AN ABSTRACT OF THE THESIS OF

David S. Walker for the degree of Master of Science in Civil Engineering presented on December 1, 1989.

Title: Groundwater Flow Model for the Lower Malheur Basin Near Ontario, Oregon

Abstract approved: Redacted for Privacy

Jonathan D. Istok

The shallow, poorly-confined, sand and gravel aquifer underlying the lower Malheur River basin is contaminated by nitrate and metabolites of the herbicide dimethyl tetrachloroterephthalate (DCPA), first detected in 1983 and 1985, respectively. The aquifer supplies water for the Ontario municipal water system, domestic and industrial uses, and irrigation. In response to this finding, the Oregon Department of Environmental Quality (DEQ), the Oregon Water Resources Department (WRD), and Oregon State University (OSU) have initiated a study of the groundwater quality and hydrogeology of the aquifer. The study includes analyzing groundwater samples for nitrate and DCPA metabolites (by DEQ); hydrogeological characterization, including measuring water levels and aquifer hydraulic parameters (by WRD); and developing a numerical groundwater flow and solute transport model (by OSU). This paper presents a regional groundwater flow model that will be used to estimate aquifer parameters and groundwater velocities for input into a solute transport model, and to identify additional data needs.

A two-dimensional, steady state, finite element model was calibrated to a set of 36 water level measurements taken by WRD personnel in late September 1988. The

model incorporates recharge from infiltration of surface irrigation water and leakage from irrigation ditches and withdrawals by production wells. Transmissivity was estimated from five pumping tests and using aquifer thickness data from well logs. The model was calibrated using a trial and error procedure. Surface irrigation recharge rates were estimated by minimizing the mean-squared-error (MSE) between predicted and measured water levels at the 36 wells.

The calibrated model produced an MSE of 25.3 ft^2 and a mean error of 1.0 feet. The estimated recharge from surface irrigation is about eight times that from leakage through unlined ditches; however, many small, unlined ditches were not modeled as separate line sources and were lumped with the irrigation recharge term. Therefore the model suggests both irrigation water infiltration and irrigation ditch leakage contribute significant quantities of recharge. The model suggests that Dork Canal and Arcadia Drain can be approximated as specified head boundaries and that recharge may occur from uplands aquifers.

Model predictions would be most improved by better definition of leakage from irrigation supply ditches, groundwater interception by major drainage ditches and shallow drainage systems on individual farm tracts, and the rate of withdrawal by producing wells. For transient flow analyses, better definition of the aquifer storativity is needed.

Groundwater Flow Model
for the Lower Malheur Basin
Near Ontario, Oregon

by

David S. Walker

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GROUNDWATER FLOW MODEL FOR THE LOWER MALHEUR BASIN NEAR ONTARIO, OREGON

INTRODUCTION

The shallow, poorly confined, sand and gravel aquifer underlying the lower Malheur River basin is contaminated by nitrate and metabolites of the herbicide dimethyl tetrachloroterephthalate (DCPA), first detected in 1983 and 1985, respectively. The aquifer supplies water for the Ontario municipal water system, domestic and industrial uses, and irrigation. In response to this finding, the Oregon Department of Environmental Quality (DEQ), the Oregon Water Resources Department (WRD), and Oregon State University have initiated a study of the groundwater quality and hydrology of the aquifer. The study includes analyzing groundwater samples for nitrate and DCPA metabolites (by DEQ); hydrogeological characterization, including measuring water levels and aquifer hydraulic parameters (by WRD); and developing a numerical groundwater flow and solute transport model (by OSU). This paper presents a regional groundwater flow model that will be used to estimate aquifer properties and groundwater velocities for input into a solute transport model, and to evaluate additional data needs.

The study area comprises the lowlands in an area between Ontario, Nyssa, and Vale, Oregon (Figures 1 and 2). Little published information is available on groundwater flow conditions at the study area. Bruck (1988) estimated groundwater flow directions based on topography. Whistler and Lewis (1916) presented water level cross-sections along two lines about two miles long that are perpendicular to and terminate at the Malheur River. They also provide a description of an installed shallow drainage system. Additional unpublished data, including well logs and observation well water level measurements, are on file with WRD. Two papers presently in preparation by Gannett will report the results of the aquifer testing and the hydrogeological characterization, respectively, performed by WRD.

Data are also available on water quality. Smyth (1988) used geostatistics to develop contour maps of DCPA and nitrate concentration from point measurements of these quantities. The extent of the 250 milligram per square meter DCPA plume calculated by Smyth is shown on Plate 2. Smyth's data are from samples taken from May 1985 to September 1986. Bruck (1988) presented nitrate concentration contours for June 1983, August 1983, and March 1986, and DCPA concentration profiles for March 1986.

The overall objective of the research at OSU is to develop regional groundwater flow and solute transport models that can be used to aid in evaluating the effects of various agricultural and water use practices on groundwater contamination. The specific objectives of this study were:

1. To develop a conceptual model of hydrogeologic conditions at the study area,
2. To develop and validate a numerical model for groundwater flow,
3. To estimate transmissivities and recharge rates within the study area,
4. To compute groundwater flow velocities required for use in a solute transport model,
5. To identify limitations of the available data and make recommendations for future data acquisition.

METHODS

Overview

The initial step of this study was to create a tabular and graphical database using a geographical information system. The governing flow equations and estimates of aquifer parameters were then selected based on analysis of these data. The flow equations were solved using the finite element method. The finite element model was calibrated by comparing computed water levels with water levels measured by WRD at 36 observation well locations. The mean-squared-error was used as the criteria to select the set of parameters that resulted in the best calibration.

Well Locations and Water Levels

A data base was created using a geographical information system (GIS). The data base comprised 103 wells, including 83 wells within the study area (Figure 3; Table 1). The Universal Transverse Mercator (UTM) coordinates of the well locations were determined by field-locating the wells on United States Geological Survey (USGS) topographic maps and digitizing the maps and well locations using the GIS. WRD field-located the wells to within 250 feet (M. Gannett, WRD, personal communication). Other sources of error in well location are 1) map accuracy (±67 ft for a 7.5-minute quadrangle), 2) transformation of digitizer coordinates to UTM coordinates (±90 ft accuracy), and 3) manual digitizing of well locations (±20 ft accuracy). The maps used were Payette (1974), Nyssa (1965), Malheur Butte (1974), Cairo (1967), Vale East (1975), and Moore's Hollow (1951). Moore's Hollow is a 15 minute quadrangle, the rest are 7.5-minute quadrangles.

Table 1. Field-located Well Data.

Well No.	Township	Range	Section	X, ft	Y, ft	Surface Elevation	Top of Aquifer Elevation	Aquifer thickness	9/20/88 Depth	9/20/88 Elevation	3/21/89 Depth	3/21/89 Elevation	11/6/86 Depth	11/6/86 Elevation	5/17/88 Depth	5/17/88 Elevation	6/29/88 Depth	6/29/88 Elevation	10/19/88 Depth	10/19/88 Elevation	12/16/88 Depth	12/16/88 Elevation
									Water level data by measuring date													
1	18	45	6d	263607	251068	2295	-	-	-	-	87.5	2207	-	-	-	-	-	-	-	-	-	-
2			12	288020	241114	2340	2225	-	84.8	2255	-	-	-	-	-	-	86.0	2254	-	-	-	-
3			14b	281582	238772	2290	2222	-	44.5	2245	54.8	2235	-	-	-	-	44.2	2246	-	-	-	-
4			14d	283860	235283	2220	2206	17	14.4	2206	13.2	2207	-	-	-	-	11.5	2208	-	-	13.5	2206
5			15ab	276910	239627	2285	2260	30	17.4	2268	19.9	2265	-	-	-	-	-	-	-	-	-	-
6			15ac	276937	237190	2250	2208	5	17.3	2233	22.6	2227	-	-	-	-	19.0	2231	-	-	-	-
7			15b	276196	240072	2360	-	-	80.1	2280	79.4	2281	-	-	-	-	79.6	2280	-	-	-	-
8			15b	276120	239635	2360	-	-	-	-	76.2	2284	-	-	-	-	25.7	2334	-	-	-	-
9			15d	279334	236200	2233	2208	9	14.8	2218	14.0	2219	-	-	-	-	15.4	2218	-	-	-	-
10			20aa	268015	233043	2240	2221	9	7.7	2232	8.4	2232	-	-	-	-	-	-	-	-	-	-
11			20ad	267887	232025	2235	2210	3	9.0	2226	-	-	8.6	2226	-	-	-	-	-	-	-	-
12			20c	265605	229947	2242	-	-	11.9	2230	9.2	2233	-	-	-	-	-	-	-	-	10.9	2231
13			20d	267964	230054	2230	2215	8	11.1	2219	10.0	2220	-	-	-	-	-	-	-	-	-	-
14			20d	268060	229986	2230	-	3	11.4	2219	11.0	2219	-	-	-	-	-	-	-	-	-	-
15			21bb	271445	230345	2240	2223	10	8.9	2231	9.8	2230	-	-	-	-	-	-	10.2	2230	11.6	2228
16			21ca	271465	230345	2230	2214	10	10.0	2220	12.5	2217	-	-	-	-	-	-	-	-	-	-
17			22	279490	230175	2215	2197	7+	10.8	2204	10.2	2205	-	-	-	-	-	-	-	-	-	-
18			28a	273578	228155	2265	2215	8	46.4	2219	48.7	2216	-	-	-	-	-	-	-	-	-	-
19			28a	273236	227942	2280	-	108	72.5	2207	70.0	2210	-	-	-	-	-	-	-	-	-	-
20			29bb	264205	227957	2240	2231	15	7.3	2233	4.9	2235	-	-	-	-	-	-	-	-	-	-
21			29bc	264615	227172	2240	2230	15	-	-	-	-	8.8	2231	-	-	-	-	-	-	-	-
22	18	46	1a	320980	246850	2160	2145	3	9.1	2151	7.9	2152	-	-	4.6	2155	-	-	-	-	-	-
23			1d	321500	246220	2160	2141	2	13.2	2147	-	-	-	-	11.1	2149	-	-	-	-	-	-
24			12a	320093	242330	2185	2157	10	20.8	2164	24.0	2161	-	-	23.4	2162	-	-	-	-	-	-
25			12a	320395	242600	2170	2154	22	11.9	2158	14.6	2155	-	-	-	-	-	-	-	-	-	-
26			12c	318856	240860	2193	2158	25	34.2	2159	37.6	2155	-	-	36.9	2156	-	-	-	-	-	-
27			13b	317293	238495	2190	2168	-	25.7	2164	27.9	2162	-	-	27.9	2162	-	-	-	-	-	-
28			14d	314678	233765	2203	2181	21+	11.8	2192	11.5	2193	-	-	-	-	12.5	2192	-	-	-	-
29			14d	315050	233776	avg w/ above		20	9.5	avg w/ above	9.3	avg w/ above	-	-	-	-	10.3	avg w/ above	-	-	-	-
30			15b	307635	238525	2180	2161	5	10.7	2169	8.0	2172	-	-	-	-	9.9	2170	-	-	-	-
31			15c	307125	233955	2197	2177	10	10.7	2186	9.5	2187	-	-	-	-	8.5	2189	-	-	-	-
32			17c	295850	233785	2198	2182	12	7.3	2191	10.2	2188	-	-	-	-	-	-	-	-	-	-
33			19b	290250	233540	2208	2191	8	10.7	2198	12.7	2196	-	-	-	-	8.2	2200	-	-	-	-
34			19b	290967	233500	avg w/ above		11	9.5	avg w/ above	10.5	avg w/ above	-	-	-	-	-	-	-	-	-	-
35			19c	290943	229400	2210	2193	11	-	-	11.9	2198	-	-	11.0	2199	-	-	-	-	12.4	2198
36			20cd	296890	229100	2215	2195	12	-	-	7.0	2208	-	-	-	-	4.9	2210	-	-	-	-
37			21c	300575	229573	2220	2196	24	12.3	2208	14.9	2205	-	-	13.9	2206	11.8	2208	13.3	2207	14.7	2205
38			22c	305800	230808	2220	-	-	13.1	2207	13.2	2207	-	-	14.6	2205	-	-	-	-	-	-
39			22d	309106	230505	2229	2215	14	7.1	2222	7.8	2221	-	-	-	-	7.0	2222	-	-	-	-
40			23	314050	228790	2250	2222	22	16.4	2234	17.7	2232	-	-	-	-	17.1	2233	-	-	18.5	2131
41	19	46	13d	320450	202290	2260	2230(?)	15	33.0	2227	37.0	2223	-	-	-	-	35.4	2225	-	-	-	-
42	17	47	32dd	331435	249597	2145	2138	18	6.1	2139	3.9	2141	-	-	-	-	5.2	2140	-	-	-	-
43			33cb	332310	252104	2143	2128	5	7.3	2136	-	-	-	-	-	-	7.6	2135	-	-	-	-
44	18	47	3	341241	244464	2150	2125(?)	6	15.0	2135	12.6	2137	-	-	-	-	-	-	-	-	-	-
45			4a	336386	248777	2150	2140	17	12.5	2137	10.8	2139	-	-	-	-	12.2	2138	-	-	-	-
46			4ca	334134	246170	2170	2147	24	24.2	2146	23.7	2146	-	-	-	-	-	-	-	-	-	-
47			4cc	332716	245405	2183	2146	20	35.3	2148	35.5	2147	-	-	-	-	36.8	2146	-	-	-	-
48			4cd	334240	244300	2183	2146	4	24.9	2148	24.0	2159	-	-	-	-	-	-	-	-	-	-
49			4d	335897	244793	2150	2134	13	7.7	2142	6.7	2143	-	-	-	-	-	-	-	-	-	-
50			5	329565	247555	2180	2148	27	33.9	2146	35.5	2144	-	-	-	-	39.3	2141	-	-	-	-
51			6	325765	244228	2185	2145	14	-	-	33.3	2152	-	-	-	-	32.5	2153	-	-	-	-

Notes.
1. Well coordinates are UTM feet.
2. Elevations are feet above mean sea level.

Table 1. Field-located Well Data (continued).

Well No.	Township	Range	Section	X, ft	Y, ft	Surface Elevation	Top of Aquifer Elevation	Aquifer thickness	Water level data by measuring data: 9/20/88		3/21/89		11/6/86		5/17/88		6/29/88		10/19/88		12/16/88	
									Depth	Elevation	Depth	Elevation	Depth	Elevation	Depth	Elevation	Depth	Elevation	Depth	Elevation	Depth	Elevation
52	18	47	7	325802	241855	2190	2150	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
53			7	326023	242838	2188	2163	25	-	-	-	-	-	-	-	-	-	-	-	-	19.2	2169
54			7	325330	240743	2183	2148	20	-	-	-	-	-	-	-	-	-	-	-	-	-	-
55			7	323183	241868	2180	2157	18	-	-	-	-	-	-	-	-	-	-	-	-	-	-
56			7	323441	241871	2180	2149	7	18.4	2162	-	-	-	-	-	-	17.2	2163	19.2	2162	-	-
57			7	323517	241852	2180	2147	16	18.4	2162	18.6	2161	-	-	-	-	17.1	2163	-	-	-	-
58			7	324486	241545	2180	2148	21	-	-	-	-	-	-	-	-	-	-	-	-	-	-
59			9ab	335530	243342	2150	2129	10	9.6	2140	-	-	-	-	-	-	-	-	-	-	-	-
60			9bc	333367	241890	2174	2147	20	-	-	-	-	-	-	-	-	-	-	-	-	-	-
61			9bd	334780	242100	2165	2146	18	19.3	2146	17.5	2147	-	-	-	-	20.3	2145	-	-	-	-
62			10b	339433	242290	2151	2132	6	10.6	2140	9.8	2141	-	-	-	-	10.7	2140	11.0	2140	10.9	2140
63			10b	339920	242615	2149	2123	4	12.8	2136	9.1	2140	-	-	-	-	-	-	-	-	-	-
64			11	344804	243788	2146	2145	25	19.6	2126	18.3	2128	-	-	-	-	20.3	2126	-	-	-	-
65			11	344700	244102	-	-	28	-	-	-	-	-	-	-	-	-	-	-	-	-	-
66			11	344936	244377	-	-	29	-	-	-	-	-	-	-	-	-	-	-	-	-	-
67			11	344859	243488	-	-	26	-	-	-	-	-	-	-	-	-	-	-	-	-	-
68			11	344973	244093	-	-	12	-	-	-	-	-	-	-	-	-	-	-	-	-	-
69			11	345126	243816	-	-	36	-	-	-	-	-	-	-	-	-	-	-	-	-	-
70			15ab	340141	237816	2150	2131	10	-	-	-	-	-	-	-	-	-	-	-	-	-	-
71			15ac	341094	236376	2150	2142	6	13.4	2137	9.8	2140	-	-	-	-	-	-	-	-	-	-
72			16	332248	233700	2177	2147	15	9.9	2167	10.0	2167	-	-	-	-	11.4	2166	-	-	-	-
73			17a	330745	237068	2190	2154	20	18.1	2172	20.5	2169	-	-	-	-	22.1	2168	-	-	-	-
74			17cb	327910	235600	2183	2163	35	4.8	2178	2.6	2180	-	-	-	-	7.7	2175	5.4	2178	5.8	2177
75			19ccc	321700	229800	2219	-	-	9.3	2210	5.6	2213	-	-	-	-	-	-	10.2	2209	-	-
76			19ccb	321748	229066	2230	2210	92	15.2	2215	13.5	2216	-	-	-	-	15.1	2215	-	-	-	-
77			19d	324865	230967	2202	2158	16	-	-	-	-	-	-	-	-	-	-	-	-	-	-
78			19d	324880	230395	2203	2154	9	15.6	2187	16.1	2187	-	-	-	-	17.6	2185	-	-	-	-
79			20c	326975	228755	2196	2163	7	10.8	2185	9.1	2187	-	-	-	-	10.7	2185	11.8	2184	-	-
80			21cc	333735	230067	2179	2146	12	17.7	2162	16.5	2162	-	-	-	-	20.3	2160	17.2	2162	17.8	2161
81			21ca	332235	229669	2180	2140	20	18.0	2161	20.3	2160	-	-	-	-	21.0	2158	19.0	2161	-	-
82			29	328577	223824	2192	2162	20	19.2	2173	20.2	2172	-	-	-	-	-	-	18.4	2174	20.3	2172
83			30a	324290	227100	2209	-	-	12.8	2196	13.4	2196	-	-	-	-	-	-	14.0	2195	14.5	2194
84			30b	321986	228176	2255	2195	6	35.7	2219	37.5	2217	-	-	-	-	37.0	2218	-	-	-	-
85			30b	322100	228180	2250	2183	6	-	-	-	-	-	-	-	-	-	-	-	-	-	-
86			32a	330650	221130	2175	2145	17	15.4	2160	16.2	2159	-	-	-	-	16.3	2159	-	-	-	-
87			32b	328582	223040	2193	2163	28	20.2	2173	-	-	-	-	-	-	22.9	2170	-	-	-	-
88			32d	330133	217967	2160	2106	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
89	19	47	7	321710	212532	2240	2210	50	16.1	2224	19.3	2221	-	-	-	-	17.9	2222	-	-	-	-
90			8ac	329840	210006	2182	2161	24	19.3	2163	24.0	2158	-	-	-	-	-	-	20.6	2161	23.9	2158
91			8cb	326863	209949	2180	2159	20	9.2	2171	8.7	2171	-	-	-	-	-	-	-	-	8.3	2172
92			20	327282	197183	2178	2166	17	6.8	2171	5.9	2172	-	-	-	-	7.5	2171	7.1	2171	7.1	2171
93	18	47	16	333500	235500	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
94			17	327150	233950	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
95			17	330250	234300	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
96			18	324250	237650	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
97			20	328300	229650	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
98			29	329400	226500	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
99			30	325500	227100	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
100			17bc	327300	236850	2187	2162	28	-	-	5.1	2182	-	-	-	-	-	-	7.4	2180	-	-
101	18	46	21bc	300625	231440	2205	-	-	-	-	10.1	2195	-	-	-	-	-	-	10.1	2195	10.5	2194
102			24db	320910	230405	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
103	18	47	17bb	327110	238670	2180	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Notes.
1. Well coordinates are UTM feet.
2. Elevations are feet above mean sea level.

In this report, wells are referred to by the well number (Table 1). Well locations are given in the township-range-section system as well as the UTM coordinate system. In the former system, each one mile square section is subdivided into four quarter sections, designated "a" through "d" starting from the northeast corner and proceeding counter-clockwise. The quarter sections may be further subdivided into quarter-quarter and quarter-quarter-quarter sections in the same manner. For example, "18.47.12abc" designates the southwest quarter of the northwest quarter of the northeast quarter of section 12, township 18, range 47.

Well logs were available for 72 wells within the study area and these were used for detailed stratigraphic information. Water levels are measured biannually in 54 wells and at intervals of 1 to 3 months in 14 wells. Water level measurements are made by WRD using either an electric water level measuring tape or a chalked steel tape. A summary of the well data is in Table 1. Water level contouring (Figures 4a and 4b) was done by a program called SURFER (Golden Graphics, 1987) using a simplified kriging procedure.

At least 120 additional wells within the study have well logs filed with WRD but have not been field-located. Many of these wells were approximately located using information from the well logs and were used to interpret the stratigraphy of the study area (Figure 5).

HYDROGEOLOGIC SETTING

The aquifer is located in the lower Malheur River basin area in northern Malheur County, Oregon. It underlies the low-lying lands from the Snake River west past Vale in the Willow Creek and Malheur River drainages; south past Nyssa; and north past Ontario. The study area was limited to Nyssa in the south, the

Malheur River in the north, and to where the higher lands and the Malheur River nearly intersect near Vale in the east. The study area encompasses about fifty-five square miles.

Water-bearing strata exist in the higher land southwest of the study area; however, hydraulic connection of these upland aquifers to the shallow aquifer is believed to be limited (M. Gannett, WRD, personal communication).

The aquifer is described on well logs as gravel or sand and gravel and its thickness generally ranges from 3 to 30 feet, averaging about 15 feet. Depth to the aquifer generally ranges from 10 to 40 feet, averaging about 20 feet. The aquifer materials are alluvial deposits (Bruck, 1988). The aquifer is underlain by lacustrine sediments described as blue clay on well logs. The hydraulic conductivity of these strata is probably low.

Above 6 feet depth the soils the soils overlying the aquifer are typically silty or sandy loams of moderate to moderately low permeability (Lovell, 1980; Figure 6). These materials are generally described on well logs as clays and sandy clays that are continuous to the top of the aquifer. A topographic break exists about one mile from the Snake and Malheur rivers. The areas adjacent to the rivers are termed bottomlands and those past the topographic break are termed benchlands. Soils in the bottomland are generally poorly drained and more alkaline than the benchland soils. A hardpan is present in parts of the study area. The top of the hardpan is usually at a depth of two to three feet. The hardpan is found in Stanfield, Nyssa, and Truesdale soils (Lovell, 1980). Since these soils comprise a small portion of the study area the effect of hardpan on recharge to the aquifer was not considered in the model.

The climate is semi-arid. Precipitation averages about 10 inches per year. Common crops are onions, potatoes, sugar beets, and alfalfa. Most of the agriculture in the area is row crops that are furrow irrigated; lesser acreage is used for alfalfa and pasturage and is usually sprinkler irrigated or non-irrigated.

Recharge to the aquifer occurs primarily by infiltration of irrigation water and leakage from unlined irrigation ditches. The sources of the irrigation water are Owyhee Lake, Snake River, and Warm Springs Reservoir. Annual surface irrigation typically ranges from 4 to 5 feet (Irrigation records from the Ontario-Nyssa irrigation district, and Bruck (1988)). The irrigation season extends from mid-April to early October.

The aquifer discharges to the Snake and Malheur Rivers, to deep drainage ditches, including Dork Canal and Arcadia Drain, to shallow drainage systems on individual farms, and to production wells. In addition, some of the bottomland soils (Umapine and Stanfield) develop a salt crust during part of the year (Lovell, 1980), indicating groundwater evaporation at the land surface during high water level periods. Water is produced in high-capacity wells for irrigation, industrial, and municipal use. Smaller amounts are withdrawn for domestic use.

The water level gradient ranges from about 5 to 25 feet per mile (Figures 6a and 6b). Based on an average porosity of 25 percent (Fetter, 1980, p.64; Driscoll, 1986, p.67) and an average thickness of 15 feet, about 132,000 acre-feet of water are stored in the aquifer within the study area.

