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BLACK & VEATCH

5728 LBJ Freeway, Suite 300, Dallas, Texas 75240, (214) 770-1500, Fax: (214) 770-1549

February 28, 1995

Mr. Ronald Crossland, Chief
Technical Section (6H-CX)
RCRA Enforcement Branch
U.S. EPA Region 6
1445 Ross Avenue, Suite 1200
Dallas, Texas 75202-27733

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VIII

Re: Revisions to Report on the Effectiveness of the Groundwater Recovery Well System in the Upper Flow Zone
Sparton Technology, Inc.
Coors Road Facility
Albuquerque, New Mexico

Dear Mr. Crossland:

Submitted here is the revision to the draft Report on the Effectiveness of the Groundwater Recovery Well System in the Upper Flow Zone (Effectiveness Report) originally submitted to U.S. EPA on July 29, 1992. The Effectiveness Report has been revised in response to U.S. EPA comments received by Sparton Technology on December 20, 1994 and subsequent conversation with Mr. Vincent Malott. This revision is being submitted by Black and Veatch on behalf of Sparton Technology.

Only revised pages are being submitted with revised information shown in shading and the revision date in the page footer. These pages should be substituted in appropriate places in the existing Effectiveness Report. Supplemental information is also provided for inclusion into Appendix 4. A discussion of specific revisions relative to their corresponding EPA comments is detailed in this letter.

1. General. It is our opinion that the Interim Measure initiated in December 1988 is achieving the requirements specified in the Section IV.A. 1.(a)(ii) of the Administrative Order on Consent. This opinion is supported by the capture zone calculations included in this revision as more fully discussed under items 5 and 6 as well as by the results of sampling and analysis completed to date.

2. Groundwater contamination in upper flow zone. Paragraph 2 on page 8 has been deleted as requested.

3. In situ permeability. There is an apparent misunderstanding of the Hvorslev methodology. Hvorslev's report was provided in its entirety in Appendix 3 of the Effectiveness Report. With respect to his report, the use of uniform as shown in figure

2/28/95

A Report Prepared for:

Sparton Technology, Inc.
4901 Rockaway Boulevard, SE
Rio Rancho, New Mexico

REPORT ON THE EFFECTIVENESS OF THE GROUNDWATER RECOVERY WELL SYSTEM IN THE UPPER FLOW ZONE

Sparton Technology, Inc.
Coors Road Facility
Albuquerque, New Mexico

Prepared by HDR Engineering, Inc.
12700 Hillcrest Avenue, Suite 125
Dallas, Texas 75230-2096
August, 1992

Revised by Black and Veatch
5728 LBJ Freeway, Suite 300
Dallas, Texas 75240
February 1995

Pierce L. Chandler, Jr., P.E.
Senior Project Manager

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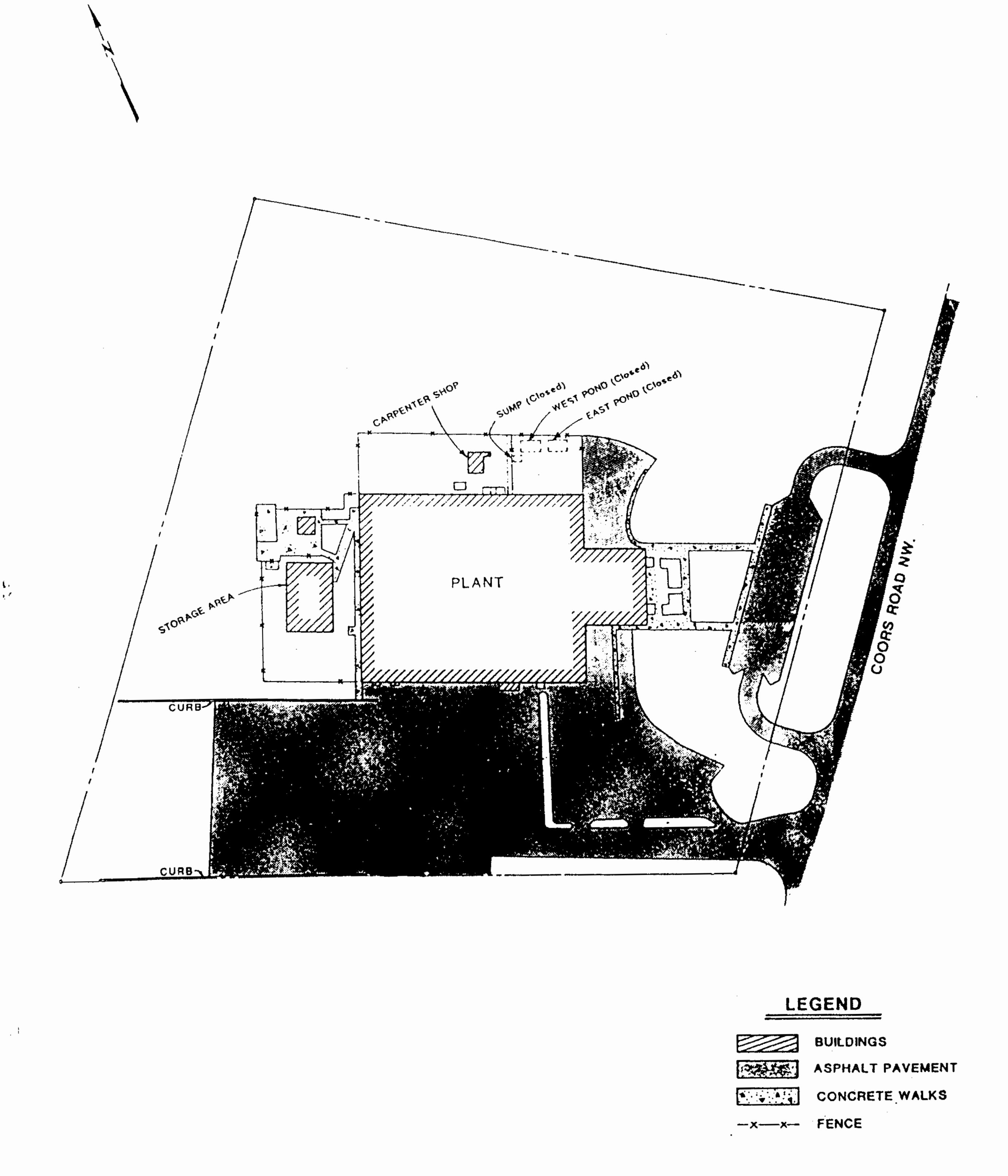
LIST OF FIGURES

I INTRODUCTION

This Report on the Effectiveness of the Groundwater Recovery Well System in the Upper Flow Zone (UFZ) is being submitted pursuant to an Administrative Order on Consent dated October 1, 1988, for the Sparton Technology, Inc. (Sparton) facility located on Coors Road in Albuquerque, New Mexico. In accordance with Section IV.A.1.(a) of the Consent Order, a groundwater recovery well network installed in the upper flow zone and a treatment/disposition system was implemented in December 1988. The purpose of this Interim Measure was to mitigate further off-site migration of contaminants in the upper flow zone. This report presents the results of an evaluation of the effectiveness of that recovery system pursuant to the requirements of Section IV.A.1.(a)ii) of the Consent Order. As required, this report is being furnished within 30 days of receipt of notification by EPA that the Final RCRA Facility Investigation (RFI) report has been approved. The EPA correspondence approving the RFI was dated July 1, 1992, and received by Sparton on July 8, 1992.

As described in the Final RFI report, the pond and sump area located on the north side of the main building is believed to be the source of soil and groundwater contamination at the site. A site layout diagram is shown on Figure 1. Although the historic content of the ponds or sump is not known, the predominant constituents can be inferred from groundwater analyses. It appears that the primary hazardous constituents include trichloroethylene (TCE) and 1,1,1-trichloroethane (TCA) with lesser amounts of methylene chloride (MeCl), 1,1-dichloroethylene (DCE), acetone, and various metals including chromium and lead.



HDR
HDR ENGINEERING, INC.
DALLAS, TEXAS

Site Layout Diagram
Sparton Technology, Inc.
Coors Road Facility
Albuquerque, New Mexico

Date MC
8/92

Figure 1

OGC-004442

II GROUNDWATER LEVELS AND FLOW DIRECTION

IN THE UPPER FLOW ZONE

To establish groundwater levels at the site, bi-weekly water level measurements have been taken at the site since early 1989. A summary of the bi-weekly readings taken during the past year (1991-1992) is included in Appendix 1. Maximum water levels occur to the north of the Sparton facility. The highest groundwater conditions, shown on Figure 2 (Figure 25 from Final RFI Report), occur at the end of the irrigation (recharge) season in November. The lowest groundwater conditions, shown in Figure 3 (Figure 26 from Final RFI Report), occur prior to the start of the irrigation season in April.

As shown on Figures 2 and 3, groundwater gradients in the upper flow zone (UFZ) are generally to the southwest across the Sparton site. Between the facility and Irving Boulevard, the gradients are generally to the west and northwest. Beyond Irving Boulevard, the gradients begin a gradual arc back to the established southwestward regional gradient.

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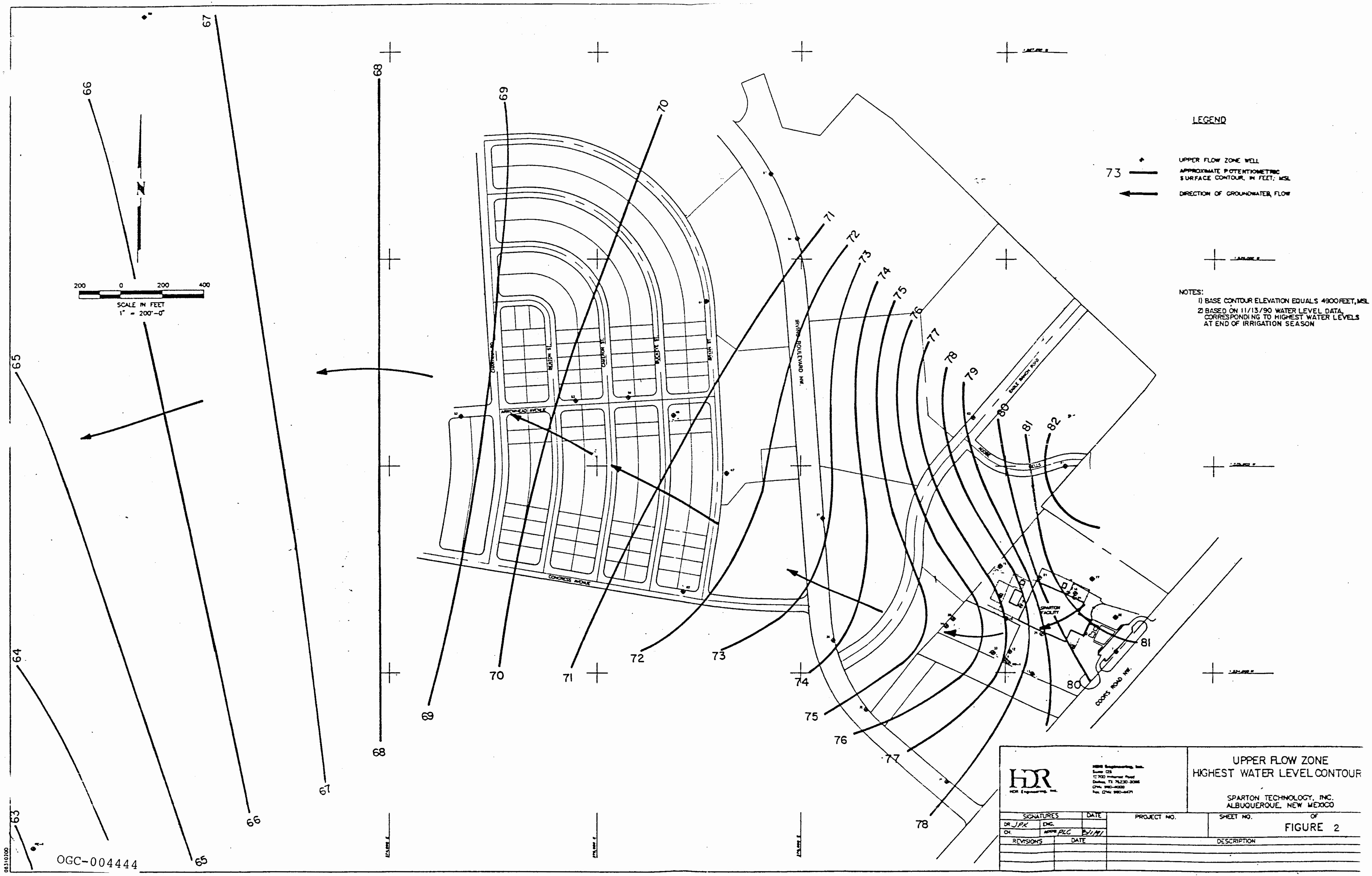

LEGEND
UPPER FLOW ZONE WELL
73
APPROXIMATE POTENTIOMETRIC SURFACE CONTOUR, IN FEET, MSL
DIRECTION OF GROUNDWATER FLOW
NOTES:
1) BASE CONTOUR ELEVATION EQUALS 4900 FEET, MSL
2) BASED ON 11/13/90 WATER LEVEL DATA, CORRESPONDING TO HIGHEST WATER LEVELS AT END OF IRRIGATION SEASON
SCALE IN FEET
1" = 200'-0"
200
0
200
400
63
64
65
66
67
68
69
70
71
72
73
74
75
76
77
78
79
80
81
82
CONGRESS AVENUE
ARROWHEAD AVENUE
IRVING BOULEVARD NW.
COORS ROAD NW.
SPARTON FACILITY
UPPER FLOW ZONE
HIGHEST WATER LEVEL CONTOUR
SPARTON TECHNOLOGY, INC.
ALBUQUERQUE, NEW MEXICO
HDR
SIGNATURES
DATE
DR JPK
CH.
APPR PLC
PROJECT NO.
SHEET NO.
OF
FIGURE 2
REVISIONS
DATE
DESCRIPTION
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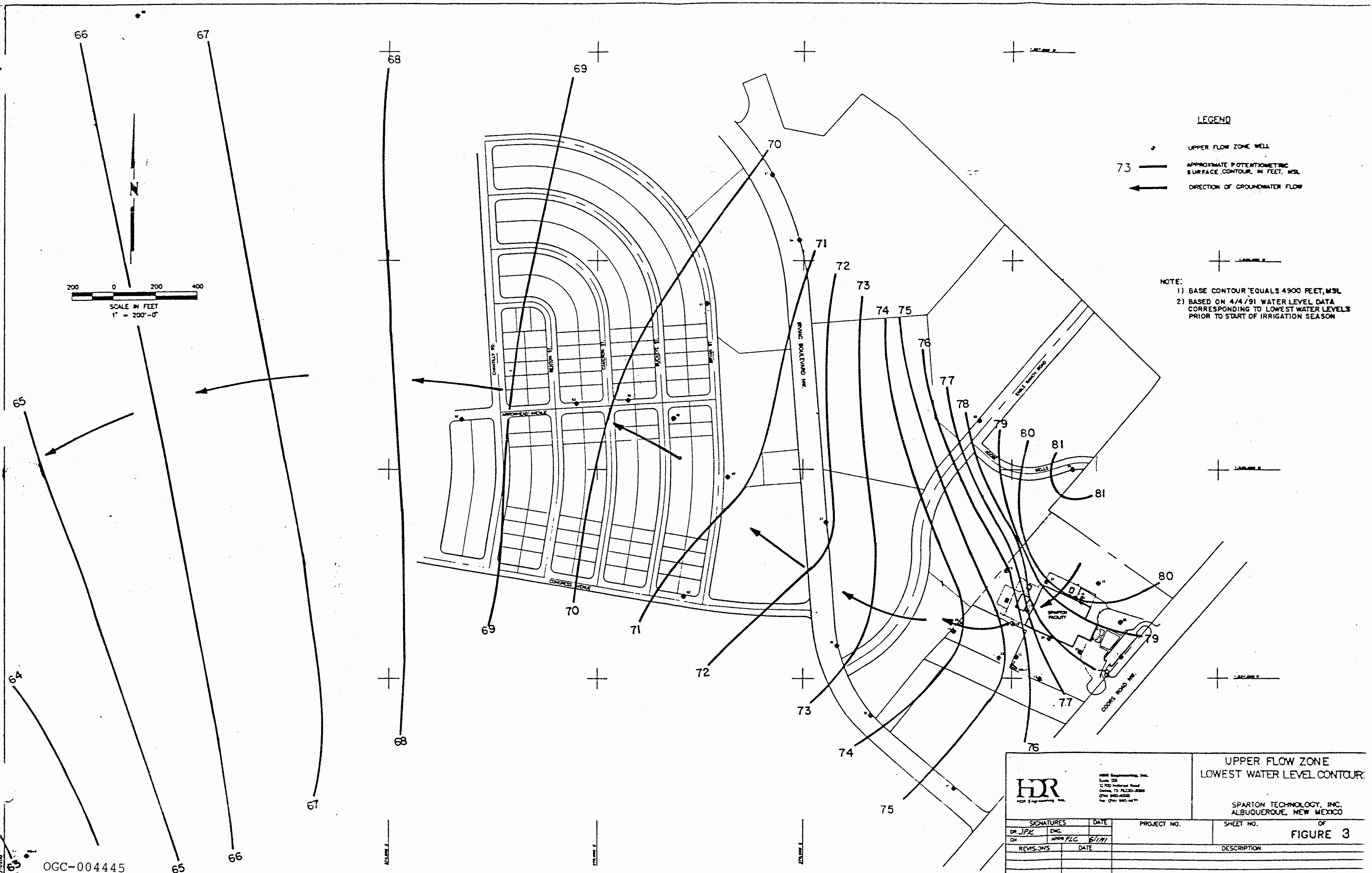
LEGEND
UPPER FLOW ZONE WELL
73 APPROXIMATE POTENTIOMETRIC SURFACE CONTOUR, IN FEET, MSL
DIRECTION OF GROUNDWATER FLOW
NOTE:
1) BASE CONTOUR EQUALS 4900 FEET, MSL
2) BASED ON 4/4/91 WATER LEVEL DATA CORRESPONDING TO LOWEST WATER LEVELS PRIOR TO START OF IRRIGATION SEASON
SCALE IN FEET
1" = 200'-0"
N
CHANTILLY RD.
ARROWHEAD AVENUE
CONGRESS AVENUE
IRVING BOULEVARD NW.
COORS ROAD NW.
SPARTON FACILITY
HDR
HDR Engineering, Inc.
UPPER FLOW ZONE
LOWEST WATER LEVEL CONTOURS
SPARTON TECHNOLOGY, INC.
ALBUQUERQUE, NEW MEXICO
SIGNATURES
DATE
DR. JPK
ENG.
CH
APPR PLC
6/1/91
PROJECT NO.
SHEET NO.
OF
FIGURE 3
REVISIONS
DATE
DESCRIPTION

III DESCRIPTION OF GROUNDWATER CONTAMINATION IN UPPER FLOW ZONE

As described in the final RFI report, routine quarterly groundwater analyses were instituted in 1985 under a state-approved program for a number of on-site monitoring wells. The analysis of groundwater from wells in the upper flow zone encountered primarily TCE and TCA with lesser amounts of acetone, DCE, MeCl, and various metals. TCE is the predominant contaminant with respect to concentration as well as areal and vertical extent. Furthermore, there is a much more extensive historical database on TCE analyses. As a result, this report will focus on the fate of TCE in the groundwater in the upper flow zone.

The general areal configuration of the TCE contaminant plume has been determined by contouring TCE concentration data from 22 upper flow zone (UFZ) wells. The TCE plume configuration as of June 1991 is graphically shown on Figure 4. The June 1991 TCE data as well as the previous TCE concentrations and sampling dates are tabulated on Figure 4 (Figure 55 from Final RFI Report). The less than 5 micrograms per liter (mg/l) isopleth or contour represents the detection limit of the perimeter of the plume. Based upon this boundary, the longitudinal length of the plume in June 1991 is approximately 2100 feet northwest from the facility's western property line. The transverse width of the plume is approximately 1400 feet.

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LEGEND

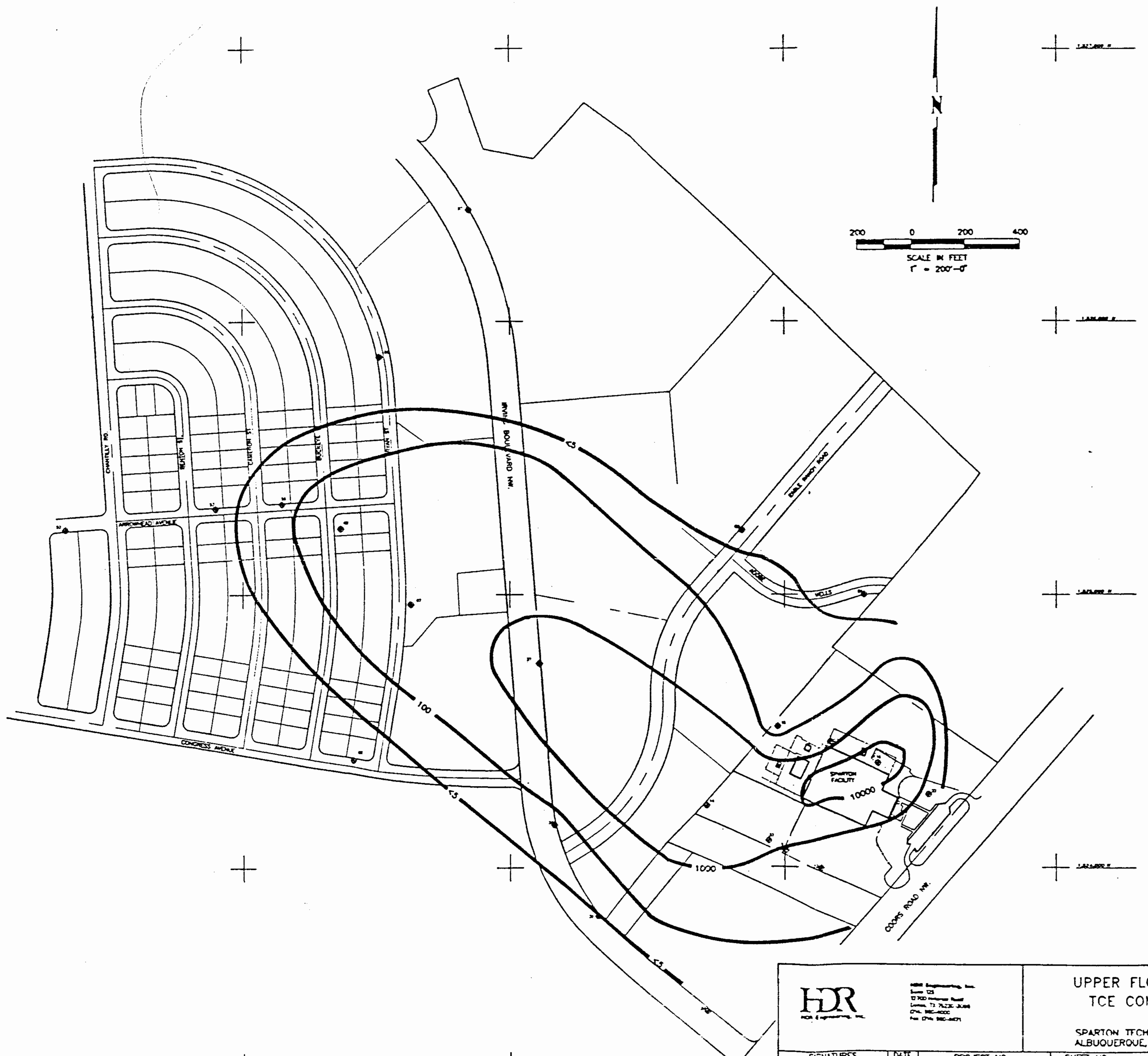
—— 100 —— TCE CONCENTRATION CONTOUR (...)

• UPPER FLOW ZONE WELL

MONITOR WELL NUMBER	SCREEN INTERVAL (FT., MSL)	JUNE, 1991 TCE CONCENTRATION (ug/l)	PREVIOUS TCE CONCENTRATION AND SAMPLING DATE (ug/l)	DATE
9	4981.61–4976.61	1400	* 1900	3rd & 4th Quarter '90
13	4983.25–4973.25	330	* 630	FEB & MAR 1989
14	4980.41–4970.41	1100	* 2400	FEB & MAR 1989
15	4967.49–4977.49	91	* 210	FEB & MAR-1989
16	4979.5–4974.88	17000	* 17500	3rd & 4th Quarter '90
21	4983.86–4978.86	500	* 760	3rd & 4th Quarter '90
22	4976.06–4971.06	110	* 111.5	3rd & 4th Quarter '90
33	4981.29–4971.29	7300	* 7250	FEB & MAR 1989
34	4977.99–4967.99	ND < 5	* ND < 5	AUG 1989
35	4979.3–4969.30	ND < 5	* ND < 5	AUG 1989
36	4977.06–4967.06	22	* 9.45	AUG 1989
37	4976.86–4966.86	2000	* 1450	AUG 1989
47	4975.83–4960.83	120	* 275	JAN 1990
48	4976.31–4961.31	410	* 1015	AUG & SEPT 1990
51	4983.86–4973.86	ND < 5	* 6.45	APR & MAY 1990
52	4975.01–4959.81	ND < 5	* ND < 1	JUN 1990
53	4974.44–4960.24	ND < 5	ND < 1	JUN 1990
57	4977.54–4962.54	ND < 5	ND < 1	AUG 1990
58	4974.89–4959.89	29	* 22	OCT 1990
61	4975.96–4960.96	ND < 5	* ND < 5	OCT 1990
62	4980.00–4965.00	ND < 5	* 2.2	OCT 1990
63	4982.74–4967.74	ND < 5	* ND < 5	OCT 1990

* TWO-SAMPLE AVERAGE

Note: ND indicates non-detection of TCE at analytical limit indicated



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UPPER FLOW ZONE
TCE CONTOURS

SPARTON TECHNOLOGY, INC.
ALBUQUERQUE, NEW MEXICO

SIGNATURES		DATE	PROJECT NO.	SHEET NO.	OF
DR JPK	ENG.			FIGURE 4	
CH	APPR PLC	5/1/91			

REVISIONS	DATE	DESCRIPTION

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TCE concentration levels in groundwater samples taken from upper flow zone (UFZ) wells in June 1991 varied from 17,000 µg/l in MW-16 to non-detection (less than 5 µg/l) in several wells. The historic maximum concentration detected in the on-site groundwater is 37,000 µg/l in MW-16.

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IV GROUNDWATER RECOVERY WELL NETWORK

IN THE UPPER FLOW ZONE

A. General

Pursuant to the requirements of the Consent Order, a groundwater recovery well network was installed in the upper flow zone as an Interim Measure. The purpose of this Interim Measure was to mitigate the spread of the shallow contaminant plume off-site. In order to maximize contaminant removal, the recovery well network utilized a number of on-site wells located in the more contaminated portions of the contaminant plume. The recovery network was designed and constructed according to the provision of the Interim Measures Workplan approved by EPA on March 1, 1989. The network became operational in December 1988.

B. Description of Recovery Well Network in Upper Flow Zone

The network is comprised of eight wells (PW-1, MW-18, MW-23, MW-24, MW-25, MW-26, MW-27, and MW-28) constructed over a four-year period and installed in the upper flow zone of the site at the locations shown on Figure 5. The wells are set in the upper flow zone (UFZ) with screened interval depths ranging from 60 to 78 feet below the existing ground surface. Recovery wells PW-1, MW-18, MW-25, MW-27, and MW-28 are screened across both the highest and lowest groundwater levels. Two of the recovery wells, MW-23 and MW-26, are screened below the lowest groundwater levels. Recovery well MW-24 is screened below the highest groundwater level and across the lowest groundwater level. Table 1 lists the pertinent construction details for each of the eight wells.

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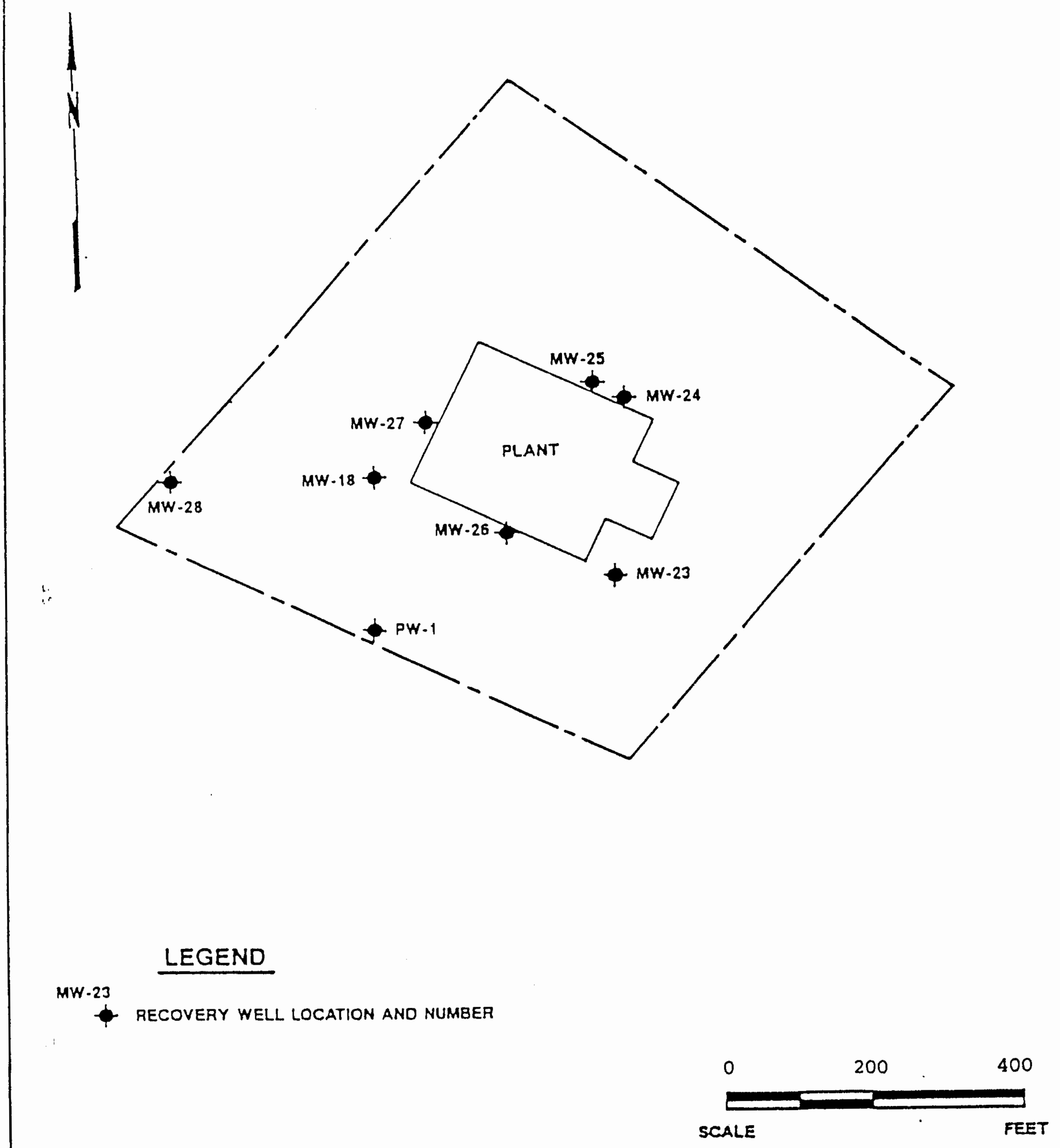

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HDR ENGINEERING, INC.
DALLAS, TEXAS

Recovery Well Location Plan
Sparton Technology, Inc.
Coors Road Facility
Albuquerque, New Mexico

Date
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Figure 5

Rev. February 1995

OGC-004450

TABLE 1
Recovery Network Well Construction Details

Well No.	Well Diameter (inches)	Well Screen Material	Riser Material	Depth of Screened Interval (feet)	Elevation at top of Screen (ft., MSL)	Construction Date
PW-1	10	PVC[(1)]	PVC	60-70	4984.54	9/84
MW-18	4	PVC	PVC	68-78	4977.58	5/86
MW-23	2	SS[(2)]	PVC	72-77	4976.51	8/86
MW-24	2	SS	PVC	68.4-73.4	4980.30	12/86
MW-25	2	SS	PVC	67.7-72.7	4981.30	12/86
MW-26	2	SS	PVC	73-78	4972.71	5/88
MW-27	2	SS	PVC	67-72	4978.50	5/88
MW-28	2	SS	PVC	65-70	4977.69	5/88

(1) Polyvinyl chloride
(2) Stainless Steel

Compressed-air-operated, positive-displacement pumps were installed at or near the bottom of each well. The compressed air is supplied by a central air compressor located in the control building. Air is pumped through piping to the well pumps and pump controllers. Four controllers are provided to control pump operations. Two pumps are controlled by each controller. Each well pump is equipped with a remote well operator to allow independent adjustment of pumping rates for each well. Each well pump discharges through flexible tubing into a common gravity drain or header. Each discharge line is equipped with a two-way sampling valve for sample collection and flow measurement.

The well pumps are operated by air supplied from the air compressor. Timing devices located in the pump controllers are present to regulate the time to fill the pump chamber and the evacuation time. The timers in the controllers initiate pneumatic signals to the remote well operator located at each wellhead via a 1/4-inch air line. Upon receiving a signal, the remote well operator actuates the pump by allowing air to enter the pump chamber, thus forcing the liquid out of the discharge tubing. Another signal to the remote well operator stops the air flow to the pump chamber. The pump chamber is then allowed to refill for another cycle. An air exhaust vent located at the well cap allows air to be vented from the pump chamber as it fills. The pumping rate of the well may be further adjusted with a throttling valve on the remote well operator. The pump operation sequence is visually depicted on Figure 6.

Groundwater extracted simultaneously at each well location is piped to an air stripper system for treatment and ultimate use in the Sparton Facility. The collection piping system consists of discharge lines encased in secondary piping to provide leak detection and containment. Table 2 describes the pumping flow rate for each recovery well as of late February 1992.

TABLE 2
Current Recovery Well Network Flow Rates

Well No.	Flow Rate (gal/hr)
PW-1	3.7
MW-18	10.0
MW-23	21.3
MW-24	1.0
MW-25	1.8
MW-26	2.0
MW-27	13.4
MW-28	2.9
TOTAL	56.1

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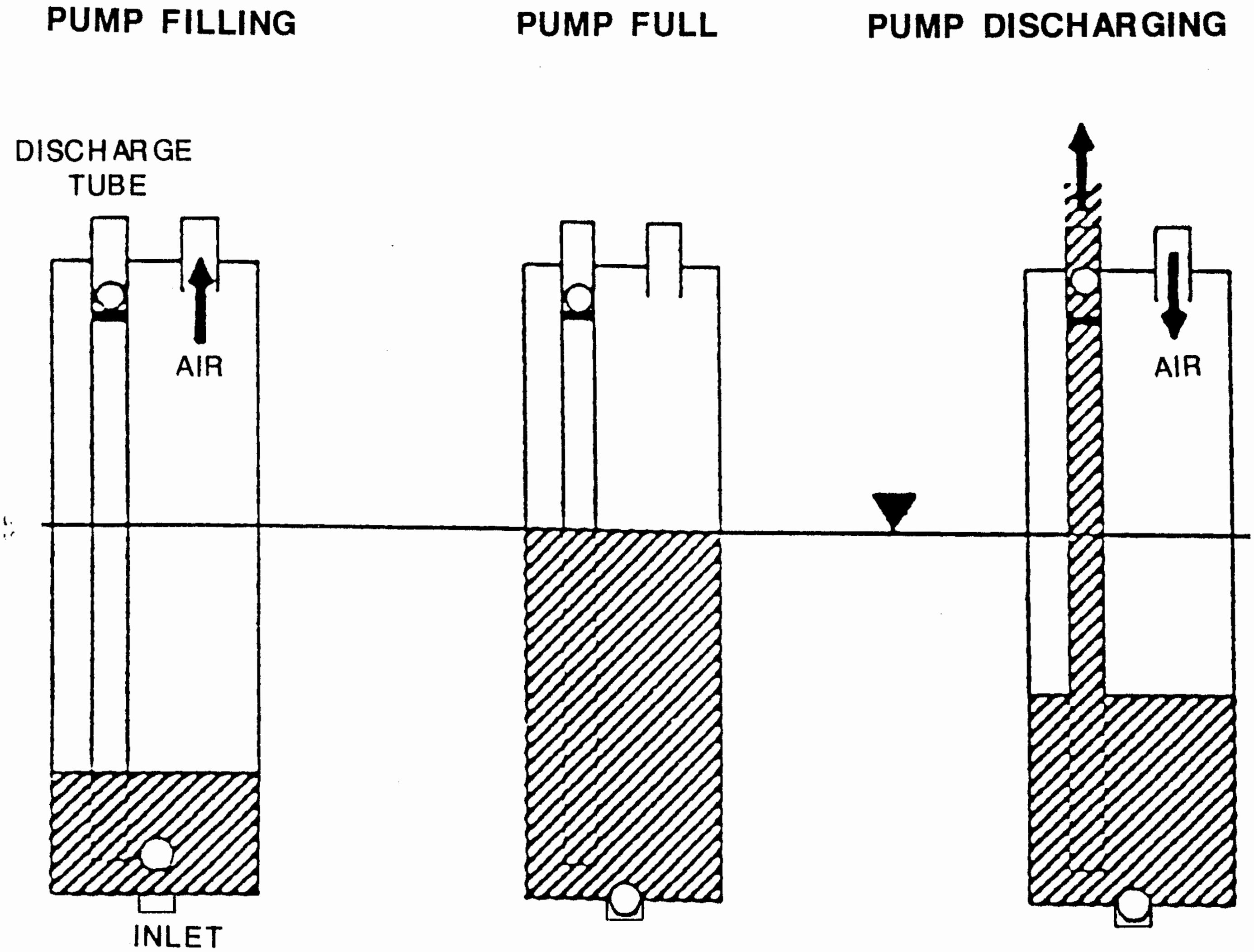

HDR ENGINEERING, INC.
DALLAS, TEXAS

WELL PUMP OPERATION

Date
MKS
8/92

Figure 6

C. Hydraulic Properties of the Upper Flow Zone

1. Aquifer Pumping Tests

Aquifer pumping tests in the upper flow zone wells were performed at the Sparton site on three separate occasions in 1987 and 1988. The tests were performed, analyzed, and reported by Metric Corporation (Metric Corporation 1987, 1988a, 1988b). Copies of these reports are included in Appendix 2.

Pumping tests were performed in all eight recovery wells and MW-16. Monitoring well MW-16 is a two-inch diameter PVC well with a screen depth interval of 68 to 73 feet below the ground surface. The elevation of the top of the screen is at 4979.50 feet. This well is screened below the highest and lowest groundwater levels. The initial aquifer test (1987) was performed in recovery wells MW-18 and MW-24 as well as monitoring well MW-16. The initial aquifer test used constant drawdown techniques on MW-16 and MW-24 and constant discharge techniques on MW-18 over a relatively long duration (49-72 hours). The pumping tests on MW-16 and MW-24 included drawdown observations in both the pumped well and adjacent observation wells (multiple well tests). The pumping test on MW-18 measured drawdown observations in the pumped well only (single well test).

The second aquifer test (1988a) was performed in recovery wells MW-25 and PW-1 using constant discharge techniques over a relatively long duration (69-72 hours). Both pumping tests included observations in both the pumped well and adjacent observation well (multiple well tests).

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The third aquifer test (1988b) was performed in recovery wells MW-23, MW-26, MW-27, and MW-28 using constant discharge techniques over a relatively long duration (70-72 hours). These pumping tests, however, only measured drawdown in the pumped wells (single well tests).

2. <u>In Situ Permeability</u>

Average flow rates during these tests varied from 0.07 to 0.32 gallons/minute. Maximum drawdown distances observed during the tests varied from approximately 2.2 to 5.0 feet. Based upon the results of the pumping tests, Metric Corporation estimated in situ field permeabilities ranging from 3.91×10^{-5} cm/sec to 4.75×10^{-3} cm/sec. These permeability values correspond to soils having a mixture of sand, silt, and clay such as clayey sands and silty sands.

An independent analysis of the pumping test data was performed using Hvorslev's (1951) formulas for determination of in situ soil permeability. A copy of the original Corps of Engineers publication describing Hvorslev's procedures is included in Appendix 3. The recovery portion of the pumping test data, taken after pump shut-down, was used for these analyses. Based upon the subsurface soils and well construction, Hvorslev's Case G, Well Point-Filter in Uniform Soil, was selected as best representing the site conditions. The recovery portion of the pumping test data represents a variable

OGC-004455

head test. As a result, the following formula was utilized in our analysis of in situ soil permeability:

$$K_h = \frac{d^2 \ln\left(\frac{2mL}{D}\right)}{8\ L\ (t_2 - t_1)} \ \ln \frac{H_1}{H_2}\ , \text{ for } \frac{2mL}{D} > 4$$

WHERE : K_h = Horizontal Coefficient of Permeability
K_v = Vertical Coefficient of Permeability
m = Transformation Ratio = $\sqrt{K_h / K_v}$
d = Diameter, standpipe
D = Diameter, intake pipe
L = Length of intake
t = time
H_1 = Drawdown at time t_1
H_2 = Drawdown at time t_2

In our analysis, the ratio of K_h to K_v was approximated as 10. In addition, the diameter of the standpipe was equal to the diameter of the intake pipe in all the tested wells.

The in situ permeability values determined with Hvorslev's equation are summarized in Table 3. Sample calculations for all wells are given in Appendix 4. These results are very similar to permeability values calculated with methods described in NAVFAC DM-7.1, Soil Mechanics (U.S. Department of the Navy, 1982) as shown in Table 3. For comparison, the values previously determined by Metric Corporation (see Appendix 2) are also listed in Table 3. Considering the methods of analysis used and the inherent assumptions involved, the Metric values compared very well with the Hvorslev values.

