10^{-4} \text{ cm/sec}
 \end{aligned}$$

Head Ratio vs. Time - Hvorslev Method
WELL 86-12



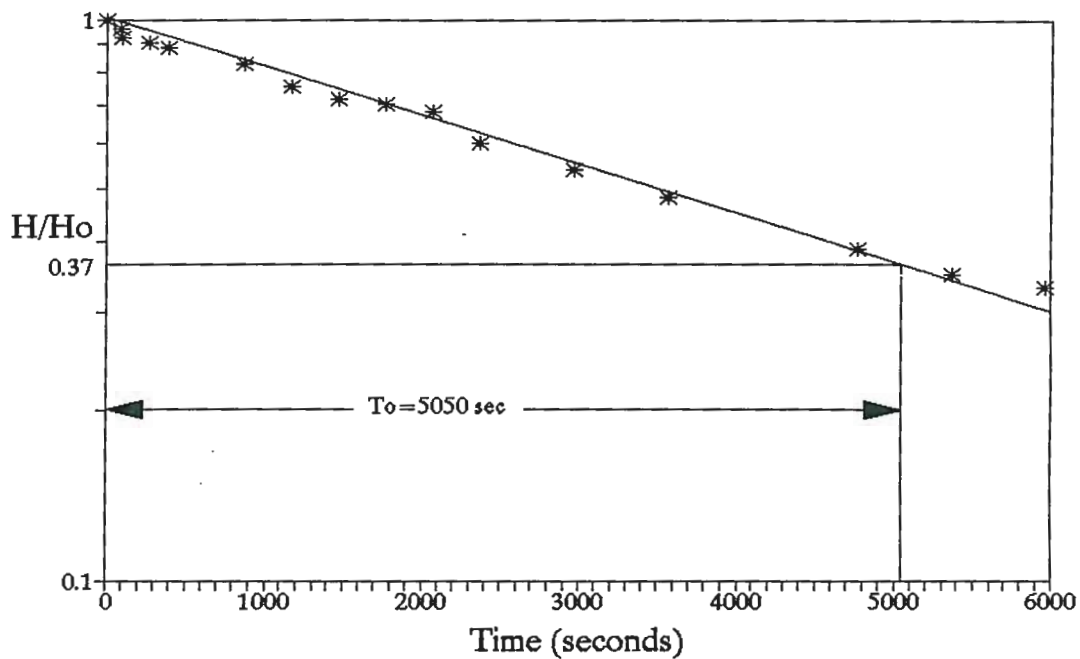
$$\begin{aligned}
 T_o &= 1380 \text{ seconds} \\
 L &= 10 \text{ feet} = 304.8 \text{ cm} \\
 L/R &= 120 \\
 K &= \frac{(2.54)^2 \ln(120)}{2 (304.8) (1380)} \\
 &= 3.7 \times 10^{-5} \text{ cm/sec}
 \end{aligned}$$

Head Ratio vs. Time - Hvorslev Method
WELL 88-4



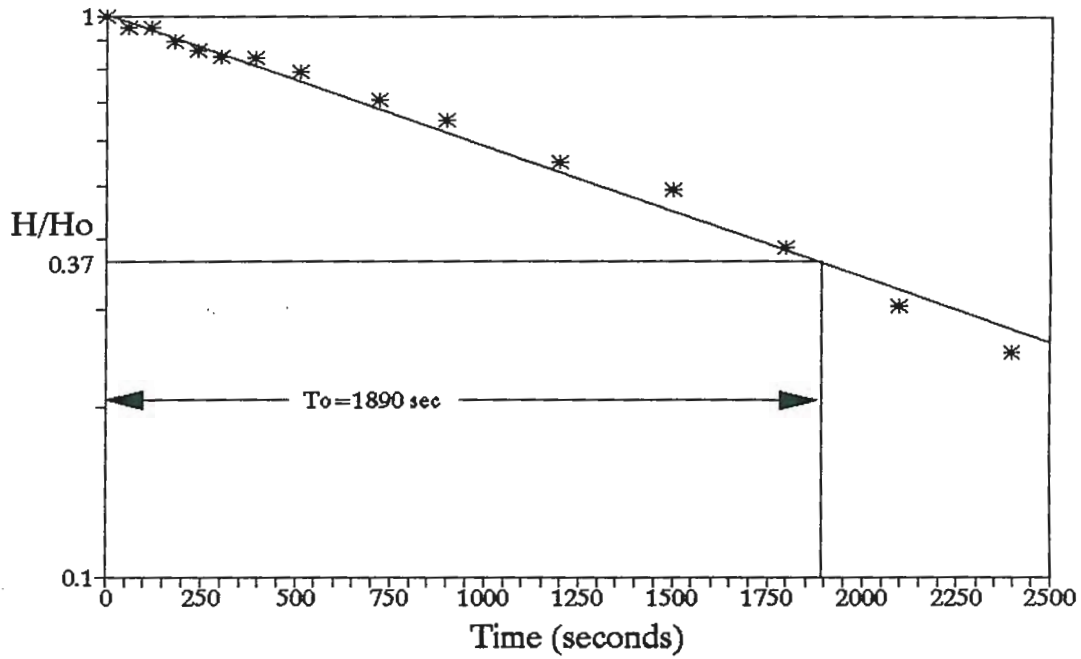
$$\begin{aligned}
 T_o &= 260 \text{ seconds} \\
 L &= 4.2 \text{ feet} = 128 \text{ cm} \\
 L/R &= 50.4 \\
 K &= \frac{(2.54)^2 \ln(50.4)}{2(128)(260)} \\
 &= 3.8 \times 10^{-4} \text{ cm/sec}
 \end{aligned}$$