MODEL DEVELOPMENT

Conceptual Model

Long-term hydrographs for three wells (locations shown on Figure 3) within the study area were examined to determine if the assumption of steady-state groundwater flow is appropriate for this aquifer. The hydrographs generally show water levels to be lowest in the first third of the year and highest during the last third. The annual fluctuation is typically two to four feet. Average annual water levels at the Pennington well (no. 35) have been very nearly constant since the early 1970's (Figure 7a). At the Weaver well (no. 103), average annual water levels have remained approximately constant since 1950 or earlier (Figure 7b). At the Teramura well (no. 40), average annual water levels have risen about 5 feet since 1962 (Figure 7c). Collins (1979) reported stable water levels in northern Malheur County. On the basis of these data, regional groundwater flow was assumed to be at steady-state.

Data from pumping tests (Table 2; Figures A.1 through A.5) were analyzed to determine the confinement conditions within the study area. Values of coefficient of storage (S) were determined by a log time-drawdown analysis (Cooper and Jacob, 1946) of observation well data. The results were 0.024 (Pennington), 0.0065 (Okuda), and 0.005 (LDS). S could not be computed at the OSU North observation well because drawdown was too small to allow an analysis. S could not be computed at the OSU East well because no observation well was available. Computed values of S are typical of confined aquifers (deMarsily, 1986, p.111). The water level response to changes in barometric pressure in well 75 (Figure 8) is typical of that obtained for confined aquifers (Freeze and Cherry, 1979, p. 234); however, it may be noted that other theories have been advanced that may explain this type of response in an unconfined aquifer (Peck, 1960; Turk, 1975). The

Table 2. Pumping Test Results.

No.	Well	x,ft[1]	y,ft	Test Duration, min. Drawdown	Recovery	Pumping Rate, gpm	Thick-ness, ft	Distance to Obs. Well, ft	T, ft^2/d	S	K, ft/d	Method	Curve[2]	Interval[3]
33	Pennington	290,250	233,540	235	75	78	8	--	5000	--	625	Cooper-Jacob[4]	D	t>2 min.
									3600	--	450	Neuman[5]	D	t>30 min.
									3600	--	462	Neuman	R	$t/t_r<10$
	obs. well							62	6000	0.024	750	Neuman	R	$t/t_r<10$
75	OSU North	321,700	229,800	300	90	77	45	--	13,000	--	290	Cooper-Jacob	D	t>50 min.
	obs. well							370	--	--	--			
83	OSU East	324,290	227,087	314	60	405	24	--	13,000	--	540	Neuman	D	t>60 min.
									14,600	--	610	Neuman	R	$t/t_r<10$
102	Okuda	320,910	230,405	705	900	426	45[6]	--	19,400	--	430	Cooper-Jacob	D	t>2 min.
									15,000	--	335	Neuman	D	t>100 min.
	obs. well							370	25,900	0.003	575	Cooper-Jacob	D	t>20 min.
									22,800	0.0065	505	Neuman	R	$t/t_r<10$
100	LDS	327,300	236,850	1835	1120	404	28	--	21,900	--	780	Cooper-Jacob	R	$t/t_r<500$
	near obs. well							79	20,400	--	730	Cooper-Jacob	D	t>30 min.
									23,800	--	850	Cooper-Jacob	R	$t/t_r<1000$
	far obs. well							1500	--	0.005	--	Cooper-Jacob	D	t>300 min.

Notes.
(1) Well coordinates are UTM feet
(2) D=drawdown, R=recovery
(3) t/t_r=ratio of total test time to recovery time
(4) Cooper-Jacob (1946)
(5) Neuman (1975)
(6) Well log does not exist. Aquifer thickness reported by well owner.

response of aquifer water levels to precipitation events and to the start-up of flow in canals at the beginning of the irrigation season is very rapid (Figure 9), indicating the confining layer is leaky.

Confinement conditions were also examined by comparing the measured water level elevation with the elevation of the top of the aquifer at field-located wells. At 8 wells, the water table resided in the aquifer, indicating unconfined conditions. At 10 wells, the water level elevation was less than 5 feet greater than the top of aquifer elevation, indicating the aquifer may become unconfined during periods of large drawdown. At 34 wells, the water level elevation was at least 5 feet greater than the top of aquifer elevation. Based on well log descriptions of the overlying silt layer, the aquifer was judged to be leaky confined at 28 of these 34 locations and two-layer unconfined at the remaining six. In both the leaky confined and two-layer unconfined cases, nearly all horizontal flow is in the sand and gravel layer. For steady-state flow, the assumption of a confined aquifer will give a very good approximation, even where a two-layer system exists. For transient flow, the system could also be assumed to be confined if suitably large storage coefficients are used where the two-layer situation exists. An alternative interpretation is that the aquifer is two-layer unconfined throughout. This would be consistent with the relatively large storage coefficients (at the high end of the range for confined aquifers), the rapid response to infiltration water inputs, and pumping test results that are similar to those obtained for unconfined aquifers with delayed gravity yield (Neuman, 1975). Although both the confined or two-layer unconfined model are consistent with the available data, the confined model was selected because a two-layer unconfined model would be much more difficult to apply and would result in little, if any, increased accuracy.

Based on material descriptions on well logs, the aquifer appears to be continuous over the study area. Flow in the aquifer will be very nearly horizontal since the areal extent of the aquifer is large compared to its average thickness of 15 feet. Vertical flow will probably occur in small areas, e.g., near discharge boundaries that do not fully penetrate the aquifer and near partially penetrating production wells. However, these small scale effects are not important for modeling groundwater flow on the scale of the study area and flow was assumed to be two-dimensional and horizontal.

The hydraulic conductivity of horizontally bedded sedimentary deposits is usually isotropic in the horizontal plane (Freeze and Cherry, 1979, p. 32). Transmissivity in this plane would also be expected to be isotropic unless the aquifer thickness varied in a regular manner that resulted in a preferred direction of flow. Groundwater flow directions indicated by water level contours drawn from measured water levels assuming isotropic transmissivity, i.e., flow directions that are orthogonal to the water level contours, generally conform to those expected considering the aquifer geometry and boundary conditions. This in itself does not demonstrate that transmissivity is isotropic; however, there is no evidence for anisotropy. Therefore, isotropic transmissivity was assumed.

Recharge from three sources is accounted for: 1) infiltrating irrigation water and precipitation, 2) leakage from Nevada Canal, Owyhee Ditch, and Ontario-Nyssa Canal, and 3) leakage from upland aquifers southwest of the study area. Recharge resulting from leakage at smaller ditches is incorporated in the surface irrigation term. Recharge from surface irrigation water is assumed uniform throughout the study area except in the Ontario area, where it is assumed zero. Estimated withdrawals from production wells are also included.

The Snake and Malheur Rivers, Dork Canal, and Arcadia Drain were assumed to act as specified head boundaries. Dork Canal and Arcadia Drain were observed to be flowing during the last week of December, 1988. Since no run-off from irrigation occurs at this time of the year and there had been no recent rainfall, much of the flow apparently was from intercepted groundwater (baseflow). Dork Canal is a deeply cut ditch and Arcadia Drain lies at the base of a slope, indicating that these features are likely to intersect the aquifer. For these reasons, as well as for improved model performance, Dork Canal and Arcadia Drain were used as specified head boundaries. Other smaller ditches may also act as discharge points, at least seasonally, but the effects were assumed to be small and these ditches were not used to define specified head boundaries. The water levels in each of the specified head boundaries fluctuate during the year; however, there is no evidence for a long-term trend of changing water levels and short-term fluctuations were neglected in the development of the regional, steady-state model presented here.

Model boundaries were initially considered to be no-flow adjacent to the uplands southwestern boundary (near the Ontario-Nyssa Canal) because hydraulic connections between the study aquifer and the upland aquifers south and west of this canal are believed to be limited. The boundary conditions were later changed to allow seepage from the Ontario-Nyssa canal as well as recharge from the upland aquifers. A no-flow boundary was used for the southern boundary near Nyssa. Although the aquifer is continuous here, the boundary used is nearly parallel to the flow-lines. In addition, the boundary was placed far from the primary area of concern to diminish the influence of any flow that may actually cross this boundary.

The important features of the conceptual model are shown in Figure 10.

Mathematical Model

The following assumptions were made in development of the mathematical model:

1. Vertical hydraulic gradients in the aquifer are zero, i.e., flow is two-dimensional and horizontal,
2. The aquifer is confined,
3. Aquifer properties are isotropic,
4. The underlying "blue clay" is impermeable,
5. Horizontal flow in the overlying silts is negligible, and
6. The rates of recharge from infiltration of irrigation water and leakage from ditches do not vary with position.

The governing differential equation for groundwater flow with these assumptions is (deMarsily, 1986, p. 343):

$$\frac{\partial}{\partial x}\left(T(x,y)\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(T(x,y)\frac{\partial h}{\partial y}\right)+r(x,y)=0 \qquad (1)$$

where

x,y = coordinates in the horizontal plane (ft),

h = water level elevation (ft),

T = transmissivity (ft^2/day), and

r = groundwater recharge (ft/day)

$= i + s + l - w$, where

i = infiltration from surface irrigation,

s = leakage from unlined ditches,

l = leakage from upland aquifers, and

w = production well withdrawals.

The term representing infiltration from surface irrigation (i) includes infiltration of precipitation and recharge from leakage through small ditches. The "s" term includes leakage from Nevada Canal, Owyhee Ditch, and Ontario-Nyssa Canal only. Leakage was estimated from Darcy's Law:

$$s = k\ i\ f \tag{2}$$

where

k = hydraulic conductivity of soil overlying the aquifer (ft/day),

i = hydraulic gradient (ft/ft),

f = fraction of the year the ditch is flowing.

The hydraulic conductivity was estimated as 1.0 inch per hour from data in Lovell (1980). A 2 foot depth of water in the ditch and a constant, saturated distance of 13 feet from the bottom of the ditch to the top of the aquifer were assumed, resulting in a hydraulic gradient slightly greater than one ft/ft. The ditches were assumed to flow during the irrigation season only (5 months each year). Recharge from upland aquifers was added to the Ontario-Nyssa canal leakage term. This was done for convenience since the canal lies very near the boundary.

The aquifer boundaries are modeled as specified head at the Snake and Malheur Rivers, Dork Canal, and Arcadia Drain, and as specified flow (or no-flow) at the southern and western boundaries (Figure 2). These are expressed mathematically as:

$$h = \text{constant (specified head boundary condition),} \tag{3}$$

$$\frac{\partial h}{\partial n} = 0 \text{ (no-flow boundary condition), and} \tag{4}$$

$$r = \text{constant (specified flow boundary condition),} \tag{5}$$

where n is the direction normal to boundary.

Water level elevations at the specified head boundaries were interpolated from USGS 7.5-minute topographic maps.

Numerical Model

Equation 1 was solved by the finite element method using a program called GW1 (Istok, 1990). The finite element method was selected over other numerical methods because this method provides: 1) greater flexibility for representing irregular aquifer boundaries and 2) greater accuracy and stability for solving the solute transport equation (deMarsily,1986, p.340). The second point is important because the velocities computed by the finite-element groundwater flow model can be input directly into a solute transport solution based on the same mesh. Linear triangle and linear quadrilateral isoparametric elements were used. The global matrix equation was solved using Cholesky's method. The mathematical formulation of the finite elment method is well documented in the literature and is not repeated here (see, for example, deMarsily, 1986; Istok, 1990). The finite element mesh consisted of 142 nodes and 114 elements (Figure 11). The mesh was constructed by placing nodes along boundaries, line recharge sources, and discontinuities in transmissivity and irrigation recharge rate. Additional nodes were added, as needed, to allow the elements to be constructed with an acceptable slenderness ratio (ratio of the longest to the shortest side of an element).

The locations of the specified head nodes are in Figure 11; the specified head values are in Table 3.

Table 3. Specified Head Values.

Node	Specified Head, ft	Node	Specified Head, ft
1	2162.5	40	2197
2	2160	47	2180
3	2157	48	2175
4	2152.5	49	2170
5	2147	50	2166
6	2143	59	2135.5
7	2140	73	2138
8	2136	79	2142
9	2133	80	2139.5
10	2130	102	2144.5
11	2193.5	106	2146
12	2191.5	109	2150
13	2190	110	2147
14	2187	113	2156
15	2183	114	2149
16	2180	117	2158
17	2174	118	2152
18	2168	121	2160
22	2162	122	2153.5
23	2158	125	2166
24	2150	126	2156
25	2145	129	2170
26	2142	130	2158
28	2131.5	133	2173
35	2133	134	2159.5
36	2220	137	2176
37	2215	138	2161
38	2209	141	2179
39	2203	142	2162

The areal (irrigation recharge) and line (leakage recharge) sources were discretized as point sources at nodes. Point values for irrigation recharge were computed by multiplying the measured recharge area for each node (Figure 11) by a uniform recharge rate. The recharge area for each node consisted of a polygon whose boundaries were determined by points located midway to each of the adjacent nodes. The polygon boundaries were digitized and the areas computed using the GIS. Point values for leakage recharge were computed by multiplying the leakage rate (equation 2) by the flow cross-section area. The flow cross-section area was computed by multiplying the width of the ditch (assumed to be five feet) by the distance between points halfway to the two adjacent nodes. Production well withdrawals were estimated by assigning fractions of the total production rate to the adjacent nodes in inverse proportion to the distance from the node to the well. A summary of specified flows at each node is in Table 4.

PARAMETER ESTIMATION

The parameters of the numerical model are transmissivity, recharge from surface irrigation, recharge from leaking irrigation ditches, and withdrawals by production wells (Equations 1 and 2). Few direct measurements of these quantities exist; as a result, the approach used was to estimate reasonable values from available qualitative data consisting of well logs, irrigation records, soil maps, and water rights records.

Transmissivity was evaluated using data from five constant discharge pumping tests performed by WRD on March 14 through 16, 1989, March 23, 1989, and May 16 and 17, 1989 (Table 2), and from well logs. The best data were for(1) drawdown in the Pennington pumped well after 2 minutes of pumping, (2) drawdown in the Okuda pumped well after 2 minutes of pumping, (3) drawdown in the Okuda observation well after 10 minutes of pumping, (4) recovery in the LDS pumped well for ratios of total test time to recovery time less than 1000, and (5) drawdown and recovery from the LDS near observation well after 3 minutes of pumping and for ratios of total test time to recovery time less than 1000, respectively. Plots of the data at the Pennington and Okuda wells resembled those obtained for unconfined aquifers with delayed gravity yield (Neuman, 1975). The values of transmissivity calculated using the Neuman method were 12 to 28 percent lower than those

Table 4. Summary of Specified Flow Nodes

Node	Irrigation Recharge	Recharge from ditch leakage: Nevada	Owyhee	Ontario-Nyssa	Leakage from Upland Aquifers	Well Discharge	Total
19	47976	0	0	0	0	0	47976
20	56352	0	0	0	0	-18900	37452
21	48297	0	0	0	0	-24120	24177
22	65542	0	0	0	0	0	65542
23	68333	0	0	0	0	0	68333
24	47711	0	0	0	0	0	47711
25	61436	0	0	0	0	0	61436
26	40557	0	0	0	0	0	40557
29	53192	0	0	0	0	-16920	36272
30	65520	0	0	0	0	-8820	56700
31	34821	0	0	0	0	-3420	31401
32	0	0	0	0	0	-3780	-3780
33	0	0	0	0	0	-11160	-11160
34	0	0	0	0	0	-9000	-9000
41	70544	14400	0	0	0	0	84944
42	65657	16608	0	0	0	-19980	62285
43	83628	0	0	0	0	-18000	65628
44	90475	15936	0	0	0	-4140	102271
45	52394	0	0	0	0	-11160	41234
46	54080	0	0	0	0	-28080	26000
47	50245	0	0	0	0	0	50245
48	46697	0	0	0	0	0	46697
49	48470	0	0	0	0	0	48470
50	60361	0	0	0	0	0	60361
51	33616	0	0	0	0	0	33616
52	78360	15792	0	0	0	-28620	65532
53	56356	0	0	0	0	-33300	23056
54	59253	0	0	0	0	-5400	53853
55	33740	0	0	0	0	-6840	26900
56	0	0	0	0	0	-26460	-26460
57	0	0	0	0	0	-20520	-20520
58	0	0	0	0	0	-8280	-8280
60	49836	17760	0	0	0	0	67596
61	60567	17568	0	0	0	0	78135
62	62367	15840	0	0	0	0	78207
63	61367	13632	0	0	0	-9000	65999
64	57980	11520	0	0	0	0	69500
65	51143	11520	0	0	0	-6300	56363
66	59979	15552	0	0	0	-8460	67071
67	51027	17280	8352	0	0	0	76659
68	51770	0	0	0	0	-19980	31790
69	59501	0	0	0	0	-35280	24221
70	30047	0	0	0	0	-27720	2327
71	0	0	0	0	0	-46440	-46440
72	0	0	0	0	0	-16740	-16740
74	47023	0	14760	0	0	0	61783
75	48651	0	13032	0	0	-3960	57723
76	55994	0	0	0	0	-26640	29354
77	45000	0	0	0	0	-19980	25020
78	37689	0	0	0	0	0	37689
81	5618	0	0	0	0	-1080	4538

Notes.
1. All flows are ft^3/day.
2. Irrigation recharge includes recharge from surface irrigation, precipitation, and leakage from small ditches.

Table 4. Summary of Specified Flow Nodes (continued)

Node	Irrigation Recharge	Recharge from ditch leakage: Nevada	Owyhee	Ontario-Nyssa	Leakage from Upland Aquifers	Well Discharge	Total
82	16317	0	0	0	0	-360	15957
83	26949	0	0	0	0	-3600	23349
84	25800	0	0	0	0	0	25800
85	44372	0	0	0	0	0	44372
86	37909	0	0	0	0	0	37909
87	30447	0	0	0	0	0	30447
88	35750	18528	0	0	0	0	54278
89	32522	16512	0	0	0	-17460	31574
90	32899	0	0	0	0	0	32899
91	35597	0	0	0	0	-9000	26597
92	42710	0	0	8736	45864	0	97310
93	45417	0	0	14880	78120	0	138417
94	32608	0	0	10896	57204	-21600	79108
95	29306	0	0	12816	67284	-30960	78446
96	38137	0	0	0	0	0	38137
97	43397	0	0	12480	65520	0	121397
98	44650	0	12816	0	0	-1980	55486
99	57755	0	0	0	0	-26640	31115
100	56501	0	0	0	0	-13320	43181
101	56412	0	0	0	0	-23220	33192
103	39836	0	0	16080	84420	0	140336
104	72095	0	14490	0	0	-39960	46631
105	77241	0	0	0	0	-39960	37281
107	44158	0	0	18552	97398	0	160108
108	101059	0	18504	0	0	0	119563
109	96639	0	0	0	0	0	96639
111	44349	0	0	18312	96138	0	158799
112	99394	0	19680	0	0	-7740	111334
113	101285	0	0	0	0	0	101285
115	44023	0	0	16992	89208	0	150223
116	94885	0	18240	0	0	0	113125
117	109483	0	0	0	0	0	109483
119	45019	0	0	17712	92988	0	155719
120	101044	0	19464	0	0	-3060	117448
121	125812	0	0	0	0	0	125812
123	40777	0	0	17208	90342	0	148327
124	83767	0	18936	0	0	0	102703
125	122288	0	0	0	0	0	122288
127	31026	0	0	16728	87822	0	135576
128	84558	0	18024	0	0	0	102582
129	105603	0	0	0	0	0	105603
131	24122	0	0	16320	85680	0	126122
132	81908	0	18336	0	0	0	100244
133	103071	0	0	0	0	0	103071
135	23038	0	0	15600	81900	0	120538
136	68344	0	16512	0	0	0	84856
137	117610	0	0	0	0	0	117610
139	13719	0	0	8016	42084	0	63819
140	32824	0	7920	0	0	0	40744
141	52096	0	0	0	0	0	52096

Notes.
1. All flows are ft^3/day.
2. Irrigation recharge includes recharge from surface irrigation, precipitation, and leakage from small ditches.

? 6,000 ?

calculated by the Cooper-Jacob (1946) method (Table 2). The interpreted transmissivities are 600, 500, and 750 ft^2/day at the Pennington, Okuda, and LDS wells, respectively. The slopes of semilog plots of drawdown and recovery data at the OSU North and OSU East wells were changing at the end of the tests, so these data were not used for evaluating the hydraulic conductivity.

Insufficient pumping test data are available to characterize the transmissivity distribution in the study area. It was concluded based on well logs and experience with aquifer response to pumping that the aquifer is relatively homogeneous, so hydraulic conductivity was assumed to be constant and to have a value of 600 ft/day. This assumption allowed the aquifer thickness data on well logs (Tables 1 and A.1) to be used for estimating transmissivities, i.e.,

$$T = K\, b \tag{6}$$

where

K = hydraulic conductivity (ft/day), and

b = aquifer thickness (ft).

The aquifer thickness was generally less than 10 feet thick for wells drilled in the bottomlands (Figure 12). The only pumping test performed in the bottomlands yielded the smallest transmissivity value of the five tests (5000 ft^2/day). Consequently, the slope break between the bottomland and benchland was used as a boundary for aquifer properties. The computed transmissivity distribution is shown on Figure 12.

Recharge to the aquifer from surface irrigation was assumed proportional to the quantity of irrigation water applied. An estimate of applied water was obtained from irrigation company records (M. Gannett, WRD, personal communication). Records are available for only 5538 acres (about 16 percent of the study area). The records cover the years 1983-88 for an area between Nyssa and Ontario (Table 5). Average applied water for this period was 4.0 ft/year and ranged from 3.90 to 4.14 ft/year. Bruck (1988) reported irrigation rates of 5 ft/year.

Table 5. Irrigation Water Use Records: Ontario-Nyssa Canal

Year	Delivered to Laterals	Lateral Waste	Lateral Losses	Delivered to Farms Total	Delivered to Farms Per Acre
1983	22873	5984	-4935	21824	3.95
1984	10574	8559	-19187	21202	3.94
1985	28697	6815	10248	22909	4.14
1986	15392	3849	7594	21649	3.90
1987	17618	2409	15209	22149	3.99
1988	24746	1621	509	22616	4.08

Notes.

1. Irrigated acreage = 5538 acres.

2. Quantities are acre-feet.

3. "Delivered to Laterals", "Lateral Waste", and "Delivered to Farms" are measured.

4. Lateral Losses are computed as (Delivered to Laterals)-(Delivered to Farms)-(Lateral Waste). Negative amounts indicate pickup from other canals.

The portion of applied irrigation water that recharges the aquifer depends on soil type, slope length and steepness, timing and duration of irrigation, and aquifer depth. The soils in the area typically have moderately low to moderate permeabilities (0.2 to 2.0 in/hr, Lovell, 1980). The runs are generally long and the slopes shallow (less than two percent). These factors indicate that a relatively small percentage of the applied water will run off and therefore a relatively large percentage will be available for recharge. Lacking detailed information on the controlling parameters, recharge was assumed uniform for the irrigated portions of the study area (except over Ontario, where little land is irrigated and much of the land is paved or built over).

Production well withdrawals were estimated from ground water rights records (Table 6). Withdrawals were estimated to be one-half of the water right during a four

Table 6. Production Well Water Rights.

Township. Range. Section	Water right (cubic feet per second)	Township. Range. Section	Water Right (cubic feet per second)
17.47.32dd	0.18	18.47.07cc	0.94
		18.47.07db	0.75
18.46.11ac	1.11	18.47.07dd	0.45
18.46.11db	0.25	18.47.09ac	0.42
18.46.12bc	1.34	18.47.09bd	0.39
18.46.12dd	0.80	18.47.09ca	0.25
18.46.13dc	0.12	18.47.09dc	0.46
18.46.14cc	0.50	18.47.10ab	0.69
18.46.14cd	0.50	18.47.10da	0.22
18.46.16dd	0.94	18.47.11bd	3.35
18.46.19ac	1.00	18.47.15ba	0.80
18.46.19cc	1.11	18.47.15ca	0.23
18.46.19cd	1.25	18.47.15cc	0.64
18.46.20cd	0.74	18.47.16bb	0.63
18.46.20da	0.46	18.47.16bc	0.28
18.46.22cc	0.50	18.47.16cb	0.21
18.46.23cd	1.21	18.47.17ad	0.19
18.46.23dd	1.72	18.47.17bc	1.22
18.46.24bb	0.70	18.47.19ca	0.33
		18.47.21dd	0.16
18.47.02ca	15.00	18.47.28ba	2.58
18.47.02cb	6.00	18.47.31dd	0.43
18.47.03ab	1.11		
18.47.03ac	1.17	19.47.08ba	0.25
18.47.03ba	1.44	19.47.08ca	0.50
18.47.04aa	0.20	19.47.29ba	0.78
18.47.04bd	0.25	19.47.29bd	0.38
18.47.04cc	0.43	19.47.29da	2.17
18.47.05cb	0.12	19.47.30aa	0.20
18.47.05dc	0.19	19.47.31bd	0.28
18.47.07ad	0.58	19.47.32ac	1.34
		19.47.33bc	2.00

month irrigation season. Production wells with water rights less than 0.1 ft^3/sec were neglected. Locations of the wells listed in the records were generally measured from section corners; occasionally only "quarter-quarter" section locations were listed.