OGC-004456

TABLE 3
Calculated In Situ Field Permeabilities

Well No.	HDR, Inc. (1) (cm/sec)	HDR, Inc. (2) (cm/sec)	Metric Corporation (cm/sec)
PW-1	3.24×10^{-4}	4.375×10^{-4}	1.00×10^{-3}
MW-16	2.39×10^{-4}	1.603×10^{-4}	4.18×10^{-3}
MW-18	3.46×10^{-4}	4.156×10^{-4}	3.26×10^{-4}
MW-23	2.53×10^{-3}	1.450×10^{-3}	8.54×10^{-4}
MW-24	4.36×10^{-4}	4.071×10^{-4}	4.75×10^{-3}
MW-25	4.50×10^{-4}	1.510×10^{-4}	2.18×10^{-3}
MW-26	3.56×10^{-4}	2.746×10^{-4}	3.91×10^{-5}
MW-27	2.90×10^{-3}	2.009×10^{-3}	9.08×10^{-4}
MW-28	2.91×10^{-5}	2.730×10^{-5}	1.07×10^{-3}

(1) Using Hvorslev's Formula for Case G, Variable Head Test
(2) Using methods described in NAVFAC DM-7.1, Soil Mechanics, case F(2) or F(3) depending on the screen length

3. Radius of Influence

Evaluation of the radius of influence for the nine wells used in the aquifer pump tests utilized Sichardt's method (U.S. Department of the Army, 1971, and Powers, 1981). Excerpts discussing Sichardt's procedures from each of these references are included in Appendix 3. The analysis was based on the permeability values determined with Hvorslev's Formula and a saturated upper flow zone thickness of 10 feet. Estimation of the radius of influence utilized the following formula:

$$r_o = C\,(H - h_w)\,\sqrt{K}$$

WHERE : r_o = Radius of Influence, feet
C = Empirical Relation of K vs. r_o
H = Height of water table (saturated thickness), feet
h_w = Head of water in well, feet
K = Coefficient of Permeability, microns/sec (1 micron = 1×10^{-4} cm)

In the analysis, C was assumed to be 3 for a single well and the term ($H-h_w$) represents well drawdown which was assumed to equal the full saturated thickness of 10 feet. Results of the analysis are summarized on Table 4.

See pg 109
of "Construction dewatering"
appndx 3

Also described in Table 4 are the minimum observed radii of influence for aquifer test locations with multiple well readings. These minimum radii of influence represent the horizontal distance between the pumped well and farthest observation well showing identifiable drawdown effects. Due to the limited number of observation wells, the actual radii of influence may exceed these minimum values.

TABLE 4
Radius of Influence

Well No.	Calculated Radius of Influence, r_o (ft)	Minimum observed Radius of Influence (ft)
PW-1	54	10
MW-16	46	50
MW-18	56	--(1)
MW-23	151	--(1)
MW-24	63	60
MW-25	64	25
MW-26	57	--(1)
MW-27	162	--(1)
MW-28	16	--(1)

(1) Single well tests

The permeability values and radii of influence vary because of the heterogeneous and anisotropic nature of the upper flow zone. Capture zone dimensions have been calculated for each well using pumping rates given in Table 2, permeability values from Table 3, and hydraulic gradient data from the Final RFI report.

Calculations utilized the methodology of Todd and Grubb (Fetter, 1994). Calculations are for single wells; grouping effects were not analyzed. Figure 7 visually shows the capture zone for each well. Sample calcualtions are included in Appendix 4.

4. Transmissivity and Storage Coefficient

Assuming an upper flow zone saturated thickness, b, of 10 feet and using the field permeability, K, values described above, transmissivity, T, values for each well location were calculated using the relation T = Kb. These values of T are given in Table 5. The aquifer storage coefficient, S, is proportional to transmissivity, T, and time, and inversely proportional to the square of the radius of influence, r_o. Using the transmissivity, T, and radius of influence, r_o, values previously calculated, the calculated storage coefficient at each well location is also listed in Table 5. The equation used to calculated the storage coefficient, S, was derived by Jacob (Lohman, 1979) to determine S from distance-drawdown graphs (see sample calculations in Appendix 4). The calculated storage coefficients indicated semi-confined conditions exist.

TABLE 5
Transmissivity and Storage Coefficient

Well No.	Transmissivity, T (gal/day/ft)	Storage Coefficient, S
PW-1	68.7	0.0205
MW-16	50.7	0.0217
MW-18	73.4	0.0144
MW-23	536.4	0.0261
MW-24	92.5	0.0214
MW-25	95.5	0.0095
MW-26	75.5	0.0207
MW-27	615.0	0.0206
MW-28	6.2	0.0045

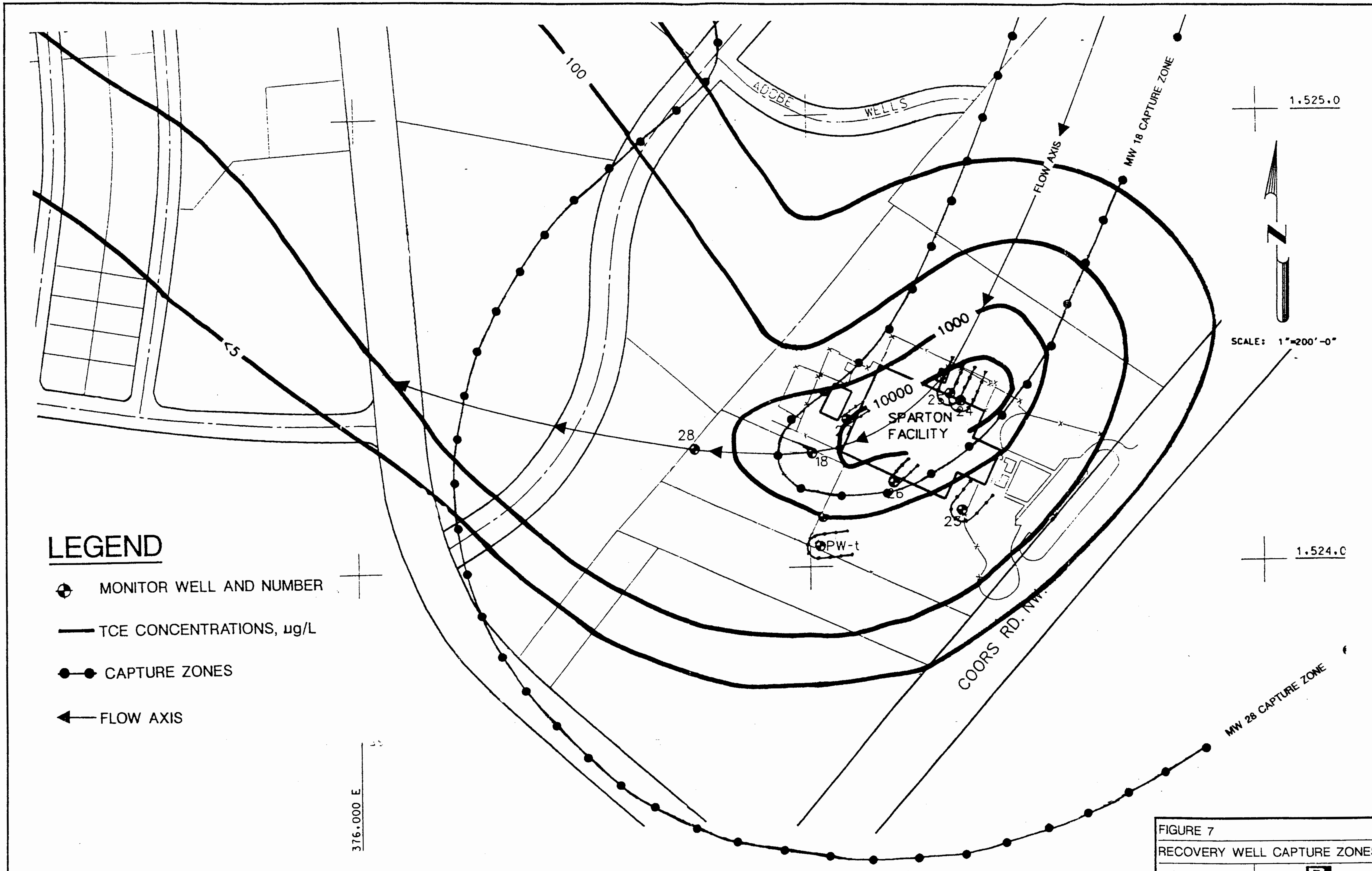


FIGURE 7
RECOVERY WELL CAPTURE ZONES
BY:
BLACK & VEATCH

OGC-004460

D. <u>Recovery Well Network Operation</u>

The recovery well network became operational in December 1988. Since start-up, approximately 3.2 million gallons of water have been pumped from the ground. The system has operated in accordance with design expectations and has required only routine maintenance.

V TREATMENT AND DISPOSITION

Groundwater pumped from the recovery wells is discharged to a collection piping system which transports the fluid to a collection tank. The collection piping system consists of discharge lines encased in secondary piping to provide leak detection and containment. Junction boxes, which house the remote well operators and sampling valves, are located at each well and at pipe junctions.

The produced groundwater is collected in a 550-gallon fiberglass-coated steel tank. The double wall tank has a leak detection system with a visual and audible alarm in the control building. A centrifugal transfer pump, which is controlled by the water level in the collection tank, transports water from the collection tank to the top of the packed tower (air stripper).

The 20 gpm packed tower, shown on Figure 8, receives untreated water from the transfer pump and discharges to the storage tank. A 400 cfm blower provides a counter-current flow of air to remove volatile organic compounds from the water. A recirculation line is provided on the packed tower discharge to allow a portion of the flow to be recirculated to the collection tank. The recirculation shortens the time between pumping cycles of the transfer pump. This procedure maintains the tower packing in a wet condition, thus improving treatment efficiency. The rate of recirculation may be adjusted by setting the butterfly valve on the recirculation line.

OGC-004462

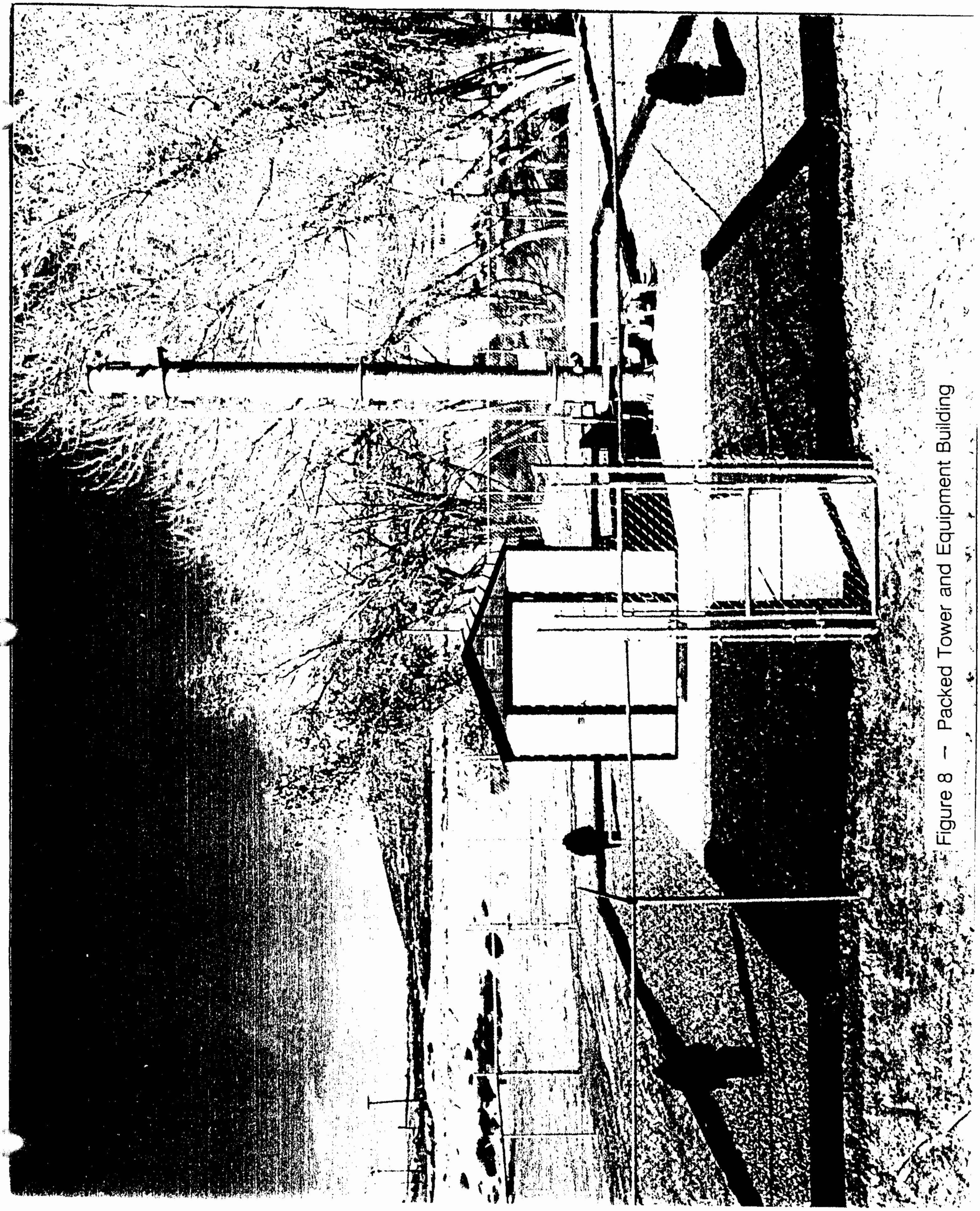


Figure 8 – Packed Tower and Equipment Building

Effluent from the packed tower is discharged to a 15,000-gallon fiberglass-coated steel tank for storage. The double-walled tank has a leak detection system with a visual and audible alarm in the control building. Water from the storage tank is used in the main plant building as cooling and flushing water and eventually discharged into the sewer system.

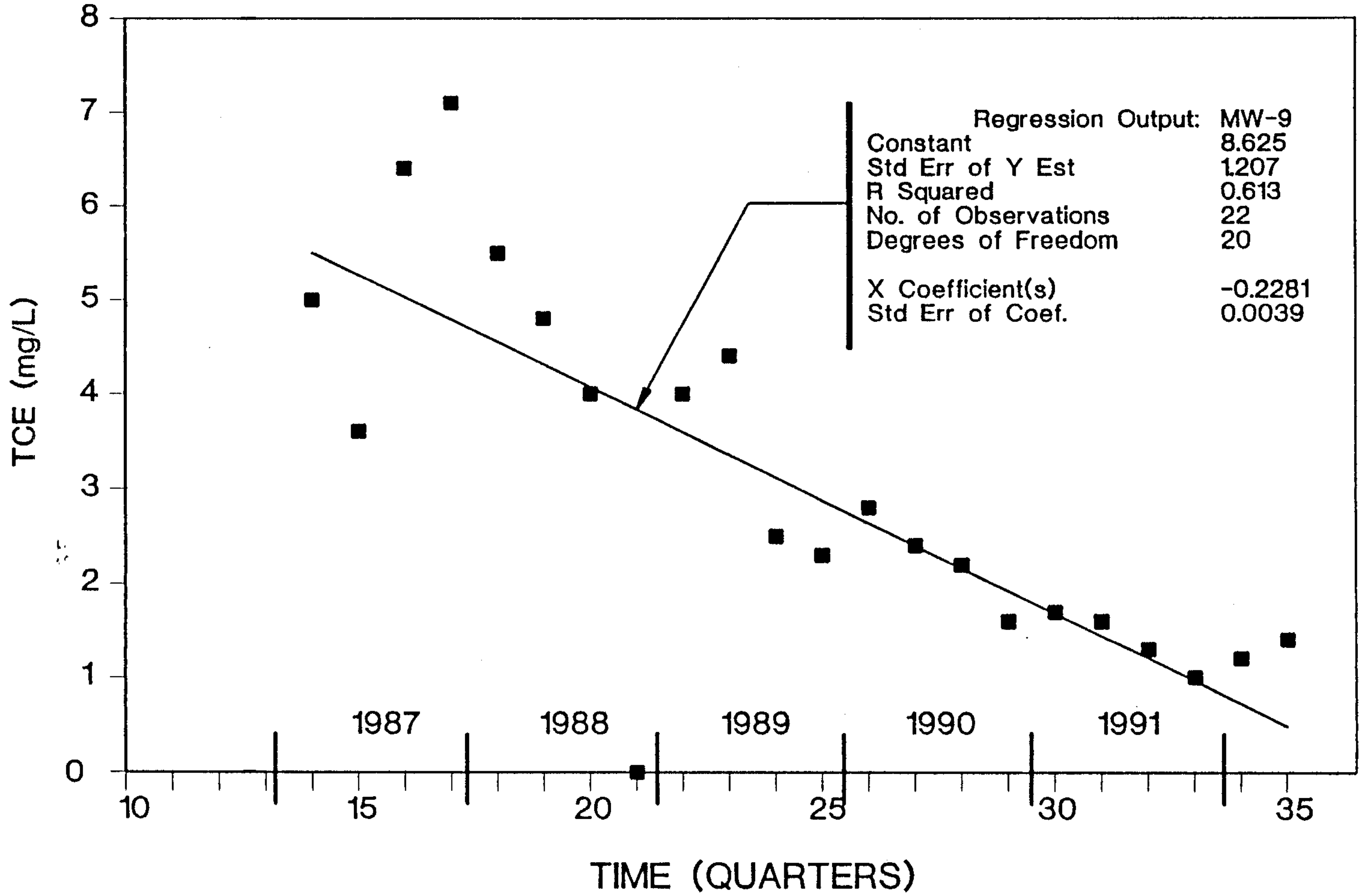
To date, approximately 3.2 million gallons of water have been treated in the packed tower. The air stripping system has demonstrated an average contaminant removal efficiency of 99 percent for the measured indicators, which include 1,1-dichloroethylene, methylene chloride, 1,1,1-trichloroethane, and trichloroethylene. Influent concentrations (total) have exceeded 1000 micrograms per liter (ppb). Air stripper treatment is producing effluent concentrations in the range of one microgram per liter (ppb) for each constituent being monitored. Monthly progress reports are sent to EPA, Region 6, describing bi-weekly water level measurements and monthly air stripper removal efficiencies.

OGC-004464

VI ANALYSIS AND CONCLUSIONS

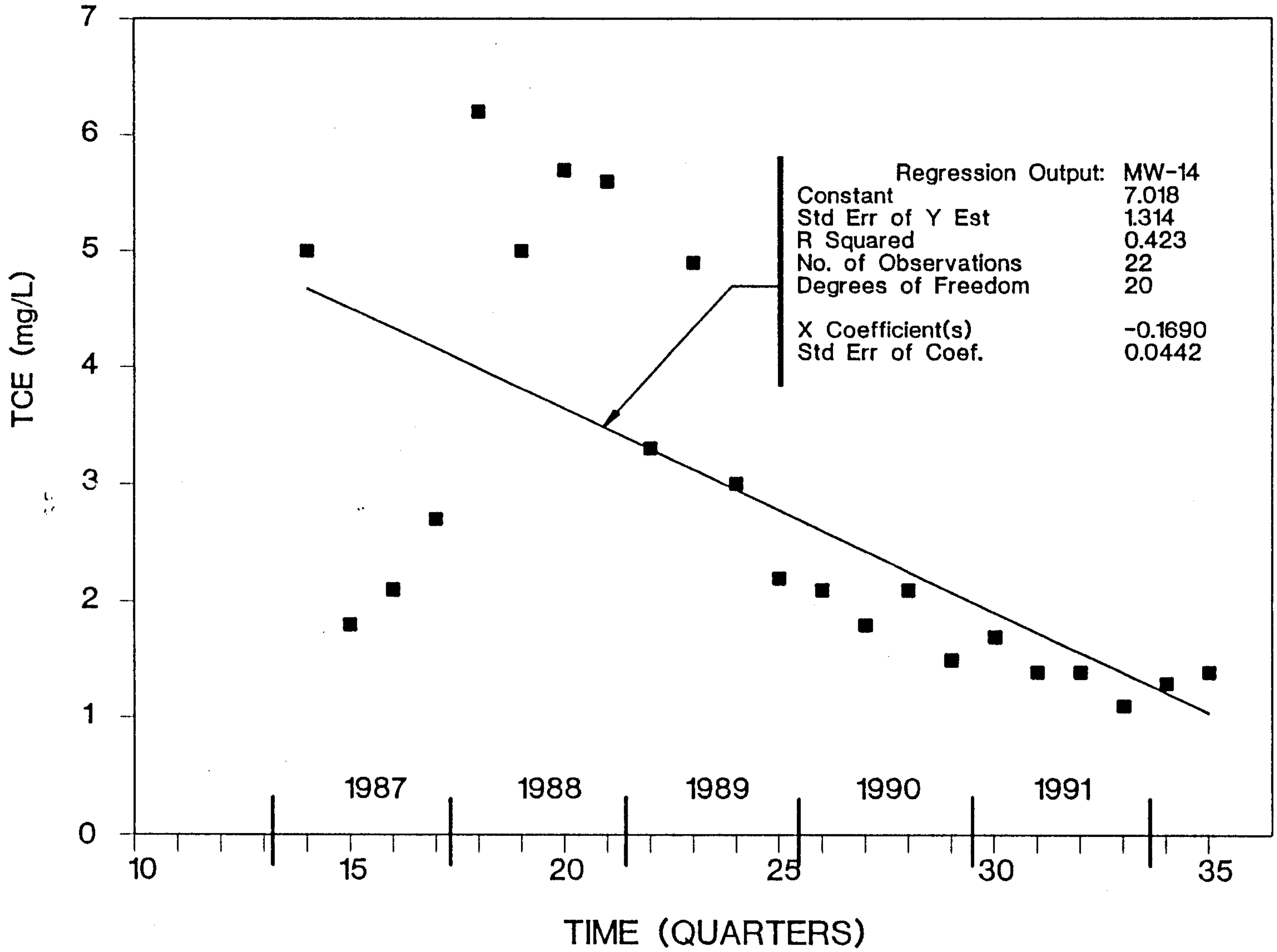
Since start-up in December 1988, the Upper Flow Zone Groundwater Recovery Well System has continuously operated in accordance with design requirements and has required only routine maintenance. The system has removed over three million gallons of contaminated groundwater and has successfully treated the water to allow beneficial use of the effluent water. The system is assisting in source removal in the immediate vicinity of the Sparton facility.

As shown on figures 9 through 14, time-history plots of TCE concentration in the upper flow zone obtained from the quarterly monitoring database indicate a steady decrease in concentration over time. Since completion of the RFI Report in July 1992, sampling and analysis was conducted in the fourth quarters of 1993 and 1994. These recent results indicate that the TCE mass has decreased approximately 30% and the average TCE concentration has dropped almost 40% since the RFI June 1991 Report Sampling. The recent results do indicate approzimately a 10% increase in the areal extent of the plume (3-zone total) to the northwest along Irving Boulevard. Reduction in TCE mass since the initial RFI sampling (1989-1990) is approximately 54% and the average TCE plume concentration has been reduced approximately 58%. Soil gas analyses conducted in 1984, 1987, and 1991 further confirm that the TCE mass has been reduced. It should be noted that other processes such as off-gassing, hydrolysis, and/or biodegradation may be contributing to the decrease in constituent concentrations.



TCE CONCENTRATION VS TIME
MW-9 (UPPER FLOW ZONE)
SPARTON TECHNOLOGY, INC.
ALBUQUERQUE, NEW MEXICO

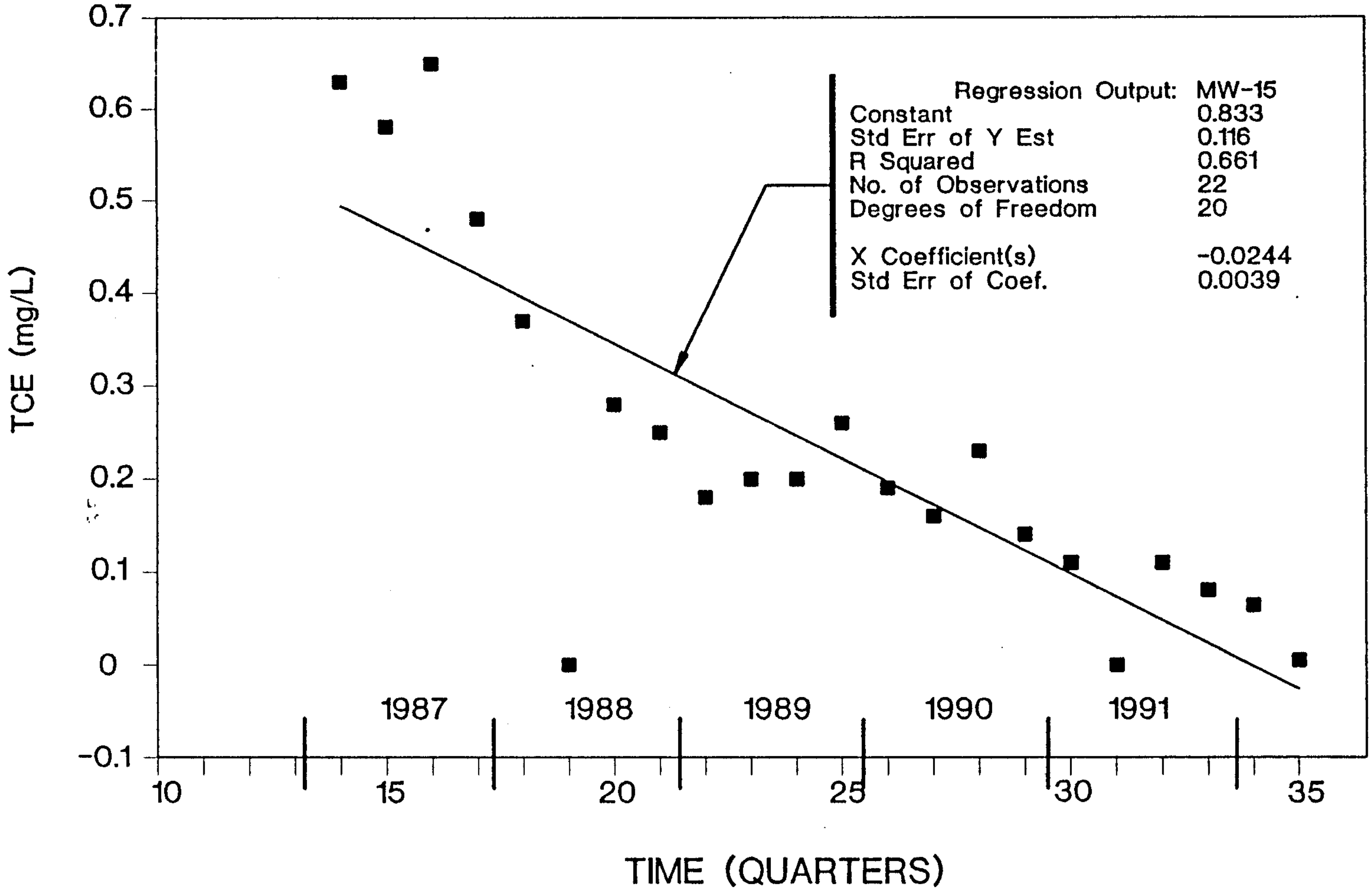
FIGURE
9



TCE CONCENTRATION VS TIME
MW-14 (UPPER FLOW ZONE)
SPARTON TECHNOLOGY, INC.
ALBUQUERQUE, NEW MEXICO

FIGURE
10

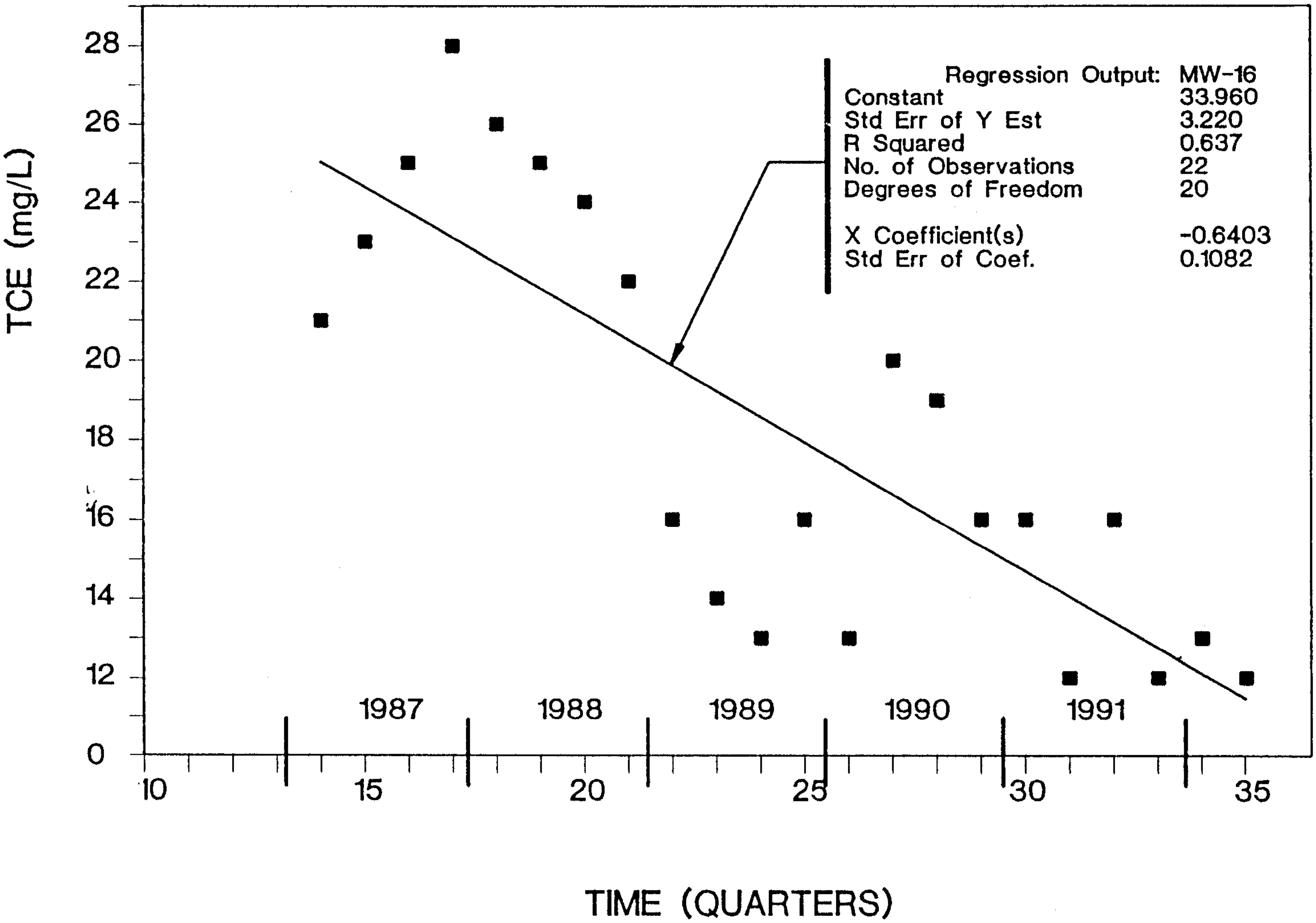
OGC-004467



TCE CONCENTRATION VS TIME
MW-15 (UPPER FLOW ZONE)
SPARTON TECHNOLOGY, INC.
ALBUQUERQUE, NEW MEXICO

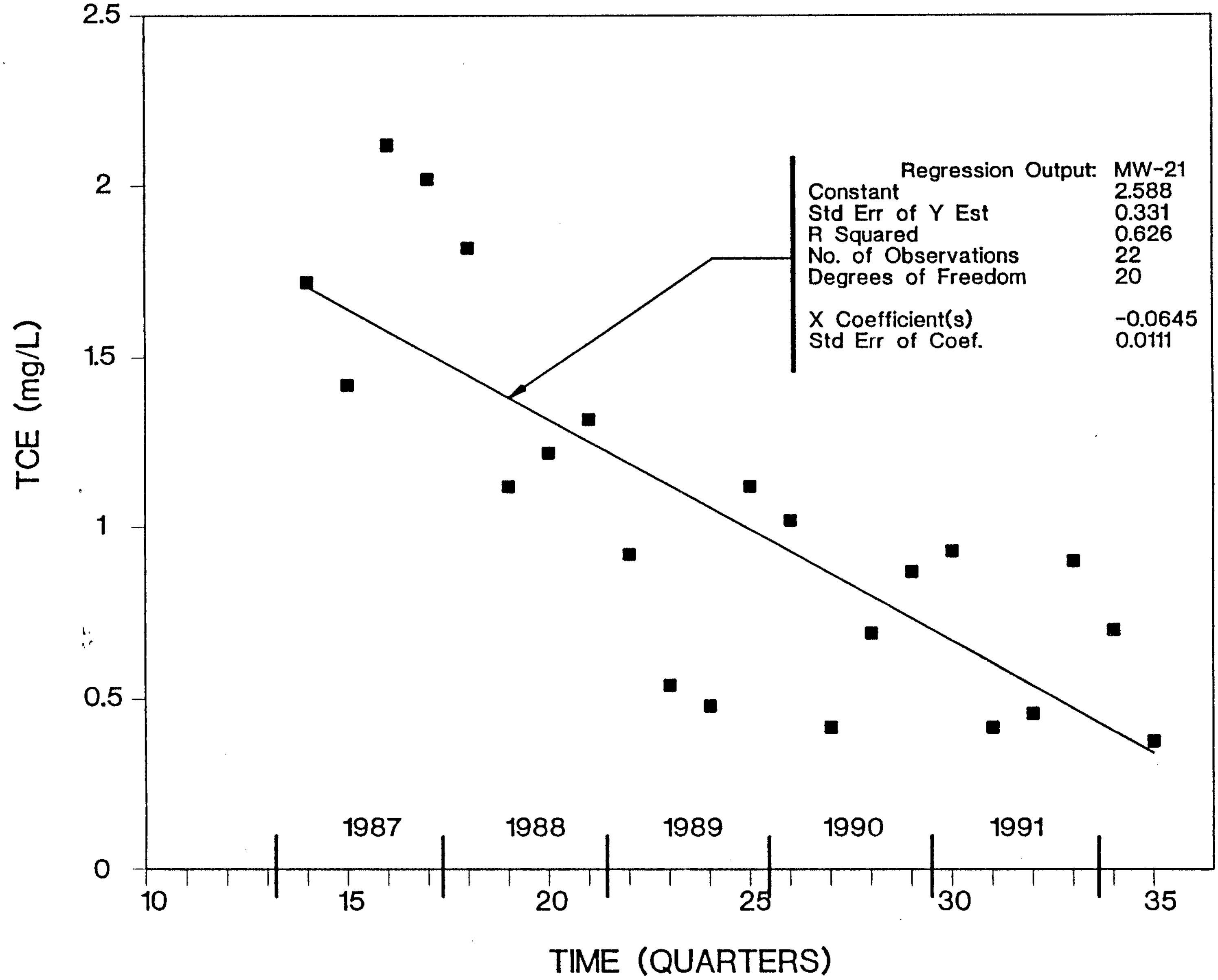
FIGURE
11

OGC-004468



TCE CONCENTRATION VS TIME
MW-16 (UPPER FLOW ZONE)
SPARTON TECHNOLOGY, INC.
ALBUQUERQUE, NEW MEXICO

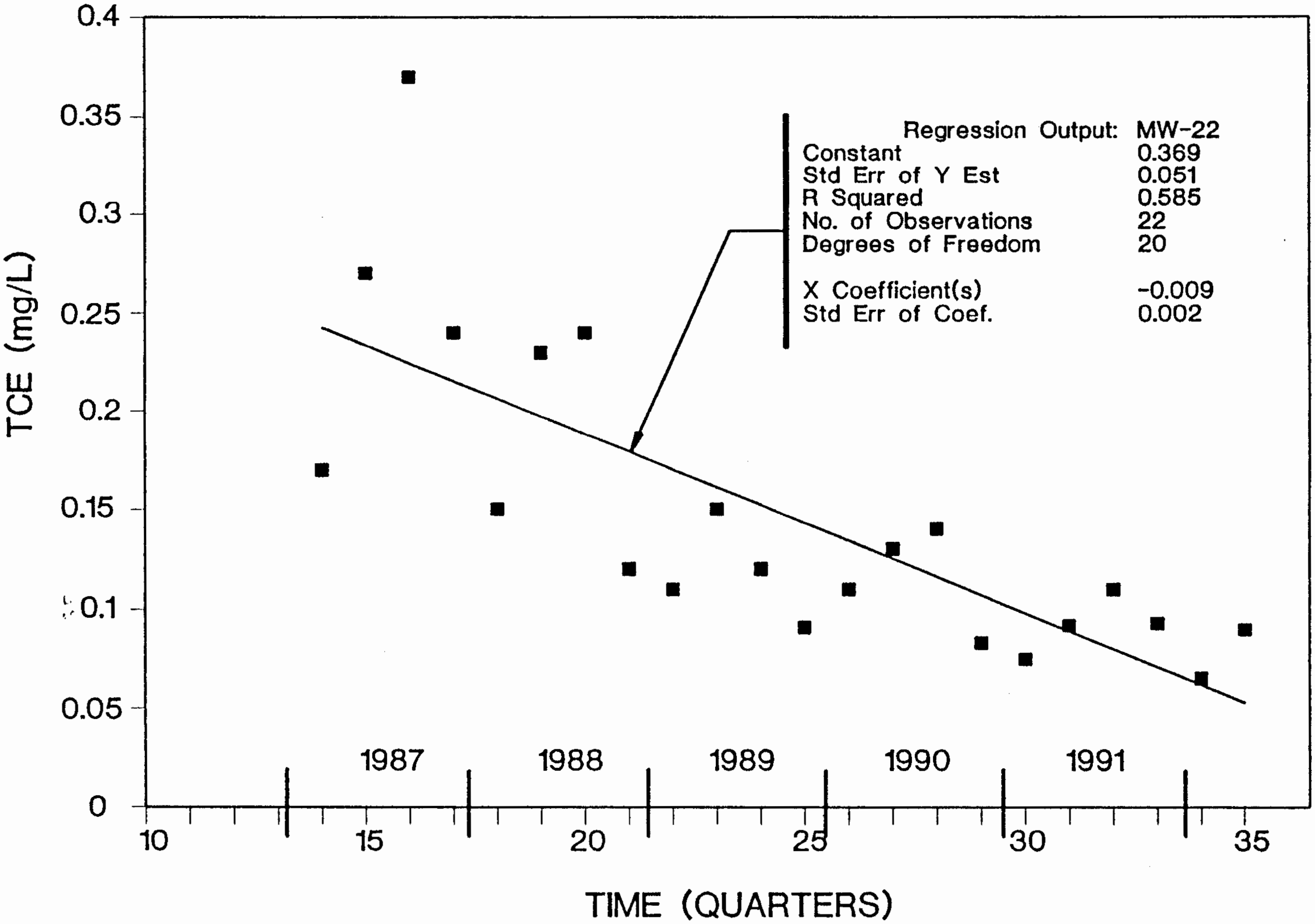
FIGURE
12



TCE CONCENTRATION VS TIME
MW-21 (UPPER FLOW ZONE)
SPARTON TECHNOLOGY, INC.
ALBUQUERQUE, NEW MEXICO

FIGURE
13

OGC-004470



TCE CONCENTRATION VS TIME
MW-22 (UPPER FLOW ZONE)
SPARTON TECHNOLOGY, INC.
ALBUQUERQUE, NEW MEXICO

FIGURE **14**

Multiple aquifer pumping tests have been conducted to evaluate hydraulic conditions in the upper flow zone. Upper flow zone aquifer parameters vary due to the heterogeneous and anisotropic subsurface conditions. A summary of parameters developed from the aquifer pumping tests is as follows:

Permeability,	$K = 2.91 \times 10^{-5}$ to 2.90×10^{-3} cm/sec
Radius of Influence,	$r_o = 35$ to 162 feet
Transmissivity,	$T = 6.2$ to 615 gal/day/ft
Storage Coefficient,	$S = 0.0045$ to 0.0261 (semi-confined conditions

These parameters seem reasonable and compare favorably with the geologic conditions observed in the upper flow zone.

In accordance with Section IV.A.1.(a) of the Administrative Order on Consent, the Upper Flow Zone Groundwater Recovery Well System was installed in December, 1988, and has been operated continuously since that time. The system is accomplishing its goal of mitigating further off-site migration of contaminants in the upper flow zone. The effectiveness of this Interim Measure is the result of locating the recovery wells in the most concentrated area of the contaminant plume and downgradient of the source. Effectiveness of the system is further confirmed by the following:

1. Recovery and treatment of approximately 3.2 million gallons with an average air stripper efficiency of ninety-nine percent.
2. Observed decrease in volatile organic constituent concentration in the on-site upper flow zone wells since early 1989.