Head Ratio vs. Time - Hvorslev Method
WELL 88-5



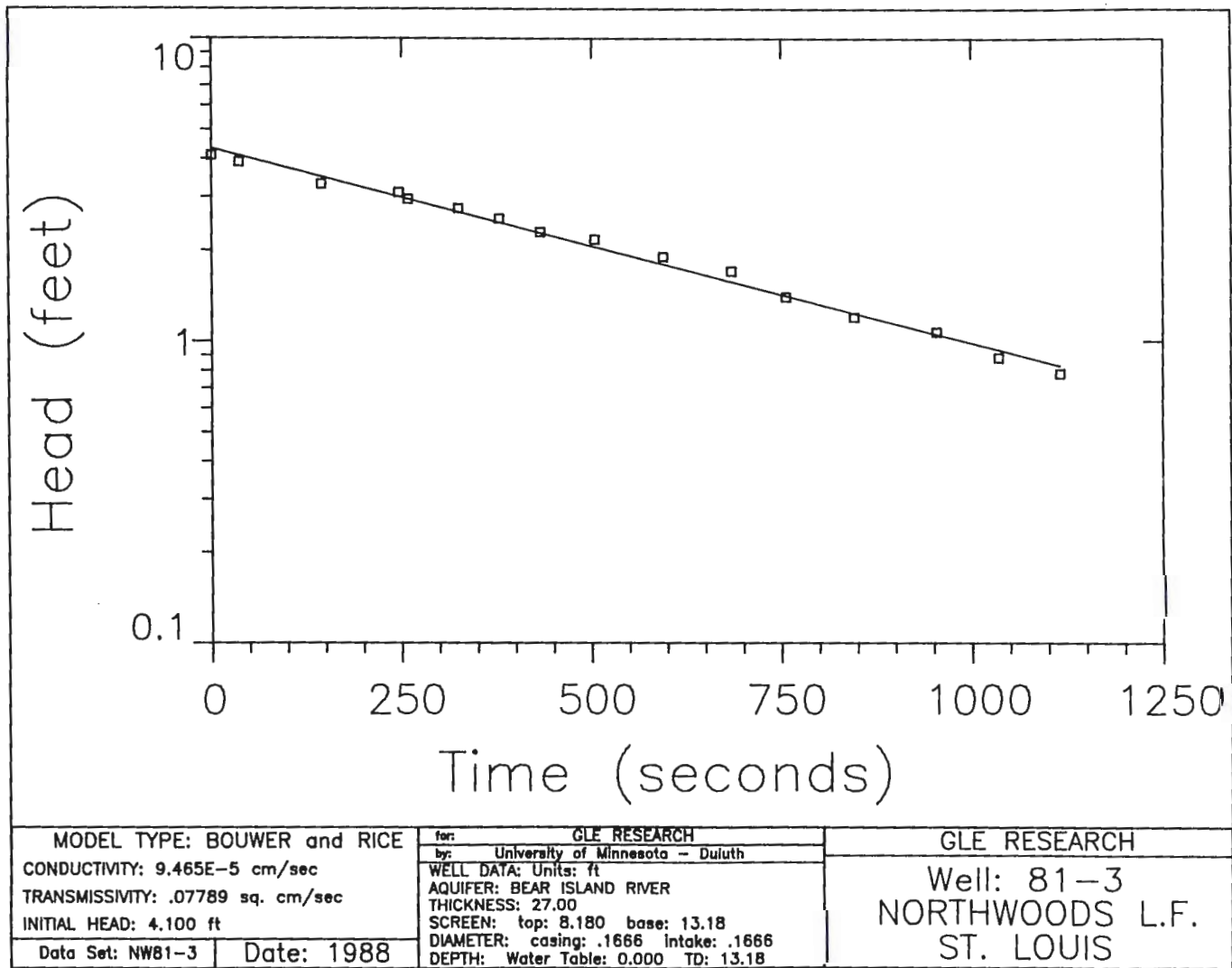
$$\begin{aligned}
 T_o &= 5050 \text{ seconds} \\
 L &= 5 \text{ feet} = 152.4 \text{ cm} \\
 L/R &= 60 \\
 K &= \frac{(2.54)^2 \ln(60)}{2 (152.4) (5050)} \\
 &= 1.7 \times 10^{-5} \text{ cm/sec}
 \end{aligned}$$

Head Ratio vs. Time - Hvorslev Method
WELL 88-6

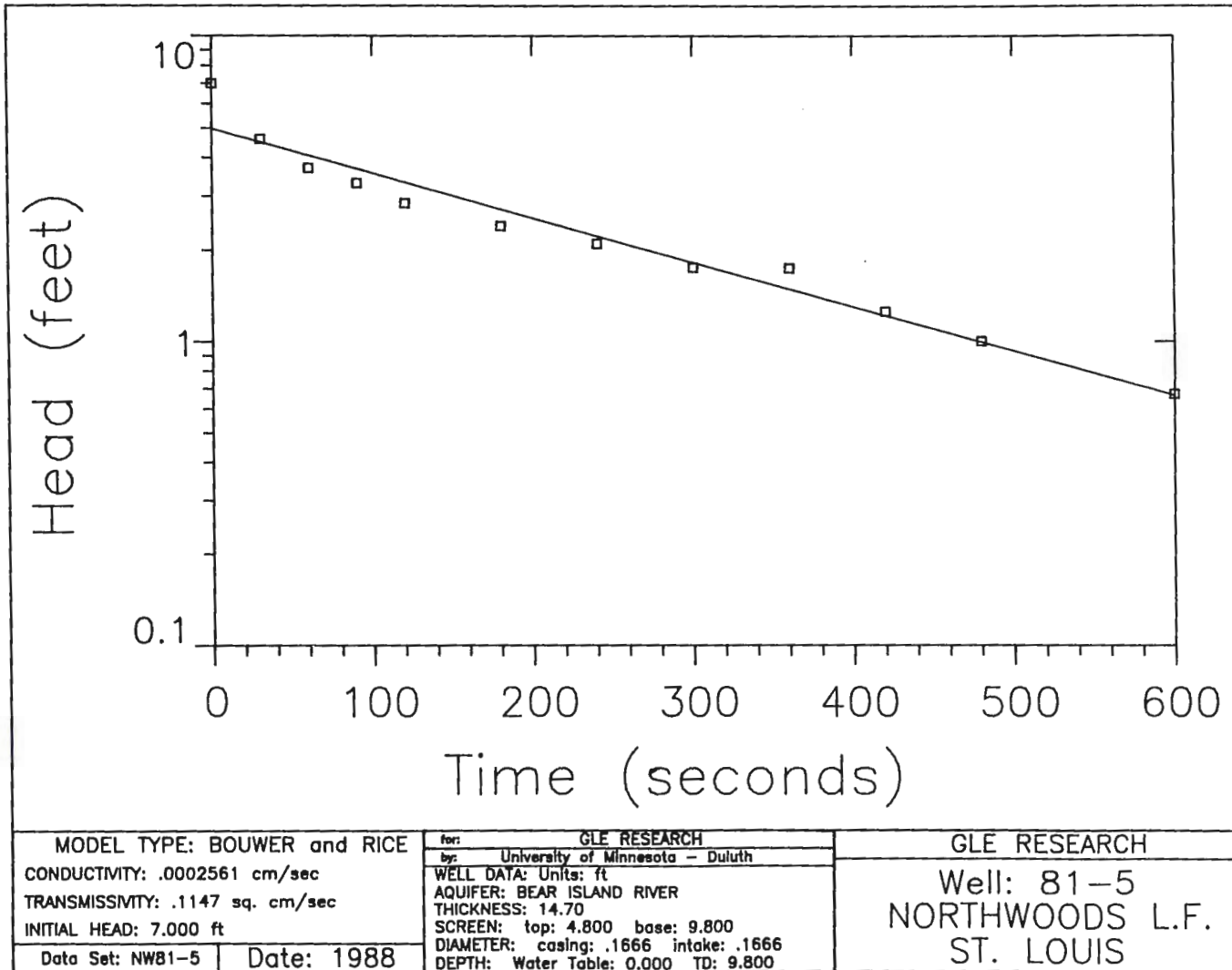


$$\begin{aligned}
 T_o &= 1890 \text{ seconds} \\
 L &= 5 \text{ feet} = 152.4 \text{ cm} \\
 L/R &= 60 \\
 K &= \frac{(2.54)^2 \ln(60)}{2 (152.4) (1890)} \\
 &= 4.6 \times 10^{-5} \text{ cm/sec}
 \end{aligned}$$