Seven additional high-capacity drainage wells were field located. Production rates are unknown and were assumed to be 500 gallons/min for four months each year. Domestic well withdrawals, assuming a discharge rate of 500 gallons/day, were small compared to high-production well withdrawals and were neglected.

MODEL CALIBRATION

Methodology

Theoretically, an infinite number of solutions to the steady-state flow problem exist for a given water level distribution (Neuman, 1973). The calibration procedure employed here was to develop reasonable estimates of the transmissivity distribution, rates of leakage to groundwater from ditches, and production well withdrawal rates, as discussed in the previous section, and then vary a uniform surface irrigation recharge rate until a satisfactory fit with the measured water levels was found.

The model was calibrated to water level measurements at 36 observation wells (Figure 13) by minimizing the mean-squared-error (MSE) between predicted and measured water levels. The mean error and maximum error were also determined.

$$MSE = \frac{1}{n} \sum_{i=1}^{n} (\hat{h}_i - h_i)^2, \text{ where} \tag{7}$$

where

n = number of calibration wells,
$\hat{h}_i$ = computed water level elevation at well i (ft), and
h_i = measured water level elevation at well i (ft).

$$\text{mean error} = \frac{1}{n} \sum_{i=1}^{n} (\hat{h}_i - h_i) \tag{8}$$

$$\text{maximum error} = \max\,(\hat{h}_i - h_i) \tag{9}$$

Model-computed water levels at the calibration well locations were found by digitizing a contour map of the computed levels into the GIS, overlaying a map of the calibration well locations and determining the computed water level by linear interpolation. Little error is introduced by the contouring and interpolation steps because the model computes a regular pattern of water levels.

Selection of Calibration Wells and Water Level Data

Several criteria were used to select calibration wells: 1) whether the land surface elevation at the well could be estimated with good accuracy from topographic maps; 2) the location relative to other wells, to avoid redundancy and cover all areas; 3) some wells near boundaries were selected to check the correctness of the boundary conditions; and 4) wells with water level measurements at several dates were preferred because this allowed comparison of calibrations made using water level data taken at different times.

Two concerns had to be addressed regarding the measured water levels: uncertainty of the accuracy of the measurements and, since water levels vary during the year dur to seasonal irrigation practices, selection of the appropriate set of measurements.

The primary measurement accuracy uncertainties are 1) the land surface elevation at the well location and 2) local transient drawdown caused by pumpage of the observation well or nearby wells. Land surface elevations at wells were interpolated from USGS 7.5-minute topographic maps. The stated accuracy of the contour lines on the topographic maps is equal to the contour interval. The contour interval used on maps covering the study area is 5 or 10 feet. Because an uncertainty in water level elevations of ten feet would detract excessively from the usefulness of the model, it was assumed that the mapping error was small compared to the error created by interpolating elevations between contour lines. The possible interpolation error was judged from the ground slope in the vicinity of the well, i.e., the interpolation error is greatest where the slopes are steepest. In general, calibration wells were selected so that this error is believed not to exceed three feet. Two wells with greater elevation uncertainty (wells 26 and 27) were included because they were useful for evaluating the boundary condition at Dork Canal. Changing water level conditions were noted by WRD personnel when the measurements were made. Considering instrument error and effects of well pumpage, depth-to-water measurements are believed accurate to within 1.0 feet (M. Gannett, WRD, personal communication).

The second concern was to select a set of measurements that approximated average water levels for the year so that constant rates of groundwater recharge and production well withdrawals, based on total annual flows, could be used. After performing calibrations to both the late September and late March data sets it was concluded that the two data sets were, for all practical purposes, statistically equivalent. The water level data measured in September 1988 were used.

Water Balance

For an assumption of steady-state to be justified, the net inflows and outflows within the study area should be approximately equal. Inflows were computed as the sum of the specified flow node inputs listed on Table 4. Outflows were computed at elements adjacent to specified head nodes. The outflows were computed by multiplying the component of the specific discharge determined by GW1 that is normal to the discharge boundary by the length of the discharge boundary:

$$Q_{out} = \sum_{i=1}^{n} (q_{xi}\, \Delta y_i + q_{yi}\, \Delta x_i) \qquad (10)$$

where

q_{xi}, q_{yi} = specific discharge in x and y direction of element i (ft^3/ft-day),

$\Delta x_i, \Delta y_i$ = length of element i along boundary in x and y directions (ft), and

n = number of elements containing a specified head node.

The interpolation functions used by the finite element method are such that the computed specific discharge is the same at every point within an element for triangular elements but is a function of position within the element for quadrilateral elements. The specific discharge used in the water balance calculations is that computed at the centroid quadrilateral of the element.

RESULTS

The calibrated transmissivity and recharge parameters (Figure 12; Table 4) produced a mean-squared error of 25.3 ft^2, mean error of 1.0 feet, and maximum error of +12 feet. Predicted water level contours are in general agreement with the measured contours (Figure 14; Table 7). Predicted and measured groundwater flow directions and water levels do not agree well in an area about 1.5 to 4.5 miles southwest of Ontario. The predicted flow direction is more northerly and the measured more easterly. This suggests the specified head boundary at Dork Canal may not in reality exert as strong an influence on groundwater flow as the model predicts. Predicted water levels in the area (wells 72, 73, 74, 75, 78, 79, 80, and 81) are generally lower than measured (Table 7). The difference is as much as -9 feet. This may suggest the pumping rates of drainage wells are less than assumed or that the rate of surface irrigation recharge and ditch leakage is relatively high in these areas.

Predicted water levels at wells near specified head boundaries (wells 22, 24, 26, 27, 30, 33, 71, 86, 90, and 92) are generally in good agreement with measured water levels (Table 7). Errors range from 5 to -5 feet. In general, water levels at wells near specified head boundaries were accurately predicted by the model. An exception is found around the municipal well field at the east end of Ontario. Measurements here indicate that the water level is lower than the river level. The municipal well field is directly adjacent to the river, i.e. it lies on a specified head node. As a result, the model computes no drawdown from the well field. An alternative approach is to use a third type (or Fourier) boundary condition (de Marsily, 1986, p. 139). Here a less permaeable layer is assumed to be present between the river and the aquifer. The flow from the river to the aquifer is computed by Darcy's Law and is therefore a function of the hydraulic conductivity of the less permeable layer and the hydraulic gradient between the river and the aquifer, i.e., the flow into the aquifer is a function of the water level in the aquifer. This condition could exist at other specified head nodes as well; however, the improvement in model predictions would be too slight to justify the additional complexity introduced by the use of an additional boundary condition.

For confined aquifers, GW1 computes a specific discharge which must be divided by the aquifer thickness to obtain the Darcian velocity and again by the effective porosity to obtain the average pore velocity:

$$v = q / b\, n_e \qquad (11)$$

where

v = average pore velocity (ft/day),

q = specific discharge (ft^2/day), and

n_e = effective porosity.

An effective porosity of 25 percent was assumed and the average aquifer thickness of 15 feet was used at every point to calculate average pore velocities. The effective porosity is equal to the total porosity used earlier to compute the volume of water in storage because the specific retention, which is small for gravels, is probably smaller than the uncertainty of the porosity estimate. The computed pore

Table 7. Measured and Predicted Water Levels for Calibrated Model.

Well no.	Township	Range	Section	Measured Water Level[1]	Predicted Water Level	Error
22	18	46	1daa	2151	2150	-1
24			12aca	2164	2161	-3
26			12cad	2159	2164	5
27			13bba	2164	2170	6
28			14dcd	2192	2194	2
30			15bab	2169	2170	1
31			15cdc	2186	2194	8
32			17ccc	2191	2199	8
33			19bbb	2198	2197	-1
37			21ccb	2208	2218	10
38			22cbb	2207	2211	4
39			22dba	2222	2211	-11
42	17	47	32ddc	2139	2141	2
43			33cbb	2136	2136	0
44	18	47	3dcd	2135	2135	0
45			4aac	2138	2136	-2
46			4cad	2146	2143	-3
49			4dcb	2142	2142	0
57			7bdc	2162	2165	3
59			9abd	2140	2144	4
62			10bda	2139	2140	1
71			15acd	2137	2141	4
72			16ccc	2167	2162	-5
73			17aca	2172	2163	-9
74			17cbd	2178	2174	-4
75			19ccc	2210	2205	-5
78			19db	2187	2194	7
79			20ccc	2185	2187	2
80			21ccb	2162	2163	1
81			21cac	2162	2158	-4
82			29cdc	2173	2171	-2
83			30abc	2196	2199	3
86			32aca	2160	2157	-3
90	19	47	8acc	2163	2166	3
91			8cbb	2171	2183	12
92			20ccd	2171	2175	4

[1]- Water level measured on September 20, 1988.

velocities are in Figure 15. Velocities calculated using the specific discharge at the centroid of quadrilateral elements were to compute the velocities in Figure 15.

Outflows computed by equation 10 exceeded inflows by about eight percent. Because the drainage ditches do not fully penetrate the aquifer and the model computes a hydraulic gradient away from the ditches on the downgradient side, it may seem that some groundwater underflow would occur beneath the ditches. However, the flow direction will probably be toward the ditches for several tens of feet on the downgradient side. This distance is very small compared to the grid size, so the flow reversal is not apparent in the model results.

Based on the model results, an estimated 59,300 acre-ft of water recharges the aquifer within the study area annually. Infiltrating irrigation water (including recharge from precipitation and leakage from small ditches), leakage from three large ditches, and leakage from upland aquifers contribute 74, 9, and 16 percent of this total, respectively. The irrigation recharge term amounts to 2.0 ft/year. The leakage from Ontario-Nyssa Canal was estimated (Equation 2) to be 1840 acre-ft annually. Compared to an average annual flow of 20,000 acre-ft (Table 5), about 9 percent of the supply is lost to leakage. The average water retention time for the aquifer is computed as the volume in storage divided by the total discharge rate. The estimated average water retention time is only slightly longer than two years.

The initial model (Model 1) used a mesh comprising uniformly-shaped elements whose locations did not correspond to irrigation supply ditches or drainage ditches. The Snake and Malheur Rivers were the only specified head boundaries, and no-flow boundaries were specified in the south and west. Seepage from irrigation supply ditches was not included. Although a generally good solution was obtained using

Model 1, the computed heads at the wells located relatively short distances upgradient from Dork Canal and Arcadia Drain were always excessively high.

Subsequent meshes (Models 2 and 3) were developed with lines of nodes along these two ditches, which were assigned specified heads equal to the land surface elevation for Arcadia Drain and five feet below land surface elevation for Dork Canal, and along Nevada Canal, Owyhee Ditch, and Ontario-Nyssa Canal. Specified flows were estimated for the latter three ditches using equation (2).

Model output after the specified head boundary changes described above (Model 2) showed a west to east gradient near the southwest no-flow boundary that was not apparent in the measured data. The specified flow at the nodes along the Ontario-Nyssa Canal was increased by 25 ft/day to improve the fit (Model 3). This amount would be extraordinary for leakage from a ditch; however, this flow may be reasonable if leakage from the large, high volume, unlined North Canal, which runs parallel to Ontario-Nyssa Canal, could migrate into the study area. Model 3 therefore suggests that some hydraulic connection may exist between these upland aquifers and the study aquifer, i.e., that a true no-flow boundary is not present. The results from Model 3 are presented in this paper.

The fit of the data could be improved further by spatially varying recharge quantities. An algorithm could be written using an iterative procedure that calculates the recharge at each node based on the difference between the measured and computed water levels. The iterations would continue until an acceptable MSE was achieved. However, considering the uncertainty in the other parameters (transmissivity, leakage recharge, and boundary conditions), the resulting velocities and recharge quantities would not be guaranteed to be any nearer the "true" values than those computed by the precedure used here. It would be better to collect data on the spatial variation of irrigation water use and combine this with data on the factors that control recharge in order to vary recharge in a more quantitative manner.

DISCUSSION

Groundwater velocities were computed by this model for predicting travel times over relatively long time periods (one year or longer) and relatively long distances (greater than two miles). Travel times for more short term and local conditions would not be reliable. For these cases the travel times should be determined from the local measured water level gradients and aquifer properties interpreted from nearby well log descriptions or pumping tests.

The relative proportions of recharge to the aquifer resulting from leakage through unlined irrigation ditches and infiltration of surface irrigation water cannot be computed directly by the model because the smaller ditches were not modeled as separate recharge sources. However, a comparison of the total recharge estimated for the study area (59,300 acre-ft/yr) to the sum of the recharges estimated for three of the largest ditches (5500 acre-ft/yr) suggests that the recharge from both surface irrigation water and leakage from unlined ditches is significant. Estimates of the ratio of water that percolates below the root zone to the total applied irrigation water for the study area were made by Oregon State University staff who specialize in irrigation. One estimate was 20 per cent (R. H. Cuenca, Dept. of Agricultural Engineering, OSU, personal communication); the other was that it could range as high as 40 per cent (J. Vomocil, Dept. of Soil Science, OSU, personal communication). These estimates indicate about 23,750 to 47,500 acre-ft/yr of this total irrigation recharge of 59,300 acre-ft/yr results directly from infiltration of applied irrigation water.

Some uncertainty remains regarding the no-flow boundaries, particularly where they are paralleled by the unlined North Canal. If a large portion of the leakage from the North Canal enters the aquifer, as the model suggests, it would have a significant effect on water levels. The leakage is inferred from water level contours interpreted from water level measurements at only 5 wells (wells 39, 41, 75, 76, and 89). A hydrogeological scenario where lower conductivity zones perpendicular to the flow direction result in relatively short, steep water level gradients alternating with longer, shallower gradients could also produce the measured contours.

Model Verification

Solutions to steady-state flow problems are non-unique; therefore, the solution presented here is only a reasonable one and others are possible. It is desireable to verify the parameters of the model in one of three ways:

1. Verify the aquifer properties by calibrating the model to a set of water level measurements taken under different conditions of irrigation and well withdrawals;
2. Verify the aquifer properties and recharge/withdrawal parameters by calibrating the model to several records of transient flow conditions at individual wells;
3. Compare the computed discharge to the measured flow rate at a discharge boundary (Dork Canal or Arcadia Drain) to the measured discharge at base-flow conditions of a section two or three miles in length.

Verification by the first method is the most straightforward but is not possible because no such set of measurements exists. The long-term hydrographs at the

Pennington, Weaver, and Teramura wells could be used for verification by the second method. Additional records will become available in the future from continuous water level recorders placed by WRD within the past year. Additional unknowns (storage coefficient and temporal variation of recharge) are introduced by this verification procedure and refinement of the finite element mesh around the monitored wells would be needed for better definition of the source and sink terms.

Verification by the third method could be accomplished by measuring flows in a two or three mile long section of Dork Canal or Arcadia drain during base-flow conditions and comparing these to model-computed flows.

Recommendations for Solute Transport Modeling

Modeling the movement of DCPA and nitrate in the aquifer can also be done by the finite element method using the velocities calculated by the calibrated model presented here. However, problems with numerical dispersion and instability must be considered when solving the mixed hyperbolic (advective transport) and parabolic (dispersive transport) solute transport equation numerically. The solution is to use a finer mesh than was used for this study. The mesh must be particularly fine around the edges of the solute plume. In addition, the mesh should be discretized such that the elements are oriented in the direction of groundwater flow (deMarsily, 1986, p. 395).

A smaller study area will probably be desireable for solute transport modeling because of the finer mesh required. The primary reason is the man-hours of labor needed to design the mesh and develop input files increases as the number of nodes and elements increases, largely because the opportunity for time-consuming errors also increases. This is particularly a problem for solute transport modeling because the solution is more dependent on the mesh design than it is for flow problems and more than one attempt may be needed to develop the final mesh. Additionally, the large amount of output data created will make data handling more time-consuming and cumbersome. Finally, the amount of computer capacity required, in both random-access-memory and disk storage, could become a consideration.

A suitable study area for solute transport modeling is shown on Figure 16. The DCPA and nitrate plumes defined by Bruck (1988) and Smyth (1988) lie within this area. A true flow boundary would not exist at the southern and western edges of this smaller study area; however, since the boundary is parallel to the groundwater flow lines, the flow across these boundaries would be small and could be estimated from the results of the model presented here.

Recommendations for Additional Data Acquisition

The ultimate objective of this study program is to develop a solute transport model for the part of the basin affected by DCPA and nitrate contamination. This objective would be best served if future data acquisition programs are designed to collect data that will improve model estimates of recharge and transmissivity within the smaller solute transport area only.

A relatively large body of data, including five pumping tests and numerous well logs, is available for estimating the aquifer transmissivity. Uncertainty remains about the transmissivity distribution because of the large area studied; however, this study indicates that additional pumping test data would not greatly decrease the uncertainty in transmissivity.

On the other hand, the process of calibrating the model has suggested some aspects of the groundwater hydrology that merit additional study. These include recharge from and discharge to irrigation ditches; leakage from the uplands aquifer to the study aquifer; lack of data on the actual discharge rates of wells; and the fraction of applied surface irrigation water that recharges the aquifer. Recharge is a parameter that is difficult to measure accurately. In addition, water usage changes yearly. However, the estimation of recharge and withdrawals could be improved if the following steps are taken.

1. Inventory the ditches in the area and identify where combinations of a high water level and a deep ditch demonstrate the ditches will act as drains and where conditions show the ditches will be recharge sources;
2. Gage a representative number of ditch sections and determine the seepage losses (or inflows) as a function of time and ditch length.
3. Measure withdrawal rates at high-capacity irrigation and industrial wells. Thirty-four individual irrigation well operators and three industries have water rights entitling them to withdraw groundwater in relatively large quantities (water right ≥ 0.2 ft^3/sec).
4. Individual tracts of land are drained by shallow drainage systems. The location and effectiveness of these systems is not known. Descriptions of these systems could be obtained from the farm operators or drainage district officials.

Additional wells could be added to the set used for model calibration if the land surface elevations of these wells were surveyed. Four of these wells (wells 47, 48, 50, and 61) would be of particular interest because they lie within the proposed solute transport model area. The reliability of the calibration presented in this paper could be improved if wells 26 and 27 were surveyed.

Numerous wells are present within the solute transport model area that have not been field-located. If some of these could be located and added to the water-level monitoring net the accuracy of water-level contouring would be improved.