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Region VI
Dallas, Texas

APPENDIX 1

BI-WEEKLY WATER LEVEL READINGS

SPARTON TECHNOLOGY, INC.
COORS ROAD FACILITY
BIWEEKLY WATER LEVEL MEASUREMENTS

DATE	MW-7	MW-9	MW-12	MW-13	MW-14	MW-15	MW-16	MW-17	MW-21	MW-22	MW-33	MW-34	MW-35	MW-36	MW-37
	Water Level Elevation - Feet Above MSL														
11/13/90	4980.98	4977.52	4976.90	4978.43	4975.15	4977.52	4981.25	4981.33	4980.92	4981.13	4976.31	4977.91	4975.52	4974.05	4972.74
11/28/90	4980.48	4977.10	4976.65	4978.01	4974.90	4977.43	4981.25	4981.33	4980.83	4980.80	4975.98	4977.35	4975.21	4973.79	4972.51
12/12/90	4980.15	4976.77	4976.32	4977.60	4974.49	4977.18	4980.83	4980.99	4980.67	4980.47	4975.98	4976.96	4974.97	4973.67	4972.48
12/27/90	4979.98	4976.60	4975.98	4977.18	4974.32	4977.14	4980.83	4980.91	4980.58	4980.22	4975.81	4976.54	4974.68	4973.42	4972.31
01/09/91	4980.15	4976.77	4976.32	4977.60	4974.49	4977.18	4980.83	4980.99	4980.67	4980.47	4975.98	4976.96	4974.97	4973.67	4972.48
01/23/91	4979.98	4976.60	4975.98	4977.18	4974.32	4977.14	4980.83	4980.91	4980.58	4980.22	4975.81	4976.54	4974.68	4973.42	4972.31
02/06/91	4979.23	4975.85	4975.48	4976.68	4973.99	4976.68	4980.42	4980.49	4980.33	4979.63	4975.23	4975.89	4974.15	4973.00	4972.07
02/20/91	4979.15	4975.85	4975.40	4976.43	4973.82	4976.35	4980.33	4980.49	4980.17	4979.55	4975.06	4975.75	4974.03	4972.86	4972.04
03/06/91	4978.90	4975.60	4975.40	4976.35	4973.65	4976.27	4980.33	4980.33	4980.08	4979.47	4975.06	4975.61	4973.90	4972.80	4971.89
03/20/91	4978.81	4975.43	4975.07	4976.35	4973.65	4976.10	4980.00	4980.16	4980.08	4979.22	4975.06	4975.53	4973.82	4972.83	4971.94
04/04/91	4978.81	4975.43	4975.07	4976.35	4973.65	4975.93	4980.00	4980.08	4980.00	4979.22	4974.81	4975.57	4973.75	4972.75	4971.86
04/18/91	4979.23	4975.85	4975.40	4976.68	4973.99	4975.93	4980.08	4980.08	4979.29	4979.47	4975.06	4975.85	4973.79	4972.78	4971.79
05/01/91	4979.65	4976.27	4975.65	4977.01	4974.07	4976.18	4980.00	4980.33	4980.08	4979.80	4975.48	4976.56	4974.28	4973.00	4972.00
05/15/91	4980.15	4976.60	4976.32	4977.60	4974.40	4976.60	4980.33	4980.49	4980.08	4980.05	4975.89	4977.09	4974.53	4973.17	4972.04
05/29/91	4980.56	4977.02	4976.65	4978.35	4974.74	4976.77	4980.42	4980.58	4980.42	4980.47	4976.08	4977.69	4974.94	4973.46	4972.22
06/12/91	4980.98	4977.27	4976.82	4978.43	4974.90	4977.10	4980.83	4980.83	4980.50	4980.80	4976.31	4977.84	4975.10	4973.60	4972.33
06/26/91	4981.31	4977.43	4976.82	4978.43	4974.90	4977.10	4980.83	4980.91	4980.58	4980.88	4977.28	4977.93	4975.26	4973.71	4972.36
07/10/91	4981.31	4977.43	4976.82	4978.51	4974.90	4977.18	4981.17	4980.99	4980.50	4980.97	4976.31	4977.94	4975.32	4973.76	4972.38
07/24/91	4981.48	4977.52	4977.15	4978.51	4975.07	4977.43	4981.25	4981.24	4980.50	4981.13	4976.39	4978.19	4975.42	4973.85	4972.46
08/07/91	4981.31	4977.10	4976.73	4978.18	4974.90	4977.27	4981.25	4981.08	4980.58	4980.80	4976.23	4977.73	4975.25	4973.75	4972.36
08/21/91	4980.98	4977.18	4976.73	4978.26	4974.90	4977.43	4981.17	4981.16	4980.58	4980.80	4976.23	4977.75	4975.20	4973.76	4972.33
09/05/91	4981.31	4977.52	4977.23	4978.85	4975.32	4977.27	4981.33	4981.33	4980.83	4981.22	4976.64	4978.42	4975.51	4973.94	4972.51
09/18/91	4981.31	4977.52	4977.15	4978.51	4975.15	4977.35	4981.33	4981.41	4980.75	4981.22	4976.56	4978.12	4975.53	4973.97	4972.54

OGC-004477

SPARTON TECHNOLOGY, INC.
COORS ROAD FACILITY
BIWEEKLY WATER LEVEL MEASUREMENTS

	[------ Water Level Elevation - Feet Above MSL ------]														
DATE	MW-7	MW-9	MW-12	MW-13	MW-14	MW-15	MW-16	MW-17	MW-21	MW-22	MW-33	MW-34	MW-35	MW-36	MW-37
10/03/91	4980.98	4977.43	4976.90	4978.43	4974.90	4977.43	4981.25	4981.41	4980.83	4981.05	4976.31	4977.81	4975.39	4973.95	4972.48
10/16/91	4981.31	4977.60	4977.15	4978.51	4975.15	4977.52	4981.50	4981.49	4980.92	4981.22	4976.48	4978.18	4975.52	4973.97	4972.54
10/30/91	4981.31	4977.52	4977.07	4978.43	4974.99	4977.60	4981.25	4981.58	4980.92	4981.22	4976.39	4977.99	4975.42	4973.94	4972.52
11/13/91	4980.90	4977.27	4976.73	4978.01	4974.90	4977.52	4981.25	4981.41	4980.75	4980.88	4976.31	4977.61	4975.27	4973.90	4972.51
11/26/91	4980.48	4977.02	4976.57	4977.93	4974.40	4977.43	4981.00	4981.24	4980.83	4980.72	4976.06	4977.25	4975.13	4973.66	4972.32
12/12/91	4980.06	4976.60	4976.15	4977.43	4974.15	4977.35	4981.00	4981.08	4980.58	4980.38	4975.89	4976.87	4974.80	4973.47	4972.23
12/27/91	4979.98	4976.35	4975.98	4977.26	4974.15	4977.18	4980.83	4980.99	4980.58	4980.13	4975.64	4976.51	4974.53	4973.25	4972.04
01/09/92	4979.65	4976.18	4975.82	4977.01	4973.82	4977.02	4980.75	4980.91	4980.50	4979.97	4975.48	4976.30	4974.42	4973.18	4971.98
01/22/92	4979.56	4976.02	4975.57	4976.76	4973.65	4976.77	4980.58	4980.74	4980.50	4979.88	4975.31	4976.15	4974.25	4973.45	4971.89
02/05/92	4979.31	4975.85	4975.48	4976.68	4973.65	4976.68	4980.42	4980.58	4980.33	4979.72	4975.14	4975.99	4974.10	4972.82	4971.16
02/19/92	4979.23	4975.77	4975.40	4976.60	4973.65	4976.35	4980.42	4980.49	4980.50	4979.55	4975.06	4975.82	4973.93	4972.65	4971.07
03/04/92	4978.98	4975.77	4975.40	4976.35	4973.57	4976.27	4980.33	4980.41	4980.08	4979.47	4975.06	4975.68	4973.84	4972.61	4971.07
03/17/92	4978.81	4975.60	4975.23	4976.35	4973.57	4976.18	4980.17	4980.33	4980.25	4979.30	4974.98	4975.54	4973.72	4972.47	4970.95
03/31/92	4978.90	4975.85	4975.40	4976.43	4973.57	4975.93	4980.08	4980.16	4980.08	4979.38	4975.06	4975.84	4973.75	4972.65	4971.66
04/16/92	4979.31	4975.93	4975.57	4976.76	4973.65	4976.10	4980.17	4980.33	4980.08	4979.63	4975.23	4976.14	4973.94	4972.74	4971.69
04/29/92	4979.56	4976.27	4975.82	4977.10	4973.90	4976.18	4980.42	4980.41	4980.08	4979.80	4975.39	4976.35	4974.07	4972.81	4971.71
05/13/92	4979.81	4976.35	4975.90	4977.18	4973.99	4976.27	4980.33	4980.49	4980.17	4979.97	4975.56	4977.55	4974.25	4972.94	4971.78
05/27/92	4980.15	4976.68	4976.23	4977.60	44974.15	4976.43	4980.67	4980.66	4980.33	4980.30	4975.81	4976.95	4974.49	4973.12	4971.94

Note: At MW-37 the water table elevations have been revised for December 91 and January 92 using the new measuring point elevation.

APPENDIX 2

METRIC CORPORATION REPORTS

Aquifer Testing
at the
Sparton Technology, Inc.
Coors Road Plant
Albuquerque, New Mexico

Prepared by

METRIC Corporation
Albuquerque, New Mexico

APRIL 1987

AQUIFER TESTING
AT THE
SPARTON TECHNOLOGY, INC.
COORS ROAD PLANT

Aquifer tests were performed in three wells at the Sparton Technology, Inc., Coors Road Plant during March 1987. The purpose of the testing was to estimate the aquifer permeability of the "upper flow zone". The resulting information will be used in design of a pollution recovery well network, and possibly a recharge well network. The "upper flow zone" consists generally of the upper 5 to 10 feet of the saturated zone at the Coors Road site separated from the remainder of the saturated zone by a fine grained aquitard unit.

Pumping tests were conducted in three wells; MW-16 and MW-24 in the pond and sump area on the northwest side of the building, and in MW-18 located about 60 feet west of the west corner of the building.

The tests were conducted as follows:

Well: MW-16
Test Type: Constant Drawdown
Test Drawdown: 2.38 ft
Available Drawdown: 5.4 ft ±
Duration of Pumping: 4325 min ≃ 72 hr
Average Discharge: 0.145 gpm
Observations Taken in Wells: MW-16 (recovery), MW-24, MW-25, MW-17

Well: MW-24
Test Type: Constant Drawdown
Test Drawdown: 3.26 ft
Available Drawdown: 8.1 ft ±
Duration of Pumping: 4390 min ≃ 73 hr
Average Discharge: 0.205 gpm
Observations Taken in Wells: MW-24 (recovery), MW-16, MW-25, MW-17

Well: MW-18
Test Type: Constant Discharge
Maximum Drawdown: 5.02 ft
Available Drawdown: 12.6 ft ±
Duration of Pumping: 2940 min = 49 hr
Average Discharge: 0.264 gpm
Observations Taken in Wells: MW-18

Since wells MW-16 and MW-24 are 2-inch i.d. wells, the tests were conducted using a 1.67-inch o.d. positive displacement pump having a maximum capacity of about 2.5 gpm. The combination of small well diameter (making it difficult to obtain reliable water levels in the pumped well with small drawdowns, i.e., less than 3.5 feet) and low well capacities i.e., less than 0.25 gpm (making it difficult to maintain a constant discharge) resulted in the selection of a constant drawdown test for wells MW-16 and MW-24. Also significant is the fact that MW-16 and MW-24 are only 11.3 feet apart, providing a close observation well for each test.

Well MW-18 is a four-inch diameter well with no close observation wells available. As a result, a constant discharge test was performed on MW-18, with drawdown and recovery measurements taken in the pumped well.

All water level measurements were made with electronic sounders and are felt to be accurate to within ± 0.01 ft.

Volatile organic samples were collected periodically during each of the tests, and metal samples were collected near the end of each test.

The water level and discharge data collected during each test is presented in APPENDIX A.

METRIC Corporation

Date: 9-27-88

Pumped Well MW-23

Measurements at Well MW-23

Pump Speed: ______ Q: 0.26417 gpm

Static Water Level ______

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min.sec/ℓ)
27 8:15:00	0			0	
15	.25			0.60	
30	.5			0.59	
45	.75			0.56	
16:00	1.0			0.11	
16:30	1.5			0.42	
17:00	2.0			0.60	
17:30	2.5			0.56	
18:00	3.0			0.66	
18:30	3.5			0.97	
19:00	4.0			0.41	
19:30	4.5			-	
20:00	5.0			0.86	
21	6			1.88	
22	7			2.06	
23	8			2.45	
24	9			2.16	
25	10			2.26	
27	12			2.37	1'09"/ℓ
29	14			2.43	
31	16			2.42	
33	18			2.19	
35	20			1.93	1'09"/ℓ
40	25			2.06	1'07"/ℓ
45	30			2.08	1'07"/ℓ
50	35			2.11	1'00"/ℓ
9:00	45			2.41	1'00"/ℓ
9:30	75			2.43	55"/ℓ
10:00	105			2.31	1'09"
10:30	135			2.16	1'05"

Date: 9-27-88

METRIC Corporation

Pumped Well MW-23

Measurements at Well MW-23

Pump Speed: ______ Q: ______ gpm

Static Water Level ______

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min·sec/ℓ)
11:00	165			2.75	1'0"
12:00	225			2.75	47" adj 56"
13:00	285			2.86	59" 58"
14:00	345			3.00	49" adj
15:00	405			3.01	49" adj
16:00	465			2.89	59"
17:00	525			2.51	1'3"
18:00	585			2.52	1'5"
19:00	645			2.46	1'16" adj 1'00"
20:00	705			2.51	1'01"
21:00	765			2.66	1'01"
22:00	825			2.55	0'59"
23:00	885			2.39	1'02"
24:00	945			2.47	1'05"
1:00	1005			2.45	1'06"
2:00	1065			2.46	1'05"
3:00	1125			2.36	1'05"
4:00	1185			2.42	1'03"
5:00	1245			2.39	1'02"
6:00	1305			2.44	1'01"
7:00	1365			2.44	1'4"
8:00	1425			2.54	1'6" adj
9:00	1485			2.60	57"
10:00	1545			2.58	55" adj
11:00	1605			2.57	56" adj
12:00	1665			2.66	1'0"
13:00	1725			2.66	58"
14:00	1785			2.70	55" adj
15:00	1845			2.62	55" adj
16:00	1905			2.63	55" adj

The results of the aquifer testing are presented in TABLE 1. The constant drawdown data from the pumped wells (MW-16 and MW-24) were analyzed by Lohman, 1972. The residual-drawdown data from the pumped wells and the time-drawdown and residual-drawdown data from the observation wells were analyzed using Jacob plots (wells MW-16, and MW-24 tests) Lohman, 1972 suggests that the recovery method is strictly applicable only to tests of constant discharge and variable drawdown or recovery, however, recovery tests generally give values of T in close agreement with constant drawdown tests.

The testing performed on MW-18 consisted of a constant discharge test with measurements taken in the pumped well. The data were analyzed using Jacob plots.

All of the plots are presented in APPENDIX B.

The time drawdown plots were checked to ensure that $u < 0.05$ and, thus, validate the use of the Jacob solution.

The MW-18 data were checked using a procedure suggested by Schafer, 1978 to establish which portions of the data are casing storage affected.

Based on the previously described testing, it is felt that the best estimate for the permeability (hydraulic conductivity) of the upper flow zone in the pond and sump area is about 5×10^{-3} cm/sec. (see TABLE 1). Likewise, the best estimate for the permeability of the upper flow zone in the vicinity of MW-18 is about 3×10^{-4} cm/sec. (see TABLE 1).

The residual-drawdown curves (APPENDIX B) show some evidence that a "recharge effect" may be occurring during the pumping period. The residual drawdown curves generally show a t/t'

TABL1

Aquifer Testing

Sparton Technology, Inc. Coors Road Plant

Pumped Well	Observations at	Curve	Apparent T (gpd/ft)	b (ft)	Apparent K (ft/day)	Apparent K (cm/sec)	Comments
MW-16	MW-16	$\frac{s_w}{Q}$ vs $\frac{t}{r_w^2}$	57.3	5.4	1.41	5.000×10^{-4}	Represents Grouted Zone Near MW-16
		Early R-D	14.1	5.4	0.35	1.23×10^{-4}	Represents Grouted Zone Near MW-16
		Late R-D	49.1	5.4	1.22	4.29×10^{-4}	Represents Grouted Zone Near MW-16
MW-16	MW-24	Early T-D	479	5.4	11.9	4.18×10^{-3}	Selected Value
		Early R-D	2734	5.4	67.7	2.39×10^{-2}	"Recharge" Affected
		Late R-D	781	5.4	19.3	6.82×10^{-3}	Good Value
MW-24	MW-24	$\frac{s_w}{Q}$ vs $\frac{t}{r_w^2}$	275	8.05	4.57	1.61×10^{-3}	Well Loss Affected
		Early R-D	22.9	8.05	0.38	1.34×10^{-4}	Well Loss Affected
		Late R-D	846	8.05	14.0	4.96×10^{-3}	Selected Value
MW-24	MW-16	T-D	773	8.05	12.8	4.53×10^{-3}	Selected Value
		Early R-D	410	8.05	6.81	2.40×10^{-3}	Good Value
		Late R-D	1203	8.05	20.0	7.05×10^{-3}	"Recharge" Affected
MW-18	MW-18	Early T-D	29.0	12.6	0.31	1.09×10^{-4}	Casing Storage Affecte
		Late T-D	87.1	12.6	0.92	3.26×10^{-4}	Selected Value
		Early R-D	131	12.6	1.39	4.90×10^{-4}	Casing Storage and Recharge Affected
		Late R-D	15.2	12.6	0.16	5.69×10^{-5}	Casing Storage Affected

value greater than 2 at zero drawdown, suggesting a "recharge effect". Possible explanations of the apparent "recharge effect" include reduction or reversal of prevailing downward vertical leakage in the cone of depression during the tests or induced flow from a more permeable burried channel(s) existing within the upper flow zone. Evaluation of the analyses of volatile organic samples collected during the pumping tests (see APPENDIX C) indicates that if a "recharge effect" is occurring the recharge water has approximately the same organic contaminant levels as the water adjacent to the well.

APPENDIX A

PUMP TEST DATA

CONSTANT DRAWDOWN AQUIFER TEST

WELL MW-16

DRAWDOWN 2.38'

Static Water Level 66.98'

time (h:m:s)	t (min)	t' (min)	t/t'	$\frac{t}{r_w^2}$ (m/ft^2)	Discharge (gpm)	Total Water Removed (gal)	$\frac{s_w}{Q}$ (ft-m/g
1:20:00	15			1532	0.251		9.5
1:28:00	23			2349	0.267		8.9
13:29:00	24			2451	0.198		12.0
13:33:30	28.5			2910	0.192		12.4
13:35:00	30			3063	0.195		12.2
3:40:00	35			3574	0.264		9.0
3:48:00	43			4391	0.227		10.5
3:51:00	46			4697	0.213		11.2
3:56:00	51			5208	0.231		10.3
4:09:00	64			6535	0.203		11.7
9:55	75			7659	0.251		9.5
28:30	83.5			8527	0.240		9.9
4:38:25	93			9497	0.236		10.1
4:48:30	103.5			10569	0.243		9.8
4:59:40	114.5			11692	0.236		10.1
5:19:00	134			13684	0.229		10.4
5:38:00	153			15624	0.227		10.5
5:03:45	179			18279	0.216		11.0
5:33:45	209			21342	0.198		12.0
7:03:33	238.5			24355	0.190		12.5
3:04:30	299.5			30584	0.186	55 gal	12.8
:03:34	358.5			36609	0.198		12.0
):02:30	417.5			42634	0.195		12.2
:03:55	479			48914	0.189		12.6
:04:18	539			55041	0.203		11.7
:03:30	598.5			61117	0.205		11.6
:10	660			67397	0.193	110 gal	12.3
:09:12	784			80059	0.211		11.3
:08:13	903			92211	0.160		14.9
:12:15	1027			104873	0.165	165 gal	14.4

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Date 3/3/87-3/07/87

CONSTANT DRAWDOWN AQUIFER TEST

WELL MW-16

DRAWDOWN 2.38'

Static Water Level 66.98'

time (h:m:s)	t (min)	t' (min)	t/t'	$\frac{t}{r_w^2}$ (m/ft^2)	Discharge (gpm)	Total Water Removed (gal)	$\frac{s_w}{Q}$ (ft-m/[illegible]
10:09:50	1265			129177	0.157	220 gal	15.[illegible]
13:02	1437			146741	0.176		13.5
14:00:00	1495			152664	0.156		15.3
15:03:00	1558			159097	0.160		14.9
16:17:00	1632			166654	0.166		14.3
17:07:00	1682			17160	0.151	275 gal	15.8
18:04:00	1739			177580	0.170		14.0
20:42:00	1897			193715	0.166		14.3
21:58:00	1973			201476	0.166		14.3
23:06:00	2041			208419	0.151		1[illegible]
:40	2075			-	-	330 gal	-
24:09:06	2104			214853	0.144		16.5
2:05:51	2221			226800	0.156		15.3
4:06:15	2341			239054	0.145		16.4
6:07:00	2462			251410	0.147	385 gal	16.2
8:07:14	2582			263664	0.147		16.2
10:44:41	2740			279799	0.145		16.4
12:06:00	2821			288070	0.145		16.3
14:04:28	2939.5			300171	0.140	440 gal	17.0
16:03:00	3058			312272	0.121		19.7
18:11:30	3186.5			325394	0.119		20.0
20:07:20	3302.5			337239	0.126		18.9
22:14:10	3429			350157	0.123	495 gal	19.4
24:9:00	3544			361900	0.122		19.5
1:57:36	3652.5			372980	0.132	535 gal	18.[illegible]
[illegible]:07:48	3783			386306	0.127		18.[illegible]
[illegible]:08:00	3903			398560	0.117		20.4
8:09:00	4024			410916	0.117		20.3
10:07	4142			422966	0.118		20.2
11:00	4195			-	-	5[illegible] gal	

CONSTANT DRAWDOWN AQUIFER TEST

WELL MW-16

DRAWDOWN 2.38'

Static Water Level 66.98"

time (h:m:s)	t (min)	t' (min)	t/t'	$\frac{t}{r_w^2}$ (m/ft^2)	Discharge (gpm)	Total Water Removed (gal)	$\frac{s_w}{Q}$ (ft·m/g
12:40:00	4295			438590	0.115	590 gal	20.7
13:03:42	4319			441040	0.090	625	26.4

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Date: 3/3/87-3/07/87

Pumped Well MW-16

Measurements at Well MW-16

Pump Speed: - Q: - gpm

Static Water Level ______

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
13:10	4325	0			
13:17:30	4332.5	7.5	578	.83	Pump OFF ~~13:10:00~~
13:17:44	4332.7	7.7	563	.76	
13:17:56	4332.9	7.9	548	.73	
13:18:22	4333.4	8.4	516	.70	
13:18:34	4333.6	8.6	504	.68	
13:18:52	4333.9	8.9	487	.65	
13:20:00	4335	10	434	.55	
22:00	4337	12	361	.42	
24:00	4339	14	310	.31	
26:00	4341	16	271	.24	
28:00	4343	18	241	.21	
13:30:00	4345	20	217	.15	
35	4350	25	174	.09	
40	4355	30	145	.06	
45	4360	35	125	.04	
50	4365	40	109	.03	
14:00:00	4375	50	87.5	.03	
14:10:00	4385	60	73.1	.02	
14:20:00	4395	70	62.8	.02	
14:30:00	4405	80	55.1	.02	
14:40:00	4415	90	49.1	.02	
14:50:00	4425	100	44.25	.02	
15:00:00	4435	110	40.3	.01	
15:20:00	4455	130	34.3	.01	
15:40:00	4475	150	29.8	.01	
15:58	4493	168	26.7	0	
16:49:56	4545	220	20.7	.01	
18:16:09	4631	306	15.1	.01	
19:08:00	4683	358	13.1	.01	

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Pumped Well MW-16

Measurements at Well MW-16

Pump Speed: - Q: - gpm

Static Water Level

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
20:26:00	4761	436	10.9	.01	
22:22	4877	552	8.8	.01	
24:10	4985	660	7.6	0	
2:08	5103	778	6.6	0	
3:27	5182	857	6.0	.01	
6:11	5346	1021	5.2	0	
9:10	5525	1200	4.6	0	
					Test Terminated

Date: 3/3/87-3/07/87

Pumped Well MW-16

Measurements at Well MW-24

Pump Speed: ________ Q: ________ gpm

Static Water Level 67.32'

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
13:05:00	0			0.00	Pump ON
13:06:15	1			0.00	
13:11:52	7			0.05	
13:18:09	13			0.06	
13:23:32	18.5			0.06	
13:26:54	22			0.06	
13:32:55	28			0.06	
13:38:03	33			0.07	
13:42:50	38			0.07	
13:48:07	43			0.07	
13:52:40	48			0.08	
13:57:55	53			0.08	
14:06:00	61			0.08	
14:17:10	72			0.10	
14:25:48	81			0.11	
14:35:30	90.5			0.11	
14:46:00	101			0.11	
14:56:57	112			0.11	
15:16:04	131			0.11	
15:35:41	151			0.12	
16:00:58	176			0.12	
16:31:03	206			0.12	
17:00:48	236			0.13	
18:01:00	296			0.14	
19:01:22	356.5			0.14	
19:59:20	414.5			0.15	
21:01:06	476			0.15	
22:01:38	536.5			0.15	
23:01:13	596			0.14	
24:05:09	660			0.14	

Date: 3/3/87-3/07/87

Pumped Well MW-16

Measurements at Well MW-24

Pump Speed: ________ Q: ________ gpm

Static Water Level 67.32'

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
2:02:30	777.5			0.14	
4:04:32	899.5			0.14	
6:03:47	1019			0.14	
9:00:46	1196			0.13	
13:06:12	1441.0			0.12	
13:55:33	1490.5			0.12	
14:59:33	1555.5			0.13	
16:13:59	1629			0.12	
17:00:00	1675			0.13	
18:01	1736			0.14	
20:37:00	1892			0.14	
21:55	1970			0.14	
23:02	2037			0.13	
24:03:42	2099			0.12	
2:01:13	2216			0.13	
4:03:16	2338			0.13	
6:04:30	2460			0.14	
8:01:40	2577			0.13	
10:00:00	2695			0.13	
12:00:54	2816			0.12	
14:07:28	2942.5			0.12	
16:15:56	3071			0.12	
18:14:40	3190			0.13	
20:10:25	3305.5			0.13	
22:17:00	3432			0.13	
24:03:49	3539			0.12	
1:53:37	3648.5			0.13	
4:02:30	3777.5			0.13	
5:59:00	3894			0.13	
8:00:00	4015			0.13	

Date: 3/3/87-3/07/87

Pumped Well MW-16

Measurements at Well MW-24

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
10:02:00	4137			0.12	
12:25:30	4281			0.11	
13:10:00	4325	0	-	0.08	Pump OFF ~~Recovery Initiate~~
13:10:15	4325.25	0.25	17301	0.08	
13:10:30	4325.50	0.50	8651	0.085	
45	4325.75	0.75	5767	0.085	
11:00	4326.00	1.0	4326	0.085	
15	4326.25	1.25	3461	0.085	
30	4326.50	1.50	2904	0.085	
45	4326.75	1.75	2472	0.085	
13:12:00	4327.0	2.00	2163.5	0.085	
12:30	4327.5	2.50	1731	0.085	
13:00	4328.0	3.00	1443	0.08	
13:30	4328.5	3.50	1237	0.08	
14:00	4329.0	4.00	1082	0.08	
14:30	4329.5	4.50	962	0.075	
15:00	4330.0	5.00	866	0.075	
16:00	4331.0	6.00	722	0.075	
17:00	4332	7.00	619	0.075	
18:00	4333	8.00	542	0.075	
19:00	4334	9.00	482	0.075	
13:20:00	4335	10.00	434	0.075	
21:00	4336	11.00	394	0.075	
22:00	4337	12:00	361	0.075	
23:00	4338	13.00	334	0.07	
24:00	4339	14.00	310	0.07	
25:00	4340	15.00	289	0.07	
27:00	4342	17.00	255	0.07	
29:00	4344	19.00	229	0.07	
13:31:00	4346	21.00	[illegible]	[illegible]	

Date: 3/3/87-3/07/87

Pumped Well MW-16

Measurements at Well MW-24

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

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time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
35:00	4350	25.00	174	0.065	
13:40:00	4355	30.00	145	0.06	
13:45:00	4360	35.00	125	0.06	
13:50:00	4365	40.0	109	0.06	
14:00:43	4376	51.0	85.8	0.06	
14:11:05	4386	61.0	71.9	0.06	
14:21:10	4396	71.0	61.9	0.05	
14:31:30	4407	82.0	53.7	0.05	
14:41:31	4417	92.0	48.0	0.045	
14:51:20	4426	101.0	43.8	0.045	
15:01:10	4436	111.0	40.0	0.045	
15:20:53	4456	131.10	34.0	0.04	
15:40:51	4476	151.10	29.6	0.04	
15:55:04	4490	165.0	27.2	0.02	
16:48:34	4544	219.0	20.7	0.02	
18:15:00	4630	305.0	15.1	0.02	
19:07:09	4682	357.0	13.1	0.02	
20:25:11	4760	435.0	10.9	0.02	
22:21	4876	551.0	8.95	0.02	
24:09:00	4984	659.0	7.6	0.01	
2:08	5103	778.0	7.6	0.01	
3:28	5183	858	6.0	0.02	
6:11	5346	1021	5.2	0.03	
9:04	5519	1194	4.6	0.04	Test Terminated

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Pumped Well MW-16

Measurements at Well MW-25

Pump Speed: ________ Q: ________ gpm

Static Water Level 67.59

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
13:05:00				0.02	
13:07:53	8			0.02	
13:13:22	8.5			-.04	
13:20:49	16			0.00	
13:24:20	19.5			0.00	
13:30:36	26			0.00	
13:35:40	31			-0.01	
13:40:27	35.5			0.00	
13:45:23	40.5			-0.01	
13:50:24	45.5			-0.01	
13:55:40	51.0			0.00	
14:06:51	62			0.01	
14:18:03	73			0.01	
14:26:40	82			0.00	
14:36:15	91			0.00	
14:46:53	102			0.01	
14:57:46	113			0.01	
15:16:50	132			0.01	
15:36:20	151.5			0.01	
16:01:40	177			0.01	
16:31:40	207			0.01	
17:01:31	236.5			0.01	
18:02:00	297			0.03	
19:02:10	357			0.03	
20:00:12	415			0.03	
21:01:55	477			0.02	
22:02:24	537			0.02	
23:01:55	597			0.02	
24:06:07	661			0.03	
2:04:02	779			0.02	

Date: 3/3/87-3/07/87

Pumped Well MW-16

Measurements at Well MW-25

Pump Speed: ________ Q: ________ gpm

Static Water Level 67.59

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
4:05:31	901			0.03	
6:06:13	1021			0.03	
9:01:47	1197			0.04	
13:06:46	1442			0.00	
13:56:12	1491			0.01	
15:00:15	1555			0.01	
16:15:00	1630			0.01	
17:01:12	1676			0.02	
18:01:30	1737			0.02	
20:38	1893			0.03	
21:55:30	1971			0.03	
23:03	2038			0.02	
24:04:43	2100			0.03	
2:01:59	2217			0.02	
4:05:15	2340			0.03	
6:06:16	2461			0.03	
8:02:18	2577			0.03	
10:40:00	2735			0.03	
12:01:25	2816			0.02	
14:06:45	2942			0.02	
16:14:00	3069			0.02	
18:13:50	3189			0.02	
20:09:05	3304			0.03	
22:16:00	3431			0.03	
24:05	3540			0.02	
1:54:18	3649			0.02	
4:03:30	3779			0.02	
6:00:00	3895			0.02	
8:02:00	4017			0.02	
10:03:00	4138			0.02	

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Pumped Well MW-16

Measurements at Well MW-25

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
12:27:40	4283			0.01	
13:10:00	4325	0		0.00	Pump OFF
13:23:27	4338	13	333.7	-0.01	
13:27:26	4342	17	255.4	-0.01	
13:32:00	4347	22	197.6	-0.01	
13:35:40	4351	26	167.34	-0.01	
13:40:40	4356	31	140.5	-0.02	
13:45:45	4361	36	121.1	-0.02	
13:50	4365	40	109.1	-0.02	
14:01:25	4376	51	85.8	-0.02	
14:12:33	4388	63	69.7	-0.02	
14:21:50	4398	73	60.2	-0.02	
14:33:00	4409	84	52.5	-0.02	
14:42:25	4418	93	47.5	-0.02	
14:52:26	4428	103	43.0	-0.02	
15:01:41	4438	113	39.3	-0.02	
15:22:11	4458	133	33.5	-0.03	
15:41:40	4478	153	29.3	-0.02	
15:58:50	4495	170	26.4	-0.02	
16:50:07	4546	221	20.6	-0.02	
18:17:02	4633	308	15.0	-0.02	
19:09:00	4685	360	13.0	-0.02	
20:27	4763	438	10.9	-0.02	
22:23	4879	554	8.8	-0.02	
24:11	4987	662	7.5	-0.03	
2:09	5105	780	6.5	-0.03	
3:30	5186	861	6.0	-0.03	
6:12	5348	1023	5.2	-0.03	
9:12	5528	1203	4.6	-0.04	Test Terminated

Date: 3/3/87-3/07/87

Pumped Well MW-16

Measurements at Well MW-17

Pump Speed: ________ Q: ________ gpm

Static Water Level 68.30

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
'3 13:05:00	0			0	
13:09:29	4.			0	
13:16:03	11			0	
13:22:25	17			0	
13:25:27	20			0	
13:31:43	27			0	
13:36:42	32			0	
13:41:34	37			0	
13:46:50	42			0	
13:51:23	46			0	
13:56:45	52			0	
14:04:37	60			0	
14:21:19	76			0	
14:30:00	85			0	
14L39:26	94			0	
14:49:00	104			0	
15:00:04	115			0	
15:20:06	135			0	
15:39:05	154			0	
16:05:15	180			0	
16:35:27	210			0.00	
17:06:00	241			0.01	
18:05:41	301			0.01	
19:08:12	363			0.02	
20:04:18	419			0.01	
21:06:30	482			0	
22:05:50	541			0	
23:05:22	600			0	
0:06:18	661			0	

Date: 3/3/87-3/07/87

OGC-004499

Pumped Well MW-16

Measurements at Well MW-17

Pump Speed: ________ Q: ________ gpm

Static Water Level 68.30

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
2:05:15	780			0	
4:04:37	900			0	
9:03:37	1099			0.01	
13:04:56	1440			-0.01	
14:01:29	1496			-0.02	
15:04:24	1559			-0.02	
16:19:11	1634			-0.01	
17:08:30	1684			0.00	
18:05:00	1740			0.00	
20:43	1898			0.00	
21:59:00	1974			0.00	
23:07	2042			0.00	
24:06:03	2101			0.00	
2:05:14	2220			0.00	
4:00:15	2335			0.00	
6:03:17	2458			0.00	
8:03:40	2579			0.01	
10:41:11	2736			0.01	
12:02:35	2818			0.00	
14:08:55	2944			0.00	
16:17:16	3072			0.00	
18:16:30	3192			0.01	
20:13:00	3308			0.01	
22:18:00	3433			0.01	
24:15:00	3550			0.01	
1:59:00	3654			0.01	
4:09:00	3784			0.01	
6:10:00	3905			0.00	
8:10:00	4025			0.00	
12:28:40	4284			0.00	

Date: 3/3/87-3/07/87

Pumped Well MW-16

Measurements at Well MW-17

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
13:10:00	4325			0.00	
13:56:05	4371			0.00	
14:13:15	4388			0	
14:23:40	4399			0.01	
14:34:33	4410			0.01	
14:44:36	4420			0.01	
14:53:43	4429			0.01	
15:02:43	4438			0.01	
15:23:47	4459			0.0	
5:42:20	4477			0.01	
15:54:00	4489			0.0	
16:47:18	4542			0.0	
18:13:46	4629			0	
19:05:39	4681			0	
20:23:45	4759			0	
22:20:00	4875			0	
24:08:00	4983			0	
2:06	5101			0	
3:20	5175			0	
6:10	5345			-0.01	
9:33	5548			-0.02	Test Terminated

OGC-004501

Date 3/10/87-3/13/87

CONSTANT DRAWDOWN AQUIFER TEST

WELL MW-24

DRAWDOWN 3.26' = S_w

Static Water Level: 68.40'

time (h:m:s)	t (min)	t' (min)	t/t'	$\frac{t}{r_w^2}$ (m/ft^2)	Discharge (gpm)	Total Water Removed (gal)	$\frac{s_w}{Q}$ (ft·m/gal)
12:16	16			1634	0.258		12.6
12:22	22			2247	0.256		12.8
12:27	27			2757	0.251		13.0
12:31	31			3166	0.243		13.4
12:36	36			3676	-		-
12:40	40			4085	0.243		13.4
12:50	50			5106	0.235		13.9
13:01	61			6229	0.241		13.5
13:13	73			7454	0.246		13.3
13:18	78			7965	0.241		13.[illegible]
13:34	94	sampled		9599	0.263		12.4
14:25	145			14807	0.260		12.5
14:38	158			16134	0.270		12.1
15:01	181			18483	0.265		12.3
15:32	212			21649	0.257	50	12.7
16:04	244			24916	0.248		13.2
16:36	276			28184	0.242		13.5
17:03	303			30941	0.258		12.6
18:03	363			37068	0.255		12.8
19:03	423			43195	0.239	100	13.6
20:03	483			49322	0.273		11.9
21:03	543			55449	0.252		13.0
22:03	603			61576	0.252		13.0
23:03	663			67703	0.240	150	13.6
24:00	720	sampled		73524	0.231		14.[illegible]
1:00	780			79651	0.245		13.3
2:00	840			85778	0.239		13.6
3:00	900			91905	0.244	200	13.4
4:00	960			98032	0.243		13.4
5:00	1020			[illegible]			

Date 3/10/87-3/13/87

CONSTANT DRAWDOWN AQUIFER TEST

WELL MW-24

DRAWDOWN 3.26'

Static Water Level: 68.40'

time (h:m:s)	t (min)	t' (min)	t/t'	$\frac{t}{r_w^2}$ (m/ft^2)	Discharge (gpm)	Total Water Removed (gal)	$\frac{s_w}{Q}$ (ft·m/ga
6:00	1080	sampled		110286	0.242		13.5
7:00	1140			116413	0.247	250	13.2
8:00	1200			122540	0.233		14.0
9:00	1260			128667	0.236		13.8
10:00	1320			134794	0.235		13.9
11:00	1380			140921	0.239	300	13.6
12:00	1440			147048	0.240		13.6
13:00	1500			153175	0.235		13.9
14:00	1560			159301	0.236	350	13.8
15:00	1620			165428	0.233		14.0
[illegible]:00	1680			171555	0.231		14.1
17:00	1740			177682	0.246		13.3
18:00	1800			183809	0.238	400	13.7
19:00	1860			189936	0.244		13.4
20:00	1920			196063	0.243		13.4
21:00	1980			202190	0.243		13.4
22:00	2040			208317	0.241	450	13.5
23:00	2100			214444	0.239		13.6
24:00	2160			220571	0.225		14.5
1:00	2220			226698	0.223	500	14.7
2:00	2280			232825	0.218		15.0
3:00	2340			238952	0.223		14.7
4:00	2400			245079	0.222		14.7
5:00	2460			251206	0.219	525	14.9
6:08	2528			258150	0.214		15.2
7:05	2585			263971	0.218	545	15.0
[illegible]07	2647			270302	0.256		12.8
9:05	2705			276225	0.253	585	12.9
10:06	2766			282454	0.251		13.0
11:00	2820			287[illegible]68	0.254		

OGC-004502

Date 3/10/87-3/13/87

CONSTANT DRAWDOWN AQUIFER TEST

WELL MW-24

DRAWDOWN 3.26'

Static Water Level 68.40'

time (h:m:s)	t (min)	t' (min)	t/t'	$\frac{t}{r_w^2}$ (m/ft^2)	Discharge (gpm)	Total Water Removed (gal)	$\frac{s_w}{Q}$ (ft·m/ga
12:06	2886	sampled		294708	0.256	635 gal	12.8
13:04	2944			300631	0.258		12.6
14:03	3003			306655	0.249		13.1
15:06	3066			313089	0.236		13.8
16:03	3123			318909	0.231		14.1
17:04	3184			325138	0.233		14.0
18:04	3244			331265	0.227		14.3
19:01	3301			337086	0.232	685 gal	14.0
20:06	3366			343724	0.235		13.9
21:03	3423			349544	0.248		13.[illegible]
22:04	3484			355773	0.233		14.0
23:03	3543			361798	0.248	735	13.2
24:00	3600	sampled		367619	0.245		13.3
1:00	3660			373746	0.241		13.5
2:00	3720			379873	0.244		13.4
3:00	3780			386000	0.242	790	13.5
4:00	3840			392127	0.240		13.6
5:00	3900			398254	0.240		13.6
6:00	3960			404381	0.244		13.4
7:00	4020			410508	0.245	845	13.3
8:00	4080			416635	0.228		14.3
9:00	4140			422762	0.227		14.4
10:00	4200			428889	0.229		14.2
11:00	4260			-	0.507	Pump speed to 75	-
11:11	4271			436139	0.226		14.4
12:00	4320			441143	0.220		14.[illegible]
13:00	4380			447270	0.209		15.6
13:10	4390	0				900	Pump OFF

OGC-004503

Date: 3/10/87-3/13/87

Pumped Well MW-24

Measurements at Well MW-24

Pump Speed: - Q: - gpm

Static Water Level 68' 4 3/4"

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
13:15	4395	5	879	0.85	
13:17	4397	7	628	0.72	
13:17:30	4397.5	7.5	586	0.64	
13:18	4398	8	550	0.57	
13:18:30	4398.5	8.5	517	0.48	
13:19	4399	9	489	0.42	
13:19:30	4399.5	9.5	463	0.38	
13:20	4400	10	440	0.32	
13:20:30	4400.5	10.5	419	0.28	
13:21	4401	11	400	0.25	
13:21:30	4401.5	11.5	383	0.22	
13:22	4402	12	367	0.20	
13:22:30	4402.5	12.5	352	-	
13:23	4403	13	339	0.16	
13:23:30	4403.5	13.5	326	0.15	
13:24	4404	14	315	0.14	
13:24:30	4404.5	14.5	304	0.13	
13:25	4405	15	294	0.10	
13:25:30	4405.5	15.5	284	0.10	
13:26	4406	16	275	0.10	
13:26:30	4406.5	16.5	267	0.10	
13:27	4407	17	259	0.09	
13:28	4408	18	245	0.09	
13:29	4409	19	232	0.09	
13:30	4410	20	221	0.08	
13:31	4411	21	210	0.08	
13:32	4412	22	201	0.07	
13:33	4413	23	192	0.07	
13:34	4414	24	184	0.07	

Date: 3/10/87-3/13/87

Pumped Well MW-24

Measurements at Well MW-24

Pump Speed: - Q: - gpm

Static Water Level 68' 4 3/4"

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
13:35:00	4415	25	177	0.06	
13:36	4416	26	170	0.06	
13:37	4417	27	164	0.05	
13:40:00	4420	30	147	0.05	
13:42	4422	32	138	0.05	
13:45	4425	35	126	0.05	
13:46	4426	36	123	0.05	
13:48	4428	38	117	0.05	
13:50	4430	40	111	0.05	
13:55:00	4435	45	99	0.05	
14:00:00	4440	50	89	0.04	
14:10:00	4450	60	74	0.04	
14:20:00	4460	70	64	0.04	
14:30:00	4470	80	56	0.03	
14:40	4480	90	50	0.02	
14:50	4490	100	45	0.02	
14:10	4510	120	38	0.02	
15:35	4535	145	31	0.01	
15:50	4550	160	28	0.01	
16:10	4570	180	25	0	
16:40	4600	210	22	0	
17:10	4630	240	19	0	
17:40	4660	270	17	0	
18:10	4690	300	16	0.01	
19:10	4750	360	13	0.02	