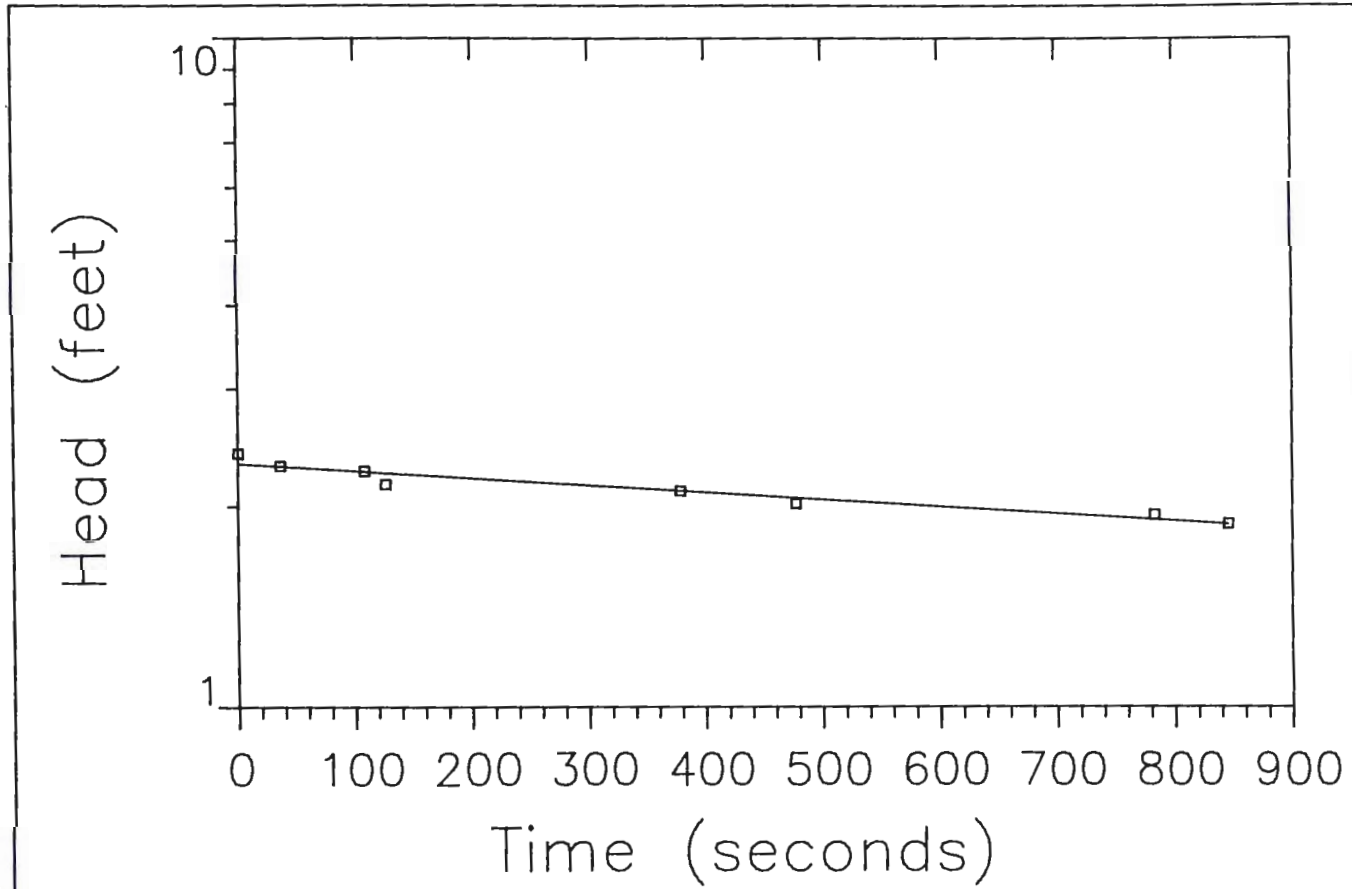
A-9



A-10

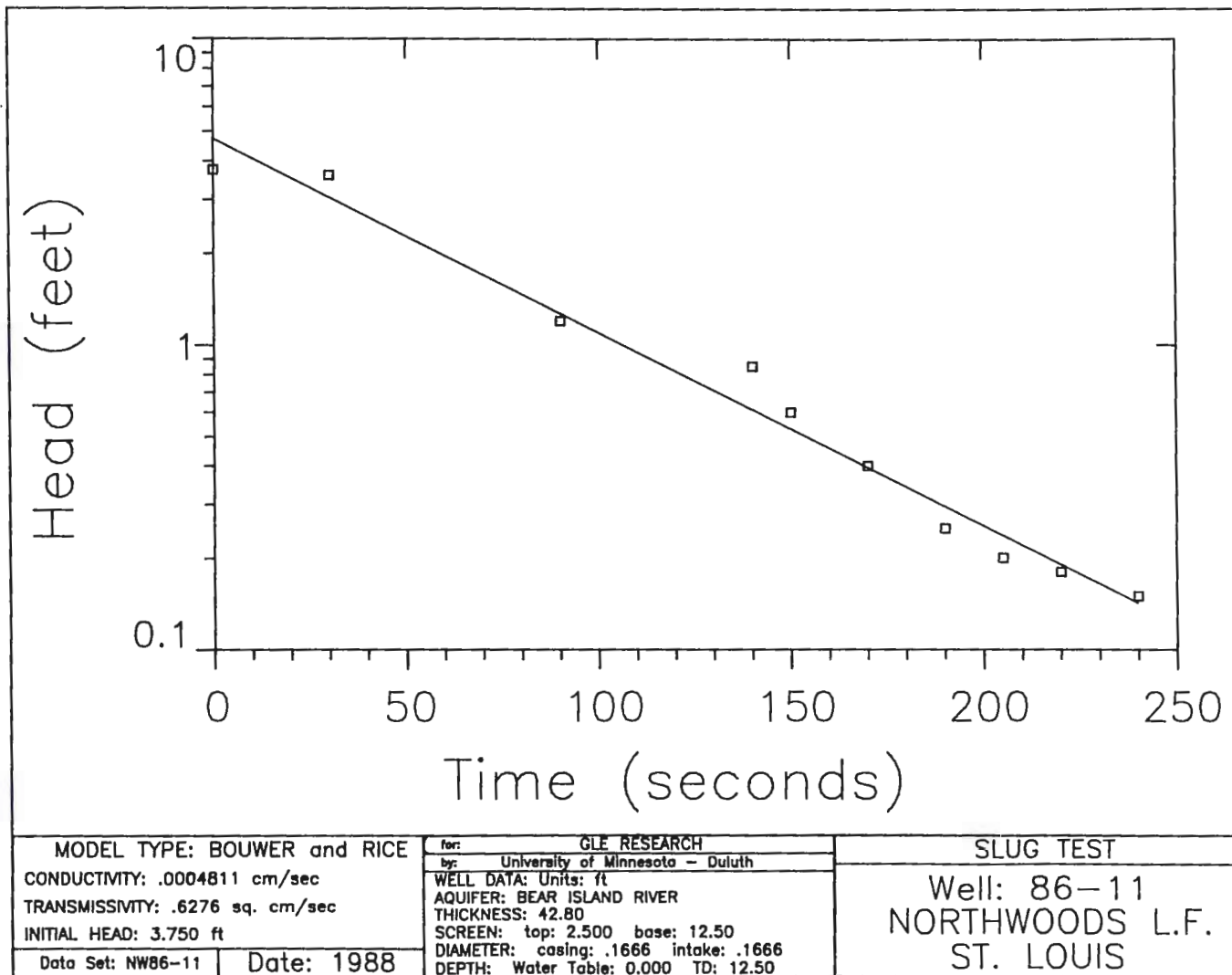


A-11

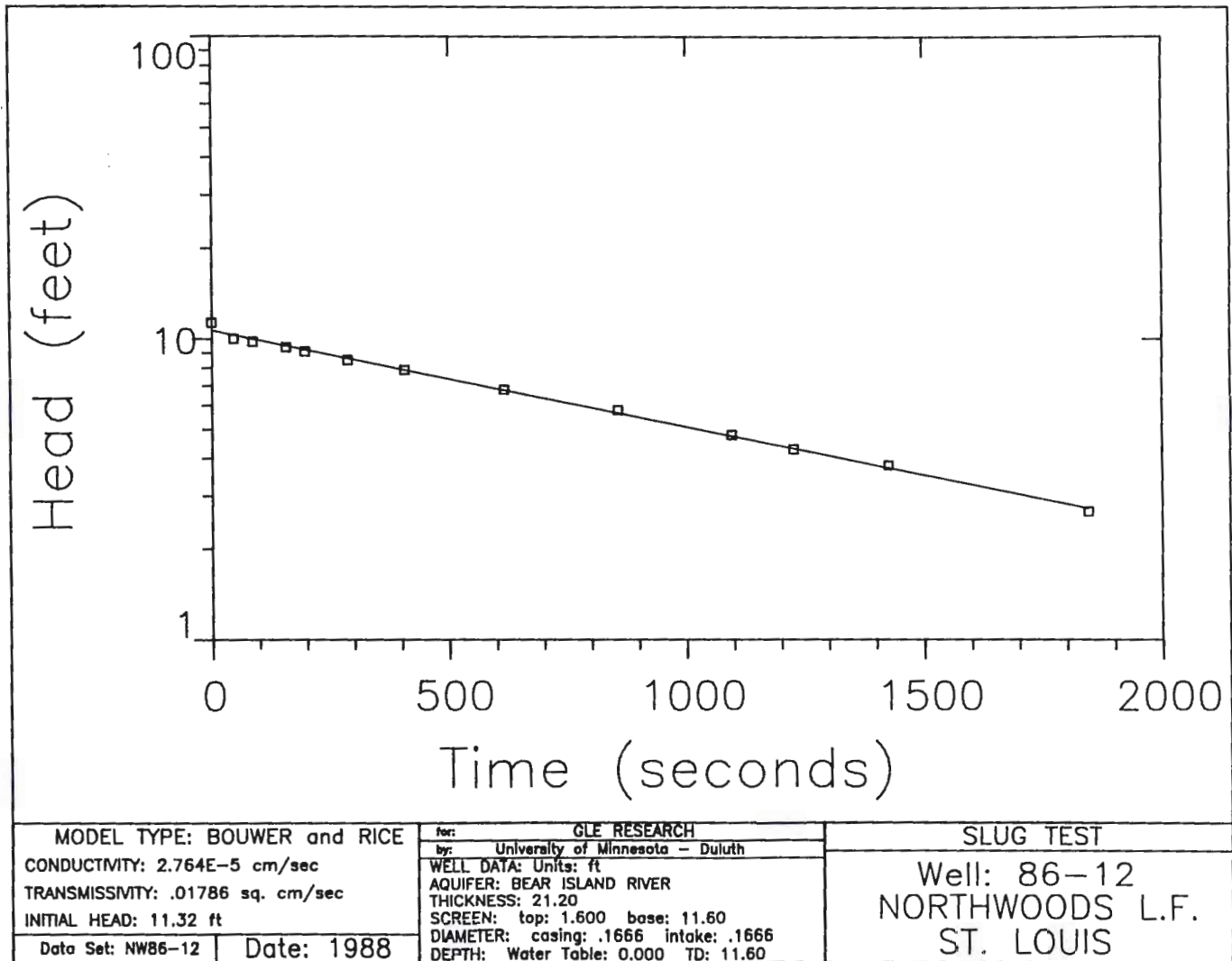


MODEL TYPE: BOUWER and RICE	for: GLE RESEARCH	GLE RESEARCH
CONDUCTIVITY: 1.685E-5 cm/sec	by: University of Minnesota - Duluth	Well: 81-6
TRANSMISSIVITY: .001438 sq. cm/sec	WELL DATA: Units: ft	NORTHWOODS L.F.
INITIAL HEAD: 2.400 ft	AQUIFER: BEAR ISLAND RIVER	ST. LOUIS
Data Set: NW81-6	THICKNESS: 2.800	
Date: 1988	SCREEN: top: -2.200 base: 2.800	
	DIAMETER: casing: .1666 intake: .1666	
	DEPTH: Water Table: 0.000 TD: 2.800	