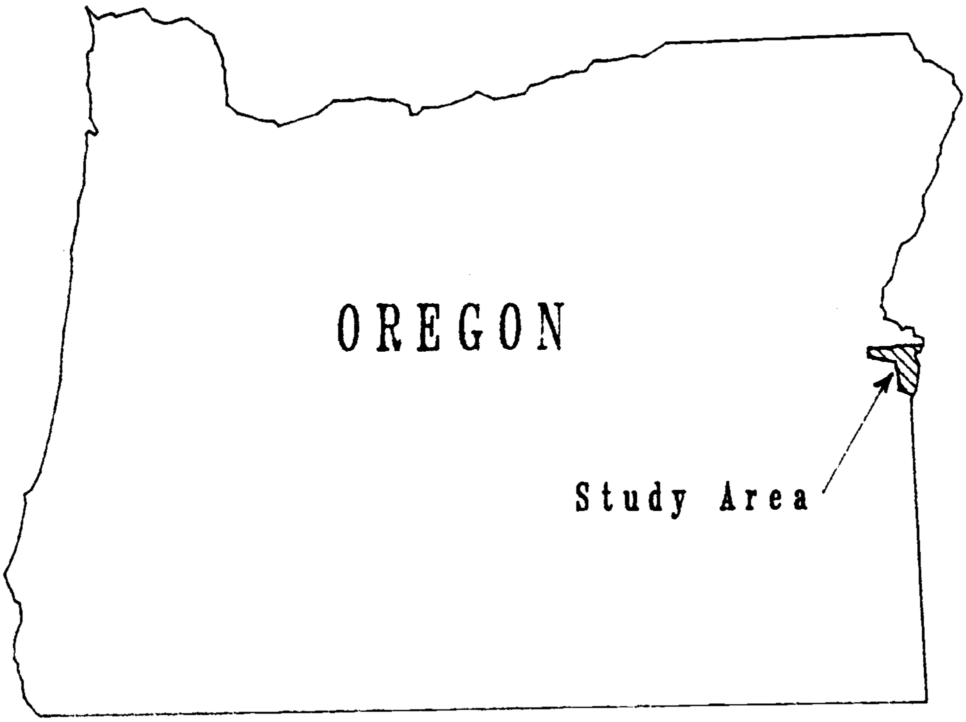


Figure 1. Location Map

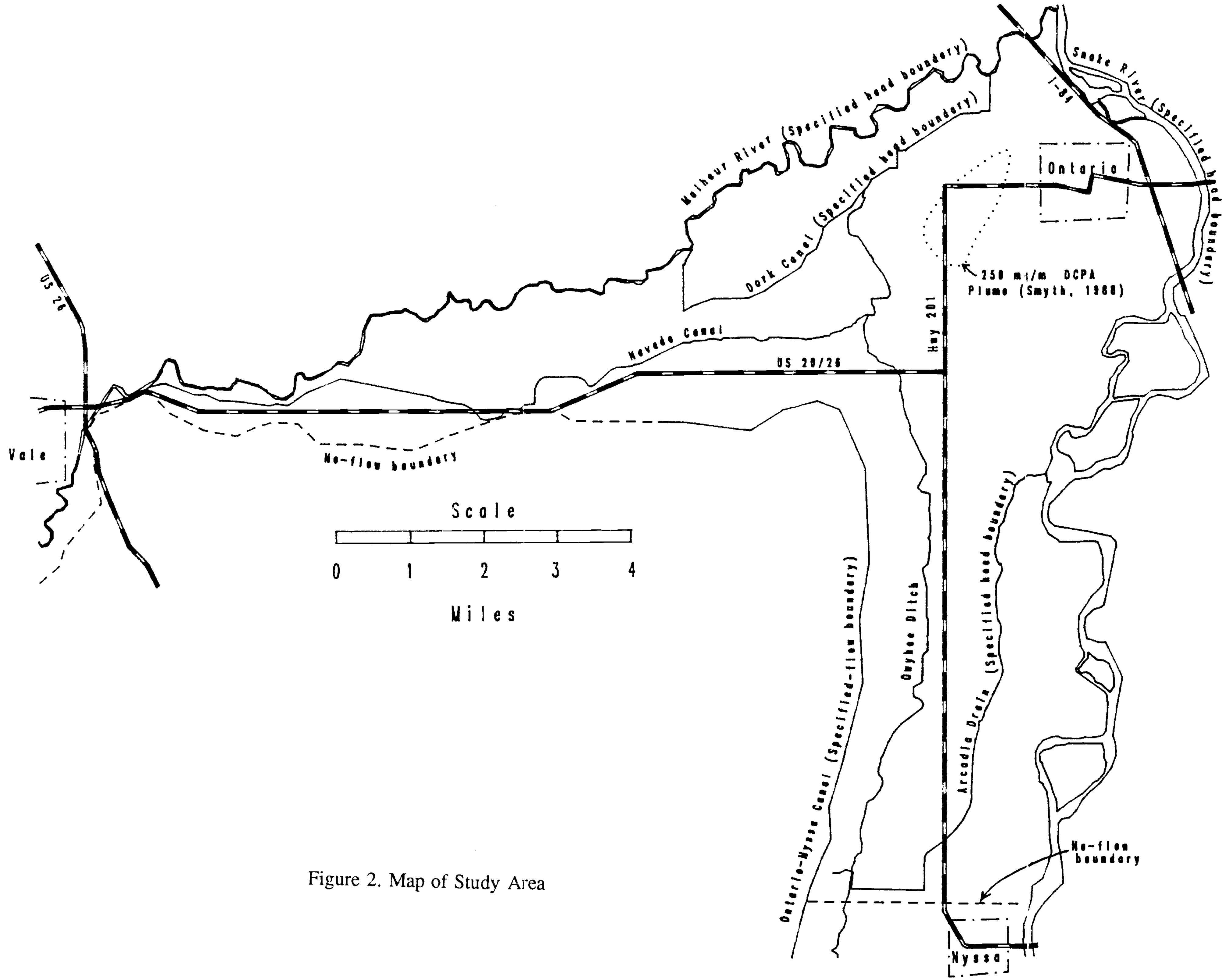


Figure 2. Map of Study Area

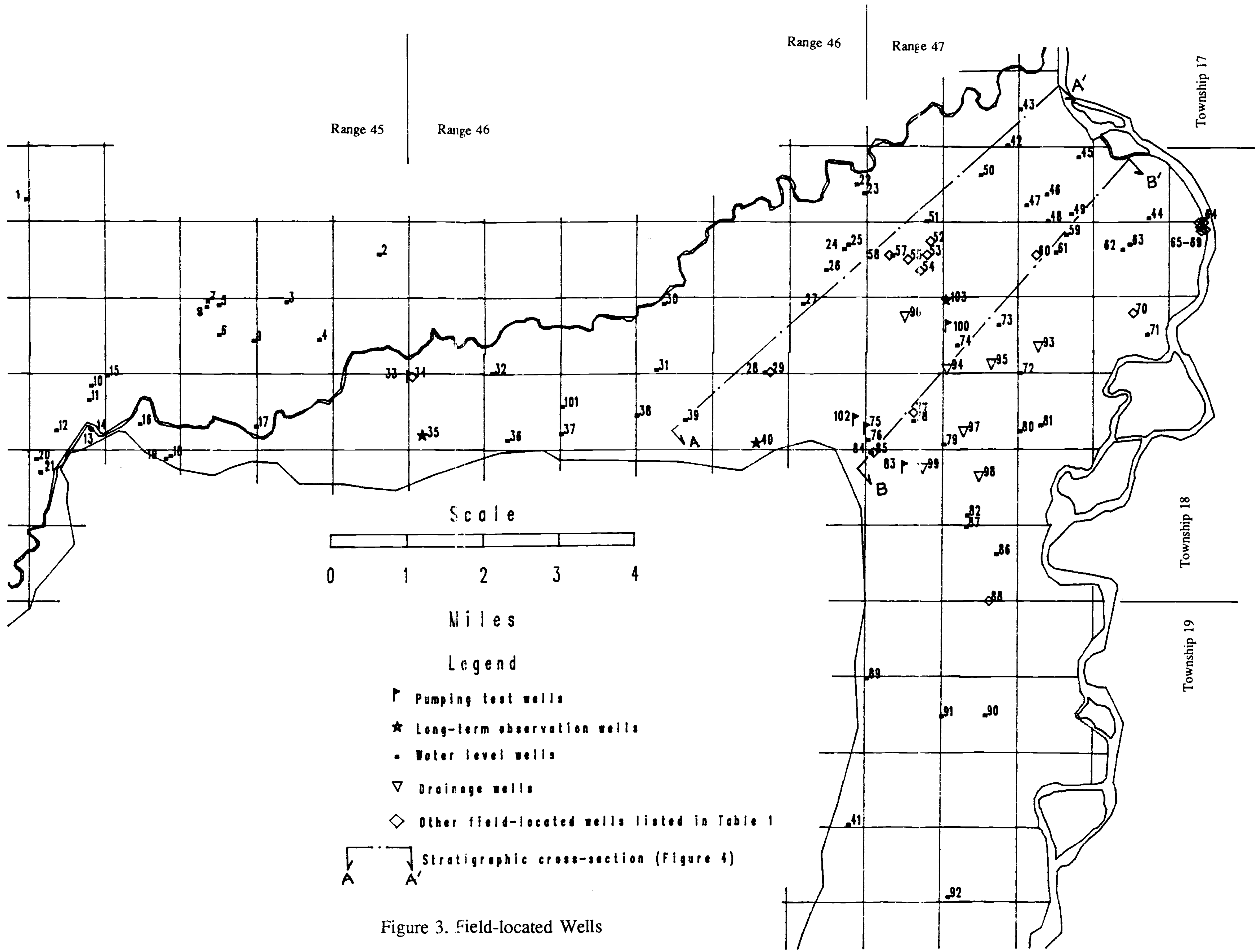


Figure 3. Field-located Wells

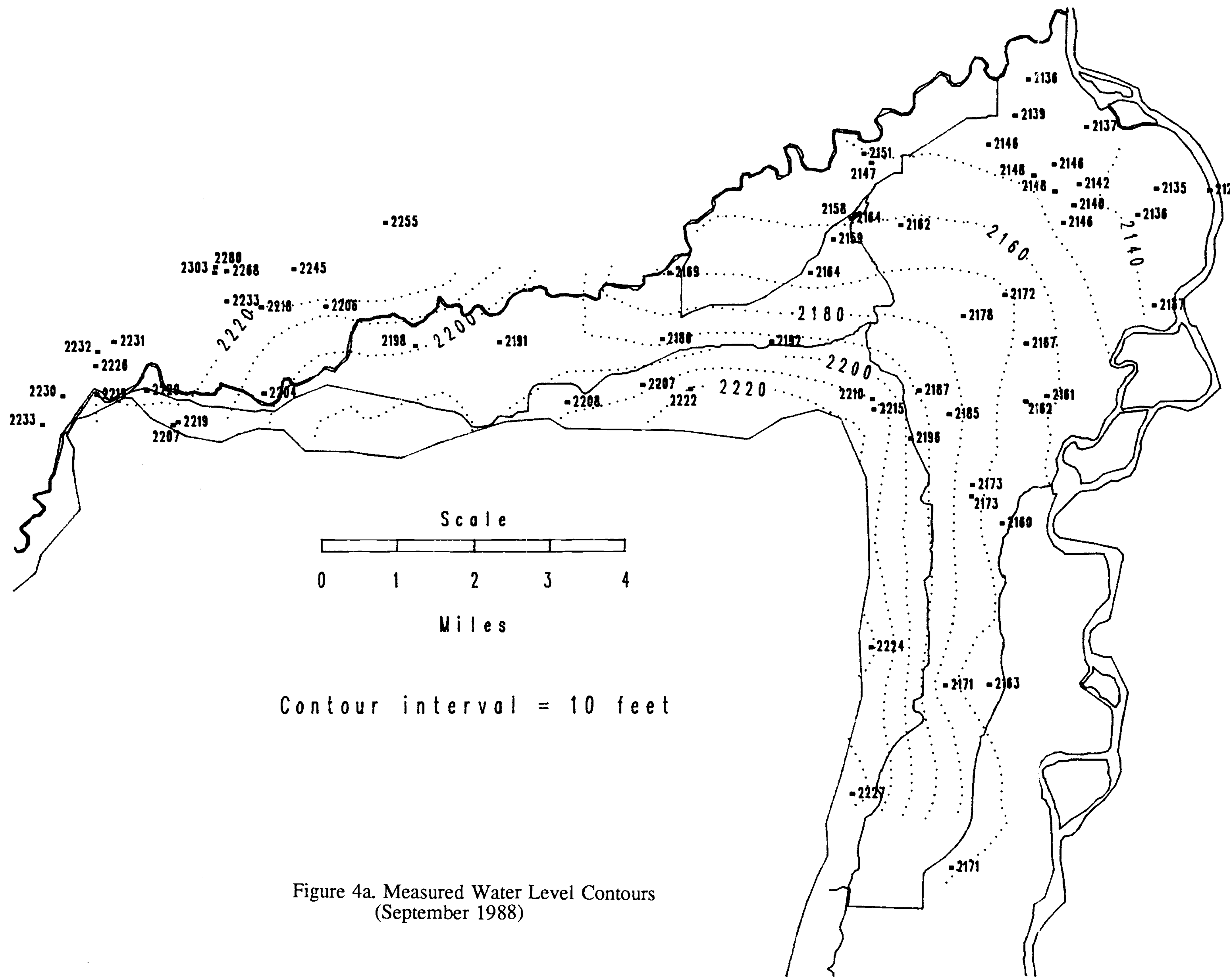


Figure 4a. Measured Water Level Contours
(September 1988)

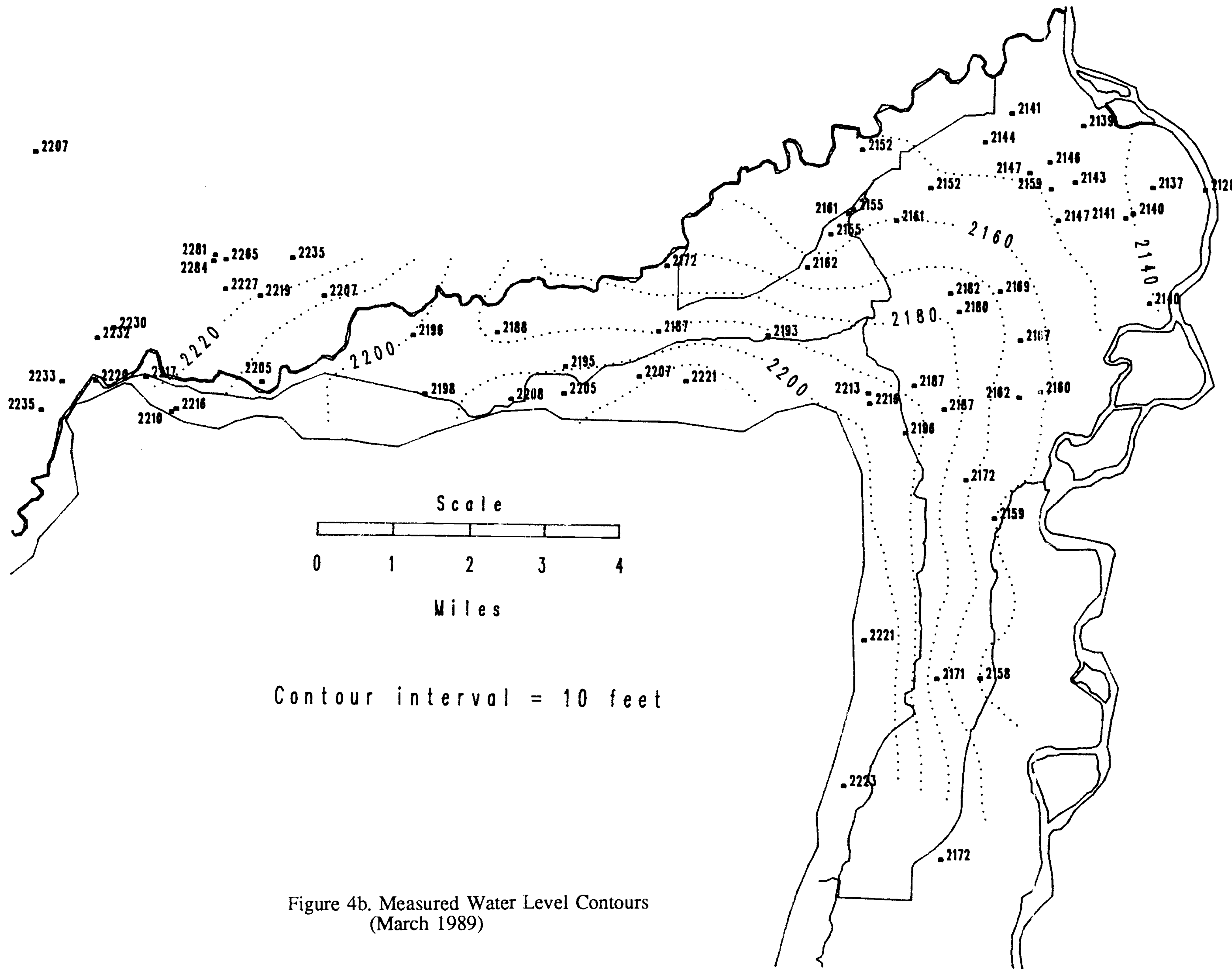


Figure 4b. Measured Water Level Contours
(March 1989)

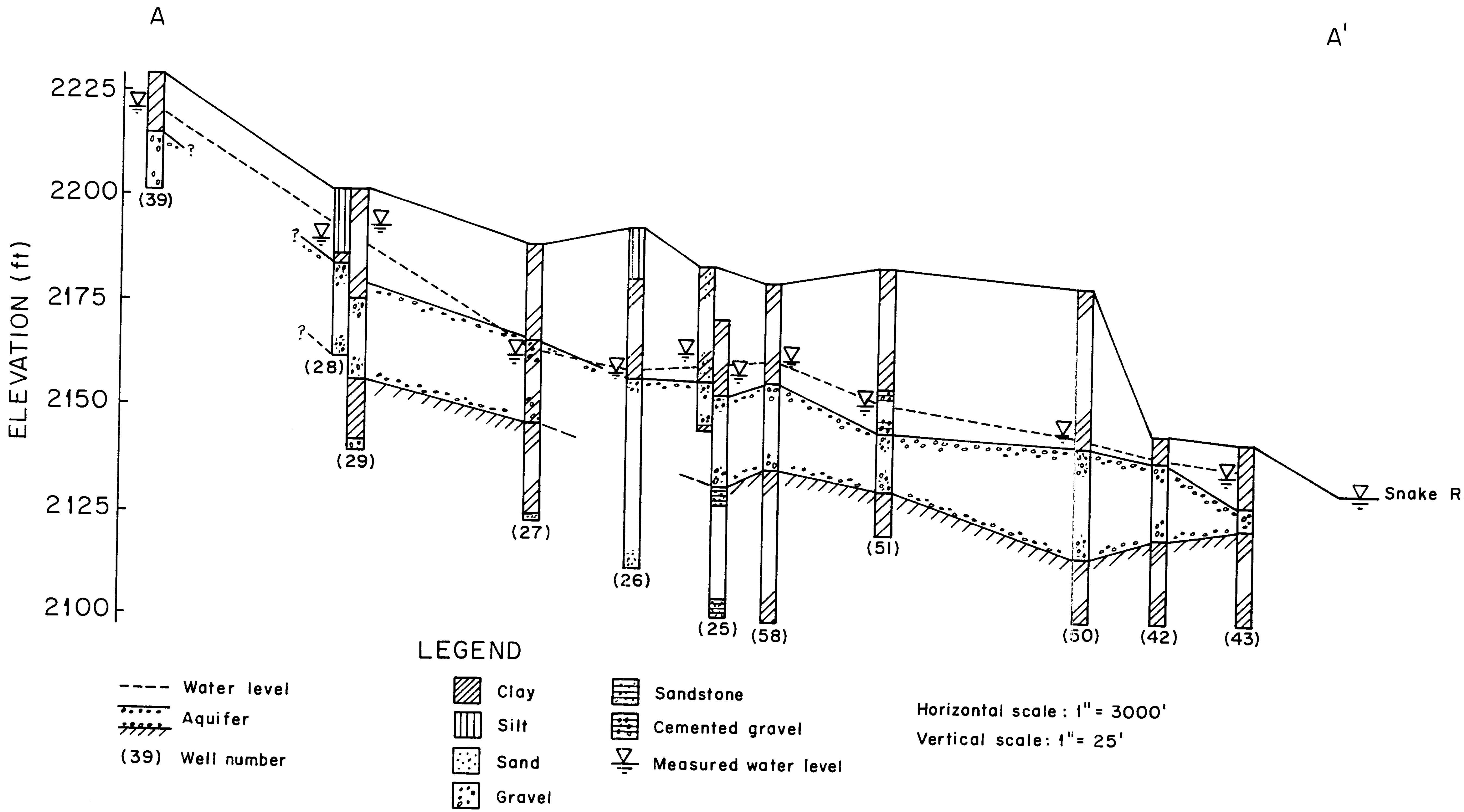


Figure 5a. Stratigraphic Cross-section A-A'

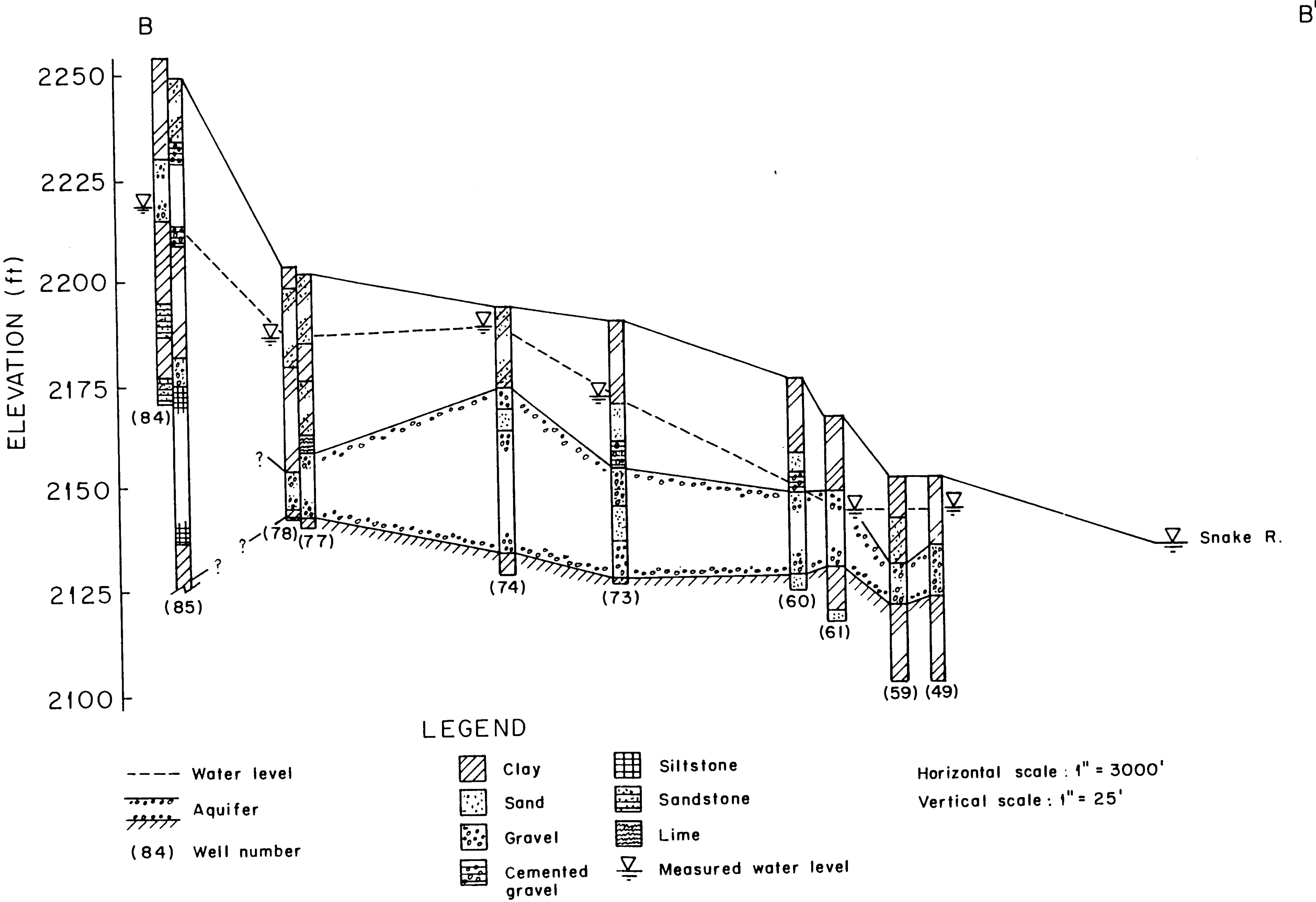


Figure 5b. Stratigraphic Cross-section B-B'

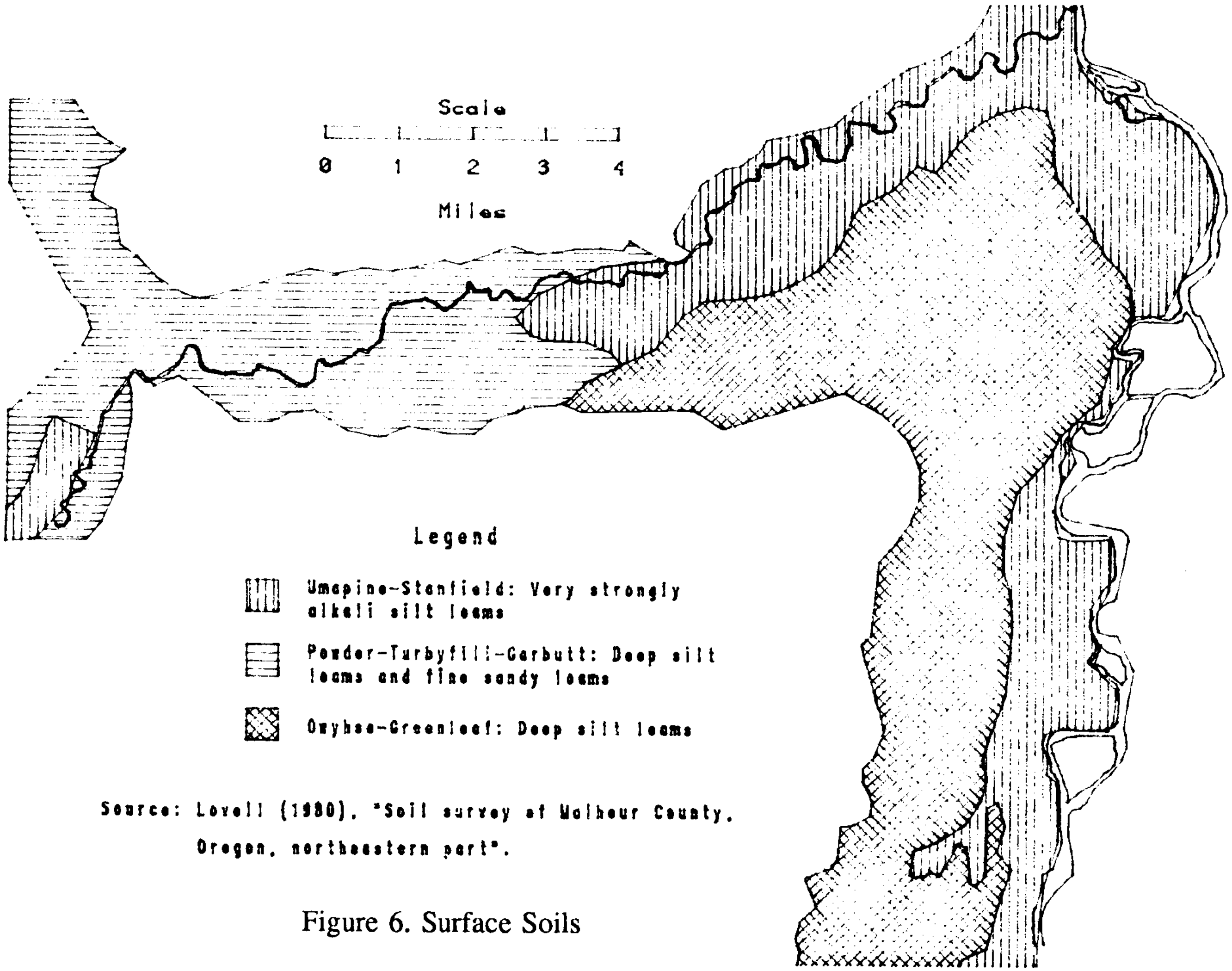


Figure 6. Surface Soils

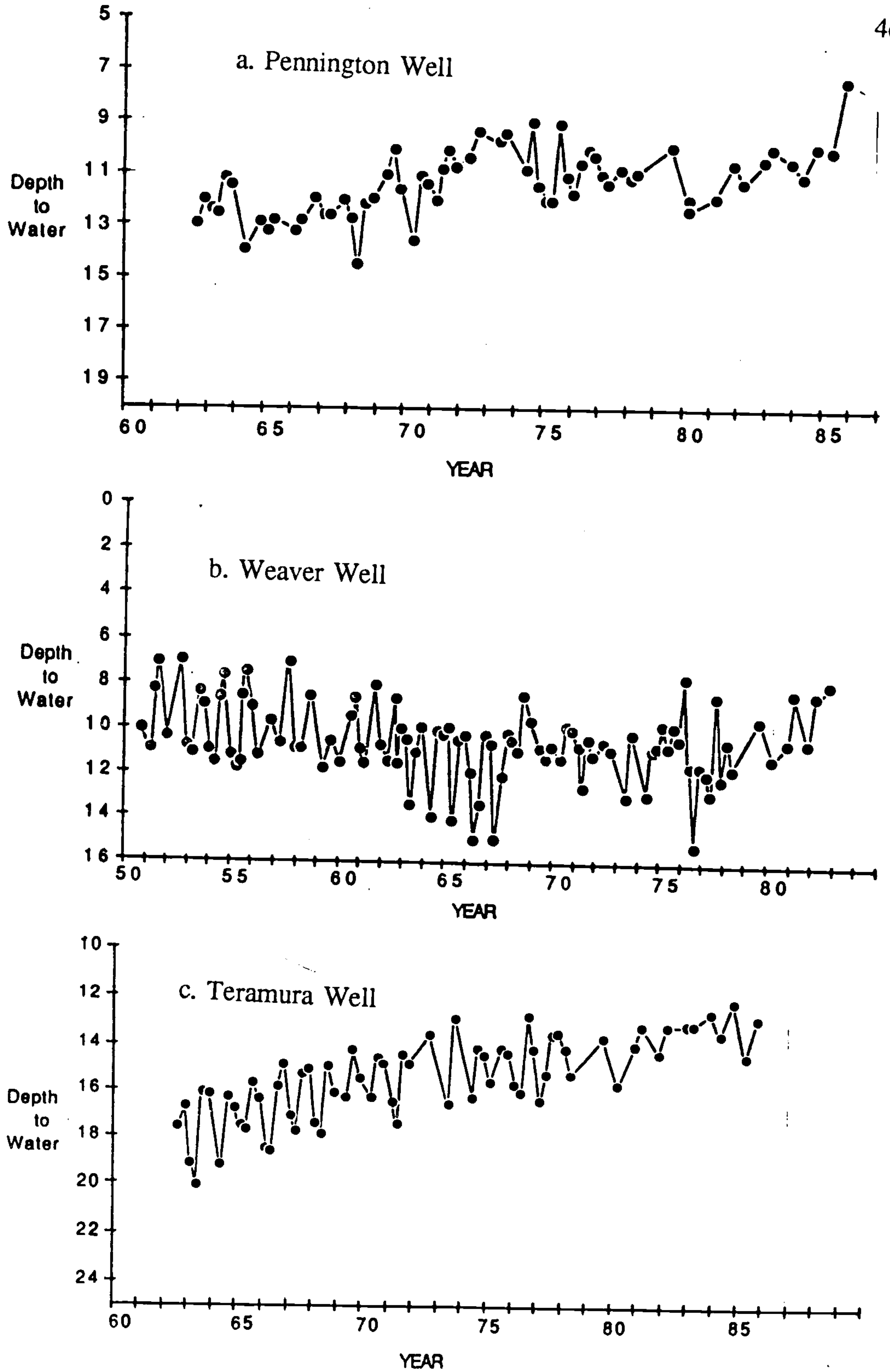


Figure 7. Long-term Hydrographs.