Date: 3/10/87-3/13/87

Pumped Well MW-24

Measurements at Well MW-16

Pump Speed: - Q: - gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
12:00:00	0.25			0.00	Pump ON
	0.50			0.00	
	0.75			0.00	
	1.0			0.00	
	1.5			0.00	
	2.0			0.00	
	3.0			0.01	
	4.0			0.02	
	5.0			0.03	
	6.0			0.03	
	7.0			0.04	
	8.0			0.04	
	9.0			0.05	
	10.0			0.06	
	12.0			0.06	
	14.0			0.06	
	16.0			0.06	
	18.0			0.06	
	20.0			0.08	
	25.0			0.08	
12:30:00	30.0			0.08	
	35.0			0.08	
	42.0			0.08	
	45.0			0.08	
	50.0			0.08	
13:00:00	60.0			0.08	
	70.0			0.09	
	80.0			0.09	
13:30:00	90.0			0.10	
	100			0.11	

Date: 3/10/87-3/13/87

Pumped Well MW-24

Measurements at Well MW-16

Pump Speed: - Q: - gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
	120			0.12	
14:20:00	140			0.12	
	160			0.12	
15:00:00	180			0.13	
	210			0.13	
16:00:00	240			0.14	
	270			0.14	
17:00:00	300			0.14	
18:00:00	360			0.15	
19:00	420			0.17	
20:00	480			0.17	
21:00	540			0.17	
22:00	600			0.18	
23:00	660			0.18	
24:00	720			0.18	
1:02	782			0.18	
2:02	842			0.17	
3:03	903			0.16	
4:02	962			0.18	
5:02	1022			0.18	
6:02	1082			0.18	
7:02	1144			0.18	
8:02	1202			0.19	
9:02	1262			0.18	
10:01	1321			0.17	
11:02	1382			0.18	
11:45	1425			0.14	Pump Speed to 40
12:00	1440			0.15	Pump Speed to 50 @ 11:46
13:00	1500			0.16	
14:00	1560			0.15	

Date: 3/10/87-3/13/87

Pumped Well MW-24

Measurements at Well MW-16

Pump Speed: - Q: - gpm

Static Water Level ______

	time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
1	15:00	1620			0.16	
	16:00	1680			0.16	
	17:00	1740			0.17	
	18:00	1800			0.18	
	19:00	1860			0.19	
	20:00	1920			0.19	
	21:00	1980			0.20	
	22:00	2040			0.20	
	23:00	2100			0.20	
	24:00	2160			0.20	
2	1:01	2221			0.20	
	2:01	2281			0.21	
	3:01	2341			0.20	
	4:01	2401			0.20	
	5:01	2461			0.20	
	6:10	2530			0.20	
	7:02	2582			0.20	
	8:04	2644			0.21	
	9:01	2701			0.22	
	10:01	2761			0.22	
	12:05	2885			0.20	
	13:00	2940			0.19	
	14:00	3000			0.19	
	15:02	3062			0.19	
	16:00	3120			0.19	
	17:03	3183			0.19	
	18:00	3240			0.21	
	19:00	3300			0.21	
	20:01	3361			0.22	
	21:00	3420			0.22	

OGC-004508

Date: 3/10/87-3/13/87

OGC-004509

Pumped Well MW-24

Measurements at Well MW-16

Pump Speed: - Q: - gpm

Static Water Level

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
22:00	3480			0.21	
23:00	3540			0.22	
0:02	3602			0.22	
1:01	3661			0.22	
3:01	3781			0.22	
4:01	3841			0.22	
5:01	3901			0.22	
6:01	3961			0.23	
7:01	4021			0.23	
8:01	4081			0.23	
9:01	4141			0.22	
10:01	4201			0.20	
11:01	4261			0.19	
12:00	4320			0.19	
13:00	4380			0.16	
13:00	4390	0	-	0.16	Pump OFF
	4390.25	.25	17561	0.16	
	4390.50	.50	8781	0.16	
	4390.75	.75	5854	0.16	
13:11	4391.0	1.0	4391	0.16	
	4391.5	1.5	2928	0.16	
13:12	4392.0	2.0	2196	0.16	
	4392.5	2.5	1757	0.16	
13:13	4393.0	3.0	1464	0.16	
	4393.5	3.5	1255	0.16	
13:14	4394.0	4.0	1099	0.16	
	4394.5	4.5	977	0.155	
13:15	4395.0	5.0	879	0.155	
13:16	4396	6	733	0.15	
13:17	4397	7	628	0.15	

Date: 3/10/87-3/13/87

Pumped Well MW-24

Measurements at Well MW-16

Pump Speed: - Q: - gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
13:18	4398	8	550	0.15	
13:19	4399	9	489	0.14	
13:20	4400	10	440	0.135	
	4401	11	400	0.13	
	4402	12	367	0.125	
	4404	14	315	0.115	
	4406	16	275	0.11	
13:30	4408	18	245	0.10	
13:20	4410	20	221	0.10	
	4415	25	177	0.085	
	4420	30	147	0.075	
	4425	35	126	0.065	
	4430	40	111	0.065	
	4435	45	99	0.065	
14:00	4440	50	89	0.065	
	4450	60	74	0.06	
	4460	70	64	0.06	
14:30	4470	80	56	0.06	
	4480	90	50	0.055	
15:00	4490	100	45	0.05	
	4510	120	38	0.05	
	4534	144	31	0.05	
	4550	160	28	0.04	
16:10	4570	180	25	0.04	
	4600	210	22	0.03	
17:10	4630	240	19	0.03	
	4660	270	17	0.03	
18:10	4690	300	16	0.05	
19:10	4750	360	13	0.06	End of Test

OGC-004510

Date: 3/10/87-3/13/87

Pumped Well MW-24

Measurements at Well MW-25

Pump Speed: - Q: - gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
12:04				0.00	
12:07				0.01	
12:09				0.01	
12:13				0.01	
12:17				0.01	
12:19				0.01	
12:26				0.02	
12:33				0.01	
12:51				0.01	
13:02				0.01	
13:08				0.01	
13:14				0.01	
13:30				0.01	
13:40				0.01	
14:01				0.02	
14:21				0.02	
14:38				0.02	
15:01				0.03	
15:31				0.03	
16:00				0.04	
16:33				0.04	
17:01				0.05	
18:03				0.05	
19:03				0.06	
20:02				0.06	
21:01				0.06	
22:01				0.07	
23:01				0.07	
24:01				0.07	
1:01				0.07	

Date: 3/10/87-3/13/87

Pumped Well MW-24

Measurements at Well MW-25

Pump Speed: - Q: - gpm

Static Water Level

	time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
1	2:01				0.06	
	3:03				0.05	
	4:02				0.05	
	5:01				0.06	
	6:01				0.06	
	7:02				0.07	
	8:02				0.07	
	9:01				0.07	
	10:01				0.07	
	11:01				0.06	
	12:01	± 4320			0.04	
	13:01				0.05	
	14:01				0.04	
	15:01				0.05	
	16:01				0.05	
	17:01				0.06	
	18:01				0.07	
	19:01				0.08	
	20:01				0.08	
	21:01				0.09	
	22:01				0.08	
	23:01				0.08	
	24:01				0.09	
2	1:01				0.08	
	2:01				0.09	
	3:01				0.09	
	4:01				0.09	
	5:01				0.09	
	6:10				0.09	
	7:01				0.09	

Date: 3/10/87-3/13/87

Pumped Well MW-24

Measurements at Well MW-25

Pump Speed: - Q: - gpm

Static Water Level ______

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
8:03				0.09	
9:00				0.09	
10:01				0.09	
12:04	2days			0.07	
13:01				0.06	
14:01				0.06	
15:04				0.05	
16:01				0.07	
17:03				0.08	
18:01				0.06	
19:01				0.07	
20:01				0.07	
21:01				0.06	
22:01				0.07	
23:01				0.06	
24:01				0.06	
1:01				0.05	
2:01				0.07	
3:01				0.07	
4:01				0.09	
5:01				0.09	
6:00				0.09 -	
7:01				0.07	
8:01				0.06	
9:01				0.07	
10:01				0.06	
11:01				0.06	
12:01				0.05	
1:00	= 4320 + 60			0.05	
1:10	438				Pump OFF

OGC-004513

Pumped Well MW-24

Measurements at Well MW-25

Pump Speed: - Q: - gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
13:44				-0.03	
13:48				-0.01	
13:56				-0.01	
14:01				-0.01	
14:06				-0.01	
14:16				-0.01	
14:31				-0.01	
14:41				-0.01	
14:51				-0.02	
15:11				-0.01	
15:37				-0.02	
15:51				-0.02	
16:11				-0.02	
16:41				-0.02	
17:11				-0.02	
17:41				-0.02	
18:11				-0.01	
19:11				0.00	End of test

Date: 3/10/87-3/13/87

OGC-004515

Pumped Well MW-24

Measurements at Well ,W-17

Pump Speed: - Q: - gpm

Static Water Level ________

	time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
0	12:00				0.00	Pump ON
	12:11				0.00	
	12:21				0.00	
	12:31				-0.01	
	12:58				-0.01	
	13:16				0.00	
	13:32				0.00	
	14:02				0.00	
	14:36				0.00	
	15:03				0.00	
	15:44				0.00	
	16:35				0.00	
	17:04				0.02	
	18:04				0.02	
	19:05				0.03	
	20:06				0.02	
	21:04				0.03	
	22:05				0.03	
	23:03				0.03	
	24:07				0.03	
1	1:05				0.03	
	2:06				0.03	
	3:06				0.03	
	4:07				0.03	
	5:07				0.03	
	6:06				0.04	
	7:07				0.04	
	8:03				0.02	
	9:03				0.04	
	10:03				0.02	

Date: 3/10/87-3/13/87

Pumped Well MW-24

Measurements at Well MW-17

Pump Speed: - Q: - gpm

Static Water Level ______

	time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
1	11:03				0.01	
	12:08				0.01	
	13:03				-0.01	
	14:03				-0.01	
	15:03				0.00	
	16:03				-0.01	
	17:03				0.00	
	18:03				0.01	
	19:03				0.02	
	20:02				0.02	
	21:02				0.03	
	22:03				0.02	
	23:04				0.02	
	24:04				0.03	
2	1:04				0.03	
	2:03				0.03	
	3:03				0.04	
	4:03				0.04	
	5:03				0.03	
	6:20				0.03	
	7:03				0.05	
	8:05				0.05	
	9:02				0.05	
	10:03				0.05	
	11:00				0.03	
	12:10				0.03	
	.3:03				-0.01	
	14:03				0.00	
	15:05				0.01	
	16:05				0.01	

Date: 3/10/87-3/13/87

OGC-004517

Pumped Well MW-24

Measurements at Well MW-17

Pump Speed: - Q: - gpm

Static Water Level

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
17:05				0.01	
18:03				0.03	
19:03				0.03	
20:03				0.05	
21:03				0.02	
22:03				0.03	
23:03				0.03	
24:03				0.03	
1:03				0.03	
2:03				0.03	
3:03				0.03	
4:03				0.03	
5:03				0.04	
6:03				0.04	
7:04				0.03	
8:03				0.02	
9:03				0.01	
10:03				0.00	
11:02				-0.01	
12:00				-0.01	
13:10					Pump OFF
14:03				-0.01	
14:22				-0.01	
15:39				0.00	
16:12				-0.01	
16:44				0.00	
17:05				0.00	
17:12				0.01	
18:12				0.01	
20:30				0.02	End of test

Date: 3/07/87-3/09-87

Pumped Well MW-18

Measurements at Well MW-18

Pump Speed: ________ Q: 0.25 gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
11:00	0			0	Speed 39
	.25			-	
	.5			0.20	
	.75			-	
11:01	1.0			0.35	
	1.5			0.55	
11:02	2.0			0.65	
	2.5			0.80	
11:03	3.0			0.90	
	3.5			1.00	
11:04	4.0			1.10	
	4.5			1.18	
11:05	5.0			1.25	
11:06	6.0			1.39	
11:07	7.0			1.51	
11:08	8.0			1.62	
11:09	9.0			1.74	
11:10	10.0			1.85	
11:11	11			1.96	1ℓ/51.04sec
11.14	14			2.30	Adj to 38
11.16	16			2.45	1ℓ/54.49
11.18	18			2.57	Adj to 37
11:20	20			2.69	1ℓ/57.30sec
11:25	25			2.85	1ℓ/60.70sec
11:30	30			3.06	
11:35	35			3.22	1ℓ/56.70
11:40	40			3.38	1ℓ/56.82
11:45	45			3.47	sampled
11:50	50			3.51	1ℓ/59.03sec
12:00	60			3.58	1ℓ/58.76sec

Date: 3/07/87-3/09/87

Pumped Well MW-18

Measurements at Well MW-18

Pump Speed: ______ Q: 0.25± gpm

Static Water Level ______

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
12:10	70			3.67	
12:20	80			3.73	1ℓ/60.61
12:30:00	90			3.83	1ℓ/56.74
12:40:00	100			3.87	1ℓ/57.83
13:00:00	120			3.89	1ℓ/58.09
13:20:00	140			3.90	1ℓ/58.45
13:40:00	160			3.96	1ℓ/58.23
14:00:00	180			4.19	1ℓ/57.48
14:30:00	210			4.19	1ℓ/56.41
15:00:00	240			4.30	1ℓ/57.03
15:47:00	287			4.20	1ℓ/60.92
16:00	300			4.24	
17:00	360			4.04	1ℓ/60.56
18:00	420			3.53	H_2O sample 66.8 taken
19:00	480			4.02	1ℓ/59.67
20:00	540			3.97	1ℓ/61.43
21:00	600			3.75	1ℓ/59.90
22:00	660			3.27	1ℓ/63.18
23:00	720			4.75	H_2O sample 61.80 taken
24:00	780			4.42	1ℓ/59.24
1:00	840			4.67	1ℓ/60.41
2:00	900			4.50	1ℓ/58.85
3:00	960			4.23	1ℓ/61.07
4:00	1020			4.20	1ℓ/63.46
5:00	1080			4.11	1ℓ/65.20
6:00	1140			4.55	1ℓ/57.15
7:00	1200			5.02	1ℓ/57.40
8:00	1260			6.56	1ℓ/49.55 Adj 61.87
9:00	1320			5.14	1ℓ/58.95 Adj 60.89
10:00	1380			4.48	57.90

Date: 3/07/87-3/09-87

Pumped Well MW-18

Measurements at Well MW-18

Pump Speed: ______ Q: 0.25 gpm

Static Water Level ______

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
11:00	1440			4.50	sampled 56.97 adj 61.36
12:00	1500			4.17	60.27
13:00	1560			4.10	1ℓ/60.71
14:00	1620			3.94	1ℓ/61.95
15:00	1680			4.15	1ℓ/58.43
16:00	1740			3.94	1ℓ/61.31
17:00	1800			4.08	1ℓ/60.46
18:00	1860			3.96	1ℓ/58.95 adj 60.06
19:00	1920			3.92	1ℓ/68.45 adj 60.30
20:00	1980			4.41	1ℓ/60.04
21:00	2040			4.06	1ℓ/59.26
22:00	2100			4.16	1ℓ/58.64
23:00	2160			4.21	H_2O sample taken 68.54 adj 62.64
24:00	2220			4.40	1ℓ/1:05:43 adj 59.
1:00	2280			4.11	61.01
2:00	2340			4.66	58.13
3:00	2400			5.09	60.58
4:00	2460			5.47	57.28 adj 60.06
5:00	2520			5.52	64.70 adj 58.67
6:00	2580			4.80	59.14
7:00	2640			4.88	60.04
8:00	2700			4.97	57.10 adj 61.39
9:00	2760			4.52	59.24
10:00	2820			5.04	58.80
11:00	2880			4.66	67.58 adj 58.95
12:00	2940			5.02	Pump OFF

Date: 3/7/87-3/9/87

OGC-004521

Pumped Well MW-18

Measurements at Well MW-18

Pump Speed: ________ Q: 0.25± gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
12:00:15	2940.25	.25	11761	4.90	
	2940.5	.5	5881	4.81	
	2940.75	.75	3921	4.72	
12:01	2941	1.0	2941	4.63	
	2941.5	1.5	1961	4.44	
12:02	2942	2.0	1471	4.26	
	2942.5	2.5	1177	4.09	
12:03:00	2943	3.0	981	3.91	
	2943.5	3.5	841	3.74	
12:04:00	2944	4.0	736	3.57	
	2944.5	4.5	654	3.40	
12:05	2945	5.0	589	3.23	
	2946	6	491	2.89	
	2947	7	421	2.57	
	2948	8	369	2.27	
	2949	9	328	2.03	
12:10:00	2950	10	295	1.79	
	2952	12	246	1.44	
	2954	14	211	1.16	
	2956	16	185	.93	
	2958	18	164	.75	
12:20:00	2960	20	148	.60	
	2965	25	119	.35	
12:30:00	2970	30	99.0	.25	
	2975	35	85.0	.20	
	2980	40	74.5	.17	
	2985	45	66.3	.15	
	2990	50	59.8	.14	
	3000	60	50.0	.11	
	3010	70	43.0	.11	

Date: 3/7/87-3/9/87

Pumped Well MW-18

Measurements at Well MW-18

Pump Speed: ________ Q: 0.25± gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
13:20	3020	80	37.8	.10	
13:30	3030	90	33.67	.09	
13:40	3040	100	30.40	.12	
14:00	3060	120	25.50	.02	
14:20	3080	140	22.0	.00	End of Test

APPENDIX B

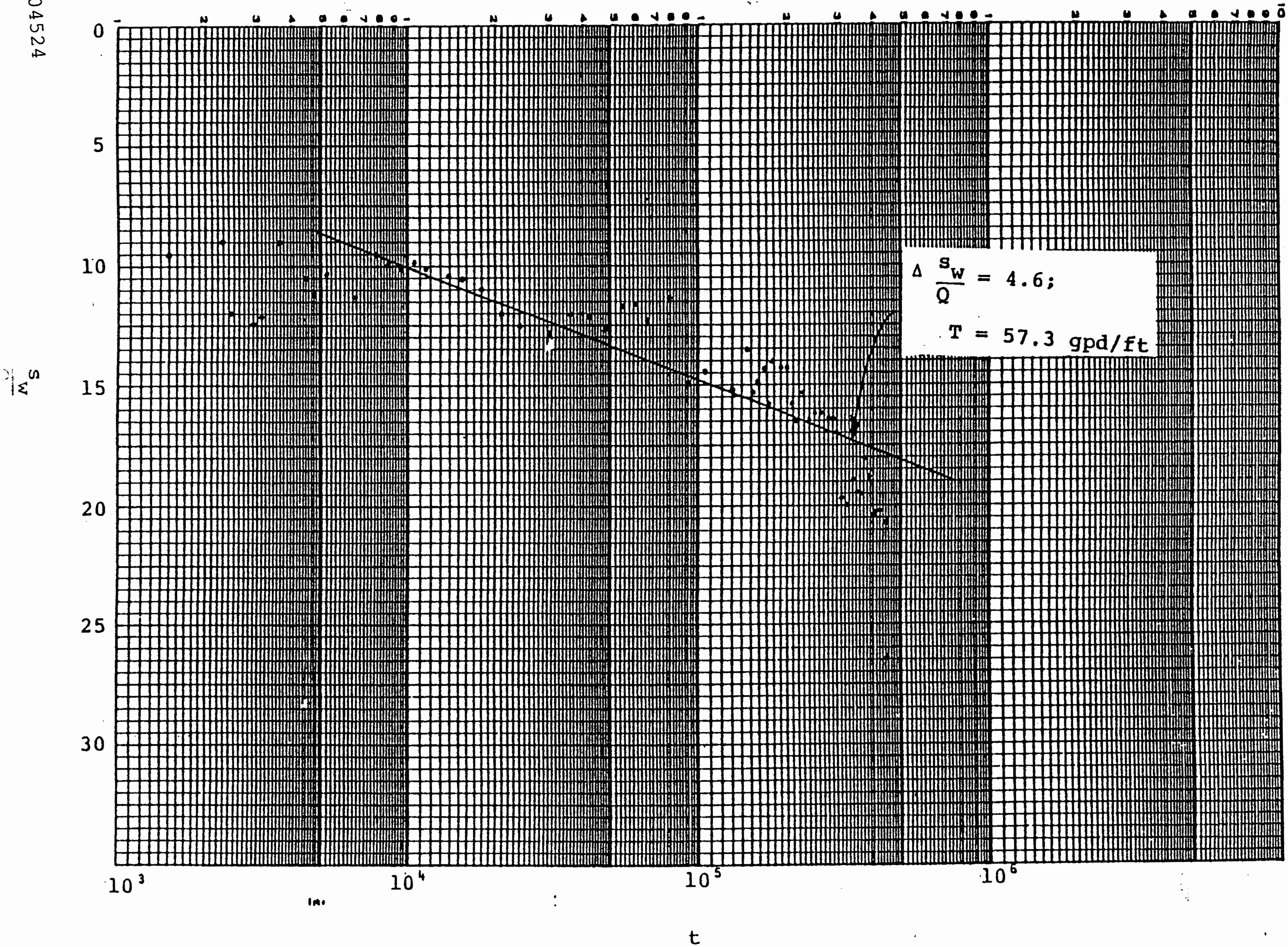
AQUIFER TEST DATA PLOTS

OGC-004524

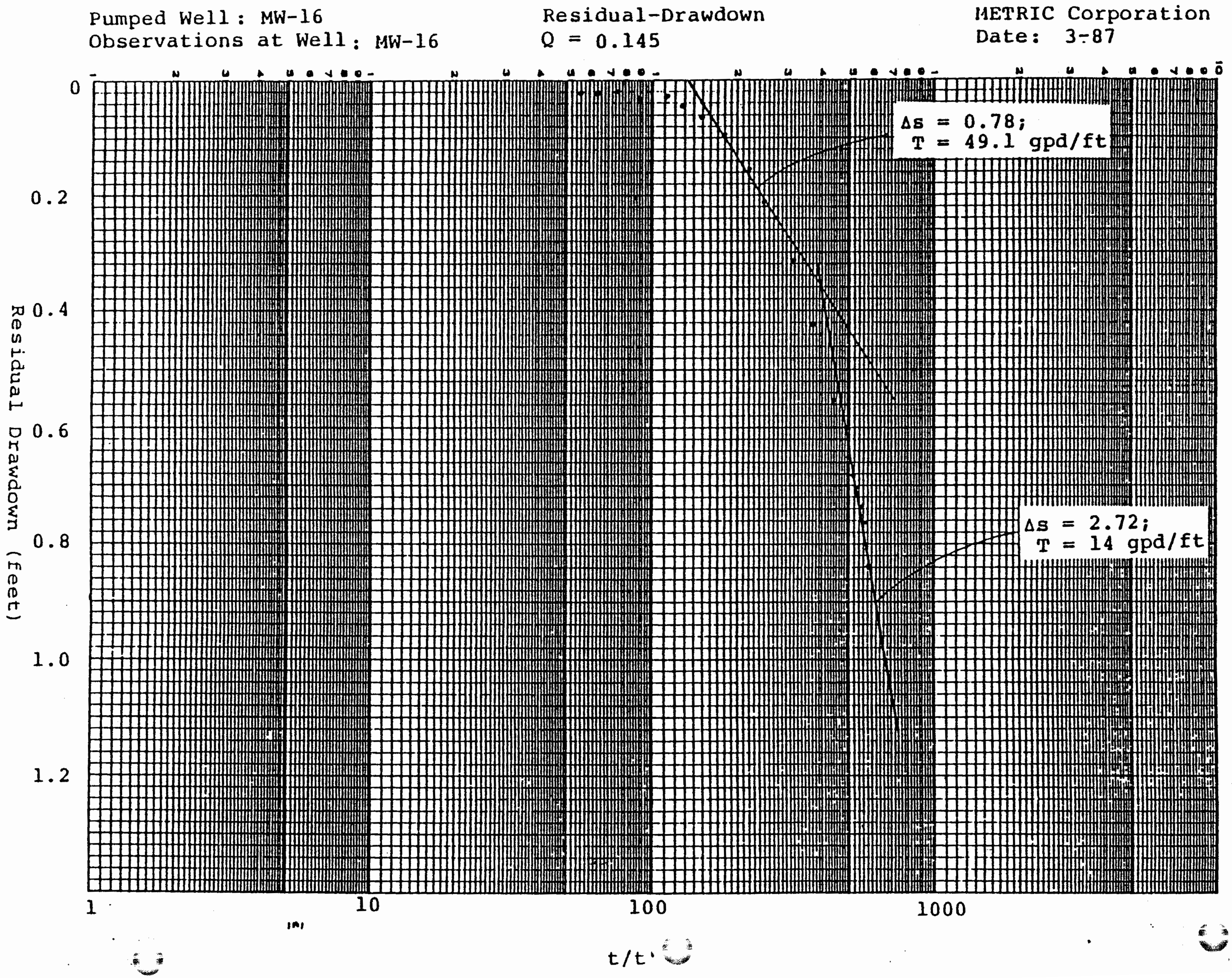
Pumped Well MW-16
Observations at Well MW-16

$\frac{s_w}{Q}$ vs $\frac{t}{r_w^2}$ d = 2 3/8"

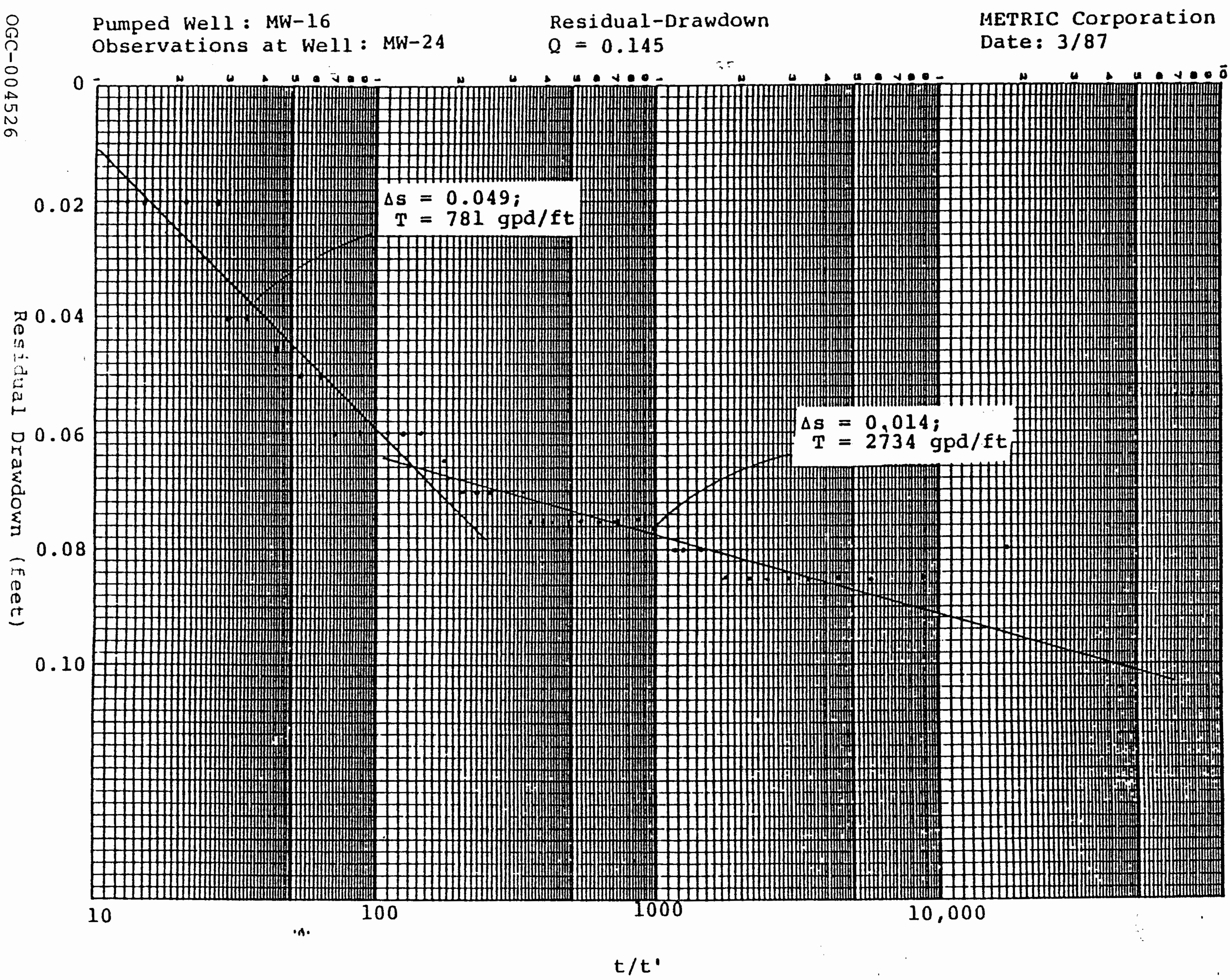
METRIC Corporation
Date: 3-87



OGC-004525



Pumped Well: MW-16
Observations at Well: MW-16
Residual-Drawdown
Q = 0.145
METRIC Corporation
Date: 3-87
Δs = 0.78;
T = 49.1 gpd/ft
Δs = 2.72;
T = 14 gpd/ft
Residual Drawdown (feet)
0
0.2
0.4
0.6
0.8
1.0
1.2
1
10
100
1000
t/t'

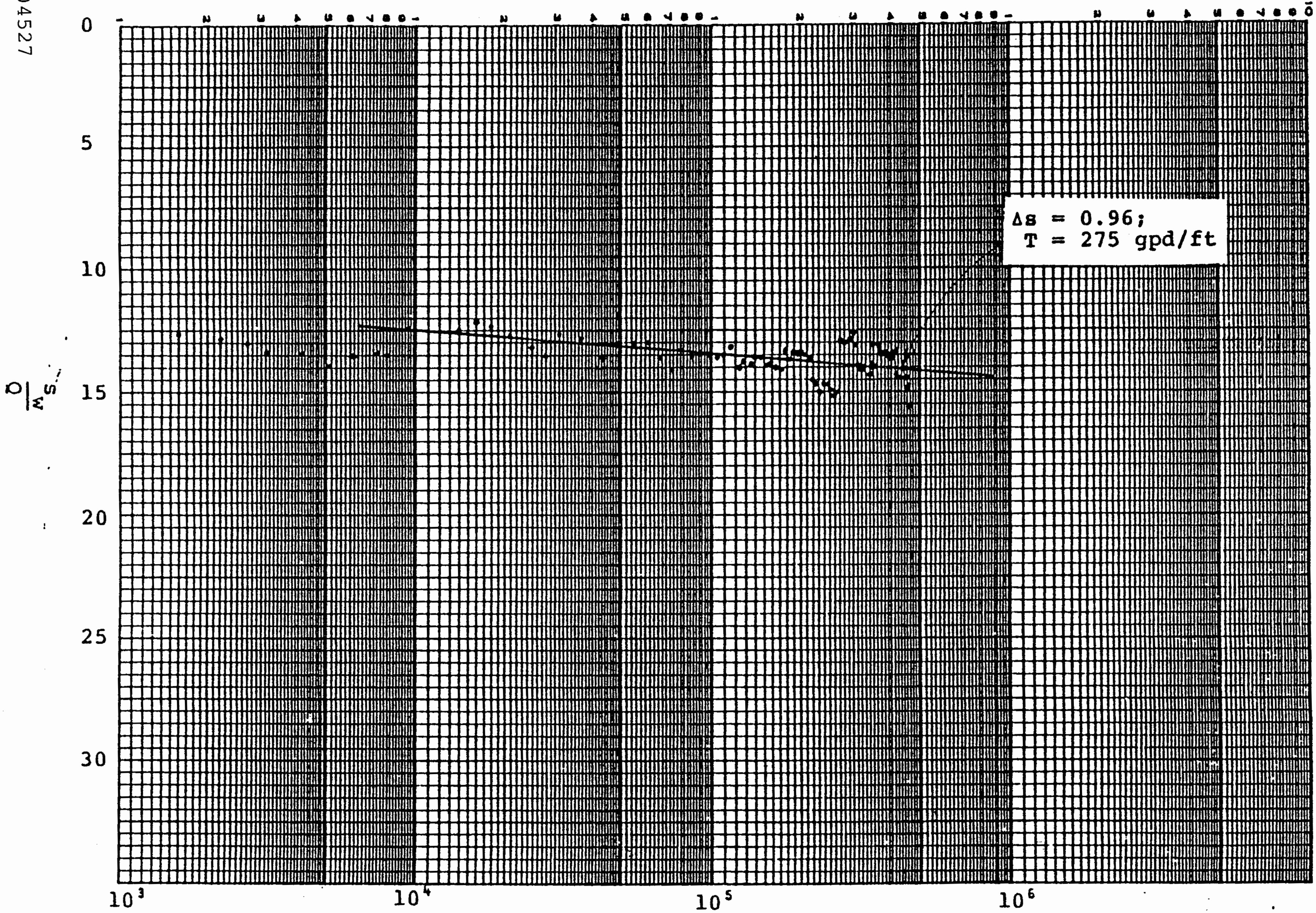

OGC-004526
Pumped Well : MW-16
Observations at Well : MW-24
Residual-Drawdown
Q = 0.145
METRIC Corporation
Date: 3/87
Δs = 0.049;
T = 781 gpd/ft
Δs = 0.014;
T = 2734 gpd/ft
Residual Drawdown (feet)
0
0.02
0.04
0.06
0.08
0.10
10
100
1000
10,000
t/t'

OGC-004527

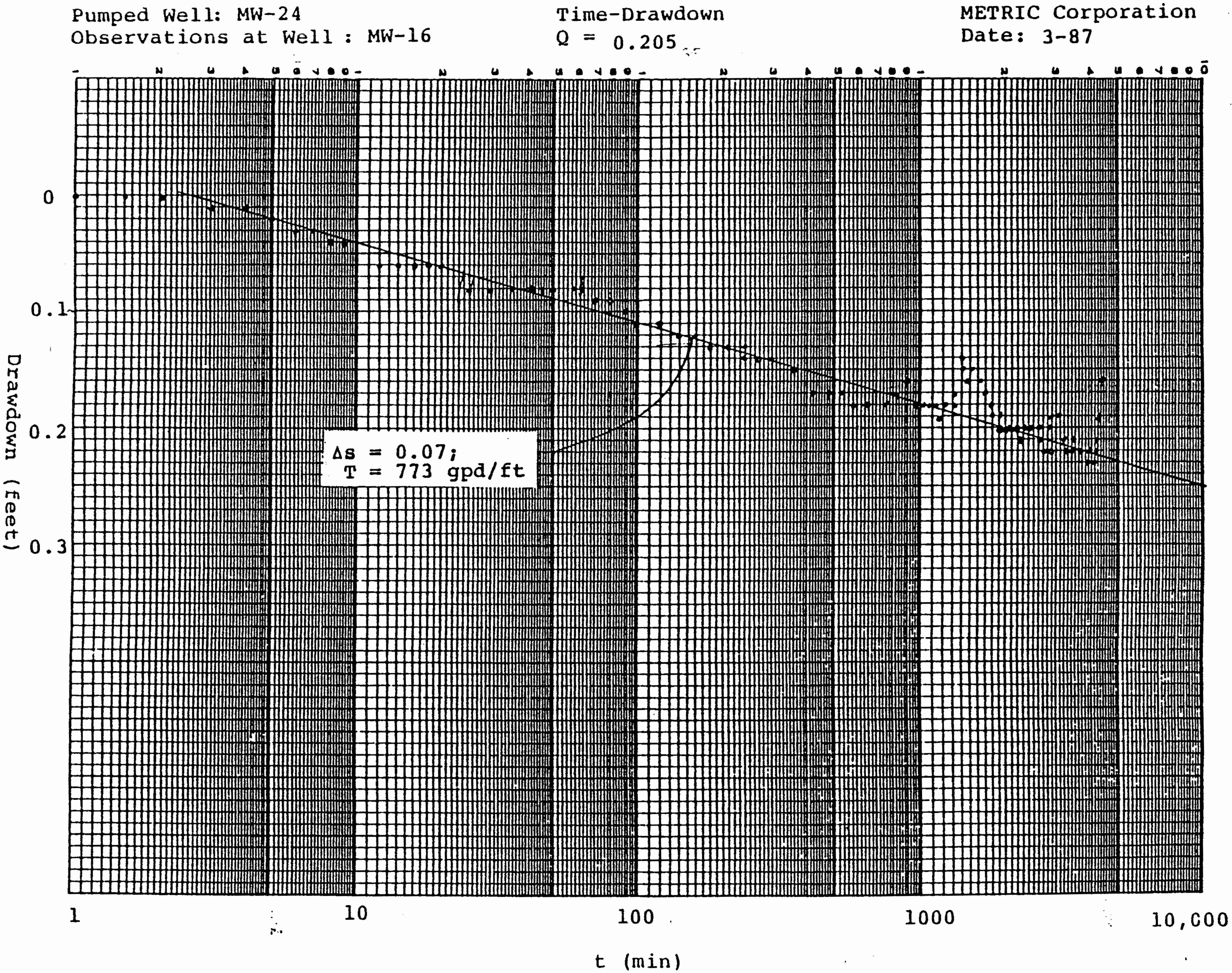
Pumped Well MW-24
Observations at Well MW-24

$\frac{s_w}{Q}$ vs $\frac{t}{r_w^2}$ d = 2 3/8"

METRIC Corporation
Date: 3-87



OGC-004528



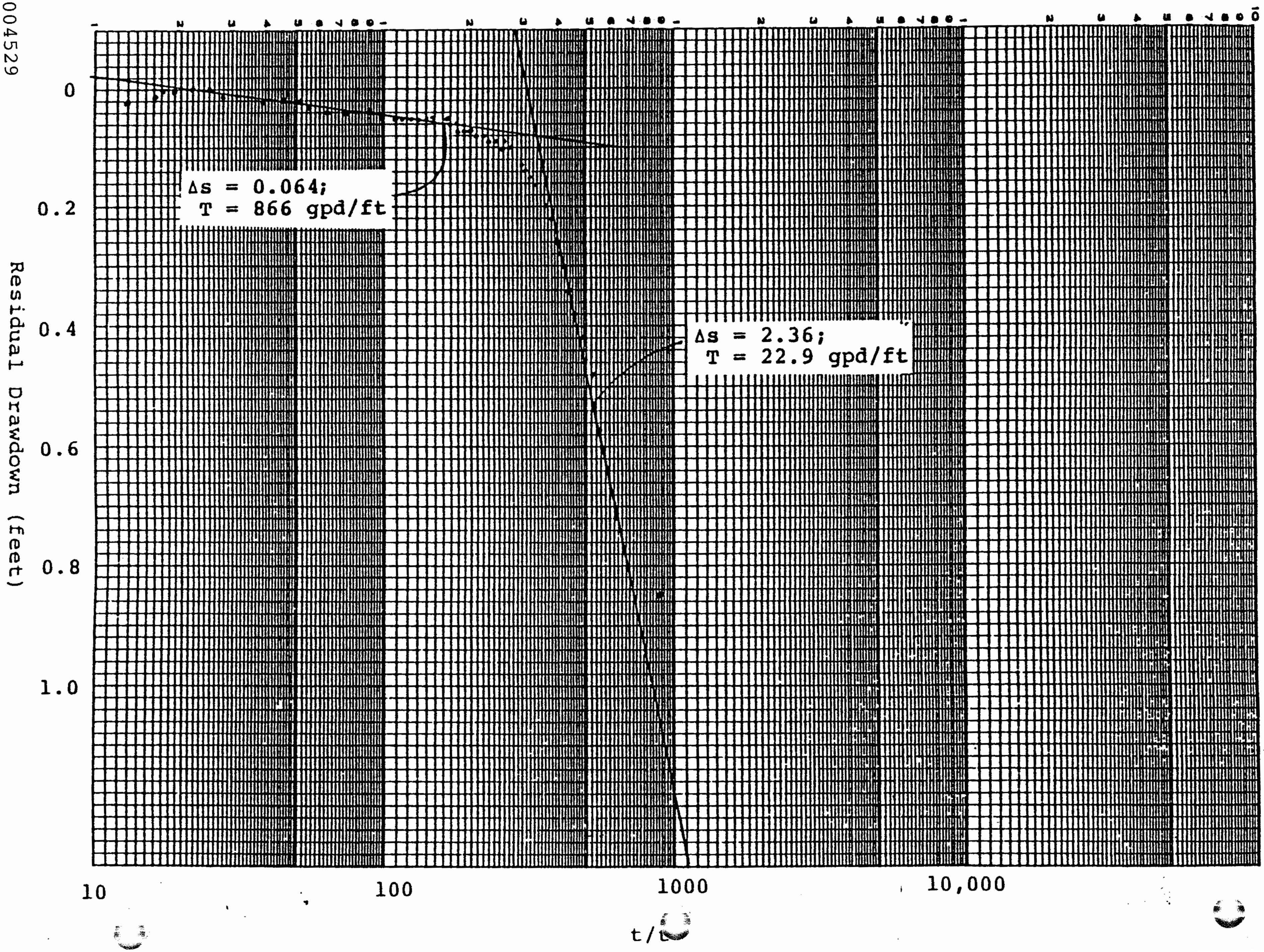
Pumped Well: MW-24
Observations at Well : MW-16
Time-Drawdown
Q = 0.205
METRIC Corporation
Date: 3-87
Δs = 0.07;
T = 773 gpd/ft
Drawdown (feet)
0
0.1
0.2
0.3
1
10
100
1000
10,000
t (min)