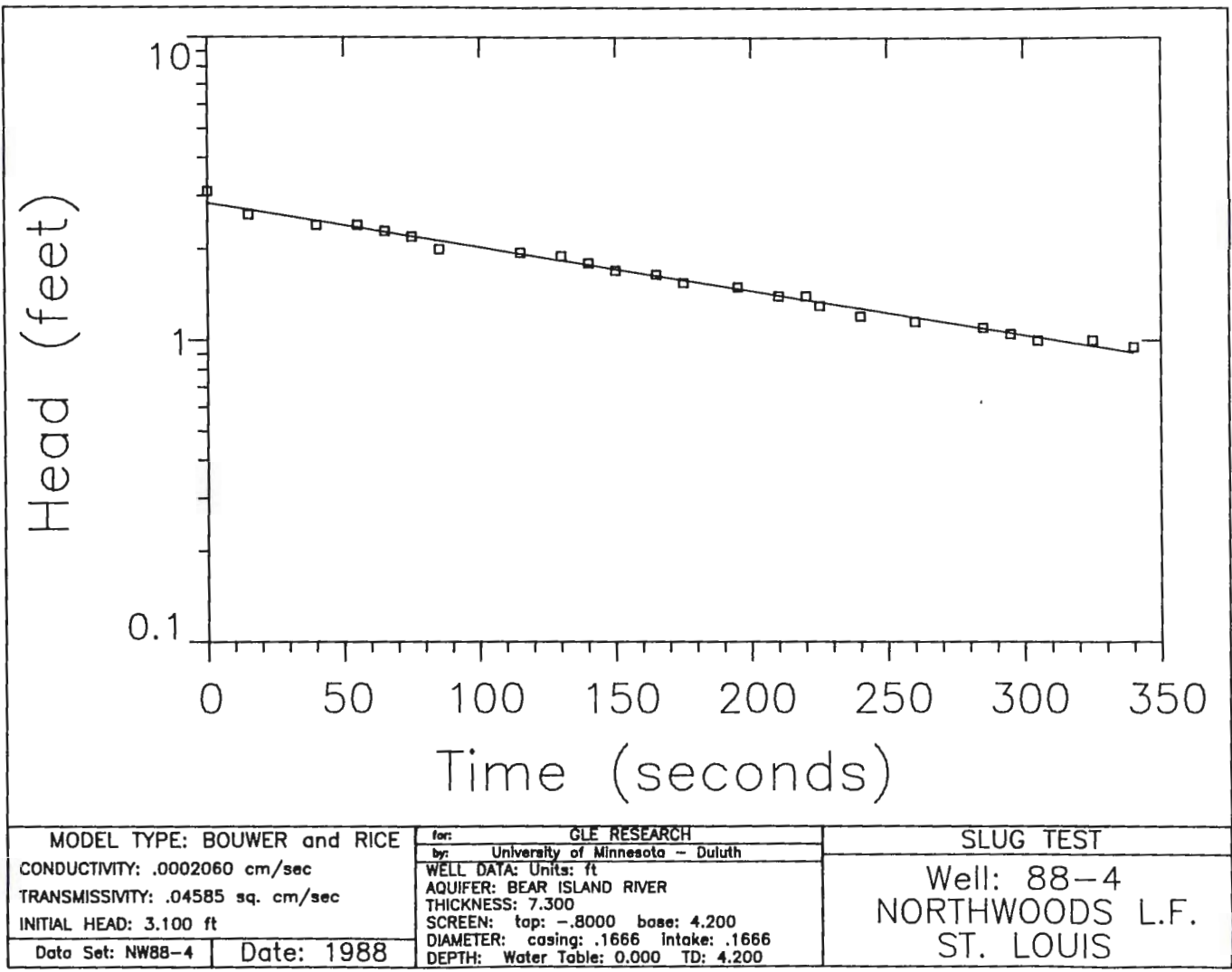
A-12



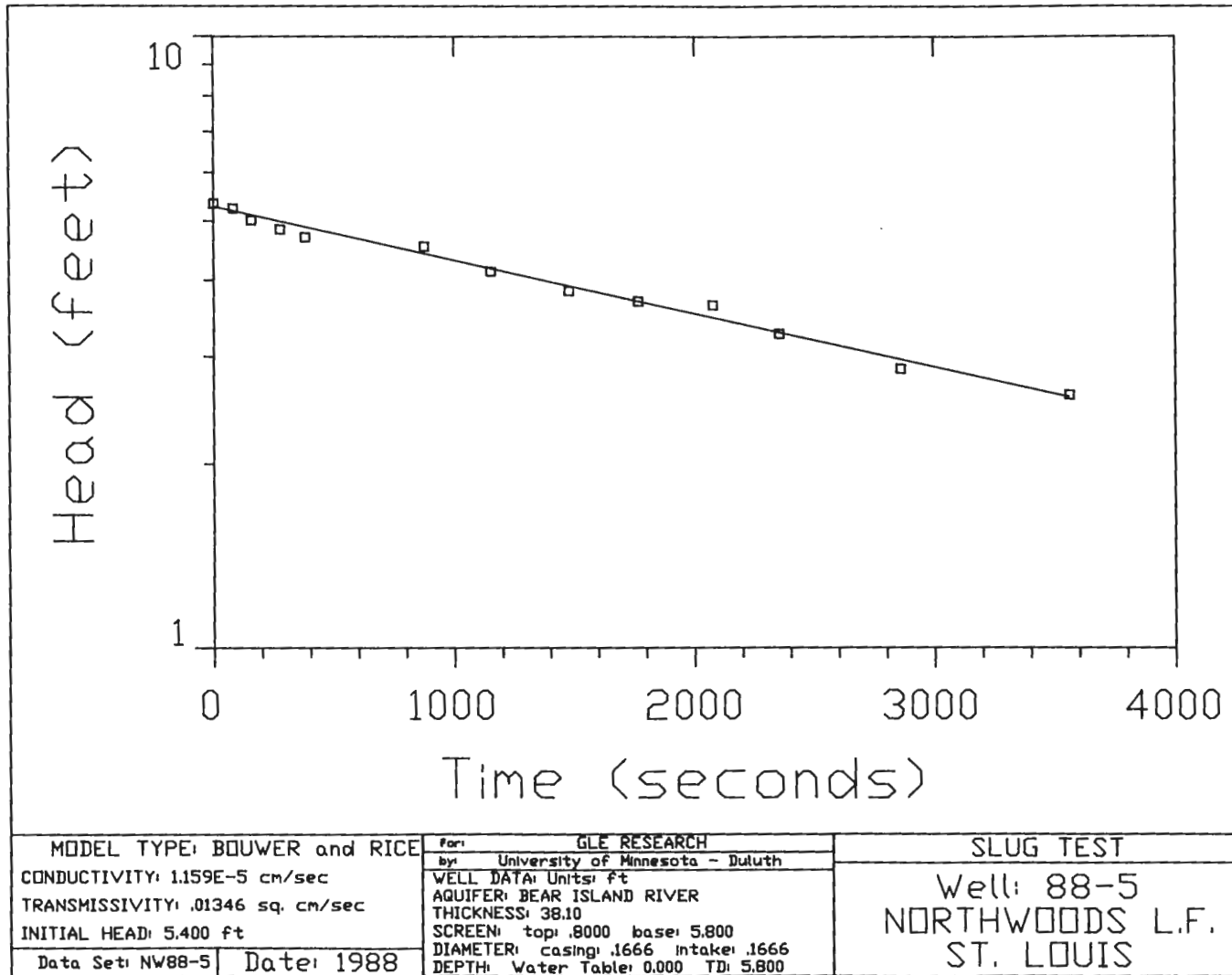
A-13



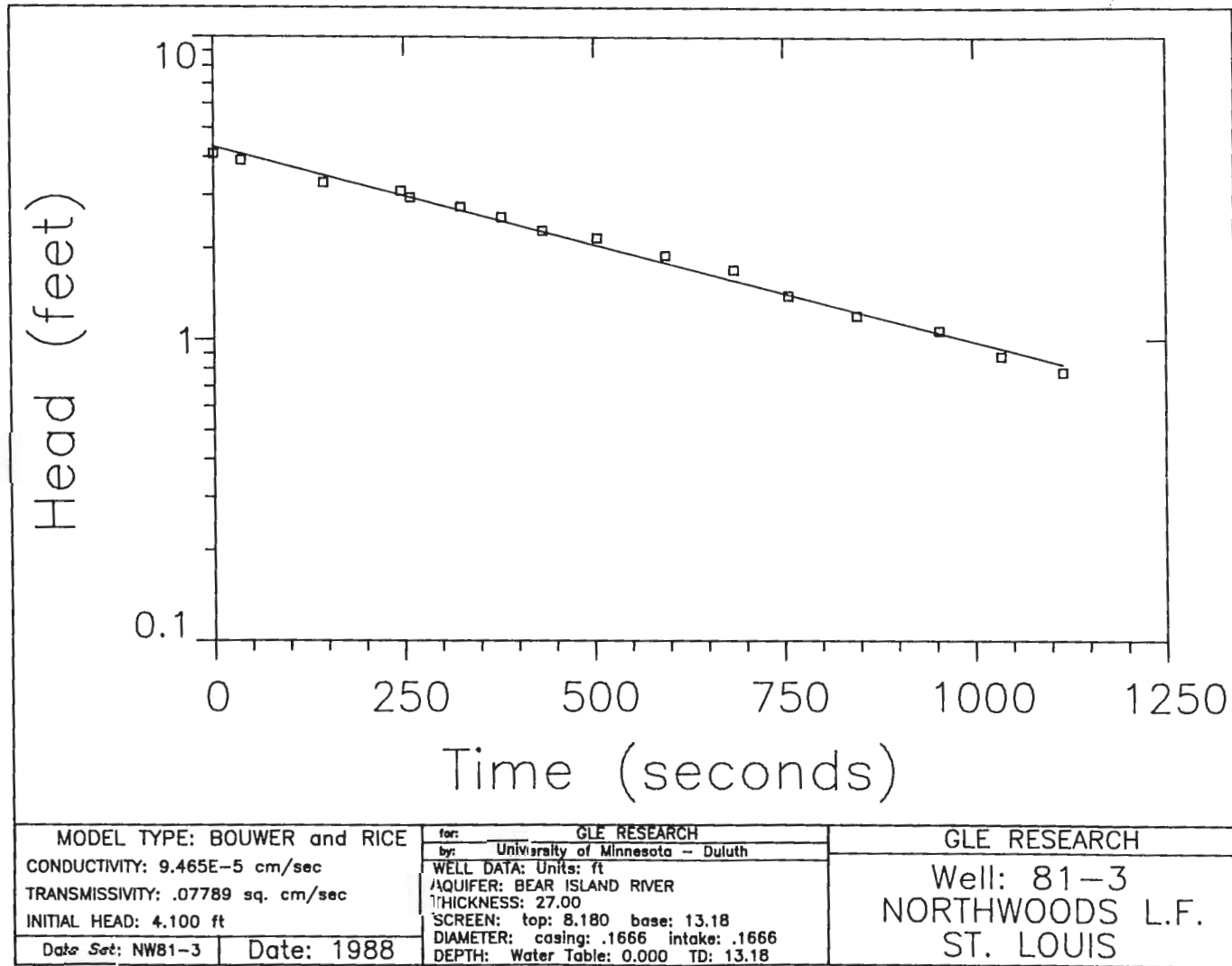