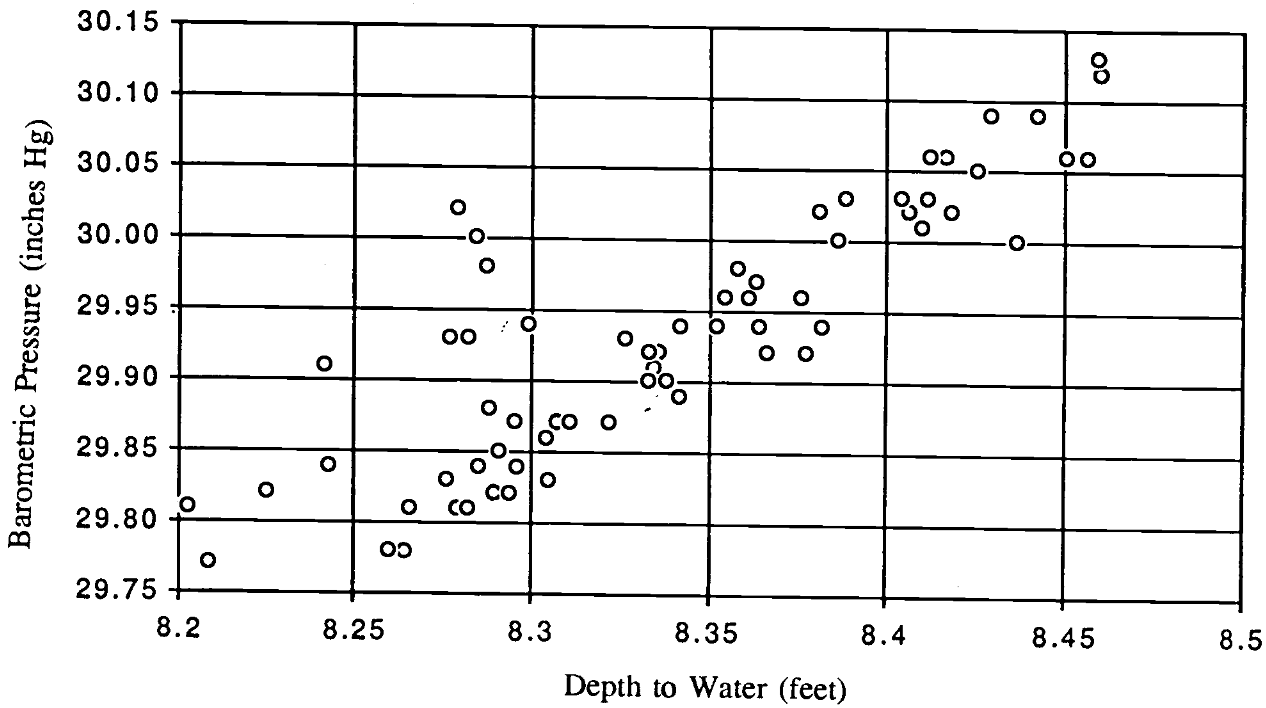
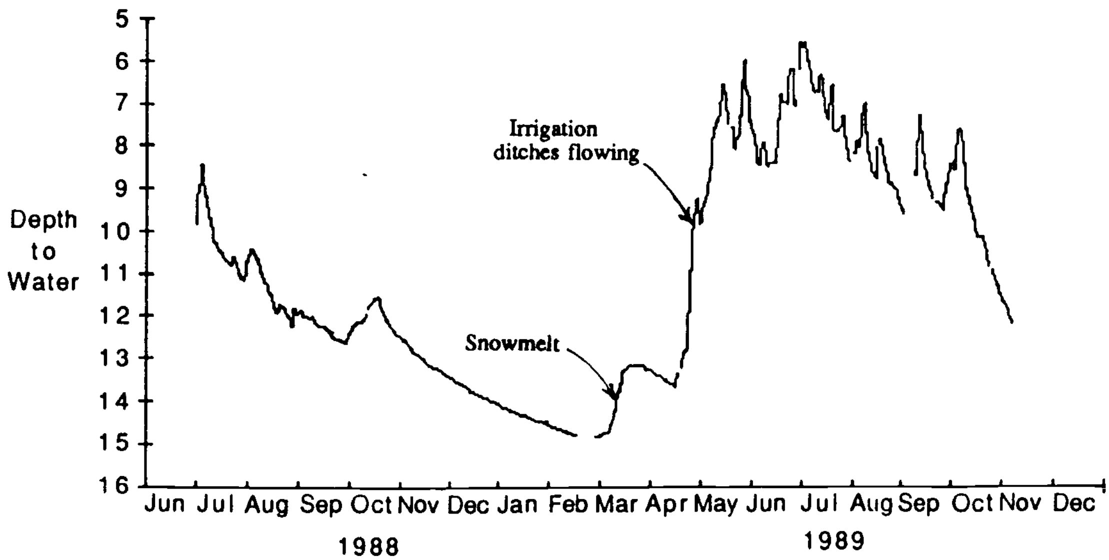
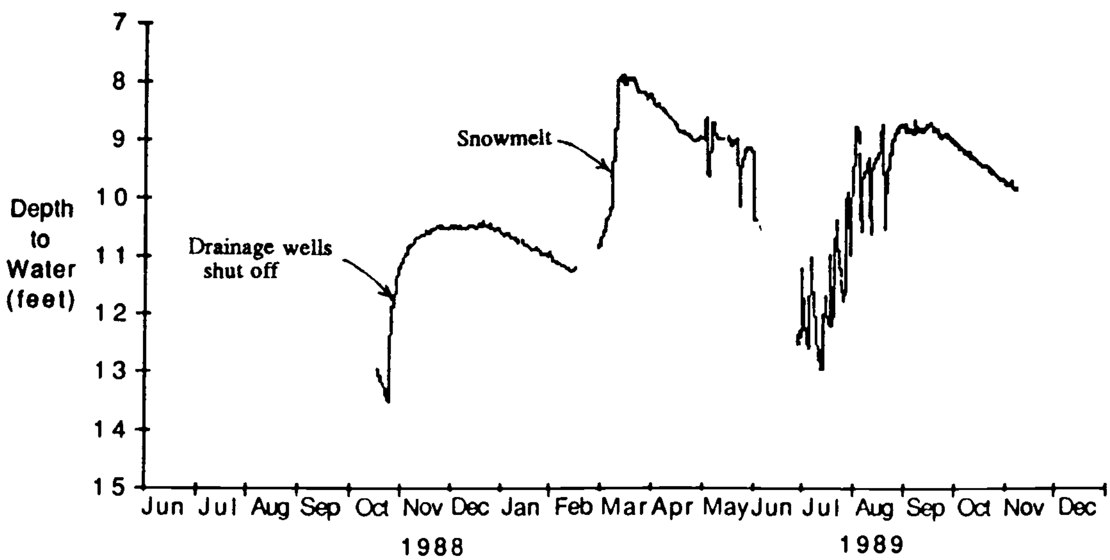


Figure 8. Barometric Response (OSU North Well)



a. Pennington Well (well 33)



b. OSU North Well (well 75)