OGC-004529

Pumped Well: MW-24
Observations at Well: MW-24

Residual-Drawdown
Q = 0.205

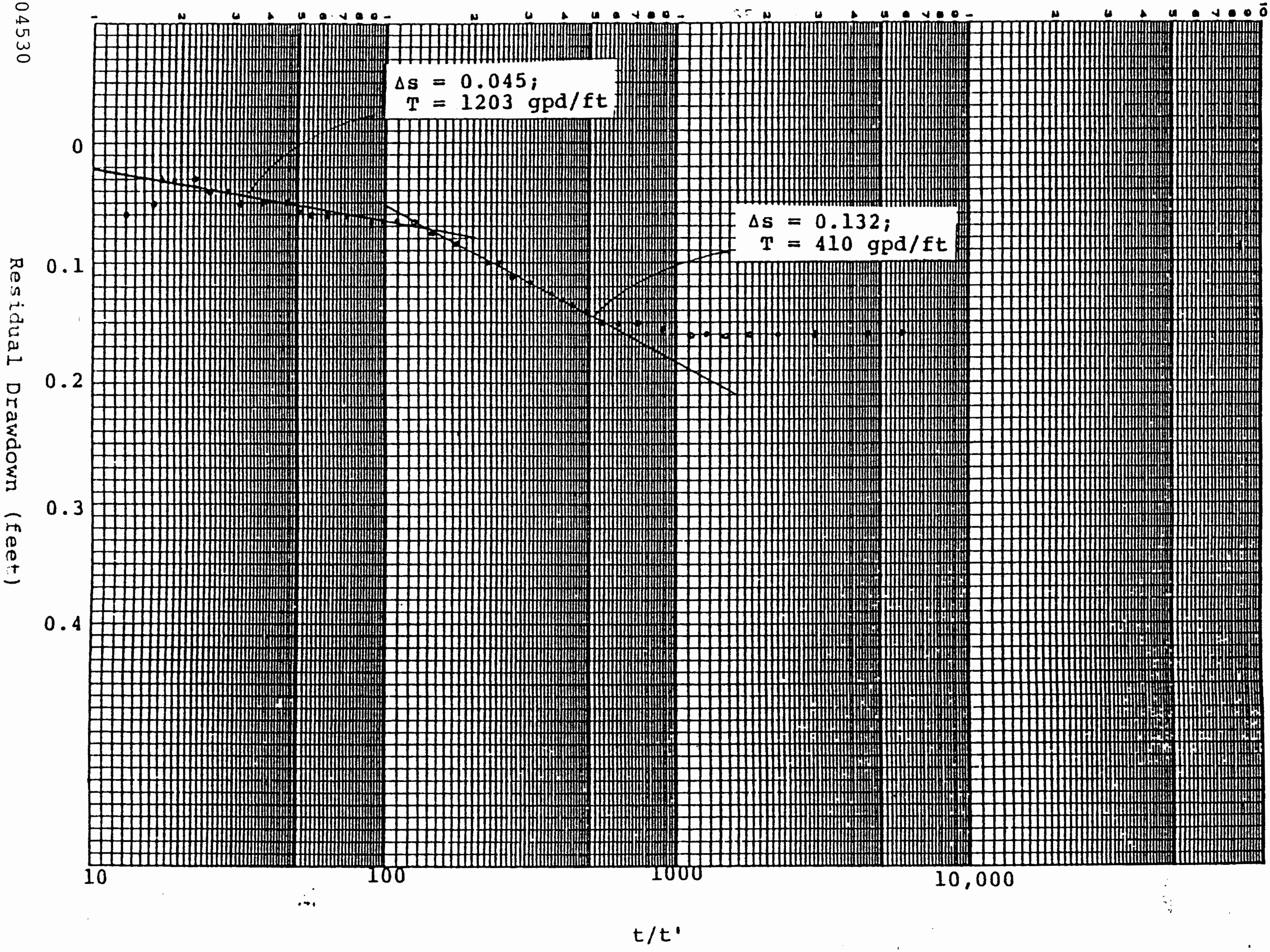
METRIC Corporation
Date: 3-87



Pumped Well: MW-24
Observations at Well: MW-16

Residual-Drawdown
Q = 0.205

METRIC Corporation
Date: 3-87

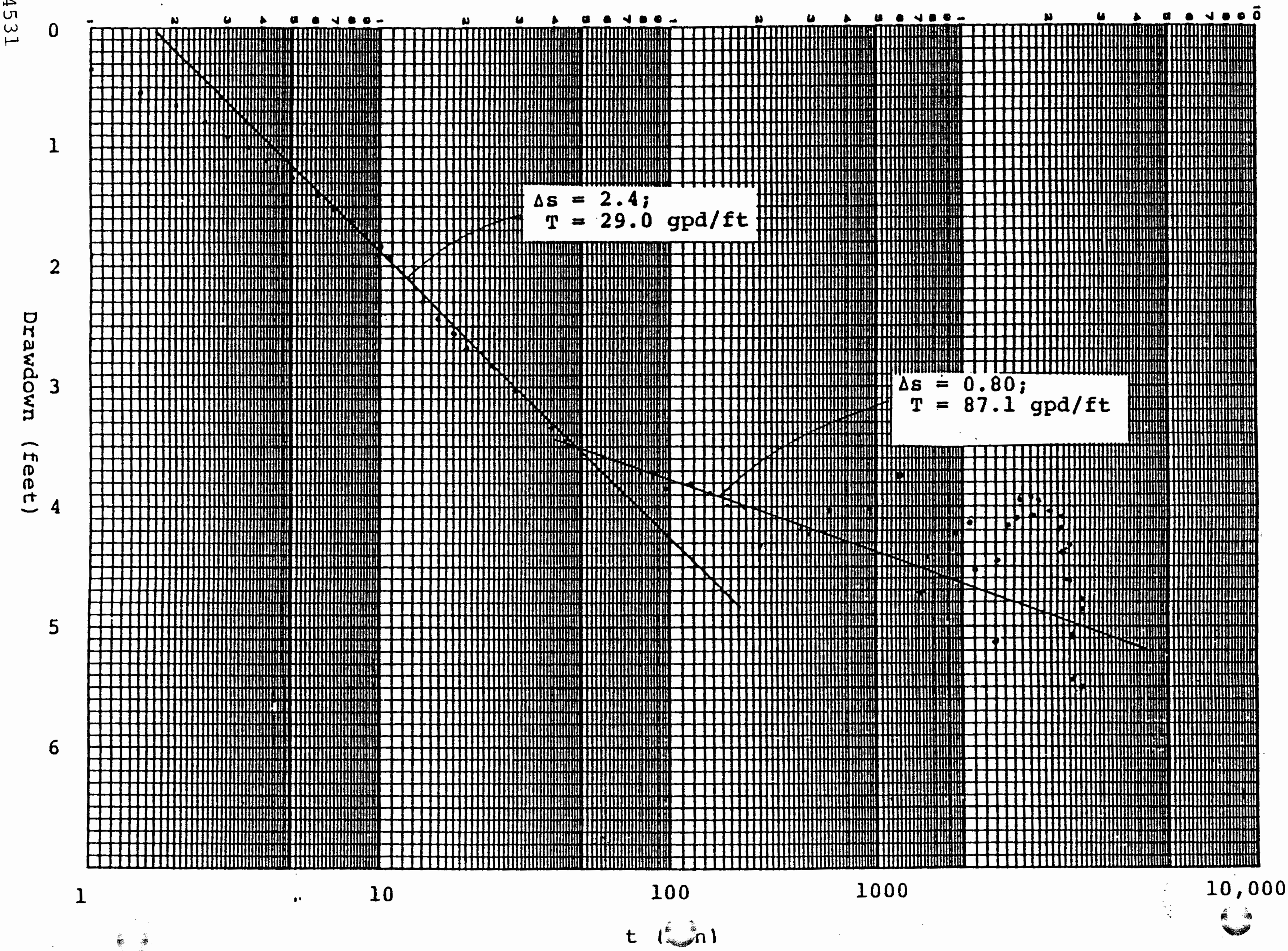


OGC-004531

Pumped Well: MW-18
Observations at Well: MW-18

Time-Drawdown
Q = 0.264

METRIC Corporation
Date: 3-87



APPENDIX C

WATER QUALITY ANALYSES

* NOT INCLUDED *

Aquifer Testing
at the
Sparton Technology, Inc.
Coors Road Plant
Albuquerque, New Mexico

Prepared By

METRIC Corporation
Albuquerque, New Mexico

MAY 1988

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AQUIFER TESTING
AT THE
SPARTON TECHNOLOGY, INC.
COORS ROAD PLANT

Aquifer tests were performed in two wells at the Sparton Technology, Inc., Coors Road Plant during February, 1988. The purpose of the testing was to estimate the aquifer permeability of the "upper flow zone". The resulting information will be used in design of a groundwater recovery system. The "upper flow zone" consists generally of the upper 5 to 10 feet of the saturated zone at the Coors Road site separated from the remainder of the saturated zone by a fine grained aquitard unit.

Pumping tests were conducted in two wells, MW-25 in the pond and sump area on the northeast side of the building and in PW-1 located near the center of the southwest property line.

The tests were conducted as follows:

Well: MW-25
Test Type: Constant Discharge
Test Drawdown: 3.2 ft.
Available Drawdown: 7.3 ft. ±
Duration of Pumping: 4129 min ≈ 69 hr.
Average Discharge: 0.32 gpm
Observations Taken in Wells: MW-25, MW-24

Well: PW-1
Test Type: Constant Discharge
Test Drawdown: 2.26 ft.
Available Drawdown: 4.2 ft. ±
Duration of Pumping: 4174 min ≈ 70 hr.
Average Discharge: 0.13 gpm
Observations Taken in Wells: PW-1, MW-9

Well MW-25, a 2-inch i.d. PVC well with a wirewound stainless steel screen, was pumped with a 1.67-inch o.d. positive displacement pump having a maximum discharge of about 2.5 gpm. Water levels in the pumped well were monitored with an airline and a water monometer using a water/anitfreeze mixture (due to freezing weather) having a specific gravity of 1.06. Water levels in the observation well (MW-24) were monitored with an electronic sounder. All water level measurements were taken to the nearest 0.01 feet.

Well PW-1, a 10-inch i.d. PVC well, was pumped with a 1/2 hp submersible pump having a maximum discharge of about 10 gpm. Water levels in both the pumped well and the observation well (MW-9) were monitored with electronic sounders. All water level measurements were taken to the nearest 0.01 feet.

Water quality samples were collected once per day at a approximate 24 hour intervals, during the aquifer testing and three days after pumping ceased. Pumping tests for both wells were begun on February 23, 1988 and ended on February 26, 1988. The samples collected on February 23 were obtained about one hour after the pumping started. The samples collected on February 26 were obtained about one hour before the pumping was stopped. The February 29 samples were collected about one hour after the pumps were restarted. The purpose of the sampling was to determine whether or not water quality changes with time might be expected when the recovery system is put into operation.

The water level and discharge data collected during each test is presented in APPENDIX A. The results of the aquifer testing are summarized in TABLE 1. The data were analyzed using the Jacob solution (semi-log plots) to the Theis equation (see APPENDIX B).

OGC-004537

TABLE 1

AQUIFER TESTING
SPARTON TECHNOLOGY, INC. COORS ROAD PLANT

Pumped Well	Observations At	Curve	Apparent T (gpd/ft)	Adjusted T (gpd/ft)	b (ft)	ft/day	cm/sec	Comments
MW-25	MW-25	Early T-D	56.3	-	7.3	1.03	3.64×10^{-4}	Near Well
		Late T-D	281.6	-	7.3	5.16	1.82×10^{-3}	Away from Well
		Early R-D	48.3	-	7.3	0.885	3.12×10^{-4}	Near Well
		Late R-D	337.9	-	7.3	6.19	2.18×10^{-3}	Away from Well <u>Selected</u>
PW-1	PW-1	Late T-D	22.8	-	4.3 1/	0.709	2.5×10^{-4}	Casing Storage Affected
		Late R-D	22.8	-	4.3 1/	0.709	2.5×10^{-4}	Casing Storage Affected
Adjusted for Casing Storage Effect				91.5	4.3 1/	2.84	1×10^{-3}	<u>Selected Value</u>

1/ PW-1 has 2' blank below aquifer

The time-drawdown data were checked to ensure that $u<0.05$ and, thus, validate the use of the Jacob solution. In the equation $u = \frac{1.87r^2S}{Tt}$, u was set equal to 0.05, and the time, t, was determined after which the Jacob solution is valid. TABLE 2 shows that the pumped well data are valid while the observation well data are not, and as a result, were not used in the analysis.

The data were also checked using a procedure suggested by Schafer, 1978 to determine which portions of the data might be casing storage affected. Only the first few minutes of the MW-25 data appear to be casing storage affected, while virtually all of the PW-1 data appears to be casing storage affected. As a result, the selected transmissivity value for PW-1 was adjusted (see TABLE 1) by a procedure also suggested by Schafer, 1978 assuming a well afficiency of 100%. This seems justified since only 0.13 gpm was being pumped from a 10-inch well screen with a substantial open area.

Based on the testing described above, it is felt that the best estimate for the permeability (hydraulic conductivity) of the upper flow zone in the vicinity of MW-25 is about 2×10^{-3} cm/sec. (see TABLE 1). Likewise, the best estimate for the permeability of the upper flow zone in the vicinity of PW-1 is about 1×10^{-3} cm/sec. (see TABLE 1).

The residual-drawdown curve (APPENDIX B) for PW-1 shows some evidence that a "recharge effect" may be occurring during the pumping period. The residual drawdown curve shows a t/t' value greater than 2 at zero drawdown, suggesting a "recharge effect". Possible explanations of the apparent "recharge effect" include reduction or reversal of prevailing downward vertical leakage in the cone of depression during the test or induced flow from a more permeable burried channel(s) existing within the upper flow zone.

TABLE 2
JACOB VALIDATION

Well	r (ft.)	T (gpd/ft.)	t (days)	t (min)
PW-1	0.63	91.5	0.03	46
MW-9	20.0	91.5	32.7	47,087
MW-25	0.29	56.3	0.011	16
MW-24	23.0	56.3	70.3	101,207

$$t = \frac{1.87\, r^2}{T} S$$

$S = 0.20$

$u = 0.05$

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Estimated well capacities have been computed for each of the wells being considered for inclusion in the groundwater recovery system (see TABLE 3). The capacities were computed based on specific capacities observed in testing to date and assuming 100% drawdown. This would tend to yield conservatively high values, however, some of the wells might respond favorably to additional development which could increase the capacities beyond the values presented.

The results of the water quality sampling and analyses are summarized in TABLES 4 and 5 and include APPENDIX C. The solvent concentrations appear to have increased with time during the pumping test of MW-25. This possibly indicates that the area of maximum solvent concentration in the ground water is some short distance away from MW-25.

The elevated, and decreasing with time values of TDS, Hardness, and pH observed in PW-1 during the pumping test (see TABLE 5) are probably the result of the bottom portion of the well having recently been plugged with portland cement.

TABLE 3

ESTIMATED WELL CAPACITY

Well	Date	Q (gpm)	Drawdown ft.	Specific Capacity (gpm/ft.)	Pumping Time (min.)	Total Water Removed (gal.)	Available Drawdown (ft.)	Estimated Capacity (gpm)
MW-16	3/7/87	0.145	2.38	0.0609	4325	627	5.4	0.33
MW-18	3/9/87	0.264	5.02	0.0526	2940	776	12.6	0.66
MW-23	10/23/86	0.48	7.22	0.07	32	15	8.76	0.61
MW-24	3/13/87	0.205	3.26	0.0629	4390	900	8.1	0.51
MW-25	2/27/88	0.317	3.0	0.106	4129	1309	7.30	0.77
PW-1	2/27/88	0.13	2.12	0.06	4174	543	4.3	0.26
MW-14								0.75*
A								0.75*
B								0.75*

* Estimated

STOP

TABLE 4

SAMPLE ANALYSIS

MW-25

Parameter	Date Sampled 2-23-88	2-24-88	2-25-88	2-26-88	2-29-88
Cyanide (mg/ℓ)	ND	ND	ND	ND	ND
TDS (mg/ℓ)	820	960	900	900	860
Hardness (mg/ℓ)	132	253	828	288	288
pH (pH units)	7.4	7.5	7.4	7.4	7.4
Total Chromium (mg/ℓ)	0.036	ND	0.036	ND	0.027
Methylene Chloride (ug/ℓ)	3,800	9,400	2,800	2,700	4,200
1,1-Dichloroethylene (ug/ℓ)	1,500	3,400	1,900	2,200	2,100
1,1,1-Thrichloroethane (ug/ℓ)	24,000	39,000	37,000	35,000	42,000
Thrichloroethene (ug/ℓ)	32,000	54,000	46,000	43,000	47,000

ND - Not Detected

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TABLE 5

SAMPLE ANALYSIS

PW-1

Parameter	Date Sampled 2-23-88	2-24-88	2-25-88	2-26-88	2-29-88
Cyanide (mg/ℓ)	ND	ND	ND	ND	ND
TDS (mg/ℓ)	1,200	1,000	840	650	680
Hardness (mg/ℓ)	469	352	321	196	179
pH (pH units)	12	11.7	11.3	10.1	11
Total Chromium (mg/ℓ)	0.033	0.029	0.031	0.026	0.029
Methylene Chloride (ug/ℓ)	16,000	16,000	11,000	10,000	10,000
1,1-Dichloroethylene (ug/ℓ)	980	990	560	670	610
1,1,1-Thrichloroethane (ug/ℓ)	2,100	2,100	1,700	1,700	1,400
Thrichloroethene (ug/ℓ)	8,000	9,000	7,200	7,400	6,200

*ND - Not Detected

BIBLIOGRAPHY

Schafer, David C., 1978, Casing Storage Can Affect Pump Testing Data, The Johnson Drillers Journal.

APPENDIX A

PUMP TEST DATA

2/23 - 2/27

Well MW-25

Q 50 sec/ℓ

Static Water Level

time (h:m:s)	t (min)	t' (min)	t/t'	Pressure (PSIG)	Drawdown (ft)	Meter (gal)	
16:11:00	0			1.85	0.000		Pum
16:11:30	.5			1.85	0.053		~~pH~~ sa
16:12:00	1.0			1.90	0.032		
16:12:30	1.5			1.90	0.021		
16:13	2.0			1.90	-0.021		
16:14	3.0			1.80	0.329		
16:15	4.0			1.75	0.445		
16:16	5.0			1.65	0.594		
16:17	6.0			1.20	1.283		
16:19	8.0			--	2.915		
16:21	10.0			--	3.265		47se
:26	15.0			--	3.710		
16:31	20.0				3.095		47se
16:34:30	23.5				2.533		
16:37:30	26.5				2.449		52se
16:41	30.0				--		
16:56	45.0				2.555		52se
17:01	50.0				--		
17:11	60.0				3.562		
17:21	70.0				3.042		
17:31	80.0				2.555		40se
17:41	90.0				3.074		49se
18:01	110.0				2.714		52se
18:21	130.0				2.544		52se
18:45	154.0				2.533		52se
19:20	189.0				2.745		52se
19:46	215.0				2.798		51se
46	335.0				2.618		
22:46	395.0				2.873		52se
23:42	451.0				2.565		52se
24:46	515.0				2.777		52se

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Well MW-25

Q 50 sec/ℓ

Static Water Level

e s)	t (min)	t' (min)	t/t'	Pressure (PSIG)	Drawdown (ft)	Meter (gal)	
47	576.0				2.777		52sec/ℓ
46	635.0				2.798		52sec/ℓ
47	696.0				2.841		52sec/ℓ
49	758.0				3.106		50sec/ℓ
47	816.0				2.915		50sec/ℓ
49	878.0				2.830		51sec/ℓ
46	935.0				2.798		52sec/ℓ
48	997.0				2.968		51sec/ℓ
43	1052.0				2.979		52sec/ℓ
55	1124.0				4.229		53sec/ℓ
:03	1132.0				3.212		50sec/
:54	1183.0				2.639		50sec/ℓ
:00	1249.0				2.597		51sec/ℓ
:57	1306.0				2.915		49sec/ℓ
:51	1360.0				3.403		50sec/ℓ
:58	1427.0				3.169		50sec/ℓ
:53	1482.0				3.180		50sec/ℓ
:48	1535.0				2.936		
:40	1649.0				2.904		52sec/ℓ
:31	1700.0				3.053		51sec/ℓ
:46	1775.0				3.032		49sec/ℓ
:46	1835.0				2.618		52sec/ℓ
:44	1893.0				2.650		53sec/ℓ
:46	1955.0				2.523		50sec/ℓ
:45	2014.0				2.767		52sec/ℓ
:45	2074.0				3.148		51sec/ℓ
:47	2136.0				3.010		50sec/
:46	2195.0				2.820		52sec/ℓ
:41	2250.0				2.724		50sec/ℓ
:47	2316.0				2.703		50sec/ℓ
:41	2370.0				3.021		50sec/ℓ

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Well MW-25

Q 50 sec/ℓ

Static Water Level

time (h:m:s)	t (min)	t' (min)	t/t'	Pressure (PSIG)	Drawdown (ft)	Meter (gal)	
8:41	2430.0				2.830		50sec/ℓ
9:45	2494.0				2.985		55sec/ℓ
10:48	2557.0						
11:41	2610.0				2.894		52sec/ℓ
13:20	2709.0				2.533		53sec/ℓ
14:50	2799.0				3.180		54sec/ℓ
15:59	2868.0				3.127		50sec/ℓ
17:49	2978.0				2.703		50sec/ℓ
18:46	3035.0				2.947		50sec/ℓ
19:54	3103.0				2.947		52sec/ℓ
21:55	3224.0				3.201		52sec/ℓ
2:56	3285.0				2.745		50sec/ℓ
24:43	3392.0				3.095		50sec/ℓ
2:40	3499.0				3.222		
4:50	3639.0				3.106		
6:11	3720.0				3.021		50sec/ℓ
6:48	3757.0				3.021		51sec/ℓ
8:45	3874.0				3.010		52sec/ℓ
10:42	3991.0				2.809		50sec/ℓ
13:00:00	4129.0	0			2.894		Pump Off
13:00:15	4129.25	0.25	16517.0		2.184		
13:00:30	4129.5	0.50	8259.0		1.664		
13:00:45	4129.75	0.75	5506.3		1.367		
13:01:00	4130.0	1.00	4130.0		1.134		
13:01:30	4130.5	1.5	2753.7		0.933		
13:02:00	4131.0	2.0	2065.5		0.763		
13:02:30	4131.5	2.5	1652.6		0.615		
13:03:00	4132.0	3.0	1377.3		0.562		
13:03:30	4132.5	3.5	1180.7		0.519		
13:04:00	4133.0	4.0	1033.3		0.466		
13:04:30	4133.5	4.5	918.6		0.424		

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Well MW-25

Q 50 sec/ℓ

Static Water Level

time (h:m:s)	t (min)	t' (min)	t/t'	Pressure (PSIG)	Drawdown (ft)	Meter (gal)	
13:05	4134.0	5.0	826.8		0.413		
13:06	4135.0	6.0	689.2		0.392		
13:07	4136.0	7.0	590.9		0.371		
13:08	4137.0	8.0	517.1		0.350		
13:09	4138.0	9.0	459.8		0.339		
13:10	4139.0	10.0	413.9		0.329		
13:12	4141.0	12.0	345.1		0.307		
13:14	4143.0	14.0	295.9		0.297		
13:16	4145.0	16.0	259.1		0.286		
13:18	4147.0	18.0	230.4		0.286		
13:20	4149.0	20.0	207.5		0.281		
.3:25	4154.0	25.0	166.2		0.265		
13:30	4159.0	30.0	138.6		0.260		
13:35	4164.0	35.0	119.0		0.249		
13:40	4169.0	40.0	104.2		0.239		
13:45	4174.0	45.0	92.8		0.233		
13:50	4179.0	50.0	83.6		0.233		
13:55	4184.0	55.0	76.1		0.228		
14:00	4189.0	60.0	69.8		0.223		
14:20	4209.0	80.0	52.6		0.217		
14:40	4229.0	100.0	42.3		0.196		
15:10	4259.0	130.0	32.8		0.175		
15:40	4289.0	160.0	26.8		0.159		
16:10	4319.0	190.0	22.7		0.164		
16:40	4349.0	220.0	19.8		9.154		
17:00	4369.0	240.0	18.2		0.154		
18:00	4429.0	300.0	14.8		0.148		
19:00	4489.0	360.0	12.5		0.148		
20:00	4549.0	420.0	10.8		0.143		
21:00	4609.0	480.0	9.6		0.138		
22:00	4669.0	540.0	8.6		0.127		

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Well MW-25

Q 50sec/ℓ

Static Water Level

time (h:m:s)	t (min)	t' (min)	t/t'	Pressure (PSIG)	Drawdown (ft)	Meter (gal)	
23:00	4729.0	600.0	7.9		0.106		
24:00	4789.0	660.0	7.3		0.106		
1:00	4849.0	720.0	6.7		0.117		
2:00	4909.0	780.0	6.3		0.085		
3:00	4969.0	840.0	5.9		0.085		
4:00	5029.0	900.0	5.6		0.095		
5:00	5089.0	960.0	5.3		0.095		
6:00	5149.0	1020.0	5.0		0.085		
7:00	5209.0	1080.0	4.8		0.085		
8:00	5269.0	1140.0	4.6		0.085		
9:00	5329.0	1200.0	4.4		0.074		
[illegible]:50	5379.0	1250.0	4.3		0.053		
12:50	5559.0	1310.0	4.2		0.021		
15:19	5708.0	1459.0	3.9		0.021		
18:00	5869.0	1620.0	3.6		0.048		
21:00	6049.0	1800.0	3.4		0.074		End of T

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Pumped Well MW-25

Measurements at Well MW-24

Pump Speed: ______ Q: 50sec/ℓ

Static Water Level ______

	time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
23	16:36	0.0			0.00	
	17:04	28.0			0.00	
	17:14	38.0			0.13	
	17:24	48.0			0.00	
	17:32	56.0			0.003	
	17:43	67.0			0.015	
	18:04	88.0			0.015	
	18:20	104.0			0.02	
	18:48	132.0			0.03	
	19:24	168.0			0.04	
	19:47	191.0			0.03	
	21:52	316.0			0.04	
	22:46	370.0			0.04	
	23:45	429.0			0.04	
4	24:46	490.0			0.04	
	1:47	551.0			0.05	
	2:46	610.0			0.05	
	3:47	671.0			0.055	
	4:50	734.0			0.06	
	5:49	793.0			0.07	
	6:51	855.0			0.07	
	7:46	910.0			0.08	
	8:48	972.0			0.07	
	9:42	1026.0			0.06	
	10:58	1102.0			0.07	
	11:48	1152.0			0.07	
	12:58	1222.0			0.10	
	13:57	1281.0			0.09	
	14:50	1334.0			0.06	
	15:55	1399.0			0.07	

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Pumped Well MW-25

Measurements at Well MW-24

Pump Speed: ______ Q: 50sec/ℓ

Static Water Level ______

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
16:51	1455.0			0.08	
17:49	1513.0			0.08	
19:45	1629.0			0.08	
20:32	1676.0			0.10	
21:46	1750.0			0.10	
22:44	1808.0			0.10	
23:44	1868.0			0.10	
24:43	1927.0			0.06	
1:42	1986.0			0.11	
2:43	2047.0			0.10	
3:43	2107.0			0.10	
4:45	2169.0			0.11	
5:40	2224.0			0.11	
6:46	2290.0			0.12	
7:42	2346.0			0.13	
8:40	2404.0			0.12	
9:47	2471.0			0.11	
10:48	2532.0			0.10	
11:43	2587.0			0.11	
13:22	2686.0			0.09	
14:52	2776.0			0.11	
15:58	2842.0			0.11	
17:48	2952.0			0.12	
18:48	3012.0			0.11	
19:53	3077.0			0.12	
21:58	3202.0			0.13	
22:57	3261.0			0.14	
24:43	3367.0			0.13	
2:41	3485.0			0.12	
4:50	3614.0			0.13	

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Pumped Well MW-25

Measurements at Well MW-24

Pump Speed: ________ Q: 50sec/ℓ

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
6:12	3696.0			0.14	
6:48	3732.0			0.15	
8:44	3848.0			0.16	
10:40	3964.0			0.14	
13:00	4104.0	0	-	0.11	Pump Off (Recove
13:00:15	4104.25	0.25	16417.0	0.11	
13:00:30	4104.50	0.5	8209.0	0.11	
13:00:45	4104.75	0.75	5473.0	0.11	
13:01:00	4105.0	1.0	4105.0	0.11	
13:01:30	4105.50	1.5	2737.0	0.11	
13:02:00	4106.0	2.0	2053.0	0.11	
13:02:30	4106.50	2.5	1642.6	0.11	
13:03:00	4107.0	3.0	1369.0	0.11	
13:03:30	4107.50	3.5	1173.6	0.11	
13:04:00	4108.0	4.0	1027.0	0.11	
13:04:30	4108.5	4.5	913.0	0.11	
13:05	4109.0	5.0	821.8	0.11	
13:06	4110.0	6.0	685.0	0.11	
13:07	4111.0	7.0	587.3	0.11	
13:08	4112.0	8.0	514.0	0.11	
13:09	4113.0	9.0	457.0	0.105	
13:10	4114.0	10.0	411.4	0.103	
13:12	4116.0	12.0	343.0	0.102	
13:14	4118.0	14.0	294.1	0.10	
13:16	4120.0	16.0	257.5	0.10	
13:18	4122.0	18.0	229.0	0.10	
13:20	4124.0	20.0	206.2	0.095	
13:25	4129.0	25.0	165.2	0.09	
13:30	4134.0	30.0	137.8	0.088	

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Pumped Well MW-25

Measurements at Well MW-24

Pump Speed: ________ **Q:** 50sec/ℓ

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
13:40	4144.0	40.0	103.6	0.085	
13:45	4149.0	45.0	92.2	0.085	
13:50	4154.0	50.0	83.1	0.083	
13:55	4159.0	55.0	75.6	0.082	
14:00	4164.0	60.0	69.4	0.082	
14:20	4184.0	80.0	52.3	0.072	
14:40	4204.0	100.0	42.0	0.073	
15:10	4234.0	130.0	32.6	0.070	
15:40	4264.0	160.0	26.7	0.062	
16:10	4294.0	190.0	22.6	0.070	
16:40	4324.0	220.0	19.7	0.075	
17:00	4344.0	240.0	18.1	0.070	
18:00	4404.0	300.0	14.7	0.070	
19:00	4464.0	360.0	12.4	0.070	
20:00	4524.0	420.0	10.8	0.070	
21:00	4584.0	480.0	9.6	0.060	
22:00	4644.0	540.0	8.6	0.050	
23:00	4704.0	600.0	7.8	0.050	
24:00	4764.0	660.0	7.2	0.070	
1:00	4824.0	720.0	6.7	0.070	
2:00	4884.0	780.0	6.3	0.070	
3:00	4944.0	840.0	5.9	0.070	
4:00	5004.0	900.0	5.6	0.070	
5:00	5064.0	960.0	5.3	0.070	
6:00	5124.0	1020.0	5.0	0.045	
7:00	5184.0	1080.0	4.8	0.038	
8:00	5244.0	1140.0	4.6	0.052	
9:00	5304.0	1200.0	4.4	0.040	
12:50	5534.0	1430.0	3.9	0.040	OGC-004554
15:[illegible]	5684.0	15[illegible]	3.[illegible]	0.015	

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Pumped Well MW-25

Measurements at Well MW-24

Pump Speed: ________ Q: 50sec/ℓ

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
18:00	5844.0	1740.0	3.4	0.015	End of Test

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Pumped Well PW-1

Measurements at Well PW-1

Pump Speed: ______ Q: 1ℓ/2min

Static Water Level ______

	time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
3	17:11:00	0			0.00	Pump On
	17:11:30	0.5			0.00	
	17:17:00	6.0			0.12	
	17:20:00	9.0			0.17	1ℓ/2:00min
	17:24:00	13.0			0.19	1ℓ/2:00min
	17:34:00	23.0			0.31	
	17:48:00	37.0			0.39	
	17:58:00	47.0			0.43	
	18:06:00	55.0			0.42	
	18:28:00	77.0			0.33	1ℓ/1:48min
	18:56:00	105.0			0.14	1ℓ/2:02min
	19:33:00	142.0			0.66	1ℓ/1:98min
	20:16:00	185.0			0.75	1ℓ/2:00min
	20:54:00	223.0			0.92	1ℓ/2:00min
	22:03:00	292.0			1.13	1ℓ/2:00min
	22:52:00	341.0			1.19	1ℓ/2:02min
	23:53:00	402.0			1.33	1ℓ/2:01min
4	24:52:00	461.0			1.24	1ℓ/2:02min
	1:52:00	521.0			0.79	1ℓ/1:99min
	2:53:00	582.0			0.71	1ℓ/1:98min
	3:49:00	638.0			0.62	1ℓ/2:00min
	4:54:00	703.0			0.66	1ℓ/2:00min
	5:54:00	763.0			0.79	1ℓ/2:00min
	6:55:00	824.0			0.87	1ℓ/2:02min
	7:58:00	887.0			0.72	1ℓ/2:00min
	8:54:00	943.0			0.95	1ℓ/2:03min
	9:45:00	994.0			1.00	New valve installe
	10:38:00	1047.0			1.29	1ℓ/2:08min
	11:58:00	1127.0			0.86	1ℓ/2:06min
	13:03:00	1192.0			0.89	1ℓ/2:02min

Date: 2/23 - 2/27

METRIC Corporation

Pumped Well PW-1

Measurements at Well PW-1

Pump Speed: ______ Q: 1ℓ/2min

Static Water Level ______

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
14:03:00	1252.0			0.96	1ℓ/2:06min
14:55:00	1304.0			0.86	1ℓ/2:04min
15:43:00	1352.0			0.70	1ℓ/2:06min
16:57:00	1426.0			0.70	1ℓ/2:00min
17:58:00	1487.0			0.99	1ℓ/2:04min
19:50:00	1599.0			1.20	1ℓ/2:04min
20:39:00	1648.0			0.68	1ℓ/2:06min
21:53:00	1722.0			1.19	1ℓ/2:03min
22:52:00	1781.0			0.39	1ℓ/2:03min
23:52:00	1841.0			0.39	1ℓ/2:04min
24:53:00	1902.0			0.99	-
1:49:00	1958.0			0.64	1ℓ/2:00min
2:49:00	2018.0			0.62	1ℓ/2:02min
3:51:00	2080.0			0.61	1ℓ/2:04min
4:52:00	2141.0			0.67	1ℓ/2:00min
5:48:00	2197.0			0.39	1ℓ/2:02min
6:52:00	2261.0			0.62	1ℓ/2:04min
7:59:00	2328.0			0.88	1ℓ/2:04min
8:48:00	2377.0			1.01	1ℓ/2:00min
9:55:00	2444.0			0.84	1ℓ/2:08min
10:56:00	2505.0			0.88	1ℓ/2:22min
11:53:00	2562.0			1.34	1ℓ/1:55min
12:53:00	2622.0			1.60	1ℓ/1:58min
13:58:00	2687.0			1.67	1ℓ/1:57min
14:53:00	2742.0			1.78	1ℓ/2:00min
15:45:00	2794.0			1.83	1ℓ/2:02min
16:38:00	2847.0			1.86	1ℓ/2:00min
17:41:00	2910.0			1.86	1ℓ/2:00min
18:55:00	2984.0			1.80	1ℓ/2:00min
19:59:00	3048.0			1.86	1ℓ/2:02min

METRIC Corporation

Date: 2/23 - 2/27

Pumped Well PW-1

Measurements at Well PW-1

Pump Speed: ________ Q: 1ℓ/2min

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
22:05:00	3174.0			1.88	1ℓ/2:00min
23:06:00	3235.0			1.88	1ℓ/2:00min
6 24:49:00	3338.0			1.92	1ℓ/2:00min
2:47:00	3456.0			1.86	1ℓ/2:00min
4:59:00	3588.0			1.72	
5:17:00	3606.0			1.94	1ℓ/2:02min
6:54:00	3703.0			2.00	1ℓ/2:00min
8:55:00	3824.0			2.26	1ℓ/2:00min
10:58:00	3947.0			2.01	1ℓ/2:00min
13:34:00	4103.0			2.12	
14:20:00	4149.0			2.07	
14:45:00	4174.0	0	--	1.98	Pump Off (Recover
14:45:15	4174.25	0.25	16697.0	1.96	
14:45:30	4174.50	0.50	8345.0	1.95	
14:45:45	4174.75	0.75	5566.3	1.94	
14:46:00	4175.0	1.0	4175.0	1.93	
14:46:30	4175.5	1.5	2783.7	1.91	
14:47:00	4176.0	2.0	2088.0	1.90	
14:47:30	4176.5	2.5	1670.6	1.88	
14:48:00	4177.0	3.0	1392.3	1.87	
14:48:30	4177.3	3.5	1193.5	1.86	
14:49:00	4178.0	4.0	1044.5	1.84	
14:49:30	4178.3	4.5	928.5	1.82	
14:50:00	4179.0	5.0	835.6	1.80	
14:51:00	4180.0	6.0	696.7	1.73	
14:52:00	4181.0	7.0	597.3	1.72	
14:53:00	4182.0	8.0	522.8	1.71	
14:54:00	4183.0	9.0	464.8	1.68	
14:55:00	4184.0	10.0	418.4	1.66	
14:57:00	4186.0	12.0	348.8	1.63	

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METRIC Corporation

Pumped Well PW-1

Measurements at Well PW-1

Pump Speed: ________ Q: 1ℓ/2min

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
14:59:00	4188.0	14.0	299.1	1.55	
15:01:00	4190.0	16.0	261.9	1.53	
15:03:00	4192.0	18.0	232.9	1.50	
15:05:00	4194.0	20.0	209.7	1.46	
15:10:00	4199.0	25.0	168.0	1.35	
15:15:00	4204.0	30.0	140.1	1.25	
15:20:00	4209.0	35.0	120.3	1.16	
15:25:00	4214.0	40.0	105.4	1.04	
15:30:00	4219.0	45.0	93.8	0.96	
15:35:00	4224.0	50.0	84.5	0.93	
15:45:00	4234.0	60.0	70.6	0.76	
16:05:00	4254.0	80.0	53.2	0.585	
16:25:00	4274.0	100.0	42.7	0.42	
16:55:00	4304.0	130.0	33.1	0.25	
17:25:00	4334.0	160.0	27.1	0.14	
17:55:00	4364.0	190.0	23.0	0.07	
18:25:00	4394.0	220.0	20.0	0.03	
18:45:00	4414.0	240.0	18.4	0.01	
19:45:00	4474.0	300.0	14.9	-0.02	
20:45:00	4534.0	360.0	12.6	-0.04	
21:45:00	4594.0	420.0	10.5	-0.05	
7 6:20:00	5109.0	935.0	5.5	-0.04	End of Test

Date: 2/23 - 2/27

METRIC Corporation

Pumped Well PW-1

Measurements at Well MW-9

Pump Speed: ________ Q: 1ℓ/2.00 min

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
17:11	0.0			0.00	Pump On
17:23	12.0			0.00	
17:34	23.0			0.00	
17:49	38.0			0.01	
17:59	48.0			0.01	
18:10	59.0			0.01	
18:26	75.0			0.00	
18:58	107.0			-0.07	
19:35	144.0			-0.02	
20:14	183.0			-0.07	
20:50	219.0			-0.02	
22:00	289.0			0.03	
22:50	339.0			0.03	
23:51	400.0			0.04	
24:50	459.0			0.03	
1:51	520.0			0.03	
2:52	581.0			0.03	
3:48	637.0			0.03	
4:53	702.0			0.03	
5:53	762.0			0.08	
6:57	826.0			0.02	
7:59	888.0			0.03	
8:55	944.0			0.03	
9:45	994.0			0.01	
10:38	1047.0			0.01	
11:59	1128.0			-	
13:05	1194.0			0.02	
14:04	1253.0			0.01	
14:56	1305.0			0.00	
15:44	[illegible]			[illegible]	

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Date: 2/23 - 2/27

METRIC
Corporation

Pumped Well PW-1

Measurements at Well MW-9

Pump Speed: ______ Q: 1ℓ/2.00 min

Static Water Level ______

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
16:58	1427.0			0.02	
17:59	1488.0			0.03	
19:48	1597.0			0.08	
20:37	1646.0			0.06	
21:51	1720.0			0.02	
22:50	1779.0			0.01	
23:50	1839.0			0.03	
24:51	1900.0			0.03	
1:48	1957.0			0.03	
2:48	2017.0			0.03	
3:50	2079.0			0.03	
4:50	2139.0			0.03	
5:47	2196.0			0.03	
6:53	2262.0			0.03	
7:58	2327.0			0.03	
9:53	2442.0			0.03	
10:58	2507.0			0.02	
11:52	2561.0			0.02	
12:54	2623.0			0.02	
13:58	2687.0			0.03	
14:55	2744.0			0.03	
15:46	2795.0			0.02	
16:39	2848.0			0.02	
17:43	2912.0			0.04	
18:53	2982.0			0.03	
19:59	3048.0			0.05	
22:06	3175.0			0.05	
23:05	3234.0			0.05	
24:49	3338.0			0.05	
2:46	3455.0			0.04	

Date: 2/23 - 2/27

METRIC Corporation

Pumped Well PW-1

Measurements at Well MW-9

Pump Speed: ______ Q: 1ℓ/2.00 min

Static Water Level ______

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
4:57	3586.0			0.04	
6:19	3668.0			0.04	
6:55	3704.0			0.04	
8:57	3826.0			0.05	
10:56	3945.0			0.04	
13:34	4103.0			0.03	
14:20	4149.0			-0.07	
14:45:00	4174.0	0		0.02	Pump Off (Recov
14:45:15	4174.25	0.25	16697.0	0.02	
14:45:30	4174.50	0.50	8345.0	0.02	
14:45:45	4174.75	0.75	5566.3	0.02	
14:46:00	4175.0	1.0	4175.0	0.02	
14:46:30	4175.5	1.5	2783.7	0.02	
14:47:00	4176.0	2.0	2088.0	0.02	
14:47:30	4176.5	2.5	1670.6	0.02	
14:48:00	4177.0	3.0	1392.3	0.02	
14:48:30	4177.5	3.5	1193.5	0.02	
14:49:00	4178.0	4.0	1044.5	0.02	
14:49:30	4178.5	4.5	928.5	0.02	
14:50	4179.0	5.0	835.6	0.02	
14:51	4180.0	6.0	696.7	0.02	
14:52	4181.0	7.0	597.3	0.02	
14:53	4182.0	8.0	522.8	0.02	
15:54	4183.0	9.0	464.8	0.02	
15:55	4184.0	10.0	418.4	0.02	
14:57	4186.0	12.0	348.8	0.02	
14:59	4188.0	14.0	299.1	0.02	
15:01	4190.0	16.0	261.9	0.02	
15:03	4192.0	18.0	232.9	0.02	
15:05	4194.0	20.0	209.7	0.02	