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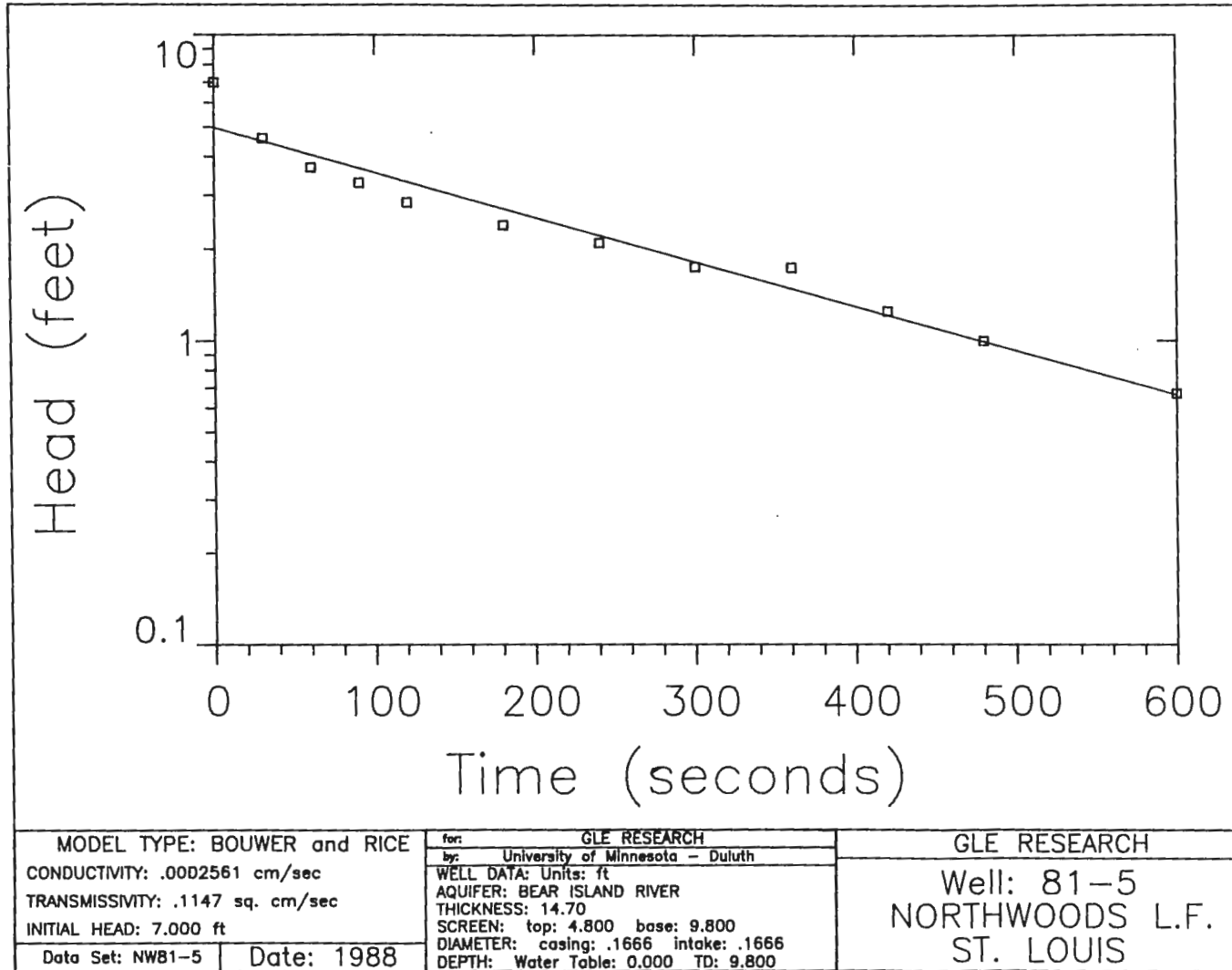
A-15