Figure 9. Continuous Hydrographs

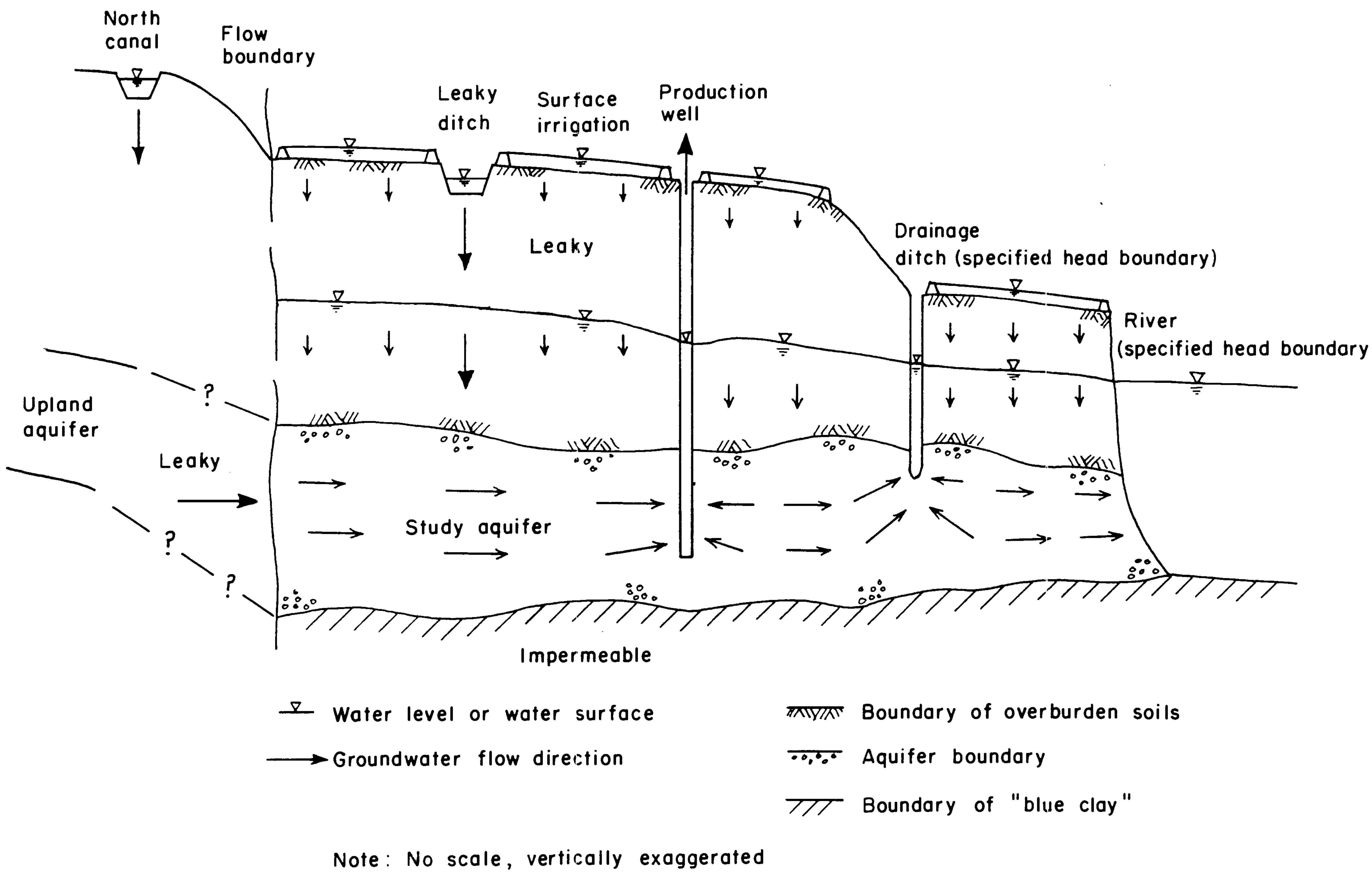


Figure 10. Conceptual Model

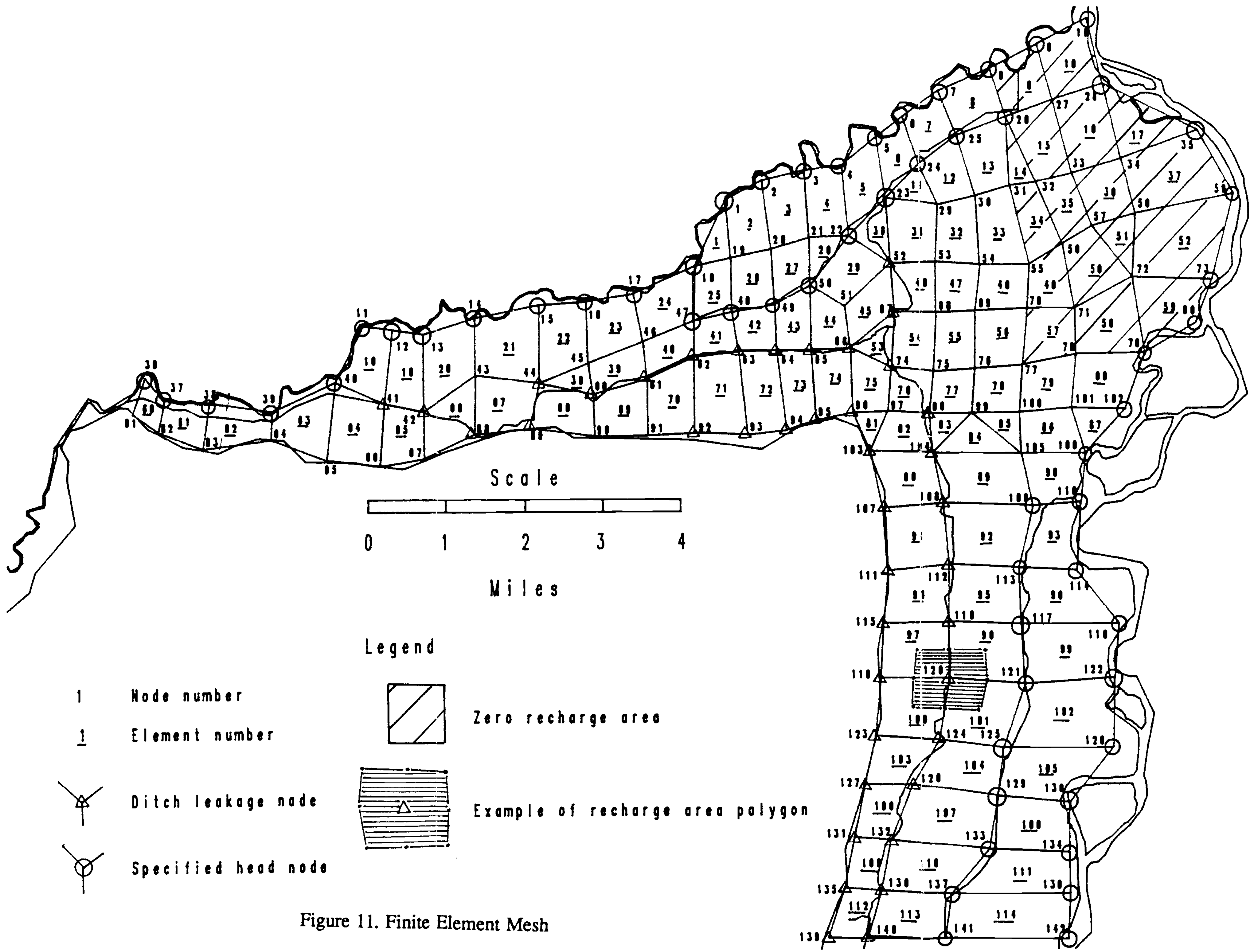


Figure 11. Finite Element Mesh

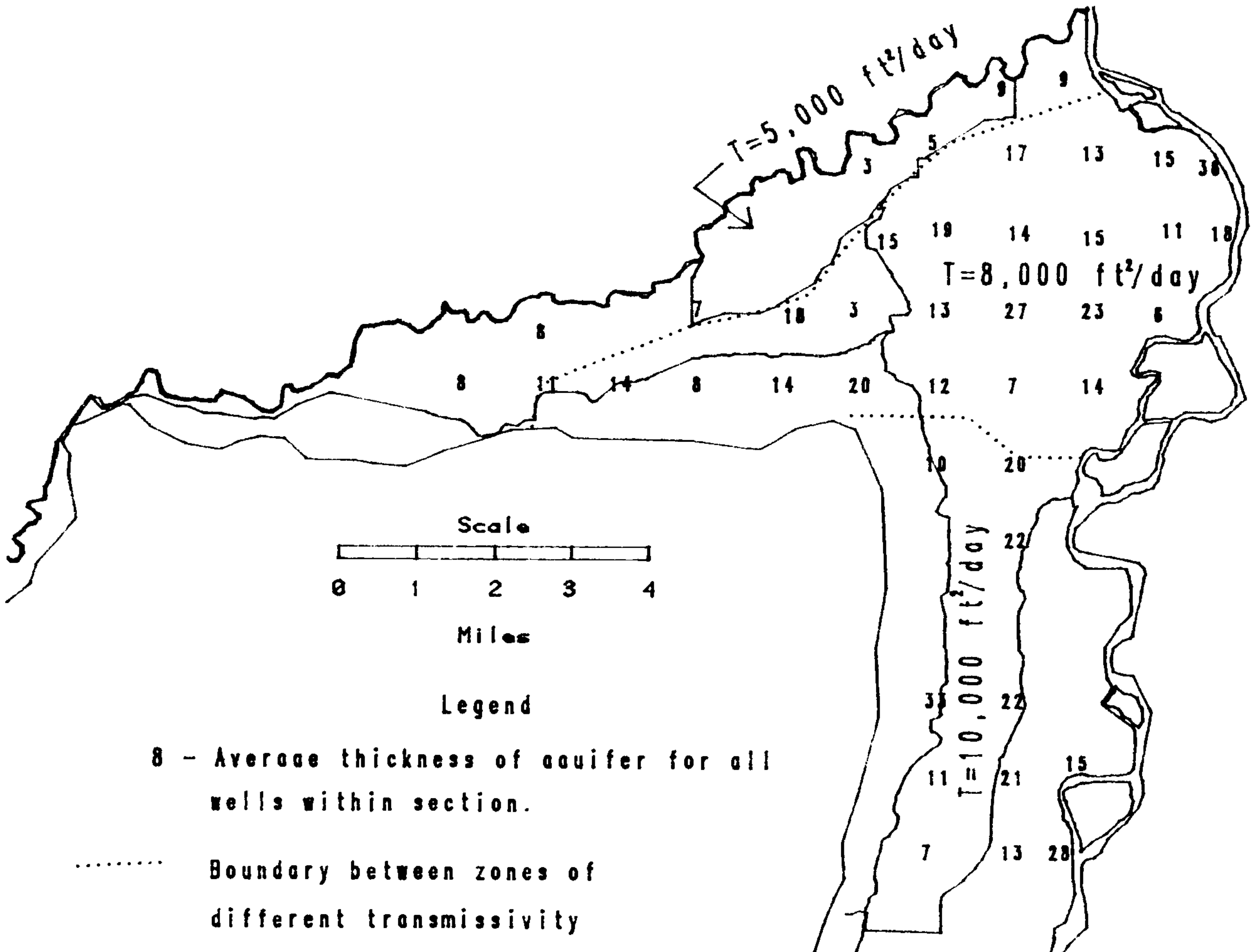


Figure 12. Calibrated Transmissivity

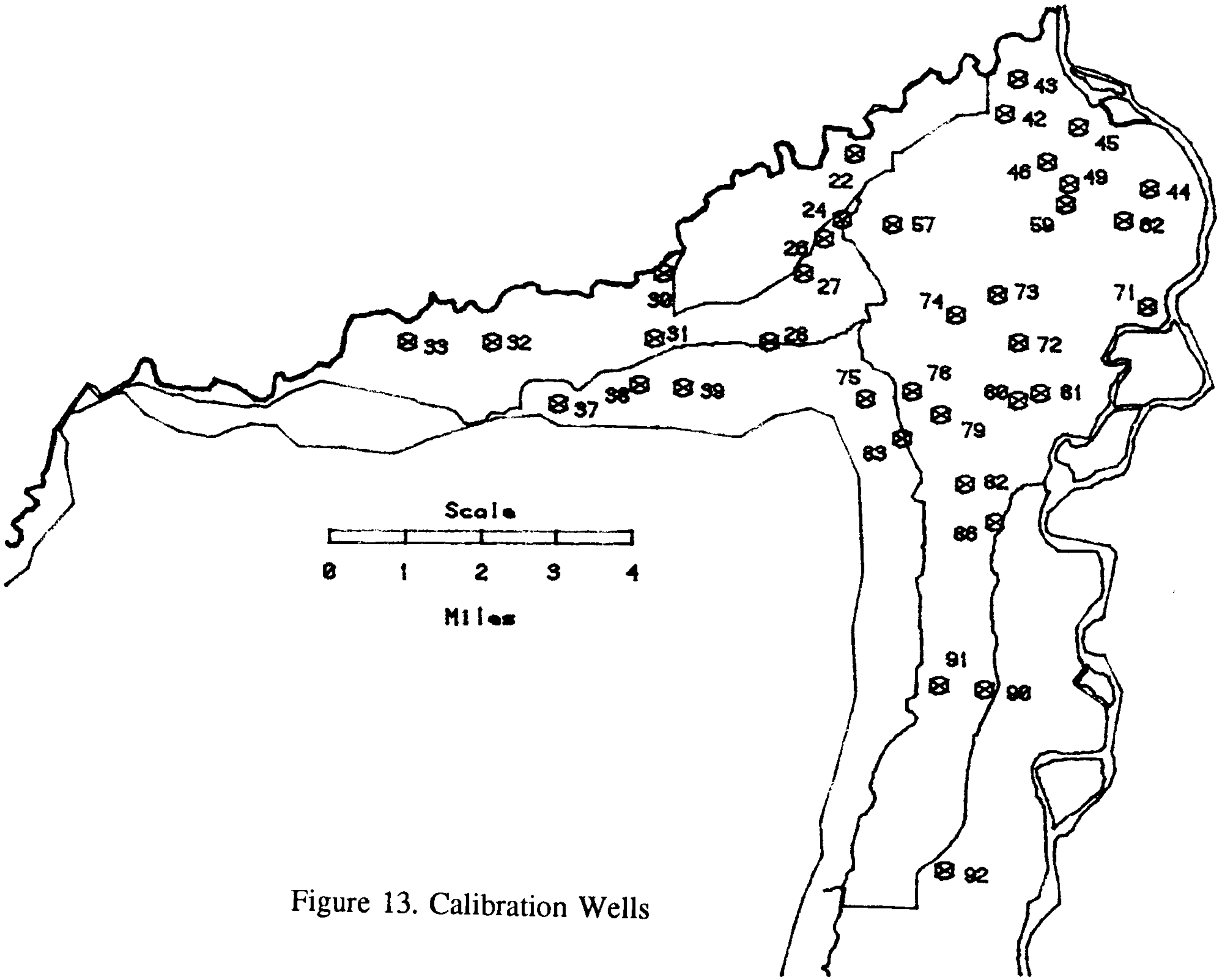


Figure 13. Calibration Wells

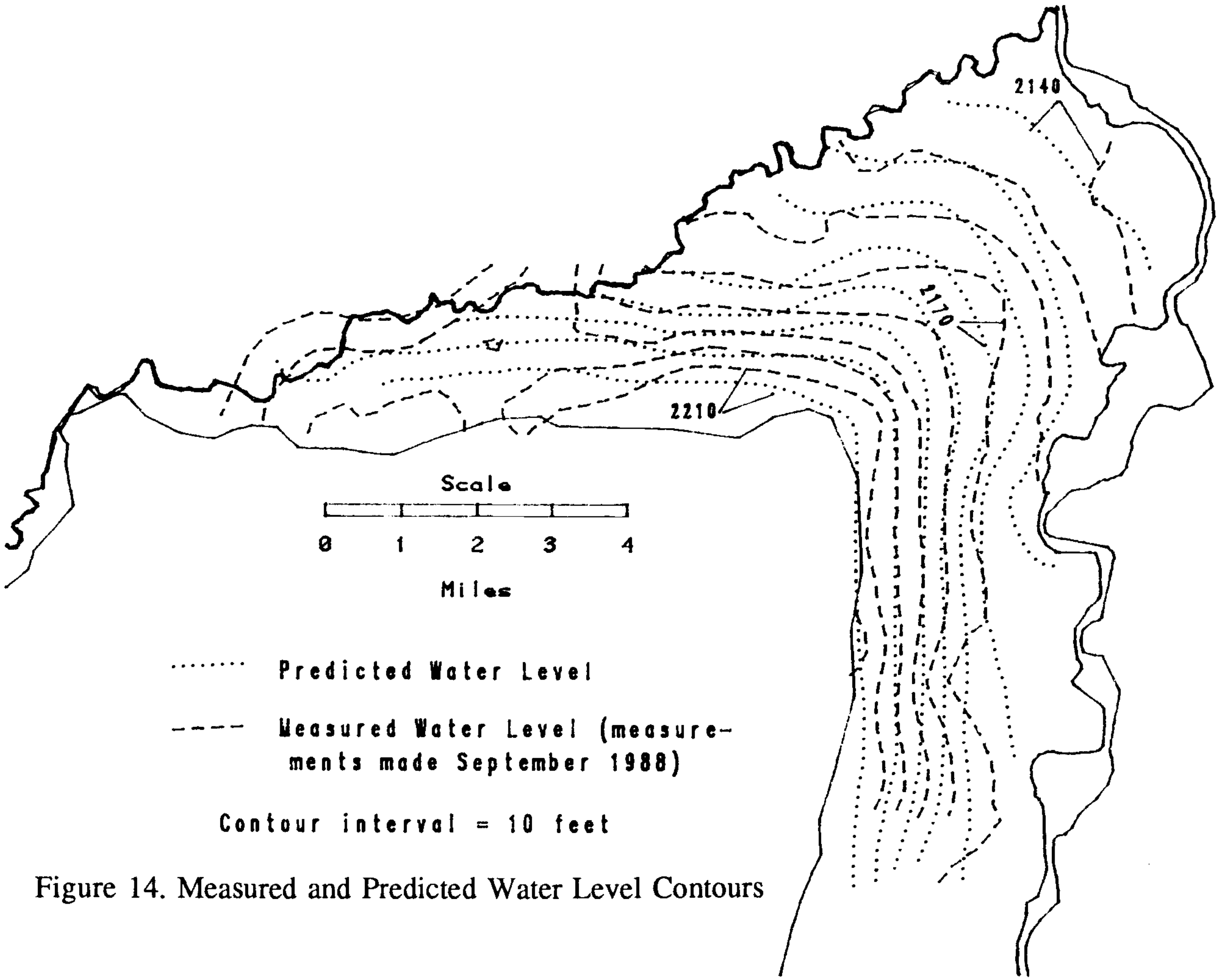


Figure 14. Measured and Predicted Water Level Contours

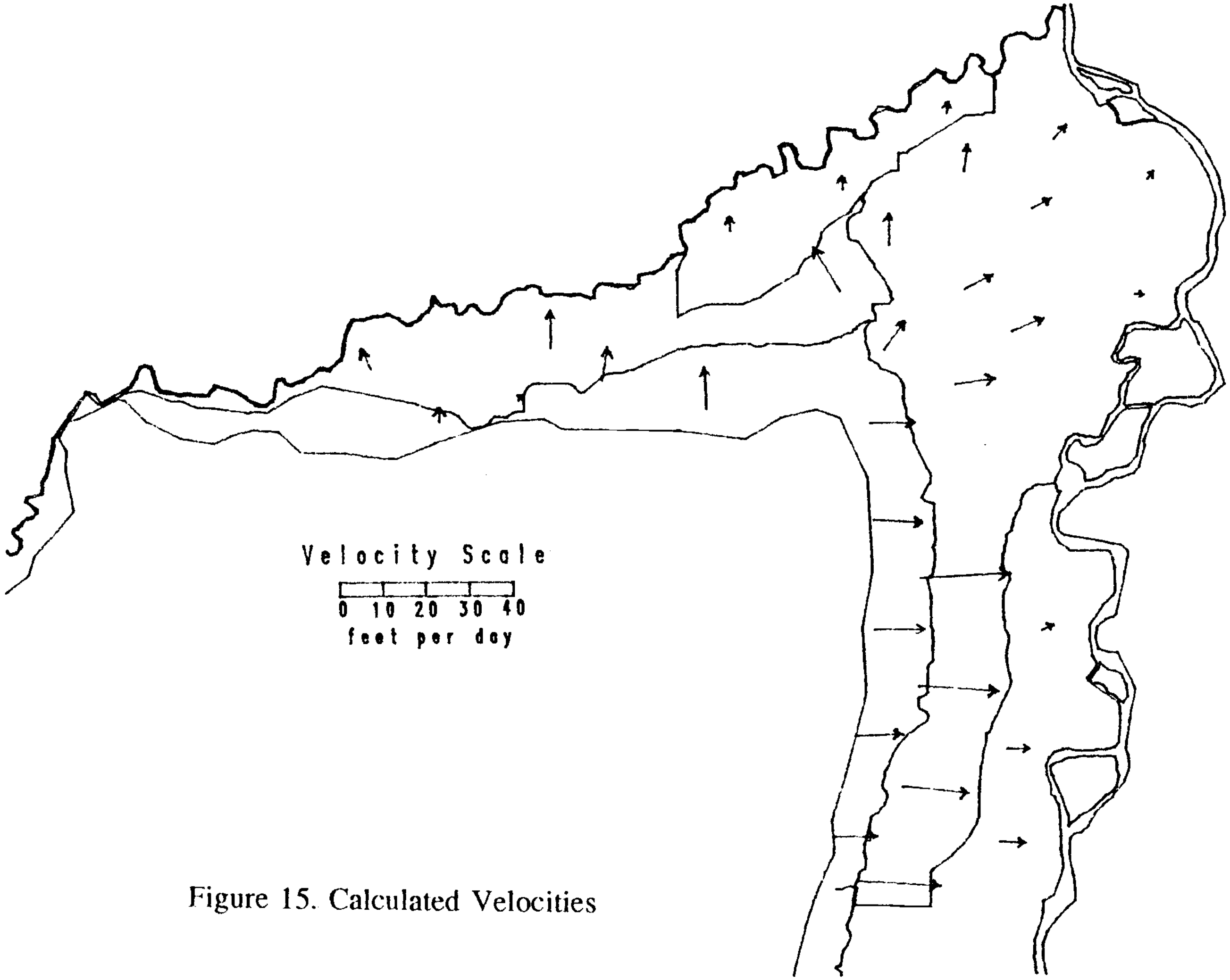


Figure 15. Calculated Velocities

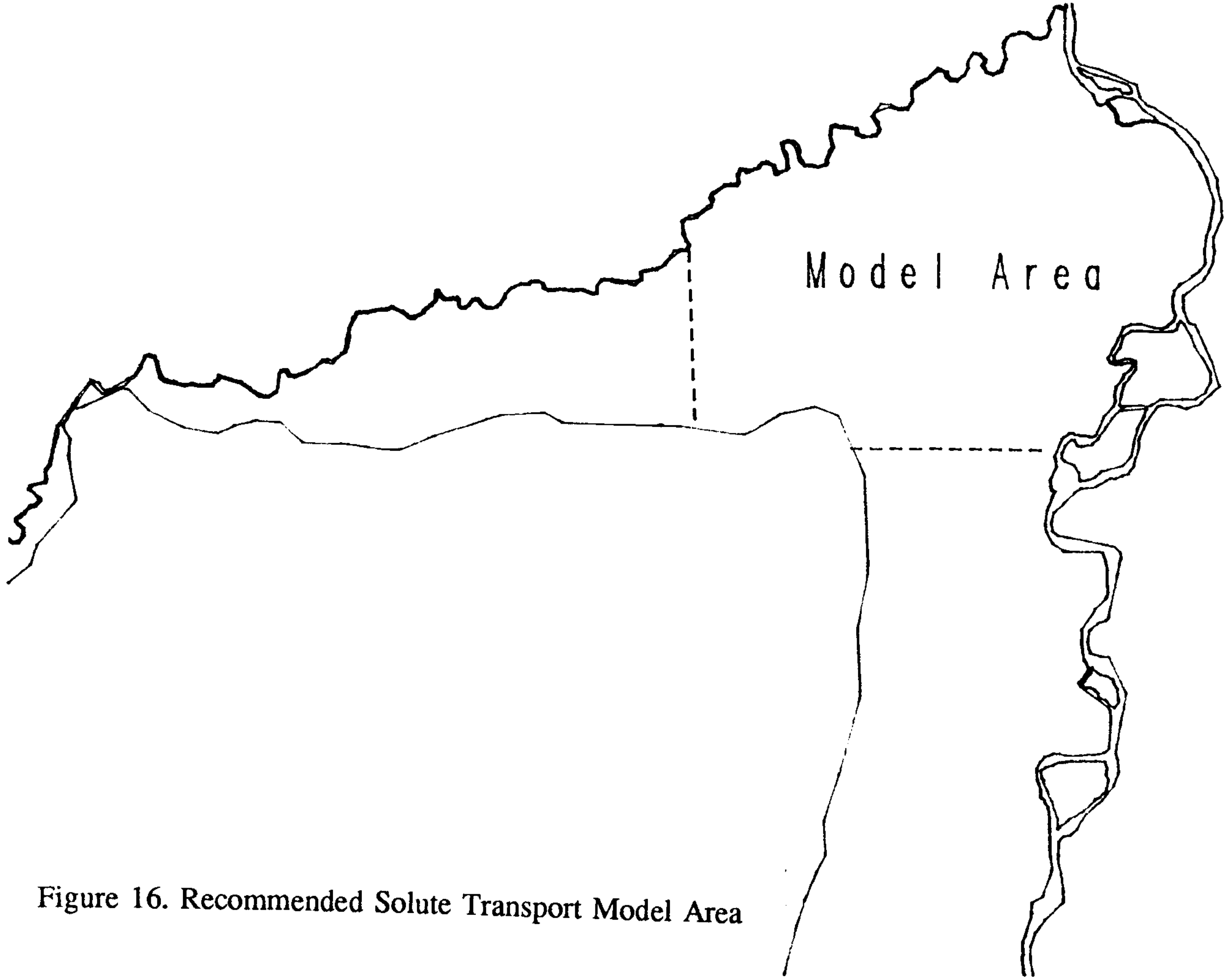


Figure 16. Recommended Solute Transport Model Area

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APPENDIX

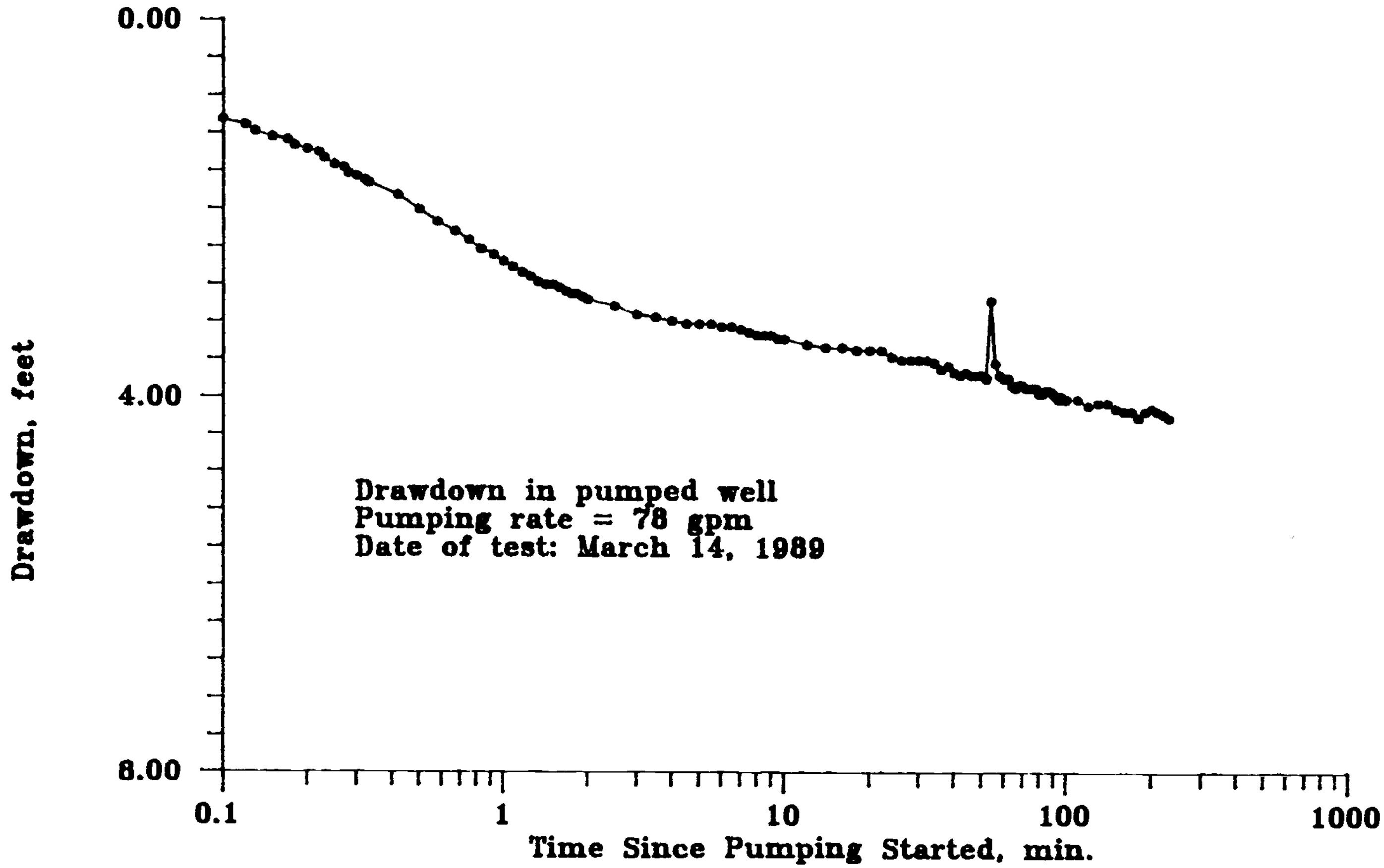

Figure A.1a. Pennington Well Pumping Test Results

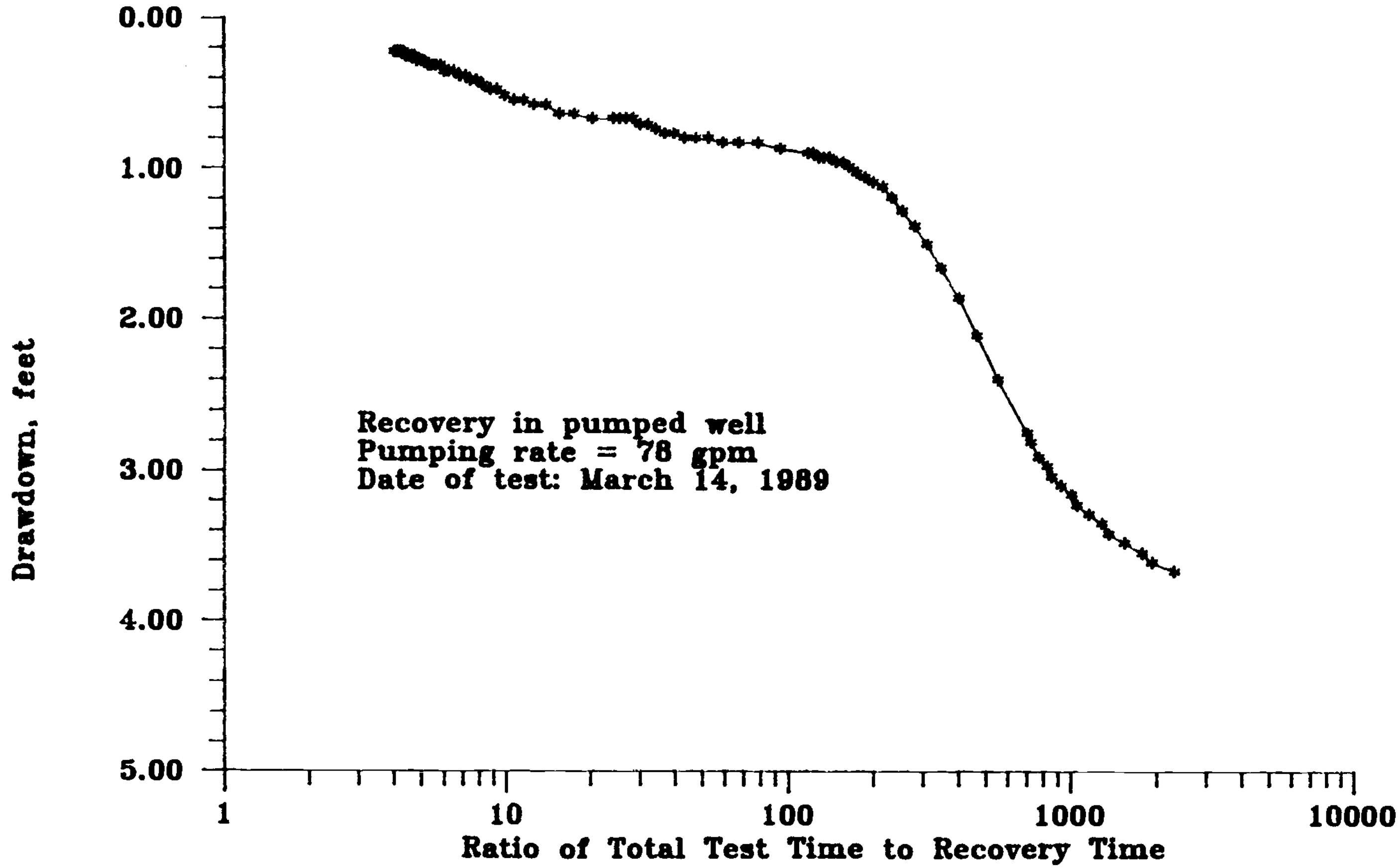

Figure A.1b. Pennington Well Pumping Test Results

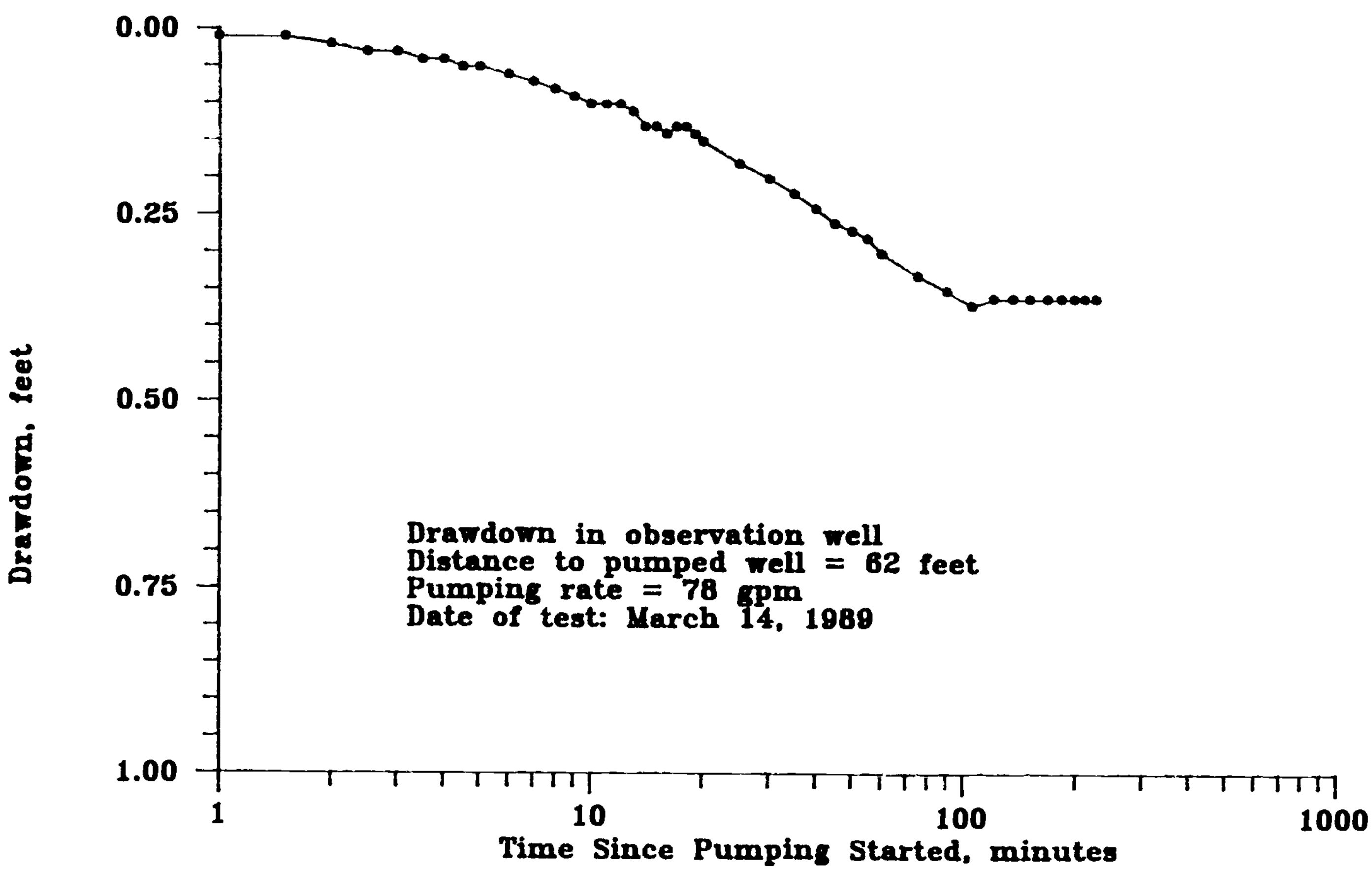

Figure A.1c. Pennington Well Pumping Test Results

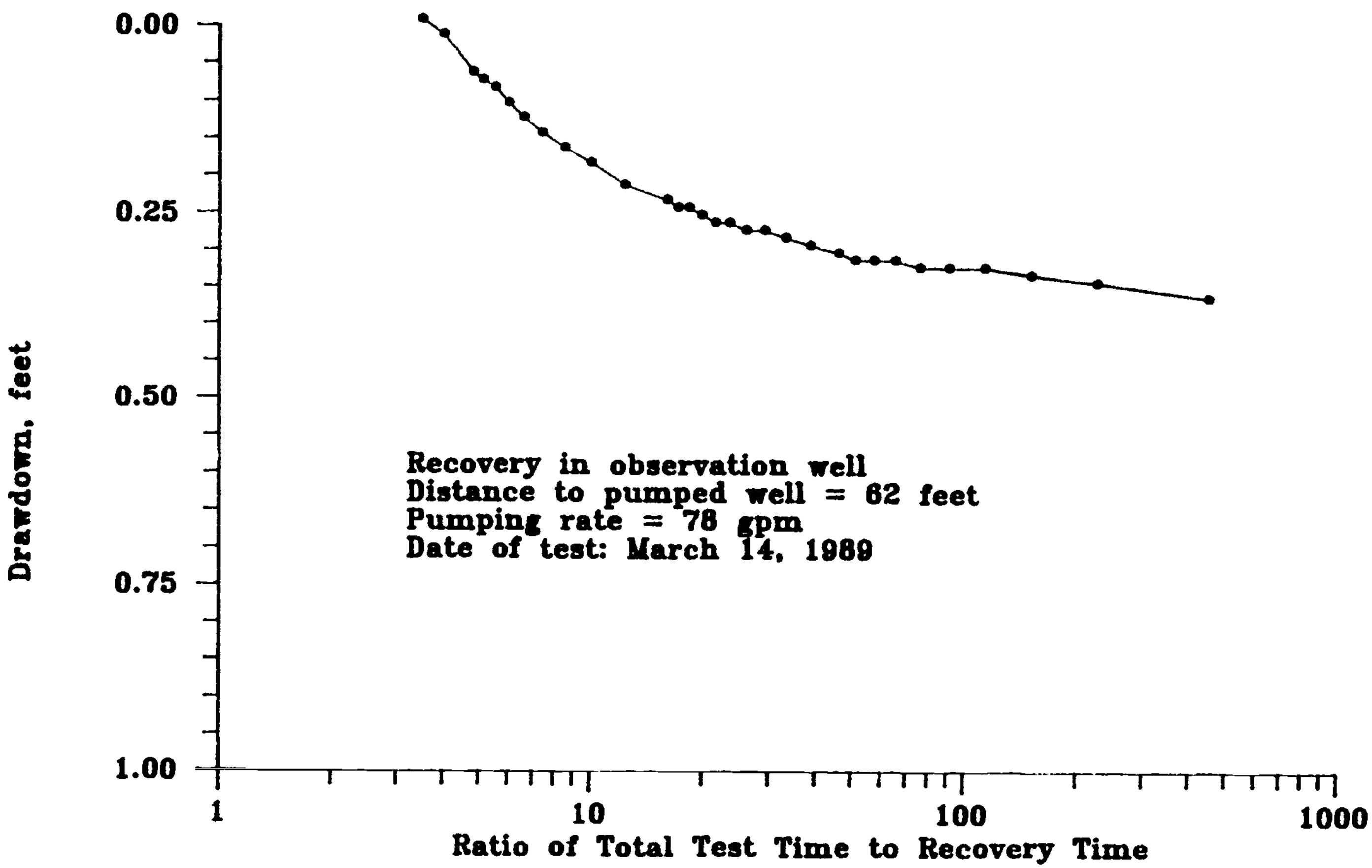

Figure A.1d. Pennington Well Pumping Test Results

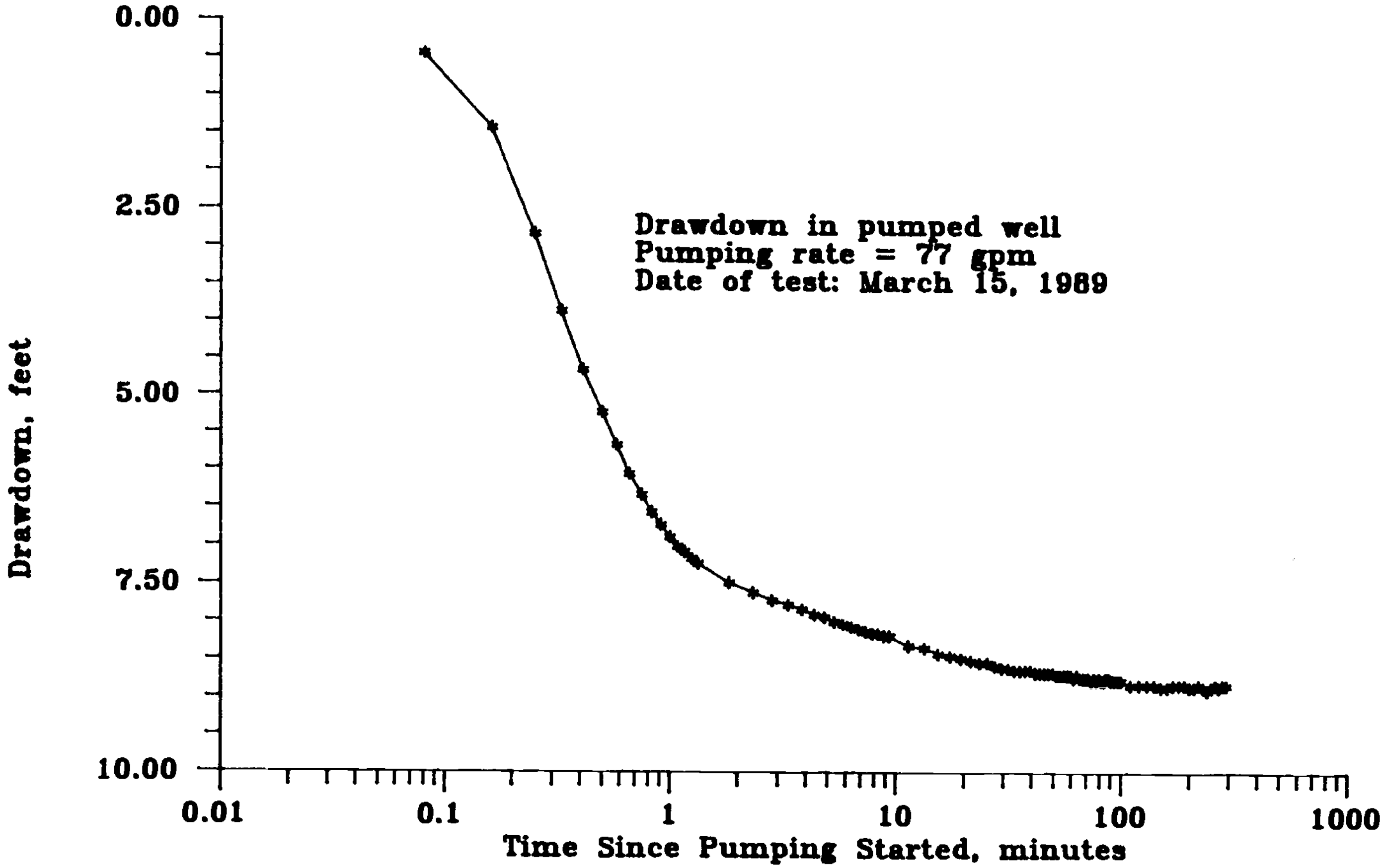

Figure A.2a. OSU North Well Pumping Test Results

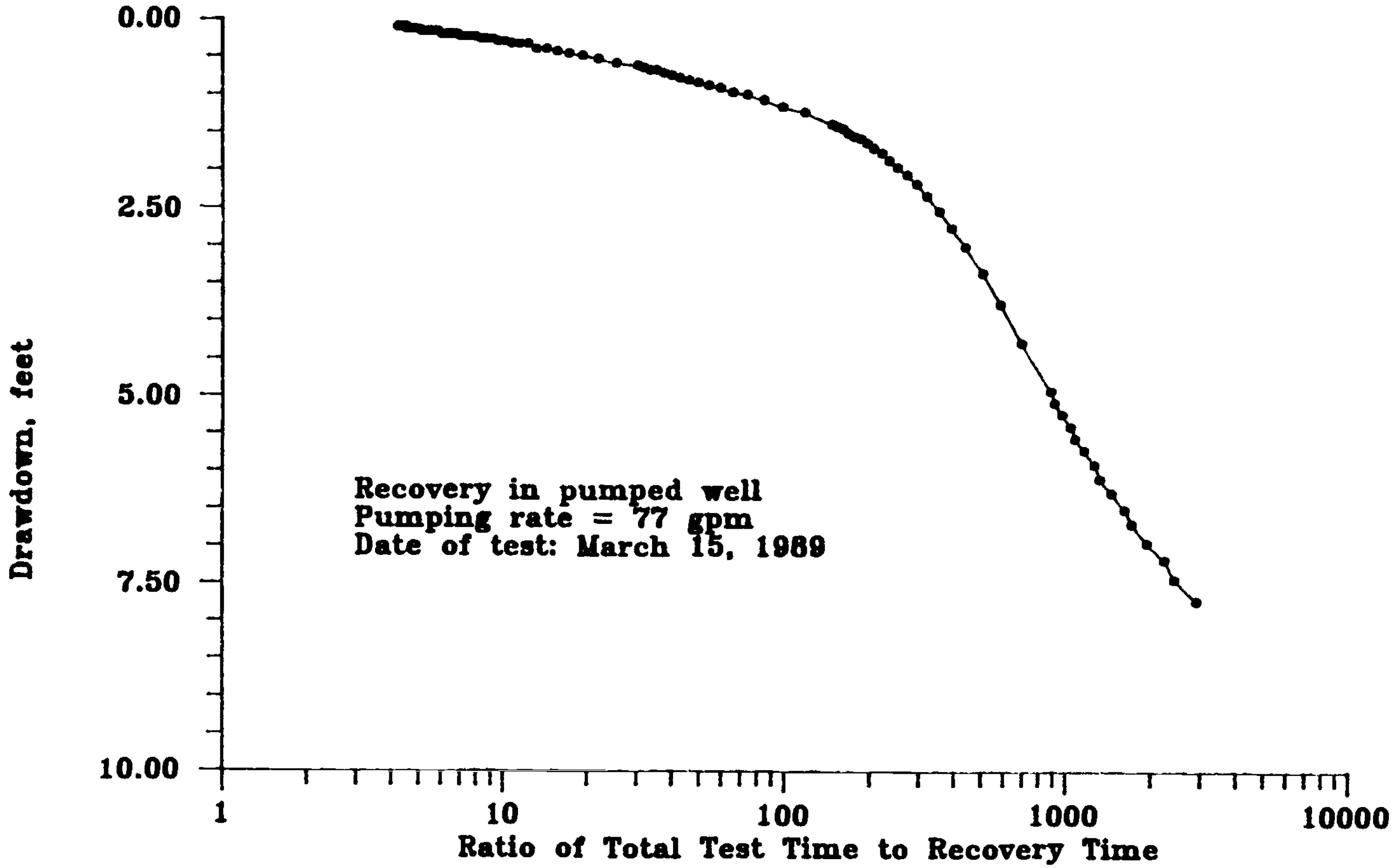

Figure A.2b. OSU North Well Pumping Test Results

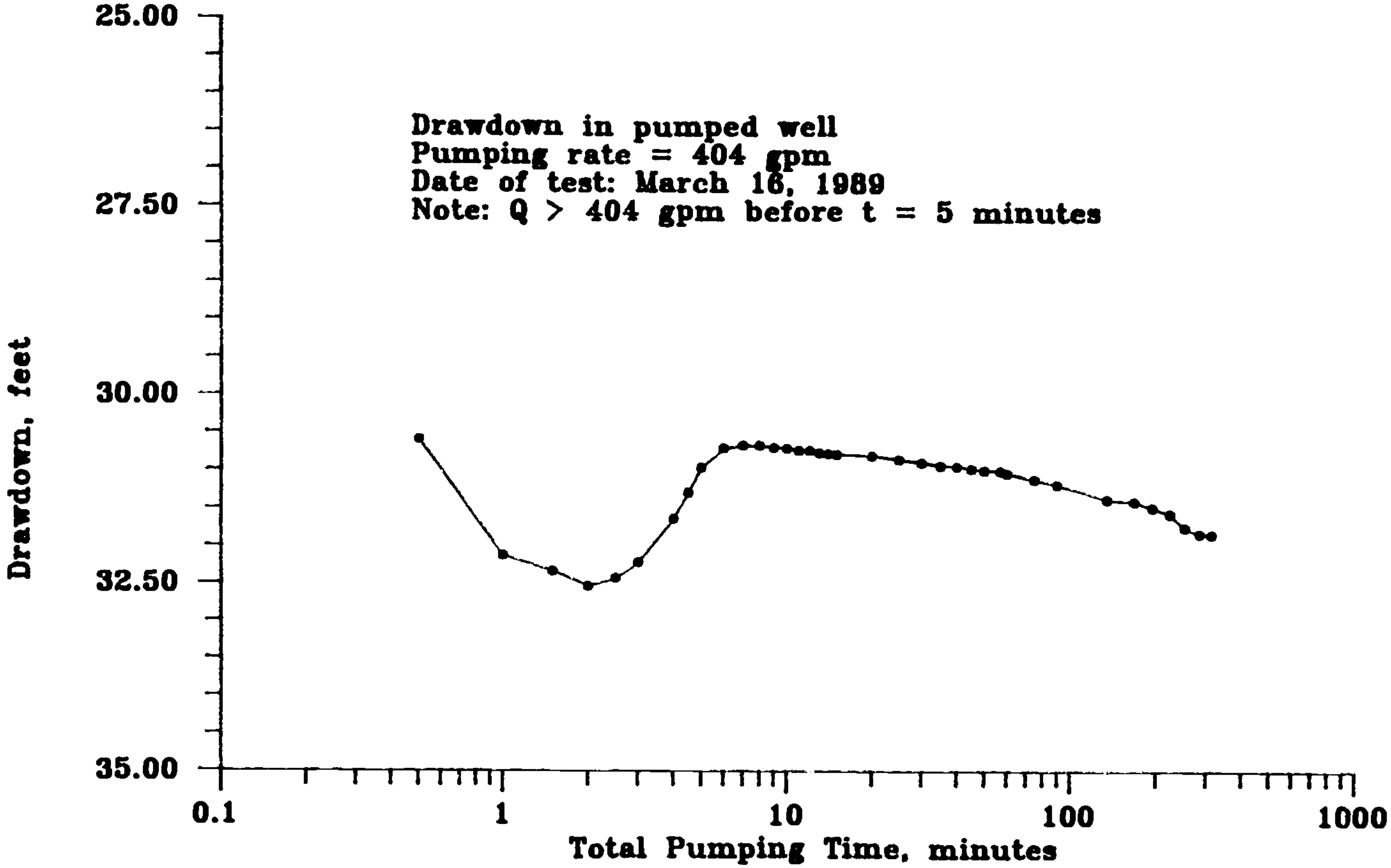

Figure A.3a. OSU East Well Pumping Test Results

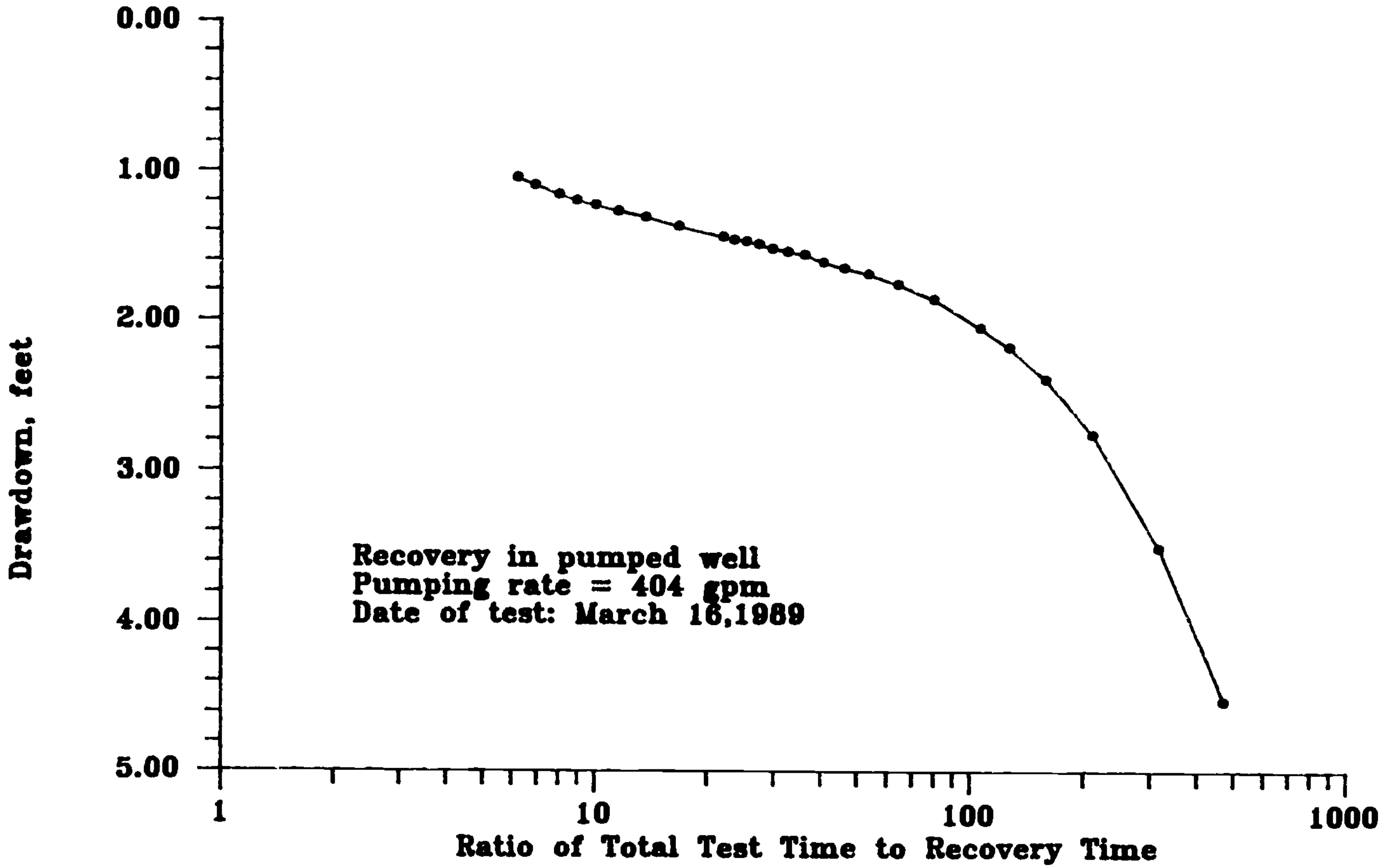

Figure A.3b. OSU East Well Pumping Test Results

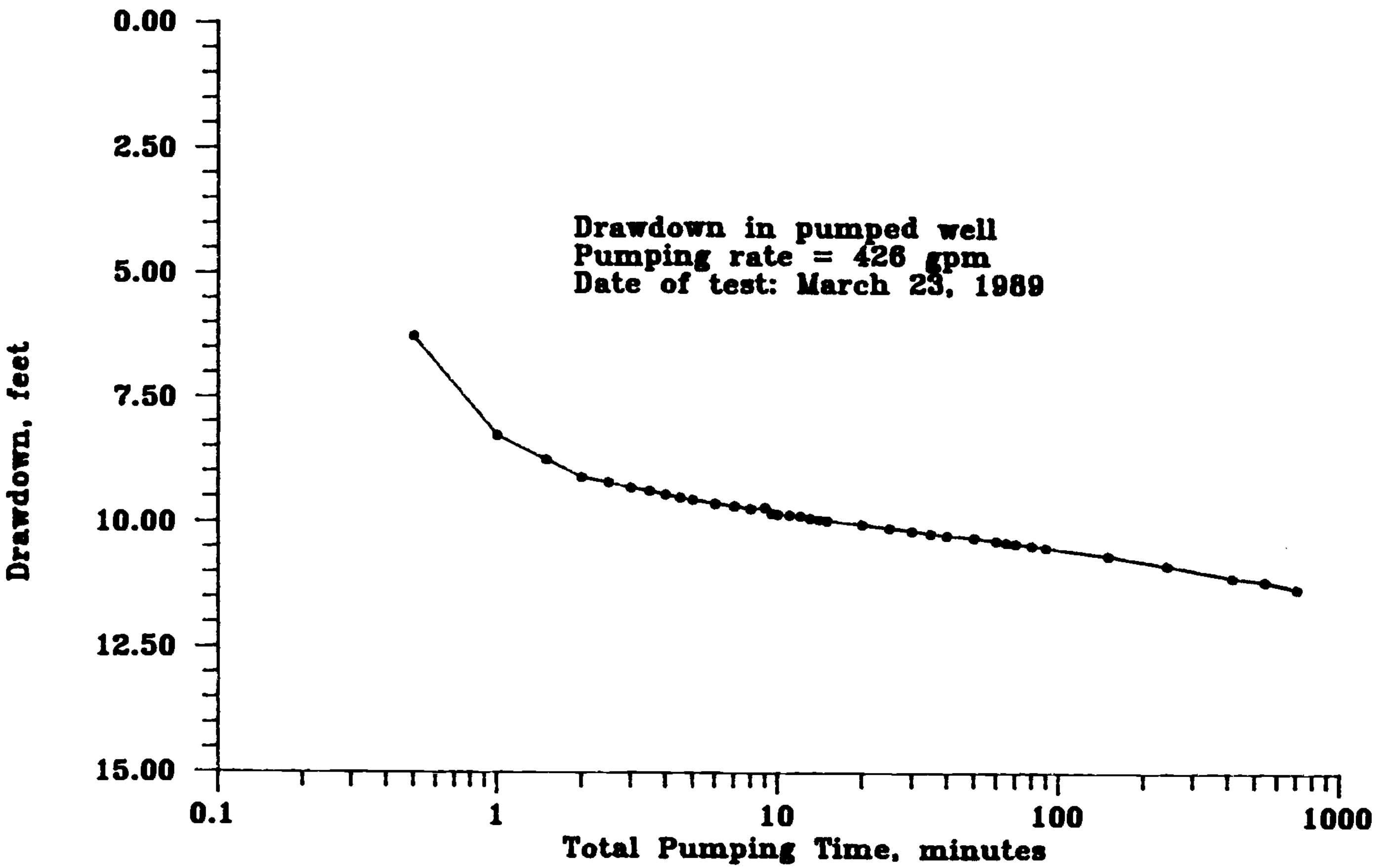

Figure A.4a. Okuda Well Pumping Test Results

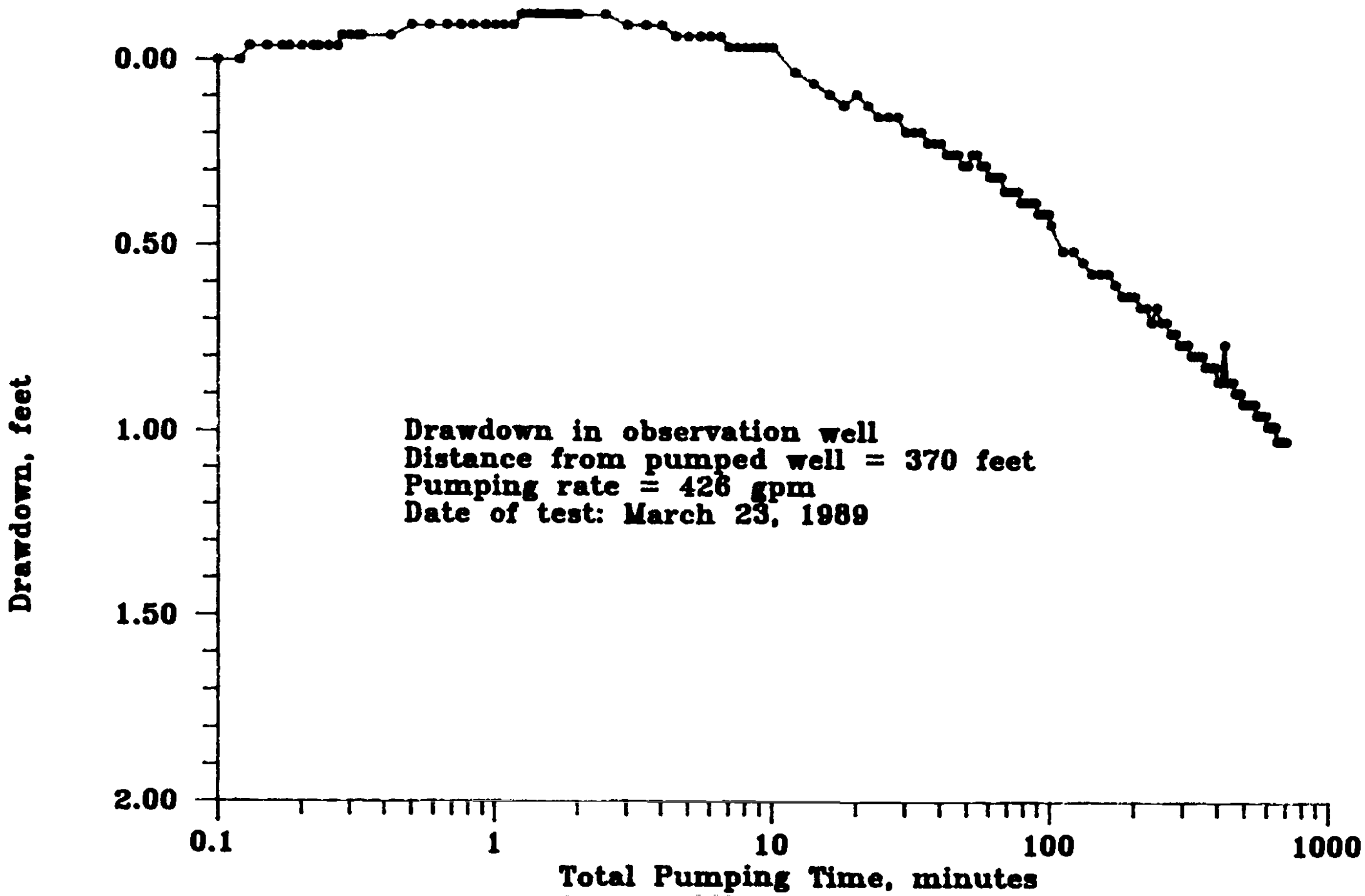

Figure A.4b. Okuda Well Pumping Test Results

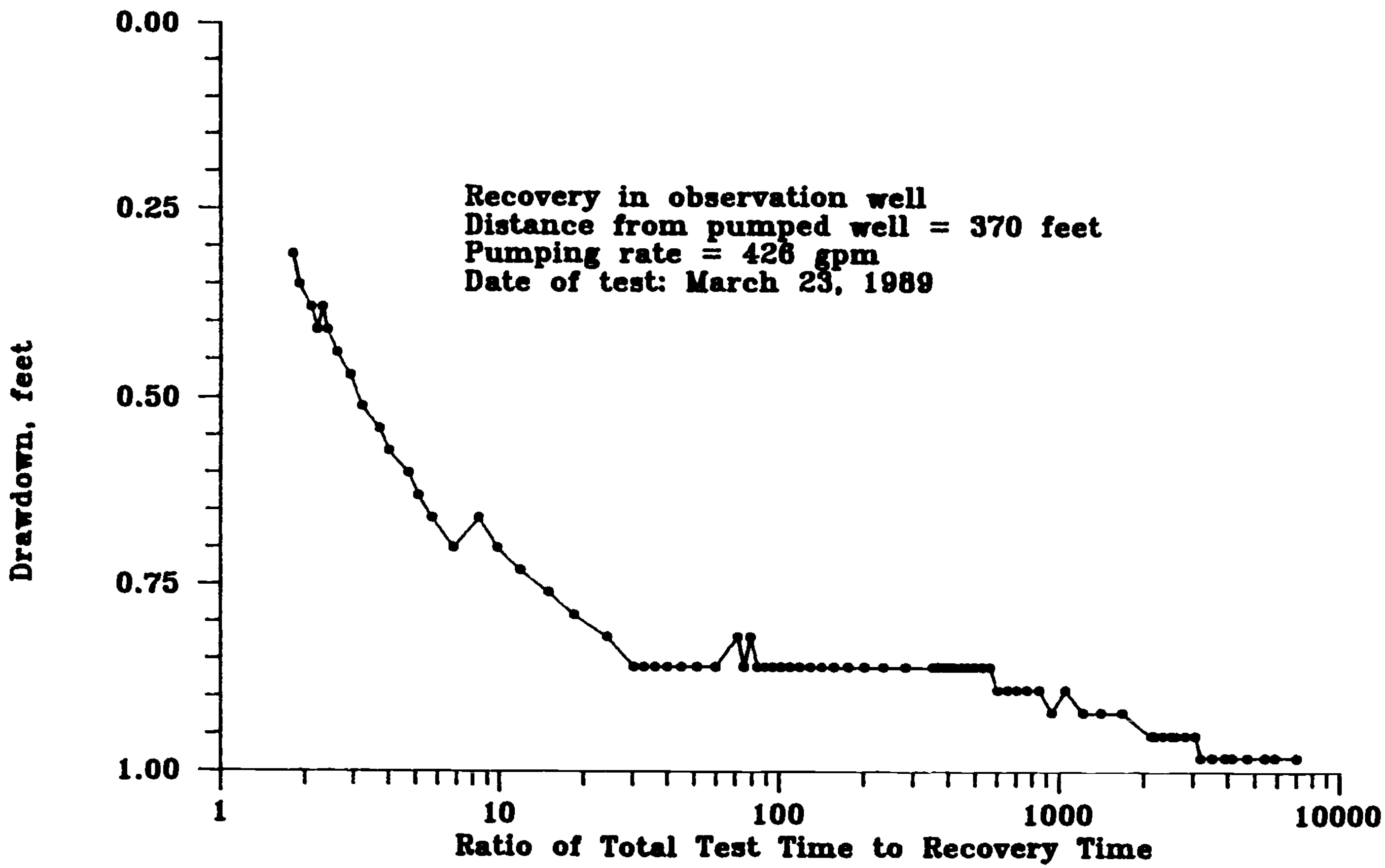