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Date: 2/23 - 2/27

METRIC Corporation

Pumped Well PW-1

Measurements at Well MW-9

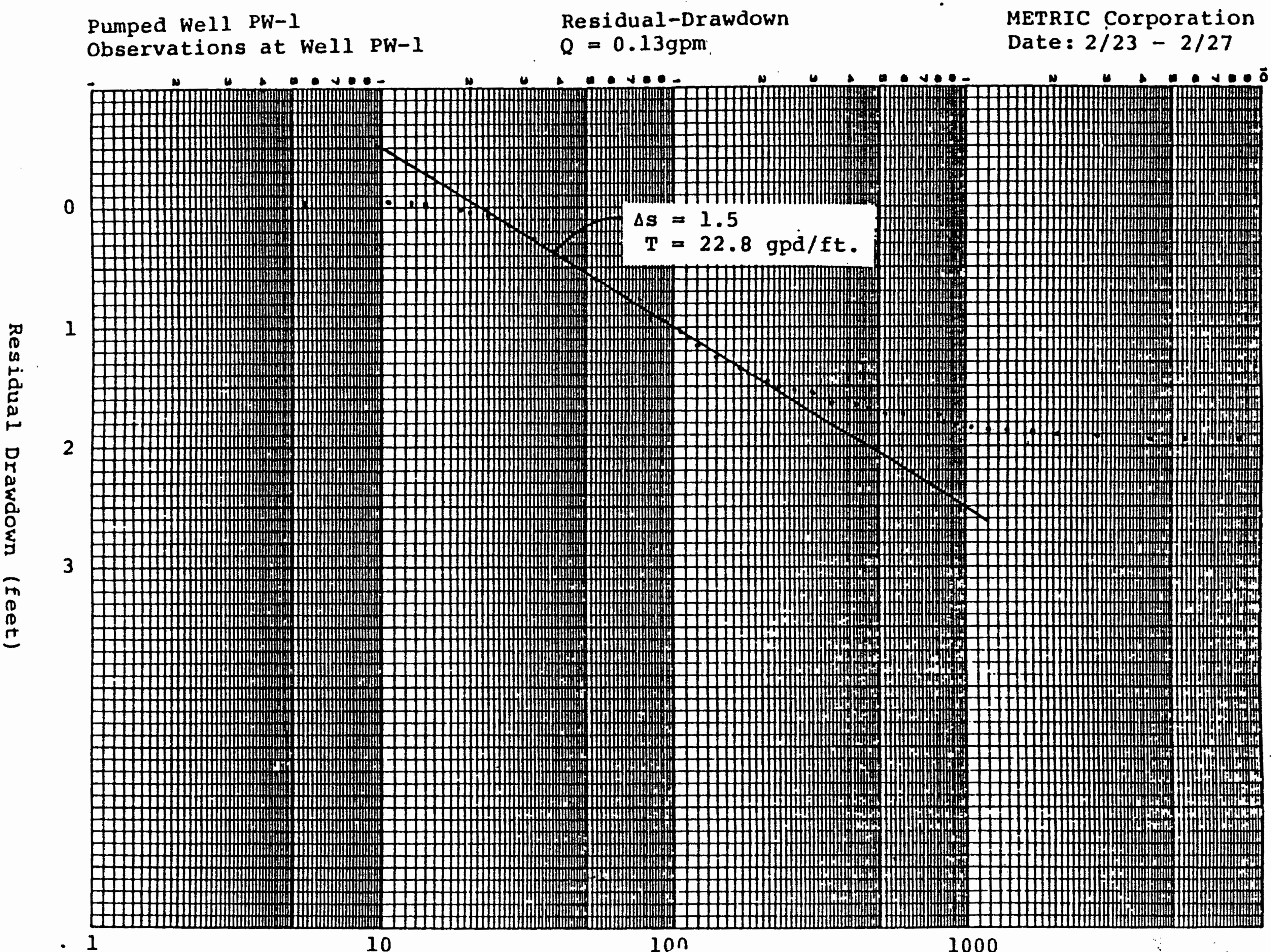
Pump Speed: ______ Q: 1ℓ/2.00 min

Static Water Level ______

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Comments
15:10	4199.0	25.0	168.0	0.02	
15:15	4204.0	30.0	140.1	0.019	
15:20	4209.0	35.0	120.3	0.014	
15:25	4214.0	40.0	105.4	0.01	
15:30	4219.0	45.0	93.8	0.01	
15:35	4224.0	50.0	84.5	0.009	
15:40	4229.0	55.0	76.9	0.009	
15:45	4234.0	60.0	70.6	0.009	
16:05	4254.0	80.0	53.2	0.00	
16:25	4274.0	100.0	42.7	0.00	
16:55	4304.0	130.0	33.1	0.00	
17:25	4334.0	160.0	27.1	0.00	
17:55	4364.0	190.0	23.0	0.00	
18:15	4384.0	210.0	20.9	0.00	
19:15	4444.0	270.0	16.5	0.00	
20:15	4504.0	330.0	13.6	0.00	
21:45	4594.0	420.0	10.5	0.00	
7 6:20	5109.0	935.0	5.5	0.00	End of Test

APPENDIX B

AQUIFER TEST DATA PLOTS



Pumped Well PW-1
Observations at Well PW-1
Residual-Drawdown
Q = 0.13gpm
METRIC Corporation
Date: 2/23 - 2/27
Δs = 1.5
T = 22.8 gpd/ft.
Residual Drawdown (feet)
0
1
2
3
1
10
100
1000

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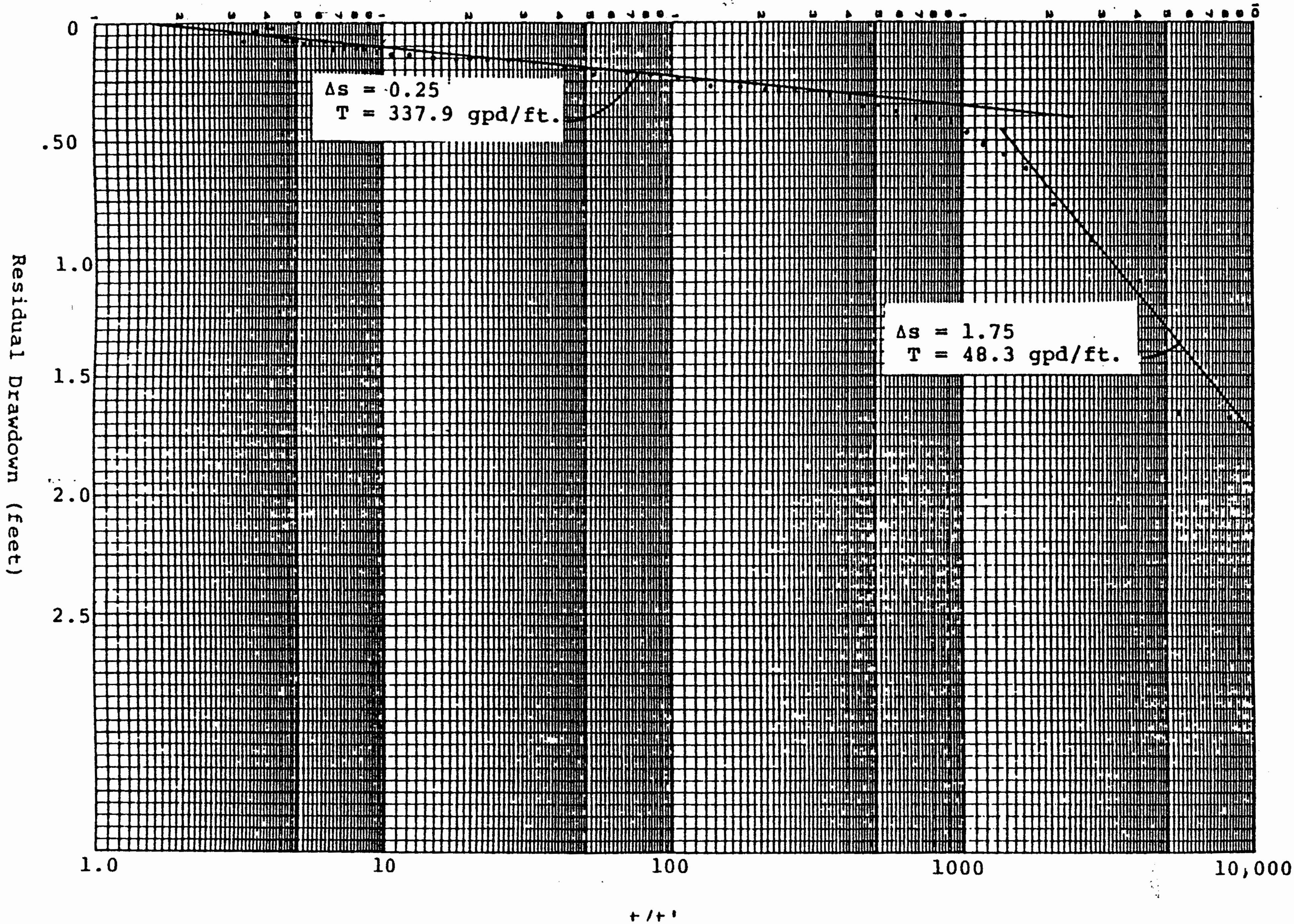
Pumped Well MW-25
Observations at Well MW-25

Residual-Drawdown
Q = 0.32 gpm

METRIC Corporation
Date: 2/88



(2.184, 16,517)

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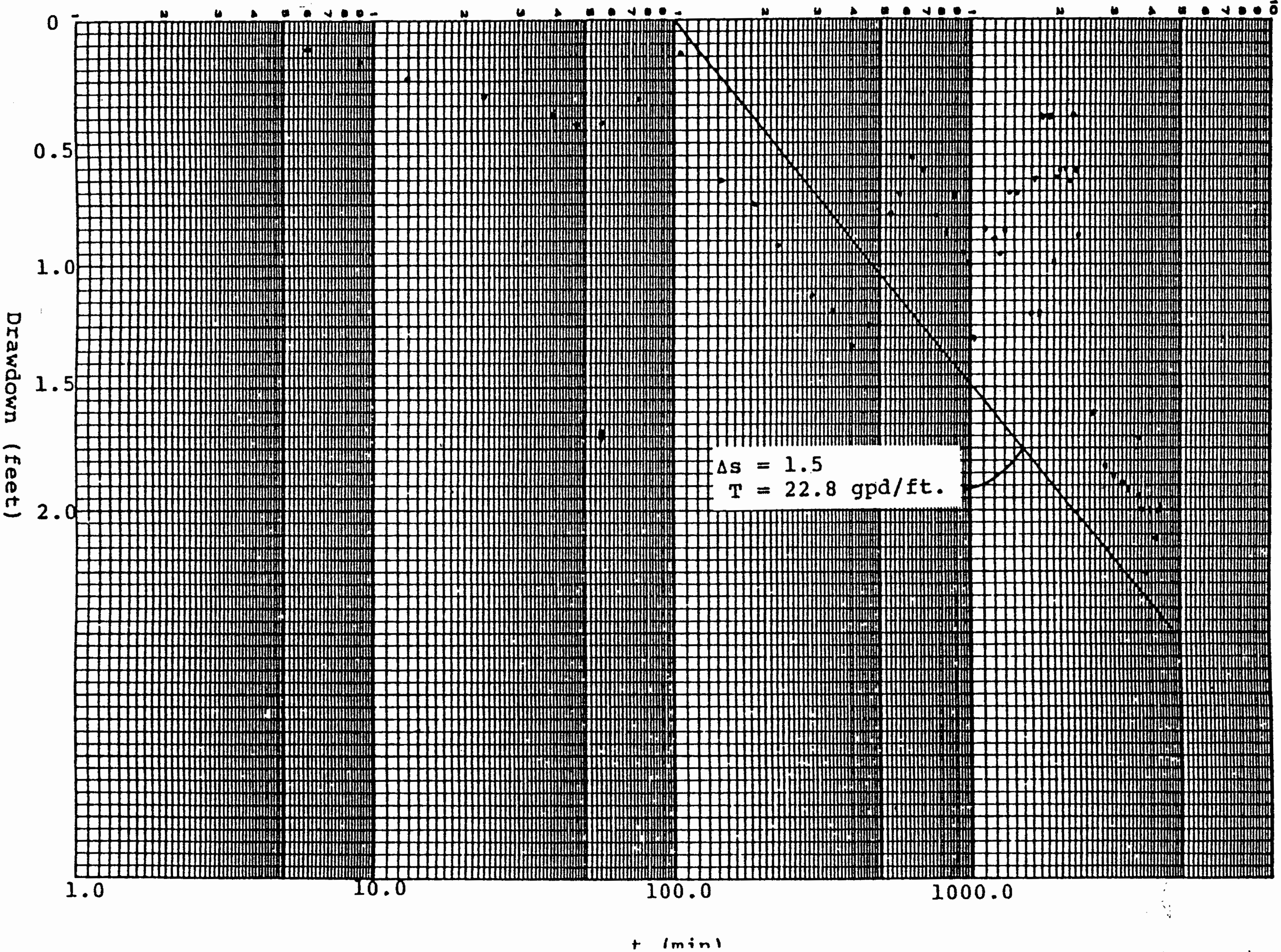
OGC-004567

Pumped Well PW-1
Observations at Well PW-1

Time-Drawdown
Q = 0.13gpm

METRIC Corporation
Date: 2/23 - 2/27



APPENDIX C

WATER QUALITY ANALYSES

* NOT INCLUDED *

AQUIFER TESTING
AT THE
SPARTON TECHNOLOGY, INC.
COORS ROAD PLANT
ALBUQUERQUE, NEW EMXICO

PREPARED BY

METRIC CORPORATON
ALBUQUERQUE, NEW MEXICO

NOVEMBER 18, 1988

T A B L E   O F   C O N T E N T S

L I S T   O F   T A B L E S

L I S T   O F   A P P E N D I C E S

AQUIFER TESTING
AT THE
SPARTON TECHNOLOGY, INC.
COORS ROAD PLANT

Aquifer tests were performed in four groundwater recovery wells at the Sparton Technology, Inc., Coors Road Plant during September and October 1988. The purpose of the testing was to estimate well capacity and further define aquifer permeability of the "upper flow zone". The well capacities were used to develope estimates of the total capacity of the groundwater recovery system for equipment sizing and water rights requirements. The "upper flow zone" consists generally of the upper 5 to 10 feet of the saturated zone at the Coors Road site separated from the remainder of the saturated zone by a fine grained aquitard unit.

Pumping tests were conducted in four wells, MW-23 and MW-26 located along the south side of the plant building, MW-27 located along the west side of the plant building and MW-28 located at the west property corner. Each of the four wells are included in the groundwater recovery system.

The tests were conducted as follows:

Well: MW-23
Test Type: Constant Discharge
Test Drawdown: 2.5 ft.
Available Drawdown: 7.7 ft.
Duration of Pumping: 72.0 hrs.
Average Discharge: 0.26gpm
Observations Taken in Wells: MW-23

Well: MW-26
Test Type: Constant Discharge
Test Drawdown: 2.5 ft.
Available Drawdown: 13.4 ft.
Duration of Pumping: 71.1 hrs.
Average Discharge: 0.019gpm
Observations Taken in Wells: MW-26

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Well: MW-27
Test Type: Constant Discharge
Test Drawdown: 2.2 ft.
Available Drawdown: 8.0 ft.
Duration of Pumping: 70.0 hrs.
Average Discharge: 0.117gpm
Observations Taken In Wells: MW-27

Well: MW-28
Test Type: Constant Discharge
Test Drawdown: 2.67 ft.
Available Drawdown: 4.1 ft.
Duration of Pumping: 72.0 hrs.
Average Discharge: 0.0705gpm
Observations Taken in Wells: MW-28

Each of the pumped wells are 2-inch, i.d. PVC wells with wire-wound stainless steel screens. The wells were installed in 7-inch diameter hollow stem auguer borings. They were pumped with a 1.66-inch o.d. positive displacement piston pump having a maximum discharge of about 2.5gpm. Water levels in the pumped wells were monitored with an airline and a water monometer. All water level measurements were taken to the nearest 0.01 feet. Discharge measurements were made with a graduated cylinder and stop watch.

The water level and discharge data collected during each test are presented in APPENDIX A. The data were analyzed using semi-log plots of time-duration and residual drawdown data (see APPENDIX B).

The time-drawdown data were checked using a procedure suggested by Johnson, 1972 to ensure that $u<0.05$ and, thus, validate the use of the Jacob solution. In the equation $u = \frac{1,87r^2S}{Tt}$, u was set equal to 0.05, and the time, t, was determined after which the Jacob solution is valid. The effective radii of the wells were assumed to be 0.29 ft. because the wells were installed in

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7-inch (0.58 ft.) diameter boreholes. TABLE 1 shows that all but the early data are valid. The selected hydraulic conductivities were all determined from data for which the Jacob solution is valid.

The data were also checked using a procedure suggested by Schafer, 1978 to determine which portion of the data might be casing storage affected. The early portion of the time-drawdown data is casing storage affected in each case as shown in TABLE 2.

For determination of aquifer permeability, the residual draw data were used rather than the time-drawdown data because the time-drawdown was affected by fluctuations in the pump discharge and because the residual drawdown data is generally considered to be more reliable when only pumped well data are available as is the case here. Additionally, the middle or late residual drawdown data were used because the early data appears to be casing storage affected.

Based on the above described testing, it is the opinion of the investigators that the best estimate for the permeability (hydraulic conductivity) of the upper flow zone in the vicinity of each of the wells tested is as follows (see TABLE 3):

Well	Hydraulic Conductivity (cm/sec)
MW-23	8.54×10^{-4}
MW-26	3.91×10^{-5}
MW-27	9.08×10^{-4}
MW-28	1.07×10^{-3}

TABLE 1

JACOB VALIDATION

Well	r (ft)	T (gpd/ft)	t days	t min.
MW-23	0.29	139	0.0045	6.52
MW-26	0.29	11.1	0.057	81.6
MW-27	0.29	154	0.0041	5.88
MW-28	0.29	93.1	0.0068	9.73

$$t = \frac{1.87 \quad r^2 \quad S}{uT} = \frac{1.87 \ (.29)^2 \ (.2)}{0.05(T)}$$

$S = 0.20$

$u = 0.05$

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TABLE 2

CASING STORAGE AFFECT

Well	Q (gpm)	S (ft)	Q/S (gpm/ft)	tc (min)
MW-23	0.264	2.2	0.12	7.6
MW-26	0.019	2.3	0.0082	112
MW-27	0.117	3.0	0.0390	23.5
MW-28	0.0705	1.1	0.0641	14.3

$d_c = 2.07$

$d_p = 1.66$

$$tc = \frac{0.6\ (dc^2 - dp^2)}{Q/S} = \frac{0.6\ (2.07^2 - 1.66^2)}{Q/S}$$

$$= \frac{0.9176}{Q/S}$$

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TABLE 3

AQUIFER TESTING
SPARTON TECHNOLOGY, INC. COORS ROAD PLANT

Pumped Well	Observations At	Curve	Apparent T (gpd/ft)	b (ft)	Hydraulic Conductivity ft/day	Hydraulic Conductivity cm/sec	Comments
MW-23	MW-23	Early T-D	33.0				Casing storage affected
		Late T-D	456				
		Early R-D	45.0				Casing storage affected
		Late R-D	139	7.7	2.42	8.54×10^{-4}	Selected
MW-26	MW-26	Early T-D	1.58				Casing storage affected*
		Late T-D	24.9				
		Early R-D	2.94				Casing storage affected
		Late R-D	11.1	13.4	0.11	3.91×10^{-5}	Selected
MW-27	MW-27	Early R-D	27.3				
		Late R-D	154	8.0	2.57	9.08×10^{-4}	Selected
MW-28	MW-28	Early T-D	18.1				Casing storage affected
		Middle T-D	62.0				
		Late T-D	19.6				Impermeable boundary
		Early R-D	27.8				Casing storage affected
		Middle R-D	93.1	4.1	3.04	1.07×10^{-3}	Selected
		Late R-D	12.0				Impermeable boundary

* Jacob Solution Not Valid

The residual-drawdown curves (APPENDIX B) for MW-23, MW-26, MW-27, and to a lesser extent MW-28, show evidence that a "recharge effect" may be occurring during the pumping period. The residual drawdown curves show a t/t' value greater than 2 at zero drawdown, suggesting a "recharge effect". Possible explanations of the apparent "recharge effect" include reduction or reversal of prevailing downward vertical leakage in the cone of depression during the test or induced flow from a more permeable burried channel(s) existing within the upper flow zone.

Estimated well capacities have been computed for each of the wells included in the groundwater recovery system (see TABLE 4). The capacities were computed based on specific capacities observed in testing to date (see METRIC Corp., April 1987 and May 1988) and assuming 100% drawdown. This would tend to yield conservatively high values, however, MW-24 has undergone additional development since it was tested. This might increase its capacity beyond that shown in TABLE 4.

TABLE 4

ESTIMATED WELL CAPACITIES

Well #	Pumping Time (hrs)	Drawdown (ft)	Discharge (gpm)	Specific Capacity (gpm/ft)	Available Drawdown (ft)	Estimated Capacity (gpm)
18	49.0	5.02	0.264	0.0526	12.6	0.66
23	72.0	2.47	0.260	0.1054	7.65	0.81
24	73.2	3.26	0.205	0.0629	8.1	0.51
25	68.8	3.0	0.317	0.106	7.3	0.77
26	71.1	2.53	0.019	0.008	13.4	0.10
27	70.0	2.21	0.117	0.053	8.0	0.42
28	72.0	2.67	0.070	0.026	4.1	0.11
PW-1	69.6	2.12	0.13	0.06	4.3	0.26
					Total	3.64

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Schafer, David C., 1978, Casing Storage Can Affect Pump Testing Data, The Johnson Drillers Journal.

APPENDIX A
PUMP TEST DATA

METRIC Corporation

Date: 9-27-88

Pumped Well MW-23

Measurements at Well MW-23

Pump Speed: ______ Q: ______ gpm

Static Water Level ______

	time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min·sec/ℓ)
28	17:00	1965			2.51	60"
	18:00	2025			2.80	1'6" adj
	19:00	2085			2.37	1'01"
	20:00	2145			2.58	1'02"
	21:00	2205			2.51	1'05"
	22:00	2265			2.59	0'58"
	23:00	2325			2.44	1'00"
	24:00	2385			2.43	1'00"
29	1:00	2445			2.46	1'03"
	2:00	2505			2.48	1'01"
	3:00	2565			2.37	1'04"
	4:00	2625			2.53	0'58"
	5:00	2685			2.49	0'57"
	6:00	2745			2.45	1'02"
	7:00	2805			2.56	0'57"
	8:00	2865			2.71	1'2"
	9:00	2925			2.58	1'0"
	10:00	2985			2.52	1'0"
	11:00	3045			2.62	1'02"
	12:00	3105			2.64	58"
	13:00	3165			2.73	1'0"
	14:00	3225			2.88	52" adj
	15:00	3285			2.55	58"
	16:00	3345			2.59	56"
	17:00	3405			2.64	1'
	18:00	3465			2.71	1'6" adj
	19:00	3525			2.60	59"
	20:00	3585			2.49	59"
	21:00	3645			2.56	1'6"
	22:00	3705			2.44	58"

METRIC Corporation

Date: 9-27-88

Pumped Well MW-23

Measurements at Well MW-23

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min·sec/ℓ)
23:00	3765			2.34	1'03"
24:00	3825			2.65	1'05"
1:00	3885			2.53	56"
2:00	3945			2.37	1'07" 1'01"
3:00	4005			2.39	1'04"
4:00	4065			2.58	1'02"
5:00	4125			2.24	1'02"
6:00	4185			2.27	1'06" adj
7:00	4245			2.47	58"
8:00	4305			2.46	1'04"
8:15:15	4320.25	.25	17,281	1.39	Pump off @8:15
:30	4320.50	.50	8,641	0.83	
:45	4320.75	.75	5,761	0.68	
8:16:00	4321.00	1.0	4,321	0.36	
8:16:30	4321.5	1.5	2,881	0.19	
8:17:00	4322.0	2.0	2,161	0.14	
8: :30	4322.5	2.5	1,729	0.09	
8:18:00	4323.0	3.0	1,441	0.07	
:30	4323.5	3.5	1,235	0.07	
19:00	4324.0	4.0	1,081	0.07	
:30	4324.5	4.5	961	0.07	
20:00	4325.0	5.0	865	0.08	
21:00	4326	6	721	0.07	
22:00	4327	7	618	0.06	
23:00	4328	8	541	0.07	
24:00	4329	9	481	0.08	
25:00	4330	10	433	0.09	
27:00	4332	12	361	0.07	
29:00	4334	14	310	0.08	
30:00	4336	16	271	0.08	

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METRIC Corporation

Date: 9-27-88

Pumped Well MW-23

Measurements at Well MW-23

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

30

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min.sec/ℓ)
33:00	4338	18	241	0.05	
35:00	4340	20	217	0.07	
40:00	4345	25	174	0.07	
45:00	4350	30	145	0.07	
50:00	4355	35	124	0.07	
55:00	4360	40	109	0.07	
9:00	4365	45	97	0.07	
9:15	4380	60	73	0.06	
9:30	4395	75	59	0.06	
10:00	4425	105	42	0.06	
11:00	4485	165	27	0.04	
11:30	4515	195	23	0.03	
13:19	4624	304	15	0.03	
15:10	4735	415	11	0.02	
17:01	4846	526	9	0.00	

Date: 9-14-88

METRIC Corporation

Pumped Well MW-26

Measurements at Well MW-26

Pump Speed: ________ Q: 0.01887 gpm

Static Water Level ________

	time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min/ℓ)
14	8:05:00	0			0	
	:15	0.25			0.23	
	:30	0.50			0.47	
	:45	0.75			0.67	
	6:00	1.00			0.93	
	:30	1.50			1.37	
	7:00	2.00			1.76	
	:30	2.50			2.20	
	8:00	3.00			2.27	
	:30	3.50			2.28	
	9:00	4.00			2.28	
	:30	4.50			2.29	
	10:00	5.00			2.33	
	11:00	6			2.37	
	12	7			2.41	
	13	8			2.38	
	14	9			2.38	
	15	10			2.41	
	17	12			2.57	
	19	14			2.53	14 min/ℓ
	21	16			2.52	
	23	18			2.50	
	25	20			2.28	
	30	25			2.44	
	35	30			2.62	14
	40	35			2.77	
	50	45			3.03	14
	9:00	55			2.71	11
	9:20	75			2.61	14
	9:40	95			2.48	12

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METRIC
Corporation

Date: 9-14-88

Pumped Well MW-26

Measurements at Well MW-26

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

	time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min/ℓ)
4	10:00	115			3.53	8
	10:30	145			3.48	10
	11:00	175			2.57	14, 12
	11:30	205			2.25	11, 14
	12:00	235			2.83	13, 11
	13:00	295			2.15	12, 15
	14:00	355			3.55	12, 11, 13, 12, 9
	15:00	415			3.72	15, 15, 11, 9, 9, 16
	16:00	475			3.25	16, 10, 12
	17:00	535			3.25	11, 10, 9, 11, 11
	18:00	595			2.85	10, 11, 11, 11
	19:00	655			3.03	10, 13, 15, 11
	20:00	715			2.55	10, 13, 15
	21:00	775			2.42	11, 13, 11, 12
	22:00	835			2.35	12, 11, 9, 9
	23:00	895			2.47	13, 12, 14, 11
	24:00	955			2.88	11, 9, 13, 10, 17
5	1:00	1015			2.07	16, 12, 17, 17
	2:00	1075			1.68	12, 11, 16
	3:00	1135			3.24	14, 13, 15, 13
	4:00	1195			1.50	11, 9, 10, 9
	5:00	1255			5.17	9, 8, 12, 10
	6:00	1315			4.75	10, 11, 17, 10
	7:00	1375			4.60	13, 11
	8:00	1435			2.78	9, 12, 17
	9:00	1495			2.26	
	9:05	1500			2.18	
	9:10	1505			2.10	15
	9:15	1510			2.76	
	9:20	1515			1.84	

METRIC Corporation

Date: 9-14-88

Pumped Well MW-26

Measurements at Well MW-26

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min/ℓ)
-15 9:25	1520			2.57	14
30	1525			3.30	
35	1530			2.46	13
40	1535			2.08	
45	1540			2.00	
50	1545			1.72	15
55	1550			2.77	
10:00	1555			2.63	
05	1560			2.78	12
10	1565			2.85	
15	1570			2.83	
20	1575			2.93	11
25	1580			3.05	
30	1585			2.82	11
35	1590			2.78	
40	1595			3.17	10
45	1600			2.76	
50	1605			2.05	
55	1610			1.84	
11:00	1615			1.92	
05	1620			1.77	22
10	1625			2.30	
15	1630			2.65	
20	1635			2.54	
25	1640			2.20	
30	1645			1.99	14
35	1650			2.90	
40	1655			3.14	
45	1660			2.07	
50	1665			2.44	14

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METRIC Corporation

Date: 9-14-88

Pumped Well MW-26

Measurements at Well MW-26

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min/ℓ)
-15 11:55	1670			2.59	
12:00	1675			2.70	12
05	1680			2.53	
10	1685			2.43	
15	1690			2.55	13
20	1695			2.53	
25	1700			2.49	
30	1705			2.52	14
35	1710			2.30	
40	1715			2.06	14
45	1720			2.70	
50	1725			2.35	
55	1730			1.89	
13:00	1735			2.10	
05	1740			2.45	16
10	1745			2.31	
15	1750			2.29	
20	1755			2.32	14
25	1760			2.32	
30	1765			2.21	
35	1770			2.39	
40	1775			2.47	12
45	1780			2.58	
50	1785			2.49	
55	1790			2.54	
14:00	1795			2.55	14
14:05	1800			2.52	
10	1805			2.80	
15	1810			1.65	
20	1815			2.80	15

METRIC Corporation

Date: 9-14-88

Pumped Well MW-26

Measurements at Well MW-26

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

15

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min/ℓ)
14:25	1820			2.18	
30	1825			2.77	
35	1830			3.36	9
40	1835			3.30	
45	1840			3.00	10
50	1845			2.83	
55	1850			1.83	
15:00	1855			2.74	12
05	1860			2.73	
10	1865			2.13	
15	1870			2.48	14
20	1875			3.48	
25	1880			3.44	
30	1885			2.92	
35	1890			3.15	15
40	1895			2.94	
45	1900			2.91	
50	1905			2.83	14
55	1910			2.32	
16:00	1915			2.07	
05	1920			2.16	15.5
10	1925			2.54	
15	1930			2.77	
20	1935			2.62	13.5
25	1940			2.68	
30	1945			2.44	
35	1950			2.37	14
40	1955			2.37	
45	1960			2.61	Start aerator
50	1965			2.55	12.5

METRIC Corporation

Date: 9-14-88

Pumped Well MW-26

Measurements at Well MW-26

Pump Speed: _______ Q: _______ gpm

Static Water Level _______

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min/ℓ)
-15 16:55	1970			2.57	
17:00	1975			2.29	
05	1980			2.09	16
10	1985			2.25	
15	1990			2.26	
20	1995			2.43	12
25	2000			3.23	
30	2005			3.24	
35	2010			3.37	
40	2015			3.15	
45	2020			3.35	10
50	2025			4.09	
55	2030			4.09	10
18:00	2035			3.78	
05	2040			3.15	
10	2045			3.05	15
15	2050			2.70	
20	2055			2.21	
25	2060			1.98	
30	2065			2.52	18
35	2070			1.89	
40	2075			2.05	
45	2080			2.13	aerator off ready to siphon
50	2085			2.52	14.5
55	2090			2.56	
19:00	2095			2.43	15
05	2100			1.92	
10	2105			1.48	
15	2110			1.28	
20	2115			1.87	12

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METRIC Corporation

Date: 9-14-88

Pumped Well MW-26

Measurements at Well MW-26

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min/ℓ)
-15 19:25	2120			2.73	
30	2125			2.49	12
35	2130			2.78	
40	2135			2.98	11
45	2140			3.10	
50	2145			3.06	
55	2150			3.00	12
20:00	2155			3.03	
05	2160			2.67	12
10	2165			2.27	
15	2170			2.25	
20	2175			2.33	
25	2180			1.84	
30	2185			2.95	11
35	2190			3.47	9
40	2195			3.54	
45	2200			2.95	
50	2205			2.93	12
55	2210			3.20	
21:00	2215			3.10	11
05	2220			3.17	
10	2225			3.37	10
15	2230			3.35	
20	2235			3.20	11
25	2240			3.09	
30	2245			2.90	
35	2250			2.78	12
40	2255			2.61	
45	2260			2.71	13
50	2265			2.79	

OGC-004592

METRIC Corporation

Date: 9-14-88

Pumped Well MW-26

Measurements at Well MW-26

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min/ℓ)
15 21:55	2270			2.72	
22:00	2275			2.85	13
05	2280			2.81	
10	2285			2.50	
15	2290			2.43	14
20	2295			2.16	
25	2300			1.93	12
30	2305			3.05	
35	2310			3.10	12
40	2315			2.96	
45	2320			3.15	
50	2325			3.12	12
55	2330			3.48	
23:00	2335			3.07	11
05	2340			2.57	
10	2345			3.26	12
15	2350			3.60	
20	2355			3.42	10
25	2360			3.15	
30	2365			3.26	11
35	2370			3.23	
40	2375			3.16	11
45	2380			3.11	
50	2385			2.97	12
55	2390			2.43	Pump Off
24:00	2395			1.88	
03	2398			2.08	
05	2400			2.20	
10	2405			2.40	
20	2415			2.35	

Date: 9-14-88

METRIC
Corporation

Pumped Well MW-26

Measurements at Well MW-26

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min/ℓ)
16 24:25	2420			2.18	15
30	2425			2.40	
35				3.01	11
40				3.36	10
45				3.39	
50				3.48	
55				3.71	10
1:00				1.85	
05				2.27	
10				2.53	14
15				2.55	
20				2.75	
25				2.80	
30	2485			2.54	13
35				2.61	
40				2.50	
45				2.65	12
50				2.71	
55				2.75	
2:00				2.72	
05				2.71	15
10				2.40	
15				2.66	
20				2.70	12
25				3.07	10
30	2545			3.35	
35				3.05	
40				2.40	15
45				2.43	
50				2.55	13

METRIC Corporation

Date: 9-14-88

Pumped Well MW-26

Measurements at Well MW-26

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min/ℓ)
55				2.45	14
3:00				2.61	
05				2.47	
10				2.59	
15				2.65	11
20				2.72	
25				1.91	11
30	2605			2.05	12
35				2.39	13
40				2.51	
45				2.37	13
50				2.33	
55				1.99	
4:00				2.60	
05				2.10	12
10				2.98	
15				3.25	10
20				3.27	
25				3.25	11
30	2665			3.15	
35				3.18	11
40				3.11	
45				3.07	
50				2.50	12
55				3.02	
5:00				2.93	13
05				2.75	
10				2.80	
15				2.60	
20				3.28	11
25				[illegible]	

Date: 9-14-88

METRIC Corporation

Pumped Well MW-26

Measurements at Well MW-26

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min/ℓ)
-16 5:30	2725			2.50	13
35				2.85	
40				2.46	
45				3.42	11
50				3.01	
55				3.54	12
6:00				2.73	
05				2.87	11
10				3.35	
15				2.92	12
20				3.25	
25				2.77	
30	2785			2.63	13
35				2.81	
40				3.26	10
45				3.57	
50				2.56	
55				3.14	12
7:00				2.36	
05				0.98	
10				2.69	
15				1.95	
20				2.82	13
25				2.68	
30	2845			2.46	
35				2.80	
40				3.08	
45				2.51	14
50				2.08	
55				1.90	

METRIC Corporation

Date: 9-14-88

Pumped Well MW-26

Measurements at Well MW-26

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min/ℓ)
16 8:00				2.00	
05				2.25	16
10				2.48	
15				3.30	10
20				3.40	
25				2.62	
30	2905			2.24	14
35				1.78	
40				1.53	
45				2.21	17
50				2.58	
55				2.90	11
9:00				3.12	
05				2.20	
10				2.03	
15				2.08	17
20				3.17	
25				2.29	
30	2965			2.32	
35				2.36	14
40				2.42	
45				2.38	
50				2.44	14
55				2.49	
10:00				2.46	
05				2.63	13
10				2.59	
15				2.83	12
20				2.59	
25				2.29	

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METRIC Corporation

Date: 9-14-88

Pumped Well MW-26

Measurements at Well MW-26

Pump Speed: ______ Q: ______ gpm

Static Water Level ______

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min/ℓ)
-16 10:30	3025			2.12	
35				2.05	15
40				2.17	
45				2.36	
50				2.44	14
55				2.27	
11:00				2.09	
05				2.17	15
10				2.30	
15				2.36	
20				2.50	
25				2.48	13
30	3085			2.57	
35				2.64	
40				2.70	12
45				2.26	
50				2.17	
55				1.93	17
12:00				1.86	
05				1.80	
10				1.63	
15				1.86	16
20				2.29	
25				2.53	
30	3145			2.74	13
35				2.43	
40				2.46	
45				2.43	14
50				2.38	
55				2.48	

OGC-004598

METRIC
Corporation

Date: 9-14-88

Pumped Well MW-26

Measurements at Well MW-26

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min/ℓ)
-16 13:00				2.35	
05				2.15	
10				2.19	
15				2.07	16
20				1.91	
25				1.18	
30	3205				Pump started-adj chattering suc
35					
40				2.80	
45				2.50	
50				2.07	18
55				2.03	
14:00				1.92	
05				1.99	16
10				1.95	
15				1.97	
20				1.93	
25				1.74	18
30	3265			2.33	
35				2.62	10
40				2.87	
45				1.81	Power off 2:43:50-2:46:06
50				2.44	12
55				2.88	
15:00				3.00	
05				2.74	
10				2.54	12
15				2.43	
20				2.44	
25				2.47	

OGC-004599

METRIC Corporation

Date: 9-14-88

Pumped Well MW-26

Measurements at Well MW-26

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

	time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min/ℓ)
9-16	15:30	3325			2.39	
	35				2.21	
	40				2.44	
	45				2.27	
	50				2.33	13
	55				2.40	
	16:00				2.39	
	05				2.18	
	10				2.28	15
	15				2.23	
	20				2.16	
	25				2.13	14
	30	3385			2.37	
	35				2.18	
	40				2.10	15
	45				2.14	
	50				2.07	
	55				2.10	15
	17:00				1.97	
	05				2.11	
	10				2.07	
	15				1.97	
	20				1.83	
	25				1.79	15
	30	3445			2.87	
	35				2.43	12
	40				2.46	
	45				2.56	
	50				2.76	
	55				2.19	15

METRIC Corporation

Date: 9-14-88

Pumped Well MW-26

Measurements at Well MW-26

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

	time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min/ℓ)
-16	18:00				1.77	
	05				1.80	
	10				1.63	15
	15				2.87	
	20				3.23	
	25				2.30	12
	30	3505			2.52	
	35				2.22	
	40				2.18	14
	45				2.16	
	50				2.35	
	55				2.35	
	19:00				2.10	15
	05				1.85	
	10				1.57	15
	15				2.60	
	20				2.02	
	25				2.09	
	30	3565			2.37	14
	35				2.23	
	40				1.85	
	45				2.47	14
	50				2.48	
	55				2.34	13
	20:00				2.78	
	05				2.45	
	10				2.07	14
	15				2.70	
	20				2.03	
	25				2.17	

OGC-004601

Date: 9-14-88

METRIC Corporation

Pumped Well MW-26

Measurements at Well MW-26

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min/ℓ)
)-16 30	3625			2.45	14
35				2.33	
40				2.18	14
45				2.32	
50				2.57	
55				2.84	12
21:00				2.95	
05				2.62	13
10				2.04	
15				2.00	
20				2.55	14
25				2.69	
30	3685			2.41	
35				2.30	13
40				2.20	
45				2.35	14
50				2.81	
55				2.72	
22:00				2.02	13
05				1.92	
10				2.10	
15				2.20	15
20				2.11	
25				2.40	13
30	3745			2.46	
35				2.74	11
40				3.38	
45				3.15	15
50				1.69	
55				1.84	

OGC-004602

METRIC
Corporation

Date: 9-14-88

Pumped Well MW-26

Measurements at Well MW-26

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min/ℓ)
23:00				1.65	
05				2.34	
10				2.43	14
15				2.28	
20				2.43	14
25				2.43	
30	3805			2.37	
35				2.60	15
40				2.50	
45				2.45	
50				2.43	14
55				2.28	
24:00				2.20	
05				2.25	14
10				2.30	
15				2.88	10
20				3.20	
25				3.18	11
30	3865			2.93	
35				2.81	
40				2.10	14
45				1.25	
50				1.94	
55				2.10	15
1:00				2.56	
05				3.50	N.R.
10				3.07	
15				2.16	
20				2.01	
25				2.07	

OGC-004603

METRIC Corporation

Date: 9-14-88

Pumped Well MW-26

Measurements at Well MW-26

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min/ℓ)
1:30	3925			1.92	N.R.
35				1.66	
40				2.15	
45				2.80	
50				1.30	14
55				2.43	
2:00				2.47	
05				2.27	
10				2.31	
15				2.26	
20				2.26	13
25				2.65	
30	3985			2.48	
35				2.23	14
40				2.57	
45				2.00	
50				2.32	14
55				2.54	
3:00				3.25	9
05				2.85	
10				2.90	
15				2.82	13
20				2.85	
25				2.29	14
30	4045			2.35	
35				2.90	11
40				2.32	
45				2.25	
50				2.08	
55				1.97	N.R.