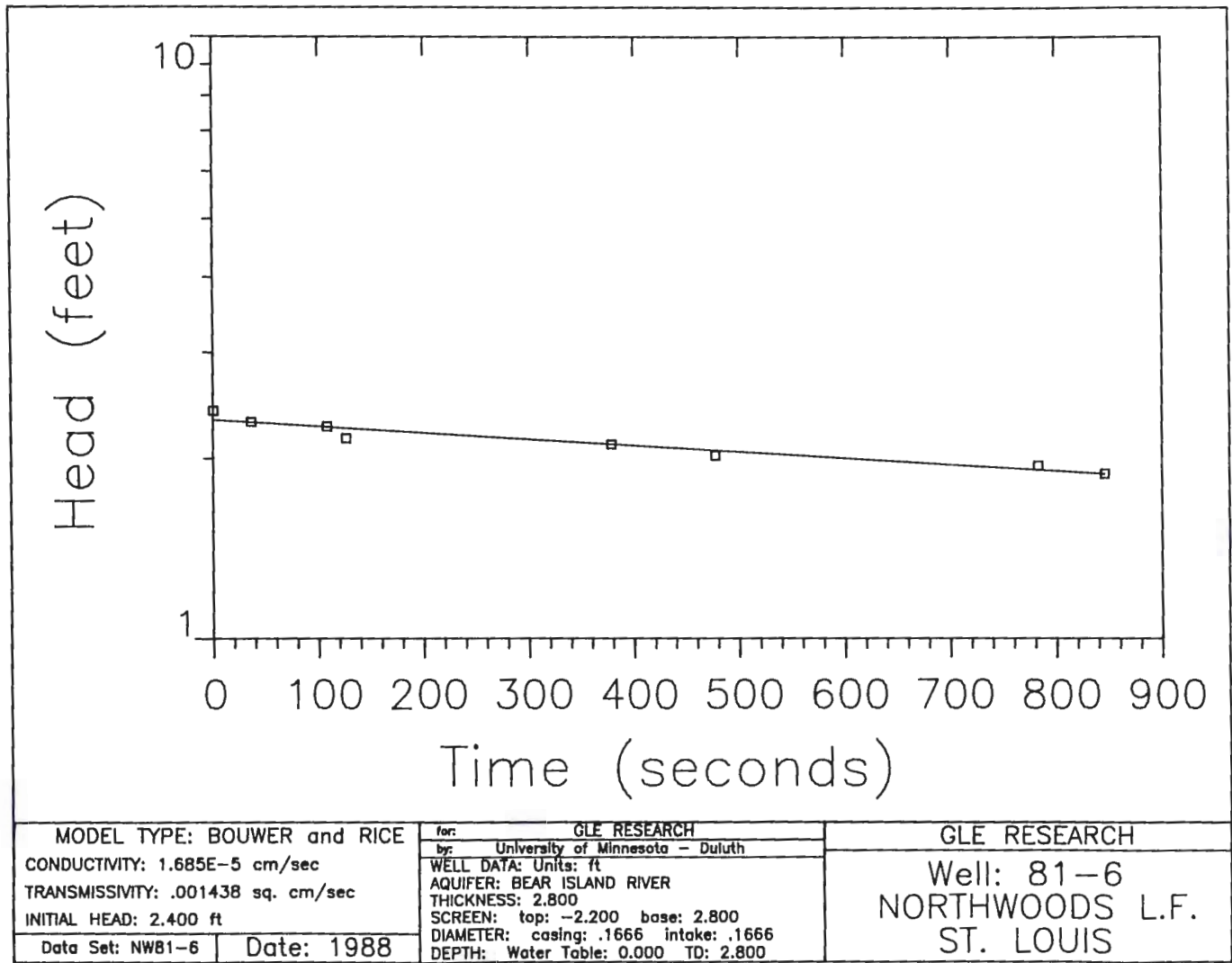
A-9



A-10

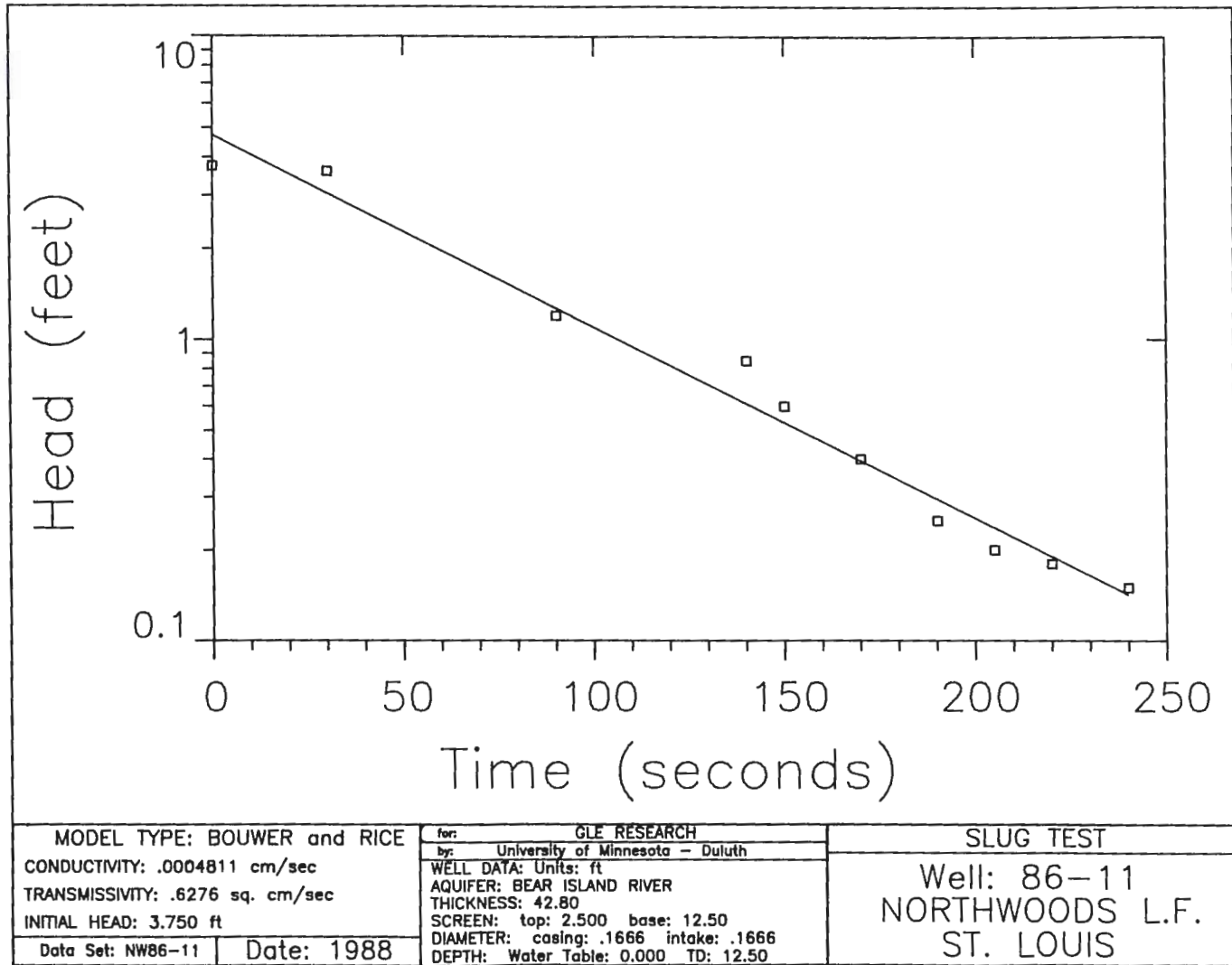


A-11

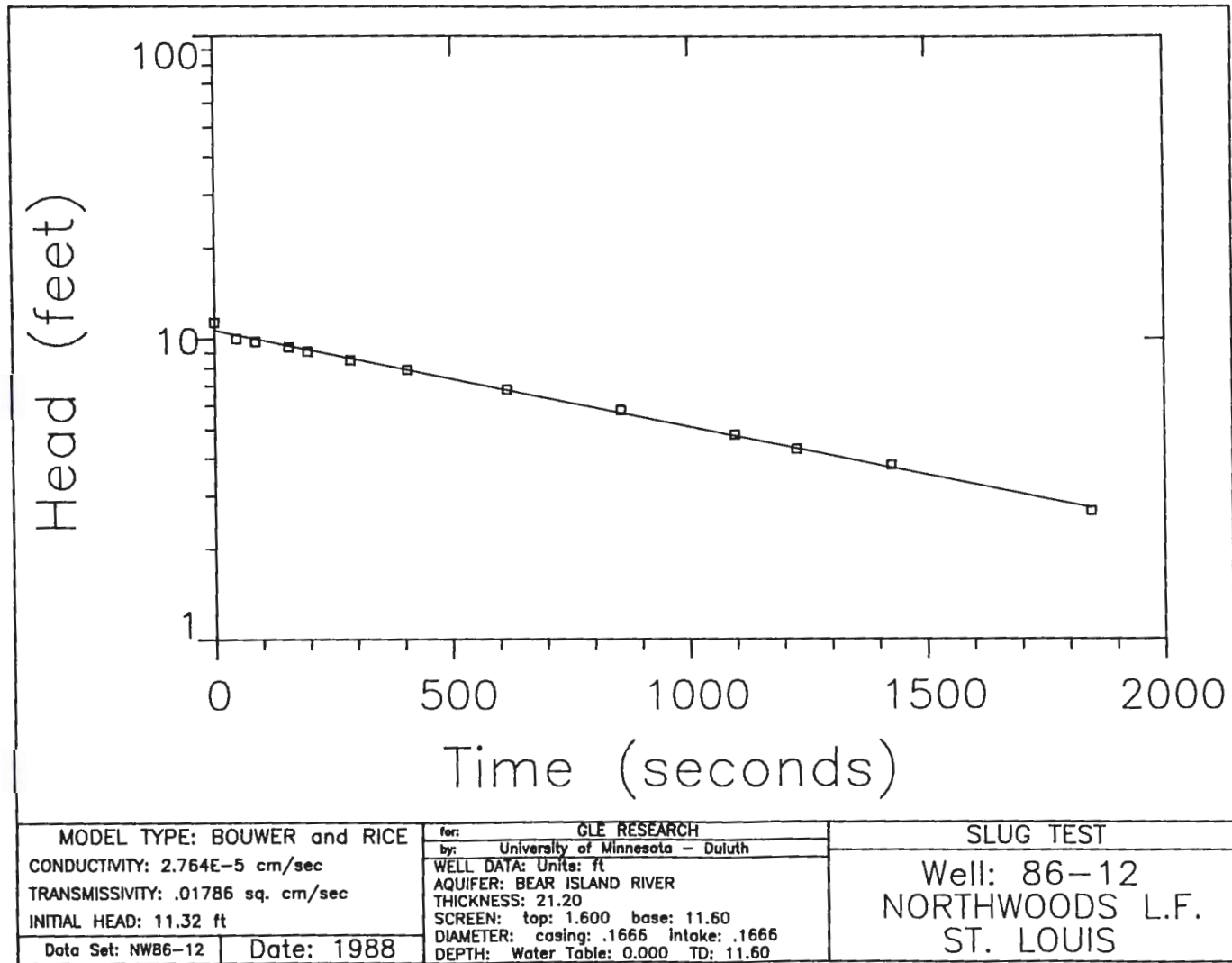


MODEL TYPE: BOUWER and RICE CONDUCTIVITY: 1.685E-5 cm/sec TRANSMISSIVITY: .001438 sq. cm/sec