Figure A.4c. Okuda Well Pumping Test Results

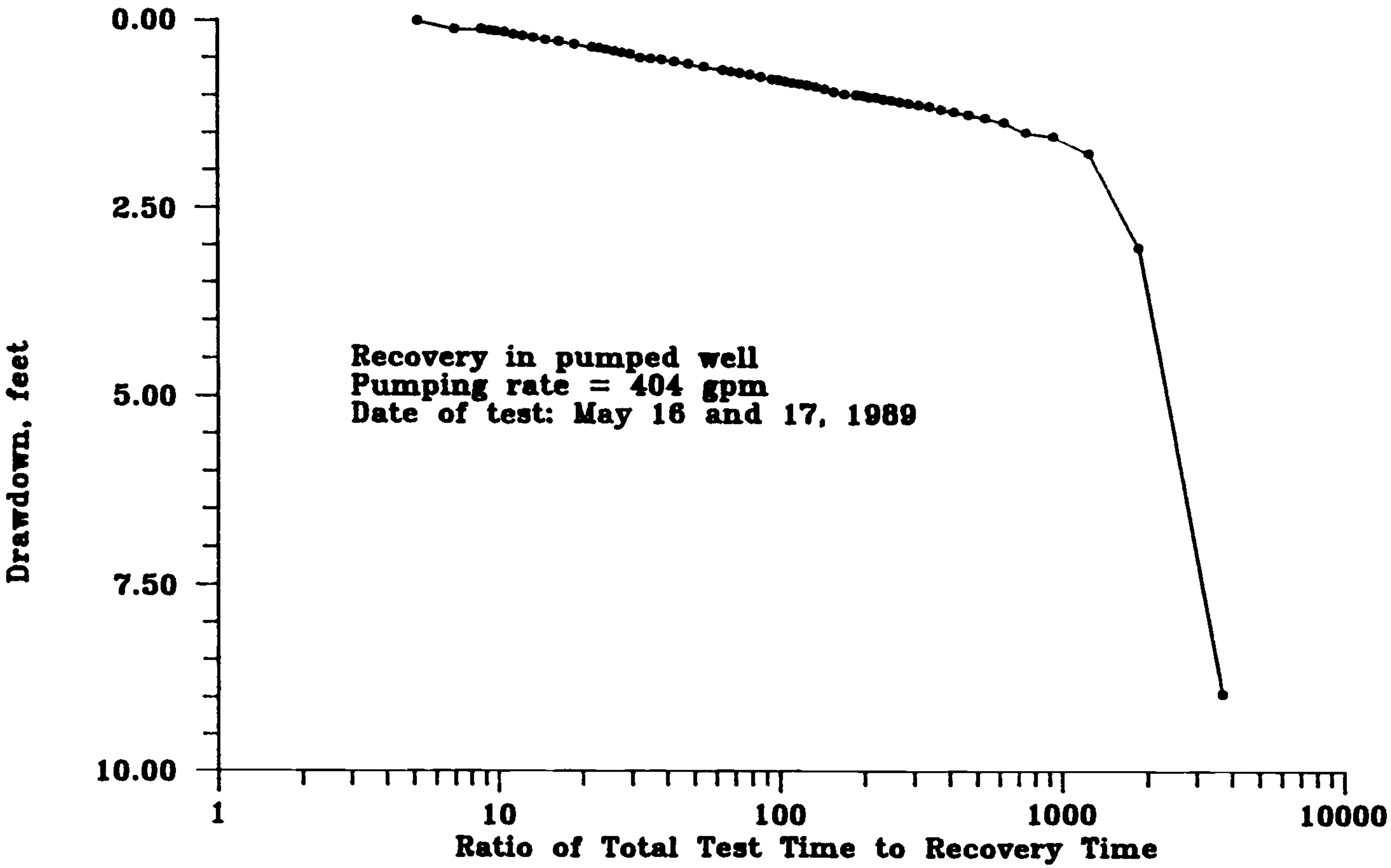

Figure A.5a. LDS Well Pumping Test Results

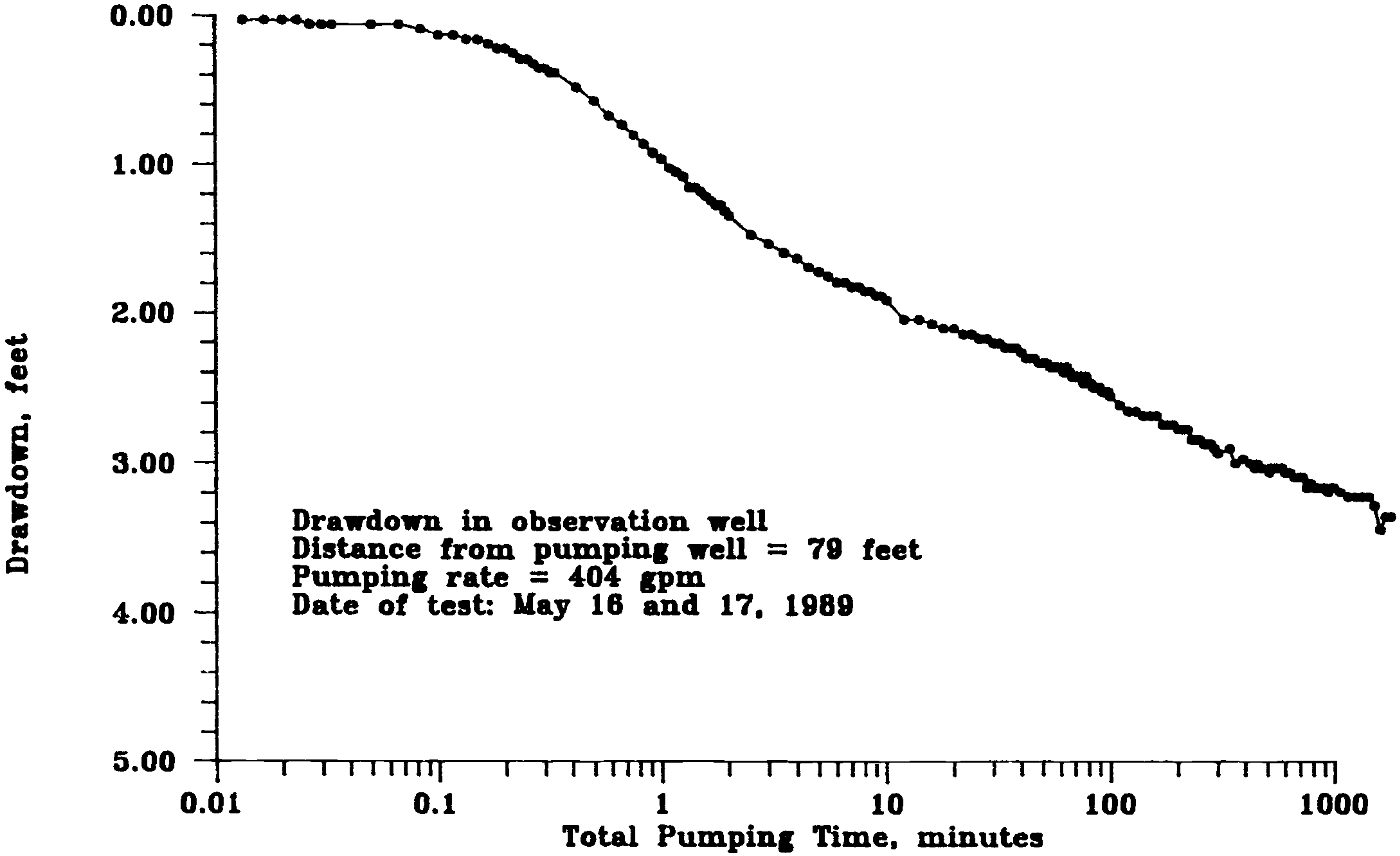

Figure A.5b. LDS Well Pumping Test Results

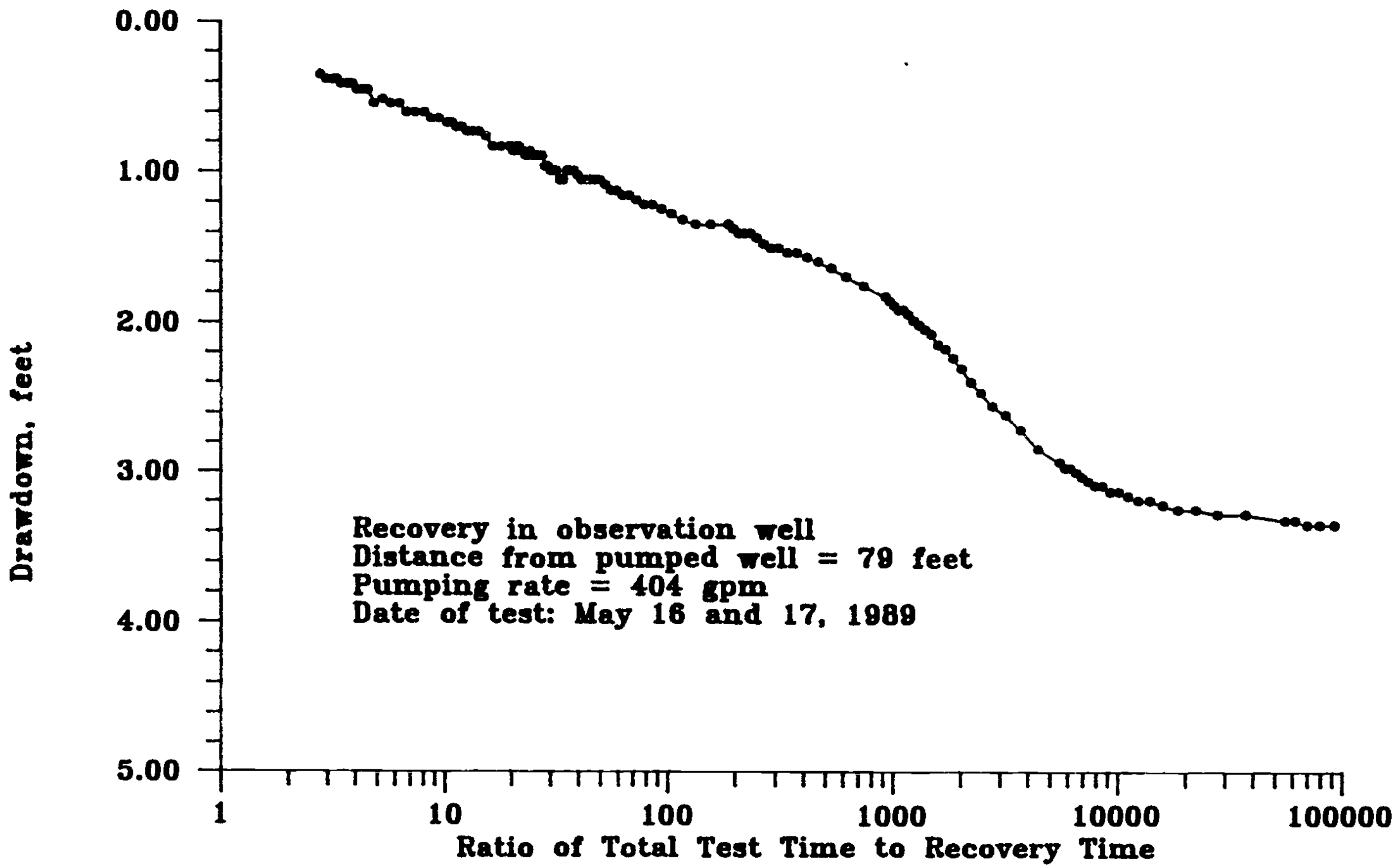

Figure A.5c. LDS Well Pumping Test Results

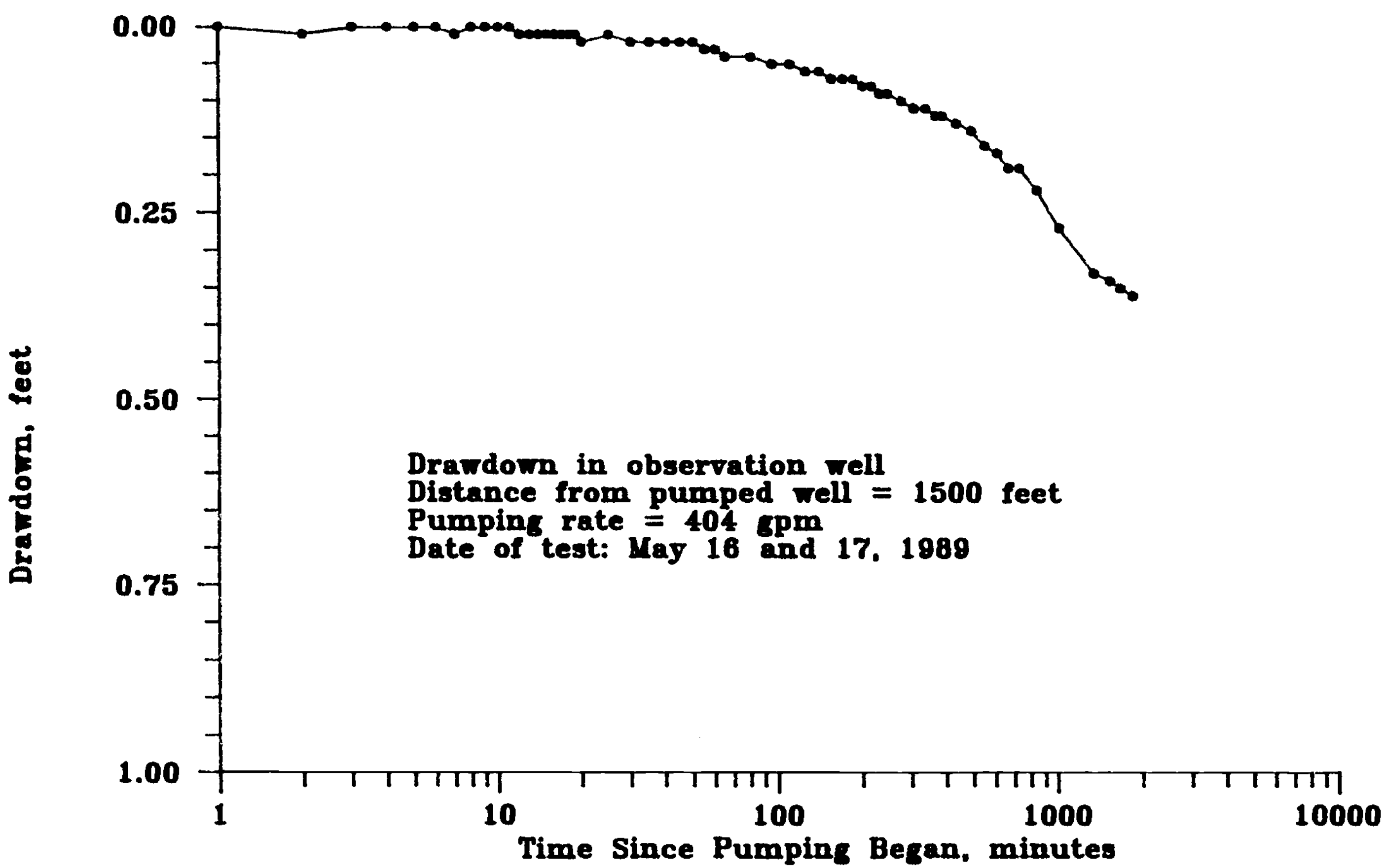

Figure A.5d. LDS Well Pumping Test Results

Table A.1. Aquifer Thickness Data.

Well Location				
Township	Range	Section	Aquifer Thickness (feet)	Remarks
17	47	32ad	14	
		32cd	4	
		32dd	7	Average for 5 wells
		33bb	10	Average for 2 wells
		33bd	8	Average for 2 wells
		33ca	8	
		33cb	12	Average for 2 wells
		33cc	5	
		33cd	9	
18	46	1da	3	
		11bb	16	Silty sand
		12cb	15	
		12cd	13	
		12dc	15	
		13ac	1	
		13cd	2	
		13dc	6	
		13dd	5	5 ft sand
		14bb	16	
		19	11	
		19ac	6	
		19cc	--	Aquifer not present
		20bc	11	
		21aa	50	Fine sand
		21ca	4	
		23ab	14	
		23dd	5	
		24ac	20	18 ft sand
		24d	21	
		27ab	8	8 ft sand
		27ad	10	
		29dd	34	
18	47	2cc	33	
		2cd	40	
		3db	24	

Notes.
1. Aquifer thickness refers to the sand and gravel aquifer, except as noted in "Remarks".
2. Wells were not field-located. Quarter-quarter sections are from the well logs.

Abbreviations.
5 ft sand = five feet of sand overlying the sand and gravel aquifer.

Table A.1. Aquifer Thickness Data (continued).

Well Location				
Township	Range	Section	Aquifer Thickness (feet)	Remarks
18	47	4bb	2	
		4bd	14	6 ft sand
		4da	9	
		5ac	18	Average for 2 wells
		5ad	9	4 ft sand
		5bb	29	
		5da	18	Average for 2 wells
		5db	16	Average for 2 wells
		5dc	12	4 ft sand
		5dd	7	6 ft sand
		6aa	3	
		6ab	2	