METRIC Corporation

Date: 9-14-88

Pumped Well MW-26

Measurements at Well MW-26

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min/ℓ)
4:00				1.88	
05				1.74	14
10				2.77	
15				2.20	
20				1.85	
25				2.53	15
30	4105			2.54	
35				2.39	14
40				2.41	
45				2.95	14
50				2.58	
55				2.60	
5:00				2.75	13
05				2.66	
10				3.45	
15				2.77	
20				2.49	
25				2.15	
30	4165			2.09	15
35				2.40	
40				1.69	
45				1.81	15
50				1.91	
55				2.09	
6:00				2.29	14
05				3.02	
10				2.15	
15				1.97	14
20				2.13	
25				1.98	

METRIC Corporation

Date: 9-14-88

Pumped Well MW-26

Measurements at Well MW-26

Pump Speed: ______ Q: ______ gpm

Static Water Level ______

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min/ℓ)
30	4225			2.03	15
35				1.82	
40				2.06	
45				2.45	14
50				2.42	
55				2.49	13
7:00				2.52	
05				2.53	Stop Pump
10	4265			2.53	
15					
20					
25					
30					
35					
40					
45					
50					
55					
8:00					
05					
10					
15					
20					
25					
30					
35					
40					
45					
50					
55					

METRIC Corporation

Date: 9-14-88

RECOVERY

Pumped Well MW-26

Measurements at Well MW-26

Pump Speed: ______ Q: ______ gpm

Static Water Level ______

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min/ℓ)
7:10:00	4265	0	-	2.53	
15	4265.25	0.25	17061	2.38	
30	4265.50	0.50	8531	2.28	
45	4265.75	0.75	5688	2.13	
11:00	4266.0	1.0	4266	2.01	
30	4266.5	1.5	2844	1.77	
12:00	4267.0	2.0	2134	1.63	
30	4267.5	2.5	1707	1.39	
13:00	4268.0	3.0	1423	1.29	
30	4268.5	3.5	1220	1.19	
14:00	4269.0	4.0	1067	1.10	
30	4269.5	4.5	949	1.00	
15:00	4270.0	5.0	854	0.95	
16	4271	6	712	0.77	
17	4272	7	610	0.65	
18	4273	8	534	0.57	
19	4274	9	475	0.53	
20	4275	10	428	0.46	
22	4277	12	356	0.33	
24	4279	14	306	0.27	
26	4281	16	268	0.23	
28	4283	18	238	0.20	
30	4285	20	214	0.15	
35	4290	25	172	0.15	
40	4295	30	143	0.10	
45	4300	35	123	0.06	
50	4305	40	108	0.04	
8:00	4315	50	86	0.09	
10	4325	60	72	0.05	
29	4335	70	62	0.07	

METRIC Corporation

Date: 9-14-88

Pumped Well MW-26

Measurements at Well MW-26

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min/ℓ)
8:30	4345	80	54	0.06	
9:00	4375	110	40	0.07	
9:30	4405	140	31	0.07	
10:00	4435	170	26	0.05	
11:00	4495	230	20	0.03	
12:00	4555	290	16	0.01	
13:00	4615	350	13	0.04	
14:00	4675	410	11	0.05	
15:00	4735	470	10	0.07	
18:00	4915	650	7.6	0.12	

METRIC
Corporation

Date: 9-20-88

Pumped Well MW-27

Measurements at Well MW-27

Pump Speed: ______ Q: 0.11741 gpm

Static Water Level ______

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min.sec/ℓ)
-20 10:10:00	0			0	
15	.25			–	
30	.5			0.43	
45	.75			1.48	
10:11:00	1			2.32	
30	1.5			3.08	
12:00	2.0			3.43	
30	2.5			3.67	
13:00	3.0			3.78	
30	3.5			3.79	
14:00	4.0			3.48	
30	4.5			3.38	
15:00	5			3.48	
16:00	6			3.64	
17:00	7			3.71	
18:00	8			3.66	
19:00	9			3.45	
20:00	10			3.28	Discharge 2'16"/ℓ
22	12			3.04	
24	14			2.85	
26	16			2.88	
28	18			2.78	
30	20			2.69	
35	25			2.65	2'16"/ℓ
40	30			2.86	
45	35			2.80	2'10"/ℓ
50	40			2.98	
55	45			2.95	
11:00:00	50			3.25	
11:10:00	60			2.25	2'07"/ℓ

Date: 9-20-88

METRIC Corporation

Pumped Well MW-27

Measurements at Well MW-27

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min·sec/ℓ)
-20 11:20	70			2.98	2'15"/ℓ
11:30	80			2.86	
11:40	90			2.25	2'19"/ℓ
11:50	100			2.58	
12:00	110			2.90	2'12"/ℓ
12:20	130			2.56	2'18"/ℓ
40	150			3.03	2'19"/ℓ
13:00	170			3.25	2'17"/ℓ
30	200			3.13	2'13"/ℓ
14:00	230			2.87	2'11"/ℓ
30	260			2.72	2'23"/ℓ
15:00	290			3.06	2'13",2'26",2'9",2'2
30	320			2.94	2'23", 2'14"
16:00	350			2.73	2'23", 2'19"
17:00	410			2.68	2'7", 26"
30	440			2.79	2'13"
18:00	470			2.68	2'35" adj 2'21"
19:00	530			2.69	2'14", 2'12"
20:00	590			2.85	2'10"
21:00	650			2.48	2'3", 2'25"
22:00	710			2.70	2'8"
23:00	770			2.66	2'15"
24:00	830			2.72	2'0" adj
1:00	890			2.54	2'20"
2:00	950			2.68	2'13"
3:00	1010			2.75	2'10"
4:00	1070			2.96	1'52" adj 2'16"
5:00	1130			2.88	2'10"
6:00	1190			3.01	2'17"
7:00	1250			3.38	1'48" adj 2'15"

METRIC
Corporation

Date: 9-20-88

Pumped Well MW-27

Measurements at Well MW-27

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

	time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min·sec/ℓ)
21	8:00	1310			3.24	2'10"/ℓ
	9:00	1370			2.39	2'15"/ℓ
	10:00	1430			2.36	2'17"/ℓ
	11:00	1490			2.78	2'15"/ℓ
	12:00	1550			2.87	2'17"/ℓ
	13:00	1610			2.51	2'18"/ℓ
	14:00	1670			2.47	2'15"/ℓ
	15:00	1730			2.95	2'18"/ℓ
	16:00	1790			3.15	2'17"/ℓ
	17:00	1850			3.18	2'11"/ℓ
	18:00	1910			3.68	2'10"/ℓ
	19:00	1970			3.19	2'14"
	20:00	2030			3.04	2'3" adj
	21:00	2090			3.00	2'11"
	22:00	2150			3.18	2'10"
	23:00	2210			2.85	2'10"
	24:00	2270			2.77	2'22"
.2	1:00	2330			2.81	2'11"
	2:00	2390			2.94	2'6"
	3:00	2450			2.83	2'15"
	4:00	2510			2.88	2'27" adj
	5:00	2570			2.66	2'14"
	6:00	2630			2.87	–
	7:00	2690			2.58	2'15"/ℓ
	8:00	2750			3.11	2'17"/ℓ
	9:00	2810			2.56	2'17"/ℓ
	10:00	2870			2.46	adj to 2'10"
	11:00	2930			2.61	2'14"
	12:00	2990			2.71	2'12"
	13:00	3050			2.98	

Date: 9-20-88

METRIC Corporation

Pumped Well MW-27

Measurements at Well MW-27

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

	time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min·sec/ℓ)
22	14:00	3110			2.61	2'19"/
	15:00	3170			2.61	2'29"/
	16:00	3230			2.86	2'8"/
	17:00	3290			2.48	2'23"/
	18:00	3350			2.54	2'9"/
	19:00	3410			2.87	2'11"
	20:00	3470			3.01	2'9"
	21:00	3530			2.83	2'28"
	22:00	3590			2.98	1'48" adj
	23:00	3650			2.75	2'18"
	24:00	3710			2.85	2'8"
23	1:00	3770			2.80	2'16"
	2:00	3830			2.54	2'17"
	3:00	3890			2.90	2'12"
	4:00	3950			3.11	2'10"
	5:00	4010			2.93	2'7" adj
	6:00	4070			2.80	2'19"
	7:00	4130			2.54	2'25" adj
	8:00	4190			2.22	2'14"
	8:10	4200			1.88	Stop pump

METRIC Corporation

Date: 9-20-88

RECOVERY

Pumped Well MW-27

Measurements at Well MW-27

Pump Speed: ________ **Q:** ________ gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min·sec/ℓ)
8:10:00	4200	0.0		1.88	
8:10:15	4200.25	0.25	16801	1.10	
30	4200.50	0.50	8401	0.73	
45	4200.75	0.75	5601	0.47	
11:00	4201.00	1.00	4201	0.35	
11:30	4201.5	1.50	2801	0.20	
12:00	4202.0	2.00	2101	0.11	
12:30	4202.5	2.50	1681	0.09	
13:00	4203.0	3.00	1401	0.07	
13:30	4203.5	3.5	1201	0.05	
14:00	4204.0	4.0	1051	0.04	
14:30	4204.5	4.5	934	0.03	
15:00	4205.0	5.0	841	0.03	
16:00	4206	6	701	0.01	
17	4207	7	601	0.01	
18	4208	8	526	0.01	
19	4209	9	468	0.00	
20	4210	10	421	0.0	
22	4212	12	351	0.0	
24	4214	14	301	0.0	
26	4216	16	264	0.0	
28	4218	18	234	+0.01	
8:30	4220	20	211	+0.02	
35	4225	25	169	+0.02	
40	4230	30	141	+0.02	
45	4235	34	121	+0.03	
50	4240	40	106	+0.03	
55	4245	45	94	+0.02	
9:00	4250	50	85	0.03	
9:10	4260	60	71	0.03	

Date: 9-20-88

METRIC Corporation

Pumped Well MW-27

Measurements at Well MW-27

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min·sec/ℓ)
9:20	4270	70	61	+0.03	
9:30	4280	80	54	+0.03	
10:00	4310	110	39	+0.03	
11:27					
13:49	3497	197	22	+0.03	
16:47	4539	339	13	+0.07	
	4717	407	11	+0.09	

METRIC Corporation

Date: 10-11-88

Pumped Well MW-28

Measurements at Well MW-28

Pump Speed: ________ Q: 0.07045 gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min·sec/ℓ)
12:20:00	0			0.05	
:15	.25			0.09	
:30	.50			0.16	
:45	.75			0.18	
21:00	1.0			0.21	
21:30	1.5			0.25	
22:00	2.0			0.31	
22:30	2.5			0.37	
23:00	3.0			0.42	
23:30	3.5			0.45	
24:00	4.0			0.49	
24:30	4.5			0.55	
25:30	5			0.56	
26	6			0.63	
27	7			0.70	
28	8			0.77	
29	9			0.83	Reduced motor spe
30	10			0.87	
12:32	12			0.93	
12:34	14			0.99	
36	16			1.04	
38	18			1.11	
40	20			1.13	
45	25			1.20	4'12"/ℓ
50	30			1.24	
55	35			1.23	
13:00	40			1.28	3'02"/ℓ
13:20	60			1.32	4'29'/ℓ
13:40	80			1.43	4'36"/ℓ
14:00	100				

METRIC Corporation

Date: 10-11-88

Pumped Well MW-28

Measurements at Well MW-28

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min·sec/ℓ)
14:30	130			1.44	4'32" adj
15:00	160			1.68	3'36" adj
15:30	190			1.58	5'26" adj
16:00	220			1.71	4'50" adj
17:00	280			1.88	2'06" adj
18:00	240			1.86	3'39" adj
19:00	400			1.73	3'49" adj
20:00	460			1.82	5'23" adj
21:00	520			1.81	3'45" tech adj.
22:00	580			1.94	3'38"
23:00	640			2.11	3'32"
24:00	700			2.20	3'48"
1:00	760			2.20	4'00"
2:00	820			2.16	3'49"
3:00	880			2.11	4'06"
4:00	940			2.18	3'55"
5:00	1000			2.19	3'47"
6:00	1060			2.30	3'47"
7:00	1120			2.21	3'47"
8:00	1180			2.33	3'47"
9:00	1240			2.30	3'43"
10:00	1300			2.43	3'44"
11:00	1360			2.52	3'46"
12:00	1420			2.59	3'45"
13:00	1480			2.69	3'43"
14:00	1540			2.45	3'42"
15:00	1600			2.48	3'43"
16:00	1660			2.50	3'46"
17:00	1720			2.52	3'43"
18:00	1780			2.54	3'44"

METRIC Corporation

Date: 10-11-88

Pumped Well MW-28

Measurements at Well MW-28

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

	time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min·sec/ℓ)
0/12	19:00	1840			2.49	3'45"
	20:00	1900			2.66	3'44"
	21:00	1960			2.59	3'46"
	22:00	2020			2.67	3'48"
	23:00	2080			2.39	3'52"
	24:00	2140			2.29	4'00"
	1:00	2200			2.17	4'08"
	2:00	2260			2.62	3'46"
	3:00	2320			2.74	3'44"
	4:00	2380			2.74	3'45"
	5:00	2440			2.63	3'48"
	6:00	2500			2.58	3'48"
	7:00	2560			2.61	3'48"
	8:00	2620			2.68	3'48"
	9:00	2680			2.42	3'47"
	10:00	2740			2.59	3'49"
	11:00	2800			2.68	3'47"
	12:00	2860			2.53	3'47"
	13:00	2920			2.53	3'46"
	14:00	2980			2.51	3'46"
	15:00	3040			2.53	3'45"
	16:00	3100			2.50	3'45"
	17:00	3160			2.69	3'44"
	18:00	3220			2.71	3'43"
	19:00	3280			2.61	3'45"
	20:00	3340			2.65	3'42"
	21:00	3400			2.70	3'44"
	22:00	3460			2.56	3'44"
	23:00	3520			2.66	3'44"
	24:00	3580			2.63	3'45"

METRIC Corporation

Date: 10-11-88

Pumped Well MW-28

Measurements at Well MW-28

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min.sec/ℓ)
1:00	3640			2.68	3'48"
2:00	3700			2.59	3'48"
3:00	3760			2.54	3'49"
4:00	3820			2.57	3'49"
5:00	3880			2.66	3'50"
6:00	3940			2.46	3'54" Hole Cleaned
7:00	4000			2.77	3'36"
8:00	4060			2.72	3'43"
9:00	4120			2.63	3'43"
10:00	4180			2.64	3'43"
11:00	4240			2.65	3'43"
12:00	4300			2.67	3'45"
12:20	4320	0		2.56	Pump Off
12:20:15	4320.25	.25	17,281	2.45	
:20:30	4320.50	.5	8,641		
:20:45	4320.75	.75	5,761	2.15	
:21:00	4321.0	1.0	4,321	2.14	
:21:30	4321.5	1.5	2,881	2.11	
:22:00	4322.0	2.0	2,161	2.10	
:22:30	4322.5	2.5	1,729	2.08	
:23:00	4323.0	3.0	1,444	2.06	
:23:30	4323.5	3.5	1,235	2.05	
:24:00	4324.0	4.0	1,081	2.03	
:24:30	4324.5	4.5	961	2.01	
:25:00	4325	5.0	865	2.00	
:26	4326	6	721	1.97	
:27	4327	7	618	1.94	
:28	4328	8	541	1.92	
:29	4329	9	481	1.89	
:30	4330	10	433	1.87	

METRIC Corporation

Date: 10-11-88

Pumped Well MW-28

Measurements at Well MW-28

Pump Speed: ________ Q: ________ gpm

Static Water Level ________

time (h:m:s)	t (min)	t' (min)	t/t'	Drawdown (ft)	Discharge (min·sec/ℓ)
12:32	4332	12	361	1.82	
34	4334	14	310	1.77	
36	4336	16	271	1.71	
38	4338	18	241	1.66	
40	4340	20	217	1.62	
50	4350	30	145	1.44	
55	4355	35	124	1.33	
13:00	4360	40	109	1.26	
13:30	4390	70	63	0.85	
14:00	4420	100	44	0.56	
14:30	4450	130	34	0.32	
15:00	4480	160	28	0.20	
16:00	4540	220	21	0.07	
17:00	4600	280	16	0.01	
18:00	4660	340	14	0.00	
19:00	4720	400	12	0.00	

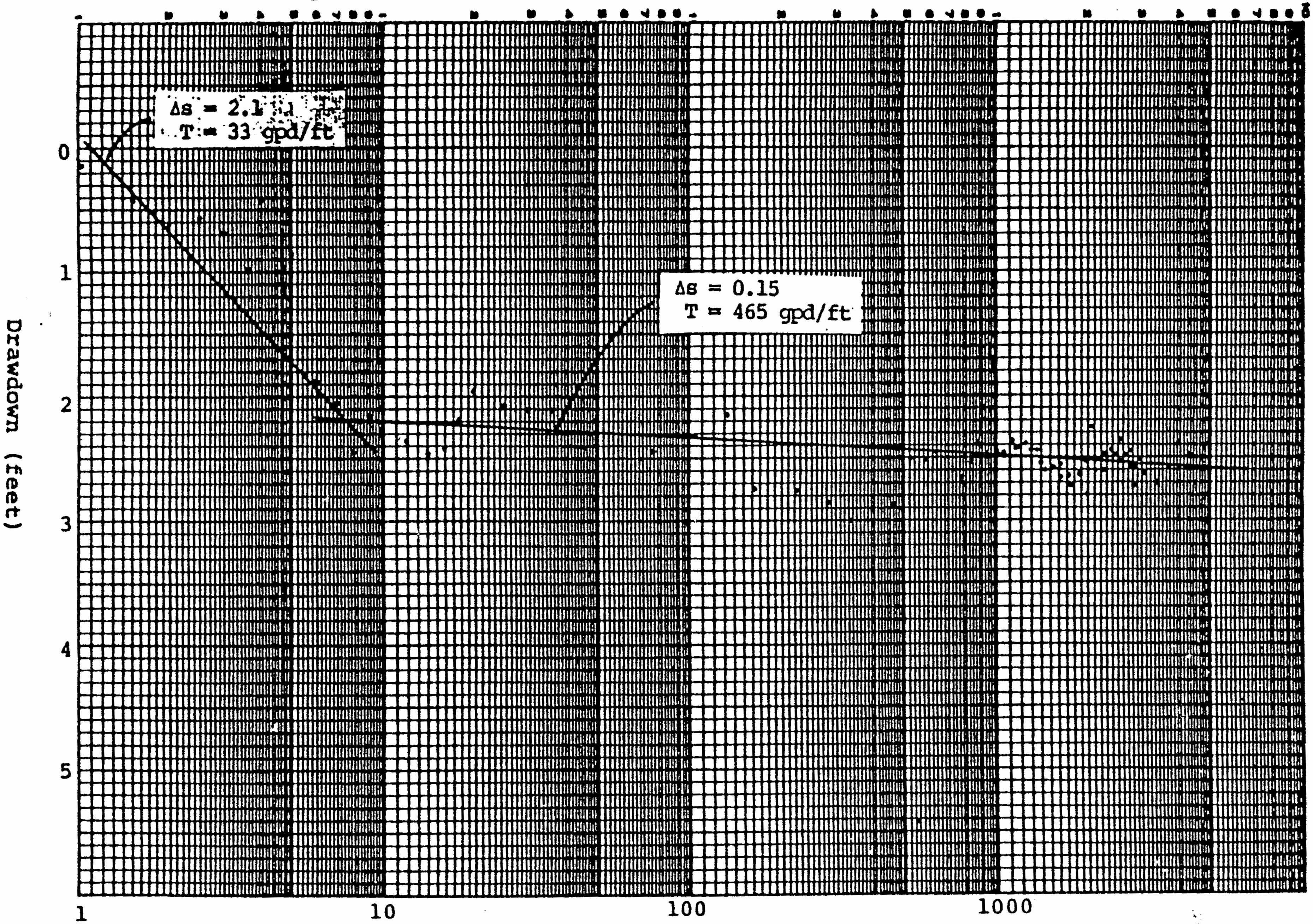
APPENDIX B

SEMI-LOG PLOTS

Pumped Well : MW-23
Observations at Well :MW-23

Time-Drawdown
Q = 0.264gpm

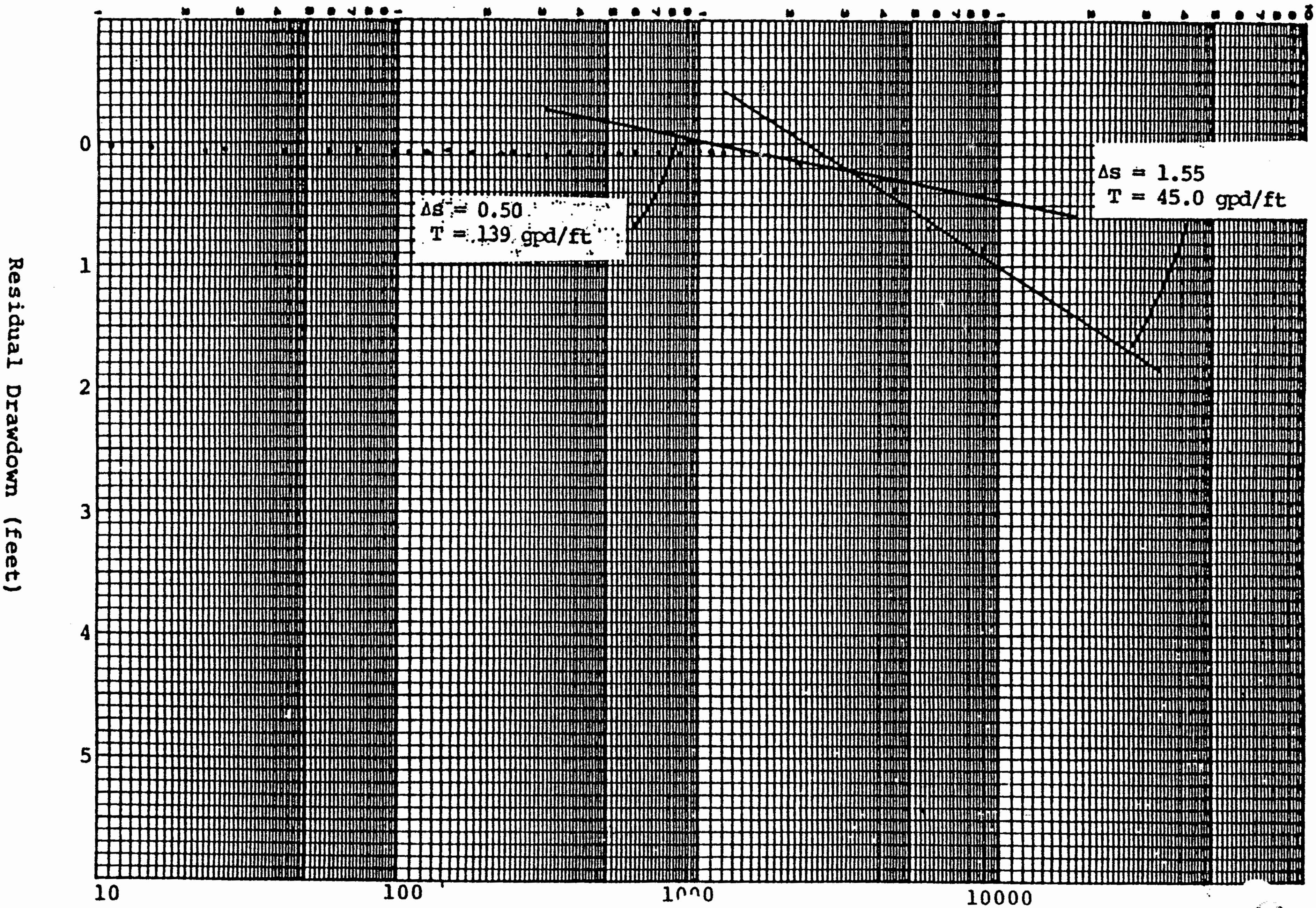
METRIC Corporation
Date: 9-27-88

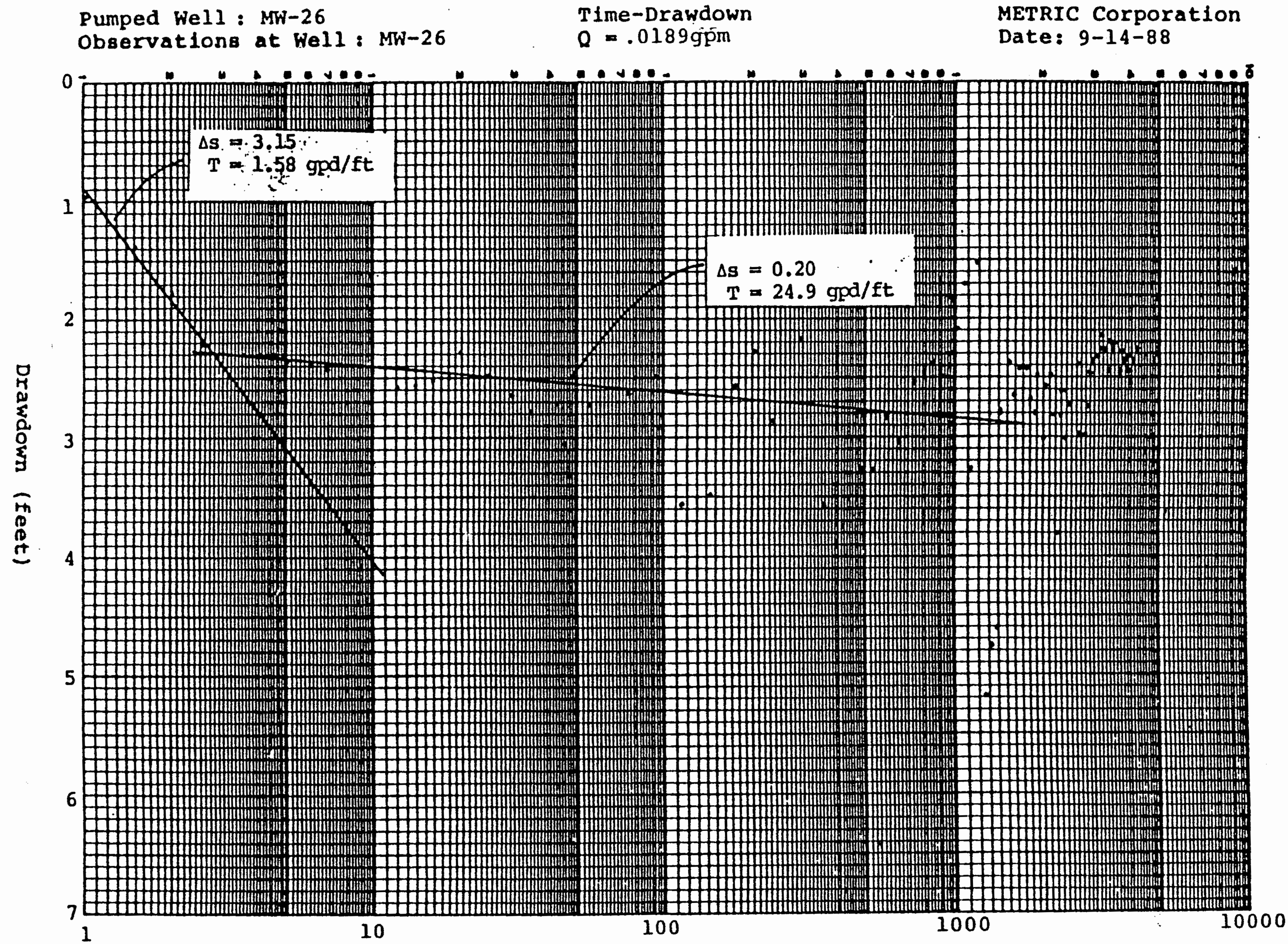


Pumped Well : MW-23
Observations at Well : MW-23

Residual-Drawdown
Q = 0.264 gpm

METRIC Corporation
Date: 9-27-88

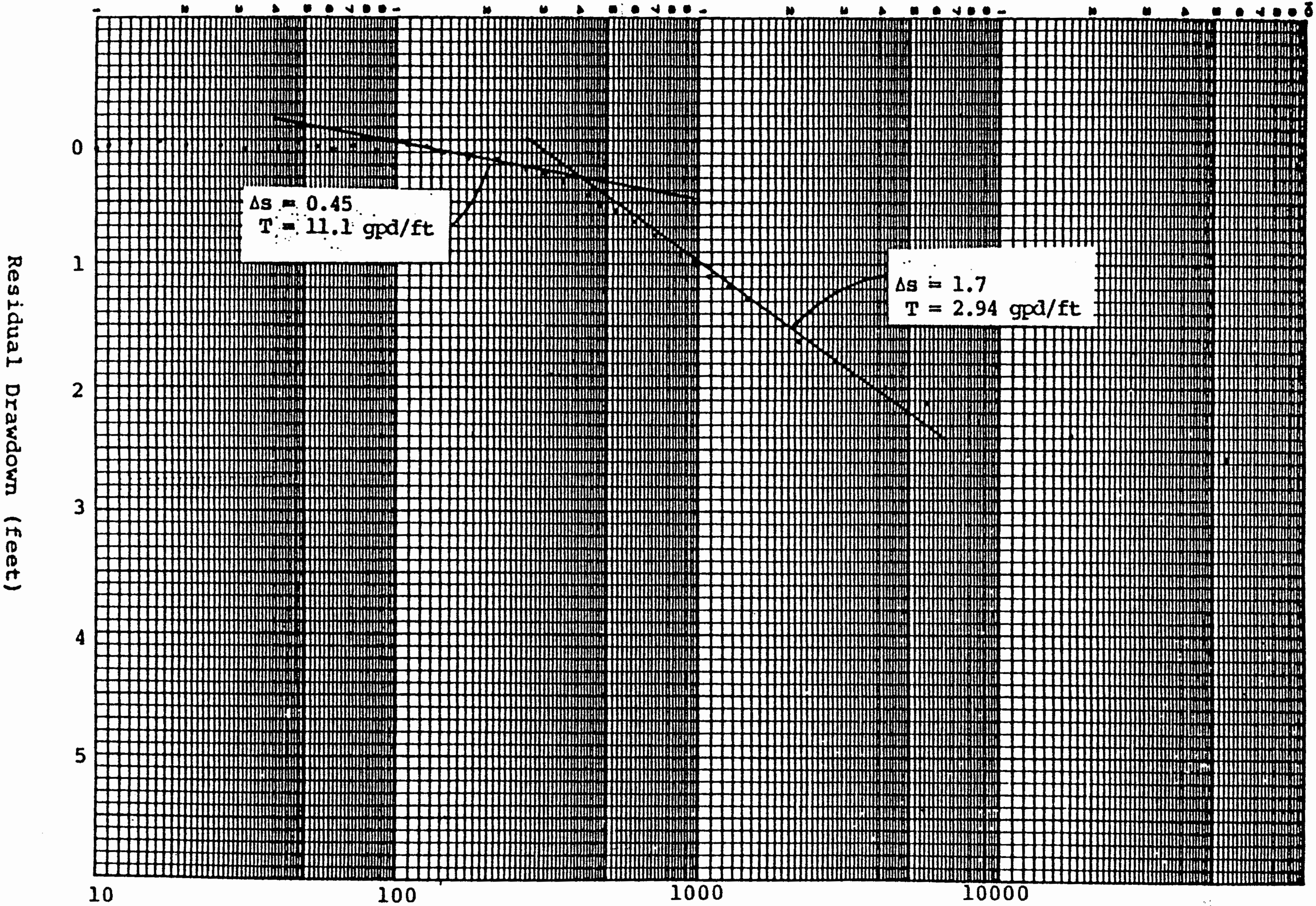



Pumped Well : MW-26
Observations at Well : MW-26
Time-Drawdown
Q = .0189gpm
METRIC Corporation
Date: 9-14-88
Δs = 3.15
T = 1.58 gpd/ft
Δs = 0.20
T = 24.9 gpd/ft
Drawdown (feet)
0
1
2
3
4
5
6
7
1
10
100
1000
10000

Pumped Well : MW-26
Observations at Well : MW-26

Residual-Drawdown
Q = .0189gpm

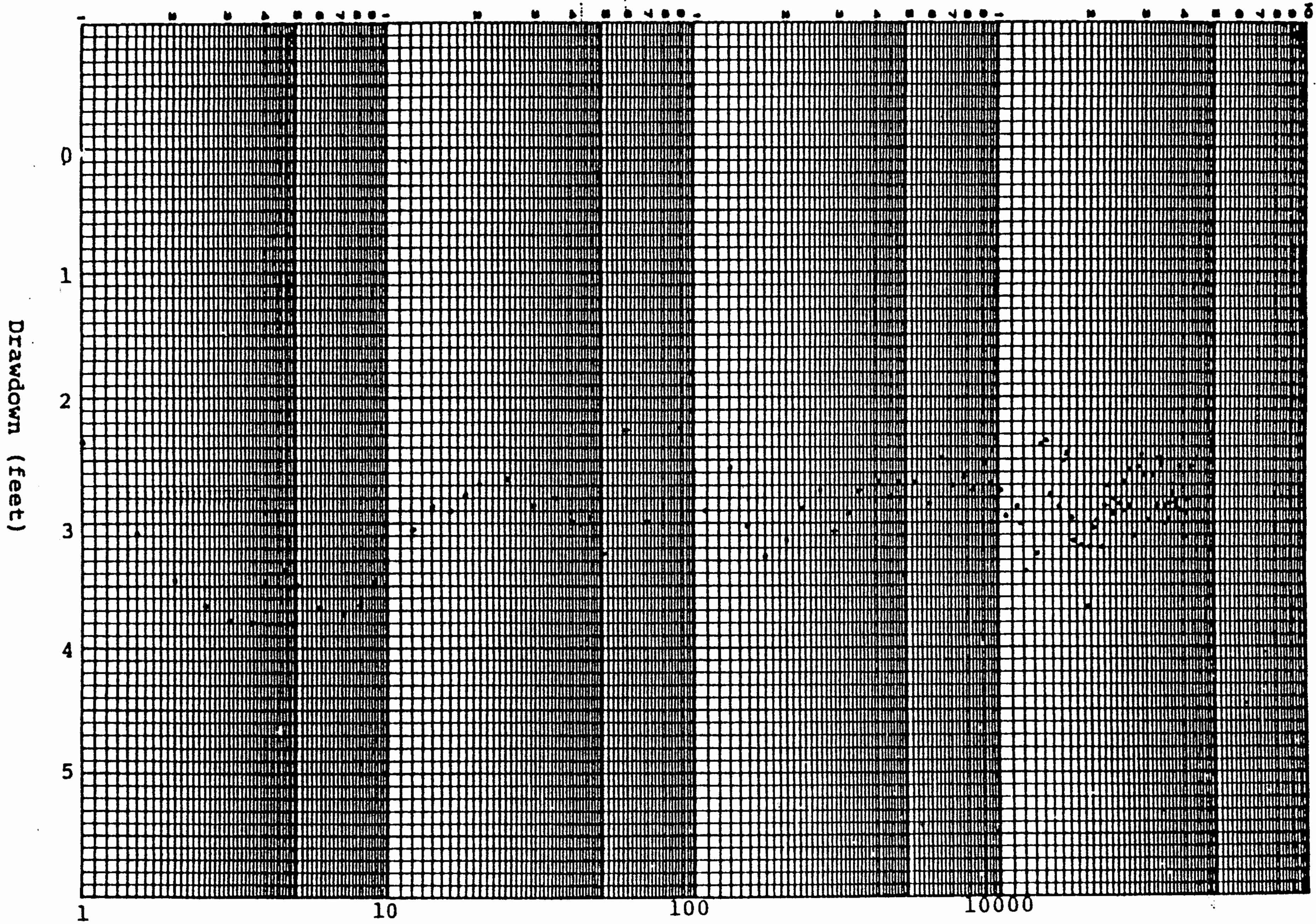
METRIC Corporation
Date: 9-14-88



Pumped Well : MW-27
Observations at Well : MW-27

Time-Drawdown
Q = 0.117gpm

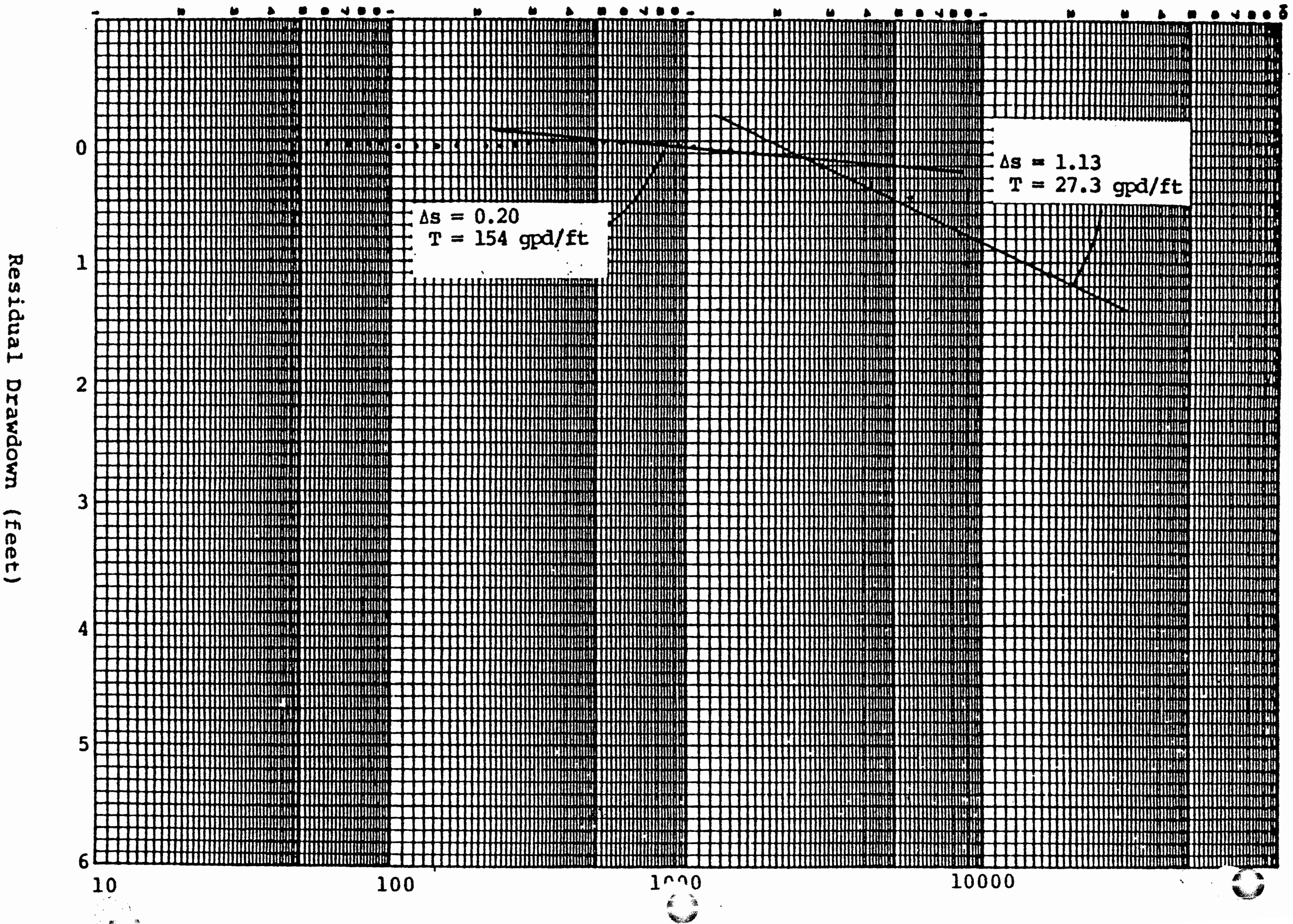
METRIC Corporation
Date: 9-20-88



Pumped Well : MW-27
Observations at Well : MW-27

Residual-Drawdown
Q = 0.117gpm

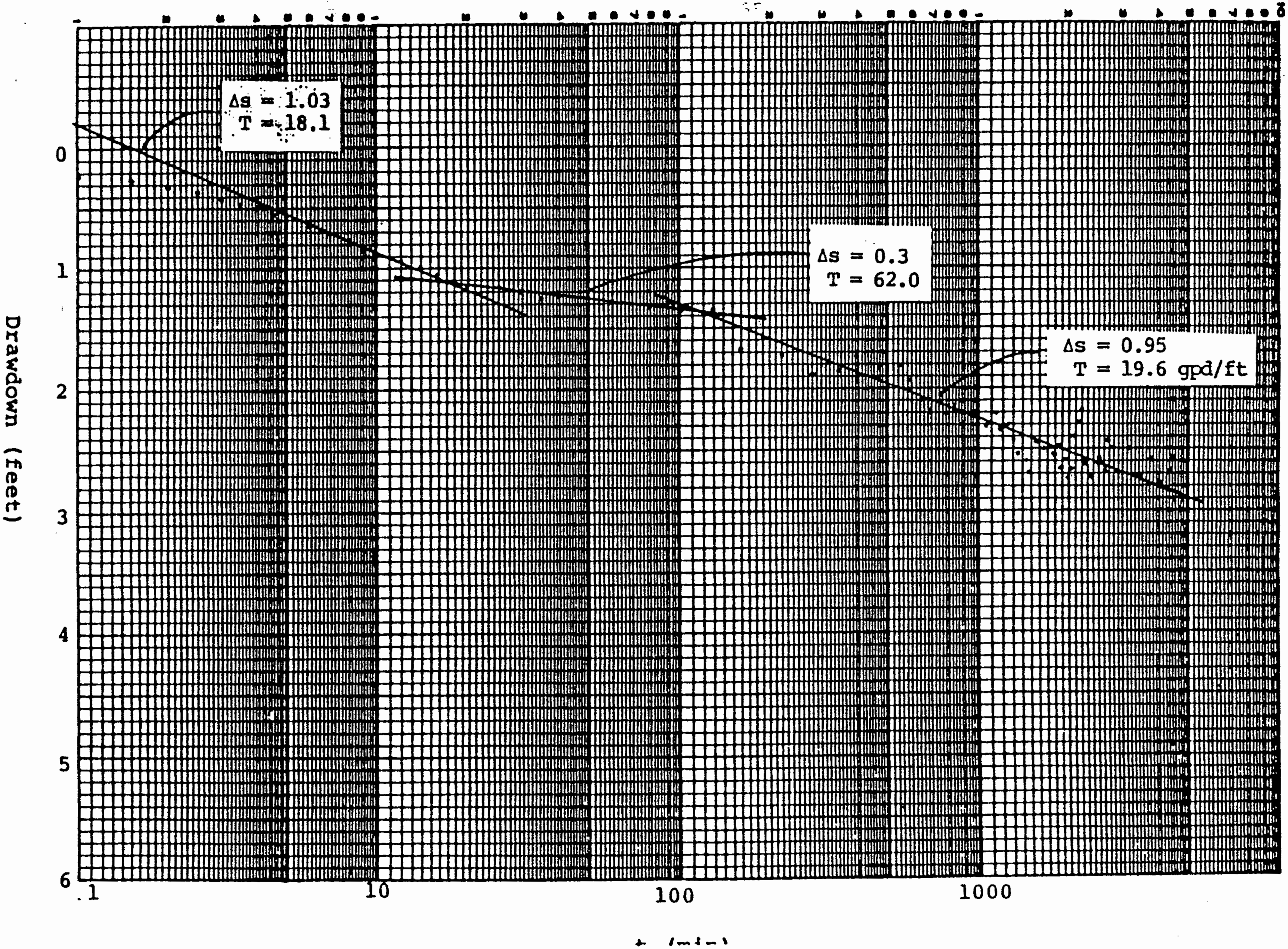
METRIC Corporation
Date: 1-20-88



Pumped Well : MW-28
Observations at Well : MW-28

Time-Drawdown
Q = .0705gpm

METRIC Corporation
Date: 10-11-88

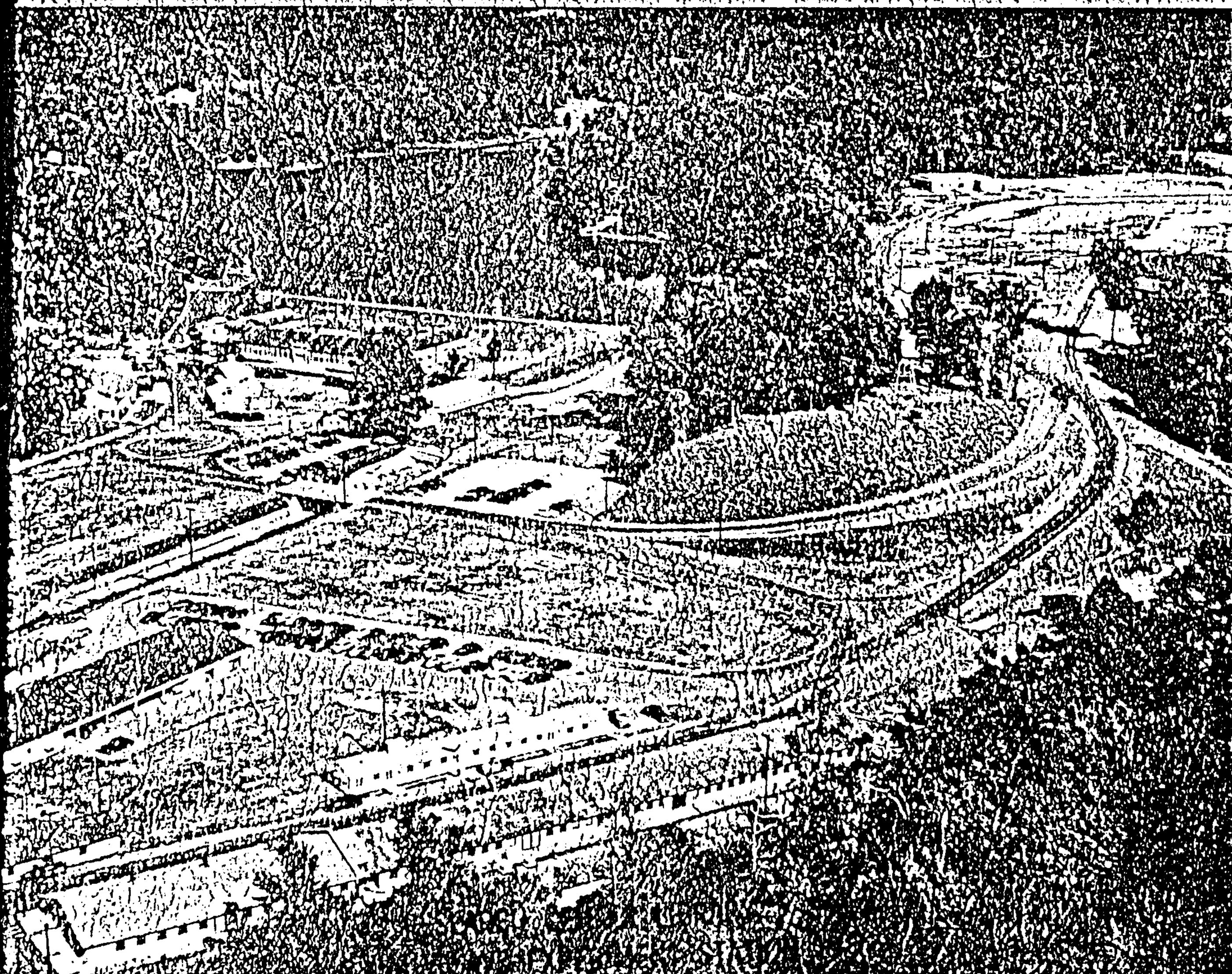


APPENDIX 3

REFERENCES

Hvorslev, M. Juul,
<u>Time Lag and Soil Permeability in Ground-Water Observations</u>,
Bulletin No. 36, Waterways Experiment Station,
U.S. Army Corp of Engineers, April 1951.