INITIAL HEAD: 2.400 ft Data Set: NWB1-6	for: GLE RESEARCH by: University of Minnesota - Duluth	GLE RESEARCH Well: 81-6 NORTHWOODS L.F. ST. LOUIS
	Date: 1988	

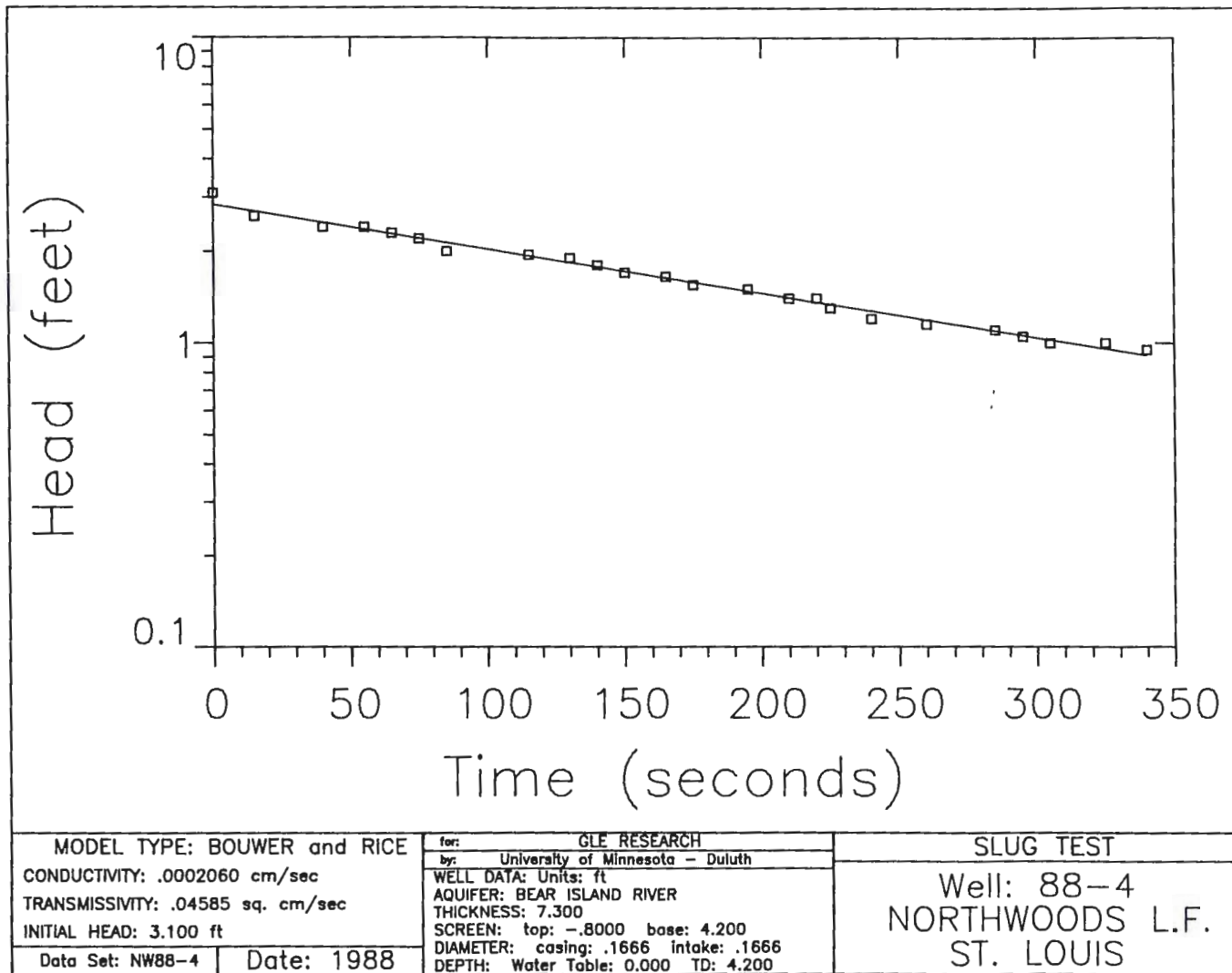
A-12



A-13



A-14



A-15