		6ba	6	Average for 2 wells
		6bb	7	
		6bd	4	
		6ca	6	Average for 2 wells
		6ca	2	
		6da	--	Aquifer not present
		7ad	43	
		7cd	11	Average for 2 wells
		7da	10	9 ft sand
		8ac	21	
		8ad	23	
		8bb	8	
		8cc	4	
		8dd	12	
		9aa	14	4 ft sand
		9ad	5	3 ft sand
		9ba	15	
		9bc	24	Average for 2 wells
		10ab	14	
		10ac	16	
		10ba	9	
		10bd	13	
		10cc	18	
		10cd	10	6 ft sand
		10dc	13	

Notes.
1. Aquifer thickness refers to the sand and gravel aquifer, except as noted in the "Remarks".
2. Wells were not field-located. Quarter-quarter sections are from the well logs.

Abbreviations.
6 ft sand = six feet of sand overlying the sand and gravel aquifer.

Table A.1. Aquifer Thickness Data (continued).

Well Location				
Township	Range	Section	Aquifer Thickness (feet)	Remarks
18	47	11db	10	
		15ac	2	
		15ad	3	Aquifer is sand
		15ad	13	
		15bc	4	4 ft sand
		16d	32	18 ft sand
		16dd	23	Aquifer is fine sand
		17aa	19	Aquifer is sand
		17bb	36	19 ft sand
		18bb	13	
		18cc	14	
		19	21	Aquifer is sand
		21cd	11	7 ft sand
		29bc	>16	
		30aa	>22	
		30ac	7	Aquifer is sand
19	46	25aa	20	16 ft sand
		25dd	7	18 ft sand
19	47	7bb	16	
		7bb	27	Aquifer is sand
		16db	15	
		17bd	21	
		18ad	11	12 ft sand
		19cc	--	Aquifer not present
		19da	12	
		19dd	8	6 ft sand
		20cc	8	8 ft sand
		20cd	17	
		20dc	10	
		20dd	15	3 ft sand
		21cc	28	
		24cb	21	
		25bd	--	Aquifer not present

Notes.
1. Aquifer thickness refers to the sand and gravel aquifer, except as noted in "Remarks".
2. Wells were not field-located. Quarter-quarter sections are from the well logs.

Abbreviations.
4 ft sand = four feet of sand overlying the sand and gravel aquifer.