TIME LAG AND SOIL PERMEABILITY IN GROUND-WATER OBSERVATIONS



VICKSBURG, MISSISSIPPI

BULLETIN NO. 36

WATERWAYS EXPERIMENT STATION

CORPS OF ENGINEERS, U. S. ARMY

VICKSBURG, MISSISSIPPI

TIME LAG AND SOIL PERMEABILITY IN GROUND-WATER OBSERVATIONS

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APRIL 1951

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PREFACE

With the advance of soil mechanics and its applications in the design and construction of foundation and earth structures, the influence of ground-water levels and pore-water pressures is being considered to a much greater extent than a decade or two ago. Rapid and reliable determination of such levels and pressures is assuming increasing importance, and sources of error which may influence the measurements must be eliminated or taken into account.

A review of irregularities in ground-water conditions and the principal sources of error in ground-water observations is presented in the first part of this paper. Many of these sources of error can be eliminated by proper design, installation, and operation of observation wells, piezometers, or hydrostatic pressure cells. However, other sources of error will always be present and will influence the observations to a greater or lesser degree, depending on the type of installation and the soil and ground-water conditions. Conspicuous among the latter sources of error is the time lag or the time required for practical elimination of differences between hydrostatic pressures in the ground water and within the pressure measuring device.

Theoretical and experimental methods for determination of the time lag and its influence on the results of ground-water observations are proposed in the second part of the paper. Simplifications are obtained by introducing a term called the basic time lag, and solutions are presented for both static, uniformly changing, and fluctuating ground-water conditions. The influence of a secondary or stress adjustment time lag, caused by changes in void ratio or water content of the soil during the observations, is discussed.

The third part of the paper contains data which will assist in the practical application of the proposed methods. Formulas for determination of the flow of water through various types of intakes or well points are summarized and expanded to include conditions where the coefficients of the vertical and horizontal permeability of the soil are different. Examples of computations and a table facilitate preliminary estimates of the basic time lag for the principal types of installations and soils, and determination of the actual time lag is illustrated by several examples of field observations and their evaluation.

Determination of the coefficients of vertical and horizontal permeability for the soil in situ by means of time lag observations is theoretically possible and is discussed briefly in the closing section of the paper. Such field determinations of permeability have many potential advantages, but further research is needed in order to eliminate or determine the influence of various sources of error.

An abstract of the paper was presented in January 1949 at the Annual Meeting of the American Society of Civil Engineers, and a limited number of copies of the first draft were distributed. In this final version of the paper the individual sections have been rearranged and amplified to some extent, and some new sections have been added.

ACKNOWLEDGEMENTS

The writer wishes to express his appreciation of the many valuable suggestions made by Messrs. Reginald A. Barron, Stanley J. Johnson, and the Reports Branch of the Waterways Experiment Station in course of their reviews of the paper.

CONTENTS

CONTENTS (Cont'd)

NOTATION

x Distance from ref. level to piezometer level for steady state, cm.
y Distance from ref. level to piezometer level, transient state, cm.
z Distance from reference level to the outside piezometric level, cm.

x_o, y_o, z_o Values of x, y, and z for $t = 0$, cm.

x_a Amplitude of fluctuating piezometer levels for steady state, cm.
z_a Amplitude of fluctuating outside piezometric levels, cm.

h Increment change in active head, cm.
H Active head, $H = z - y$, cm.
H_o Active head for $t = 0$, cm.
H_c Constant piezometric head, cm.

h' Increment change in transient differential head, cm.
H' Transient differential head, $H' = y - x$, cm.
H'_o Transient differential head for $t = 0$, cm.

A Area of casing, piezometer, manometer, or pressure cell, cm^2.
d Diameter of piezometer, manometer, or pressure cell, cm.
D Diameter of effective intake, boring, or well point, cm.
e Base of natural logarithms, no dimension.
E Equalization ratio, $E = (H_o - H)/H_o$, no dimension.
F Intake shape factor, from $q = F k H$, cm.
L Length of effective intake or well point, cm.

k Coefficient of permeability, cm/sec.
k_h Coefficient of horizontal permeability, undisturbed soil, cm/sec.
k_m Mean coefficient of permeability, $k_m = \sqrt{k_h \cdot k_v}$, cm/sec.
k_v Coefficient of vertical permeability, undisturbed soil, cm/sec.
k'_v Coefficient of vertical permeability, soil in casing, cm/sec.
m Transformation ratio, $m = \sqrt{k_h/k_v}$, no dimension.

q Rate of flow at time t and head H, cm^3/sec.
q_o Rate of flow at time $t = 0$ and head H_o, cm^3/sec.
t Time, seconds unless otherwise indicated.
- s Seconds)
- m Minutes) Used only in Figs. 14, 16, 17.
- h Hours)
- d Days)

t_s Phase shift of sinusoidal wave, seconds unless otherwise indicated.
T Basic time lag, $T = V/q$, seconds unless otherwise indicated.
T_w Period of sinusoidal wave, seconds unless otherwise indicated.
V Total volume of flow required for pressure equalization, cm^3.

α Rate of linear change in pressure, cm/sec.
γ Unit weight, g/cm^3.
ε Deflection of diaphragm in pressure cell, cm.

TIME LAG AND SOIL PERMEABILITY IN GROUND-WATER OBSERVATIONS

by

M. Juul Hvorslev*

INTRODUCTION

Accurate determination of ground-water levels and pressures is required, not only in surveys of ground-water supplies and movements, but also for proper design and construction of most major foundation and earth structures. The depth to the free ground-water level is often a deciding factor in the choice of types of foundations, and it governs the feasibility of and the methods used in deep excavations. A recent fall or rise in ground-water levels may be the cause of consolidation or swelling of the soil with consequent settlement or heaving of the ground surface and foundations. The existence of artesian or excess pore-water pressures greatly influences the stability of the soil; determination of pore-water pressures permits an estimate of the state or progress of consolidation, and it is often essential for checking the safety of slopes, embankments, and foundation structures. In general, determination of both free ground-water levels and pore-water pressures at various depths is usually a necessary part of detailed subsurface explorations, and the observations are often continued during and for some period after completion of foundation and earth structures.

Ground-water levels and pore-water pressures are determined by means of borings, observation wells, or various types of piezometers and hydrostatic pressure cells. During the advance of a bore hole or immediately after installation of a pressure measuring device, the hydrostatic pressure within the hole or device is seldom equal to the original pore-water pressure. A flow of water to or from the boring or pressure measuring device then takes place until pressure differences are eliminated, and the time required for practical equalization of the pressures is the time lag. Such a flow with a corresponding time lag also occurs when the pore-water pressures change after initial equalization. It is not always convenient or possible to continue the observations for the required length of time, and adequate equalization cannot always be attained when the pore-water pressures change continually during the period of observations. In such cases there may be considerable difference between the actual and observed pressures, and the latter should then be corrected for influence of the time lag.

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* Consultant, Soils Division, Waterways Experiment Station.

The magnitude of the time lag depends on the type and dimensions of the pressure measuring installation, and it is inversely proportional to the permeability of the soil. A preliminary estimate of the time lag is necessary for the design or selection of the proper type of installation for given conditions. The actual time lag should be determined by field experiments so that subsequent observations may be corrected for its influence, when conditions are such that corrections are required or desirable.

Theoretical and experimental methods for determination of the time lag and its influence on the results of pressure measurements are presented in this paper. These methods are based on the assumptions usually made in the theories on flow of fluids through homogeneous soils, and the results are subject to corresponding limitations. In addition to the time lag, ground-water observations may be influenced by several other sources of error and by irregular and changing ground-water conditions. Therefore, an initial review of ground-water conditions in general and of the principal sources of error in determination of ground-water levels and pressures is desirable in order to clarify the assumptions on which the proposed methods are based, and to delimit the field of application of these methods.

PART I: GROUND-WATER CONDITIONS AND OBSERVATIONS

Irregularities and Variations

Several sources of error in determination of ground-water levels and pressures occur primarily when irregular and/or rapidly changing ground-water conditions are encountered. Regular conditions, with the piezometric pressure level equal to the free ground-water level at any depth below the latter, are the exception rather than the rule. Irregular conditions or changes in piezometric pressure level with increasing depth may be caused by: (a) perched ground-water tables or bodies of ground water isolated by impermeable soil strata; (b) downward seepage to more permeable and/or better drained strata; (c) upward seepage from strata under artesian pressure or by evaporation and transpiration; and (d) incomplete processes of consolidation or swelling caused by changes in loads and stresses. For a more detailed description of these conditions reference is made to MEINZER (20)* and TOLMAN (30); a general discussion of ground-water observations is found in a recent report by the writer (16).

Ground-water levels and pressures are seldom constant over considerable periods of time but are subject to changes by: (a) precipitation, infiltration, evaporation, and drainage; (b) load and stress changes and/or seepage due to seasonal or diurnal variations in water levels of nearby rivers, lakes, estuaries, and the sea; (c) construction operations involving increase or decrease in surface loads and removal or displacement of soil; (d) pumping and discharge of water; (e) variations in temperature and especially freezing and thawing of the upper soil strata; and (f) variations in atmospheric pressure and humidity. The last mentioned variations may cause appreciable and rapid changes in ground-water levels, but the interrelationship between atmospheric and ground-water conditions is not yet fully explored and understood; see HUIZINGA (13), MEINZER (20), and TOLMAN (30). The possibility that minor but rapid changes in ground-water levels and pressures may occur should be realized, since such changes may be misinterpreted and treated as errors, and since they may affect the determination of corrections for actual errors.

Sources of Error in Measurements

The principal sources of error in determination of ground-water levels and

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* Numbers in parentheses refer to references at end of paper.

pressures are summarized in Fig. 1, and some further details are presented in the following paragraphs.

Hydrostatic time lag

When the water content of the soil in the vicinity of the bottom of a bore hole or intake for a pressure measuring device remains constant, and when other sources of error are negligible, the total flow or volume of water required for equalization of differences in hydrostatic pressure in the soil and in the pressure measuring device depends primarily on the permeability of the soil, type and dimensions of the device, and on the hydrostatic pressure difference. The time required for water to flow to or from the device until a desired degree of pressure equalization is attained, may be called the *hydrostatic time lag*. In order to reduce the time lag and increase the sensitivity of the installation to rapid pressure changes, the volume of flow required for pressure equalization should be reduced to a minimum, and the intake area should be as large as possible.

Stress adjustment time lag

The soil structure is often disturbed and the stress conditions are changed by advancing a bore hole, driving a well point or installing and sealing a pressure measuring device, and by a flow of water to or from the device. A permanent and/or transient change in void ratio and water content of the affected soil mass will then take place, and the time required for the corresponding volume of water to flow to or from the soil may be called the *stress adjustment time lag*. The apparent stress adjustment time lag will be increased greatly by the presence of air or gas bubbles in the pressure measuring system or in the soil; see Items 6 to 8, Fig. 1. This time lag and its influence on the results of observations are discussed in greater detail in Part II, pages 21-29.

General instrument errors

Several sources of error may be found in the design, construction, and method of operation of the pressure measuring installation. Among such sources of error may be mentioned: (a) inaccurate determination of the depth to the water surface in wells and piezometers; (b) faulty calibration of pressure gages and cells; (c) leakage through joints in pipes and pressure gage connections; (d) evaporation of water or condensation of water vapors; (e) poor electrical connections and damage to or deterioration of the insulation; (f) insufficient insulation against extreme temperature variations or differences, especially inactivation or damage by frost. The effect of leakage through joints and connections is similar to that of seepage along the outside of conduits, discussed below.

Seepage along conduits

Seepage along the casing, piezometer tubing, or other conduits may take

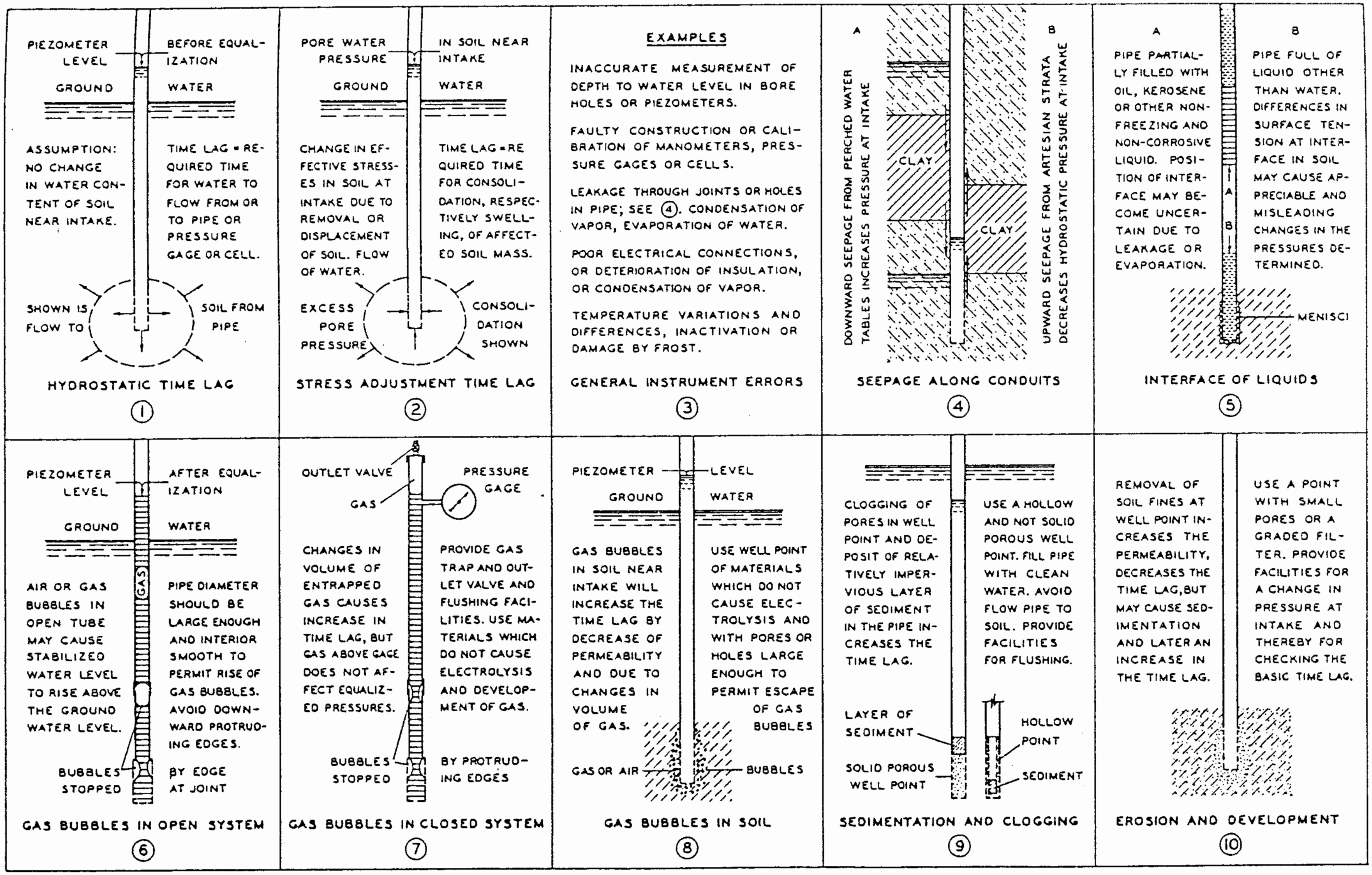

Fig. 1. Sources of error in determination of ground-water pressures

place, especially when irregular ground-water conditions are encountered. As shown in the figure, such seepage may increase or decrease the pore-water pressure in the soil at the bottom of the hole or at the intake for a pressure measuring device. Even under regular ground-water conditions seepage may occur in closed systems with attached manometer or pressure gage, and it will always affect experimental determination of the time lag of the system and of the permeability of the soil. To avoid seepage, the entire piezometer or the well point is often driven into the soil; but this method causes increased disturbance of the soil, and in many cases it is also desirable to surround the well point with a graded sand filter. When the well point is installed in an oversized bore hole, the space between the standpipe and the wall of the hole must be sealed above the well point, preferably in a fairly impermeable stratum. Puddled clay, bentonite mixtures, and cement grout have been used for sealing, but it is not always easy to obtain a tight seal and at the same time avoid stress changes in the surrounding soil because of swelling of the sealing material. A seal consisting of alternate layers of sand and clay balls, compacted by means of an annular tamping tool, has been developed and used successfully by A. CASAGRANDE (2) and (3).

Interface of liquids

To avoid corrosion or inactivation and damage by frost, manometer and pressure gages and the upper part of piezometers may be filled with kerosene or other oils. The difference in specific gravity of water and the liquid used, as well as the position of the interface, must be taken into consideration in determining the pore-water pressure. However, when observations are extended over long periods of time, the position of the interface may change because of evaporation and/or leakage and be difficult to determine. If the interface is in the wall of a well point with very fine pores, or in fine-grained soil outside the well point, additional and considerable errors may be caused by the menisci formed in the pores and by the difference in surface tension of water and the liquid in the pipe and well point.

Gas bubbles in open systems

Air or gas bubbles in an open observation well or piezometer may influence the time lag and cause the stabilized level in the pipe to rise above the ground-water or the piezometric pressure level for the soil. Therefore, the interior of the pipe should be smooth, downward protruding edges or joints should be avoided, and the diameter of the pipe should be large enough to cause the bubbles filling the cross section to rise to the surface. These requirements are fulfilled by use of seamless and jointless plastic tubing, CASAGRANDE (2) and (3), and when the inside diameter of such tubing is 3/8 in. or more.

Gas bubbles in closed systems

Air or gas bubbles in a closed pipe connected to a manometer or pressure gage will increase the time lag, but gas above the connection to the pressure gage, and small gas bubbles adhering to the walls of the pipe, will not affect the stabilized pressure indicated by the gage. Gas bubbles below the gage connection and filling the entire cross section of the pipe will influence the indicated stabilized pressure. The pipe should be provided with an air trap and outlet valve at top, and should be smooth, without protruding joints, and of a diameter large enough to permit free rise of gas bubbles. At least, facilities for occasional flushing should be provided and the entire installation should be composed of materials which do not cause development of gases through electrolysis.

Gas entrapped in the water-filled space below the diaphragm of a hydrostatic pressure cell of the type shown in Case 9, Fig. 13 -- or in the perforated cover plate or porous stone -- will not influence the ultimate pressure indicated but will greatly increase the time lag of the pressure cell. It is conceivable that a material accumulation of gas below the diaphragm may cause the time lag of a hydrostatic pressure cell to be considerably greater than that of a closed piezometer with attached manometer or Bourdon pressure gage.

Gas bubbles in soil

Air and other gases are often entrapped in the pores of the soil, even below the ground-water level, or dissolved in the water. When the gas bubbles migrate to and cluster around the well point or are released there from solution in the water, the time lag will be increased on account of volume changes of the gas and because the gas bubbles decrease the permeability of the soil. The well point should consist of materials which do not cause development of gases through electrolysis. It is also advisable to avoid an excessive decrease of the hydrostatic pressure inside the well point and a consequent decrease of the pore-water pressure in the surrounding soil, since a decrease in hydrostatic pressure may cause release of gases dissolved in the water.

Sedimentation and clogging

Sediment in the water of the standpipe or piezometer will ultimately settle at the bottom of the pipe. When a solid porous well point is used, the sediment may form a relatively impervious layer on its top and thereby increase the time lag. Therefore, a hollow well point should be used, the pipe should be filled with clean water, and facilities for occasional cleaning and flushing are desirable. An outward flow of water from the pipe and well point may carry sediment in the pipe into the pores of the walls of the point or of the surrounding soil and may thereby cause clogging and a further increase in time lag. Therefore and insofar as possible, a strong outward flow of water from well point should be avoided.

Erosion and development

A strong inward flow of water may carry fine particles from the soil into the pipe, thereby increasing the permeability of the soil in the vicinity of the well point and decreasing the time lag of the installation. An initial strong inward flow of water and "development" of the well point may in some cases be desirable in order to decrease the time lag, provided the well point and pipe thereafter are cleaned out and filled with clean water. Uncontrolled erosion or development is undesirable on account of consequent unknown changes in the time lag characteristics of the installation, and because the soil grains may cause clogging of the well point, or the soil grains may be carried into the pipe, settle at the bottom, and ultimately increase the time lag. The porosity of, or openings in, the well point should be selected in accordance with the composition and character of the soil, or the well point should be surrounded with a properly graded sand or gravel filter.

Summary comments

It should be noted that several of the above mentioned sources of error require conflicting remedial measures, and for each installation it must be determined which one of these sources of error is most serious. Those listed under Items 3, 4, 5, and 6 in Fig. 1 will affect the results of the observations, even when these are made after practical equalization of the inside and outside pressures is attained. Those described under Items 7, 8, 9, and 10 primarily influence the time lag, but they may also affect the final results when the direct field observations are corrected for influence of the time lag. It is possible that these sources of error may develop or may disappear and that their influence on the observations may vary within wide limits during the life of a particular installation. Therefore, it is desirable that facilities be provided for controlled changes of the hydrostatic pressure inside the well point, so that the time lag characteristics may be verified or determined by methods to be described in the following sections of the paper.

The time lag characteristics of a hydrostatic pressure cell may be determined by laboratory experiments, but it should be realized that these characteristics may be radically altered and the time lag greatly increased by an accumulation of gases below the diaphragm after the pressure cell has been installed. When a hydrostatic pressure cell is to be left in the ground for prolonged periods, it would be desirable but also very difficult to provide means for releasing such gas accumulations and for verifying the basic time lag of the pressure cell in place.

PART II: THEORY OF TIME LAG

The Basic Hydrostatic Time Lag

In this and the following sections concerning the hydrostatic time lag, it is assumed that this time lag is the only source of error or that the influence of the stress adjustment time lag and other sources of error, summarized in Fig. 1, is negligible. Derivation of the basic differential equation for determination of the hydrostatic time lag, Fig. 2, is similar to that of the equations for a falling-head permeameter and is based on the assumption that Darcy's Law is valid and that water and soil are incompressible. It is also assumed that artesian conditions prevail or that the flow required for pressure equalization does not cause any perceptible draw-down of the ground-water level. The active head, H, at the time t is $H = z - y$, where z may be a constant or a function of t. The corresponding flow, q, may then be expressed by the following simplified equation,

$$q = F k H = F k (z - y) \qquad (1)$$

where F is a factor which depends on the shape and dimensions of the intake or well point and k is the coefficient of permeability. This equation is valid also for conditions of anisotropic permeability provided modified or equivalent values $\overline{F}$ and $\overline{k}$ are used; see pages 32-35. It is assumed that the friction losses in the pipe are negligible for the small rates of flow occurring during pressure observations. Considering the volume of flow during the time dt, the following equation is obtained,

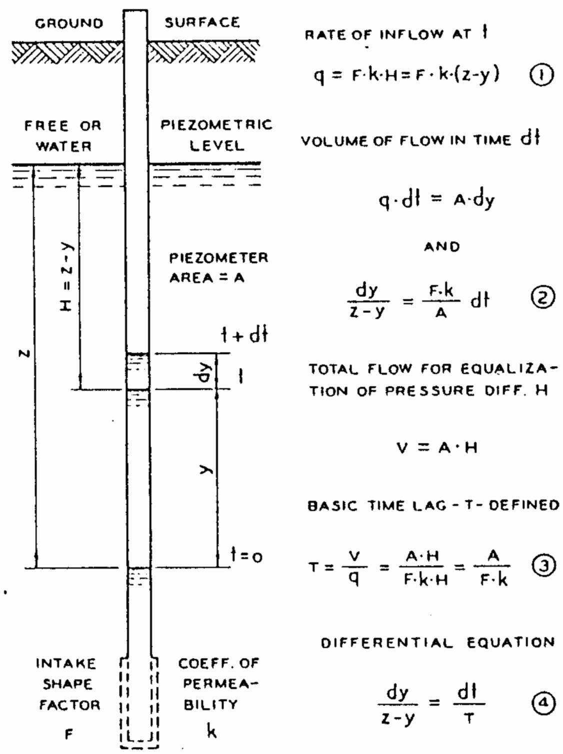

Fig. 2. Basic definitions and equations

$$q \; dt = A \; dy$$

where A is the cross-sectional area of the standpipe or an equivalent area expressing the relationship between volume and pressure changes in a pressure gage or cell. By introducing q from equation (1), the differential equation can be written as,

$$\frac{dy}{z - y} = \frac{F k}{A} dt \qquad (2)$$

The total volume of flow required for equalization of the pressure difference, H, is V = A H. The *basic time lag*, T, is now defined as the time required for equalization of this pressure difference when the original rate of flow, q = F k H, is maintained; that is,

$$T = \frac{V}{q} = \frac{A\,H}{F\,k\,H} = \frac{A}{F\,k} \tag{3}$$

and equation (2) can then be written,

$$\frac{dy}{z - y} = \frac{dt}{T} \tag{4}$$

This is the basic differential equation for determination of the hydrostatic time lag and its influence. Solutions of this equation for both constant and variable ground-water pressures are derived in the following sections, and methods for determination of the basic time lag by field observations are discussed. Examples of theoretical shape factors, F, and preliminary estimates of the basic time lag by means of equation (3) are presented in Part III, pages 30-37.

Applications for Constant Ground-Water Pressure

When the ground-water level or piezometric pressure is constant and $z = H_o$, Fig. 3, equation (4) becomes

$$\frac{dy}{H_o - y} = \frac{dt}{T}$$

and with $y = 0$ for $t = 0$, the solution is,

$$\frac{t}{T} = \ln \frac{H_o}{H_o - y} = \ln \frac{H_o}{H} \tag{5}$$

The ratio t/T may be called the time lag ratio. The head ratio, H/H_o, is determined by the equation

$$\frac{H}{H_o} = e^{-\frac{t}{T}} \tag{6}$$

and the equalization ratio, E, by

$$E = \frac{y}{H_o} = 1 - \frac{H}{H_o} = 1 - e^{-\frac{t}{T}} \tag{7}$$

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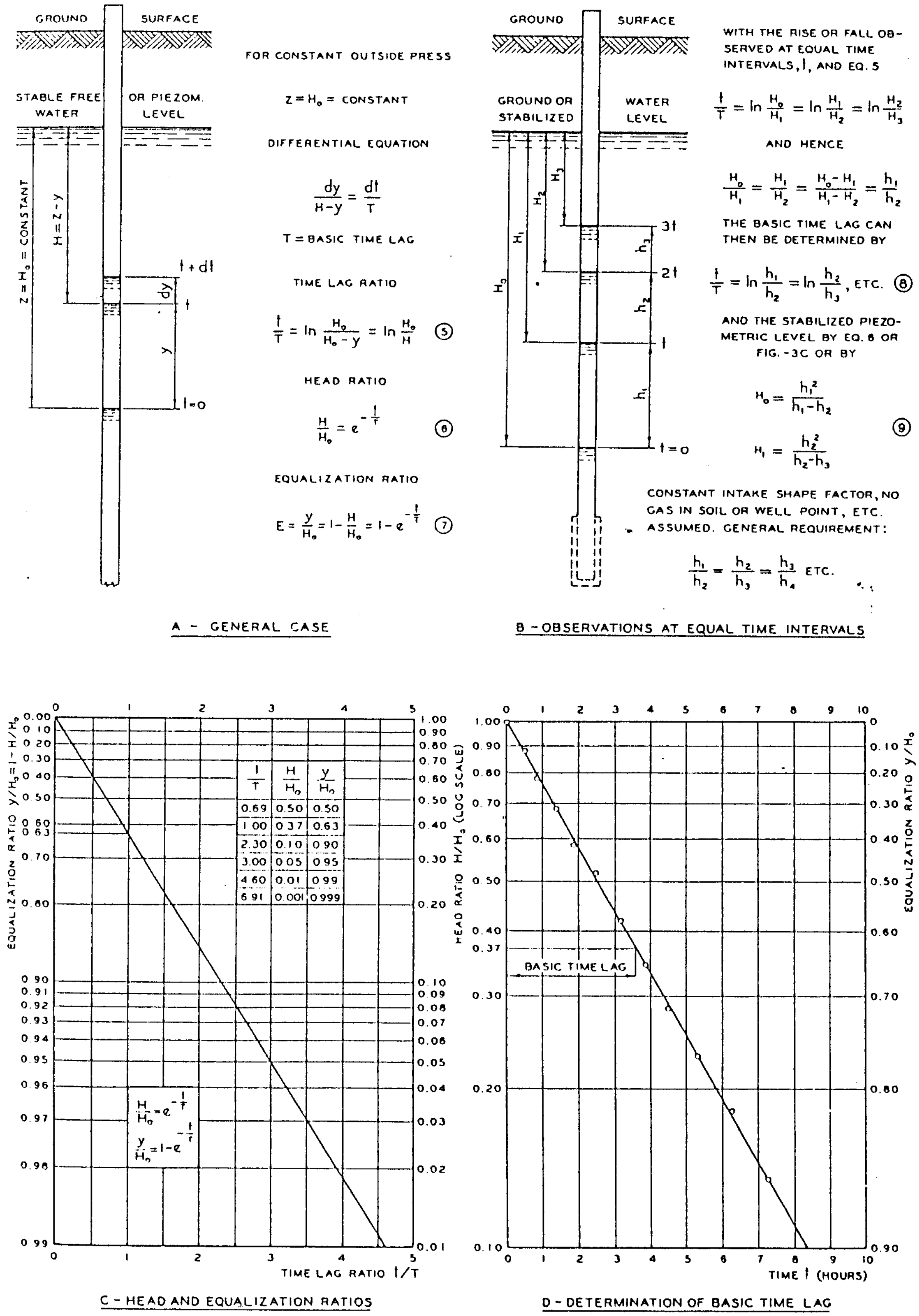

Fig. 3. Constant ground-water levels and pressures

A diagram representing equations (6) and (7) is shown in Fig. 3-C. It should be noted that the basic time lag corresponds to an equalization ratio of 0.63 and a head ratio of 0.37. An equalization ratio of 0.90 may be considered adequate for many practical purposes and corresponds to a time lag equal to 2.3 times the basic time lag. An equalization ratio of 0.99 requires twice as long time as 90 per cent equalization.

When the stabilized pressure level, or initial pressure difference, is not known, it may be determined in advance of full stabilization by observing successive changes in piezometer level, h_1, h_2, h_3, etc., for equal time intervals; see Fig. 3-B. The time lag ratio is then equal for all intervals, or according to equation (5),

$$\frac{t}{T} = \ln\frac{H_o}{H_1} = \ln\frac{H_1}{H_2} = \ln\frac{H_2}{H_3}, \text{ etc.}$$

and hence,

$$\frac{H_o}{H_1} = \frac{H_1}{H_2} = \frac{H_o - H_1}{H_1 - H_2} = \frac{h_1}{h_2}$$

or,

$$\frac{t}{T} = \ln\frac{h_1}{h_2} = \ln\frac{h_2}{h_3}, \text{ etc.} \qquad (8)$$

and since $H_1 = H_o - h_1$, $H_2 = H_1 - h_2$, etc.,

$$H_o = \frac{h_1^2}{h_1 - h_2} \quad \text{or} \quad H_1 = \frac{h_2^2}{h_2 - h_3}, \text{ etc.} \qquad (9)$$

It is emphasized that these equations can be used only when the influence of the stress adjustment time lag, air or gas in soil or piezometer system, clogging of the intake, etc., is negligible, or when

$$\frac{h_1}{h_2} = \frac{h_2}{h_3} = \frac{h_3}{h_4}, \text{ etc.}$$

Equations (9) form a convenient means of estimating the stabilized pressure level. In actual practice it is advisable to fill or empty the piezometer to the computed level and to continue the observations for a period sufficient to verify or determine the actual stabilized level.

When the head or equalization ratios, or the ratios between successive pressure changes for equal time intervals, have been determined, the basic time lag may be found by means of equations (5), (7), or (8). However, due to observational errors, there may be considerable scattering in results, especially when the pressure changes are small. In general, it is advisable to prepare an equalization diagram or a semi-logarithmic plot of head ratios and time, as shown in Fig. 3-D. When the assumptions on which the theory is based are fulfilled, the plotted points should lie on a straight line through the origin of the diagram. The basic time lag is then determined as the time corresponding to a head ratio of 0.37. Examples of both straight and curved diagrams of the above mentioned type are discussed in Part III, pages 38-43.

Applications for Linearly Changing Pressures

When the ground-water or piezometric pressure level, as shown in Fig. 4, is rising at a uniform rate, $+\alpha$, or falling at the rate $-\alpha$, then

$$z = H_o + \alpha t \tag{10}$$

and equation (4) may be written,

$$\frac{dy}{H_o + \alpha t - y} = \frac{dt}{T} \tag{11}$$

With $y = 0$ for $t = 0$, the solution of equation (11) is,

$$\frac{y - \alpha t}{H_o - \alpha T} = 1 - e^{-\frac{t}{T}} \tag{12}$$

which corresponds to equation (7) for constant ground-water pressure. Theoretically α, T, and H_o may be determined, as shown in Fig. 4-B, by observing three successive changes in piezometer level at equal time intervals, t, and expressing the results by three equations similar to equation (12). By successively eliminating $(H_o - \alpha T)$ and $e^{-\frac{t}{T}}$ from these equations, the following solutions are obtained,

$$\alpha t = \frac{h_1 h_3 - h_2^2}{h_1 + h_3 - 2h_2} \tag{13}$$

$$\frac{t}{T} = \ln \frac{h_1 - \alpha t}{h_2 - \alpha t} \tag{14}$$

$$H_o = \alpha T + \frac{(h_1 - \alpha t)^2}{h_1 - h_2} \tag{15}$$

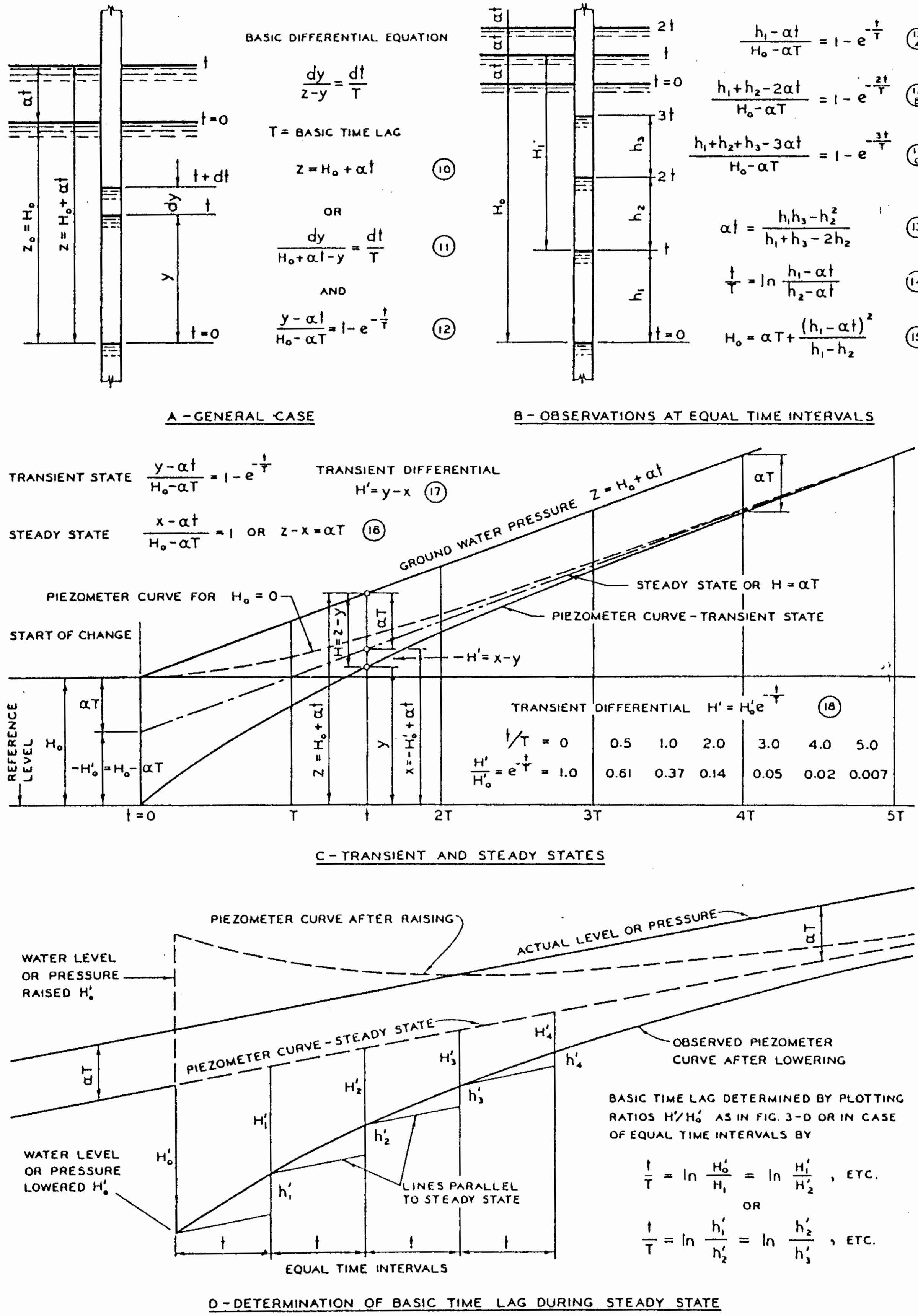

Fig. 4. Linearly changing ground-water pressures

These equations correspond to equations (8) and (9) for constant ground-water pressure. However, the form of equation (13) is such that a small error in determination of the increment pressure changes may cause a very large error in the computed value of αt. In general, it is better to determine the basic time lag and the actual ground-water pressures after the steady state, discussed below, is attained.

Referring to Fig. 4-C, equation (12) represents the transient state of the piezometer curve. With increasing values of t, the right side of this equation approaches unity and the curve the steady state. Designating the ordinates of the steady state of the piezometer curve by x, this curve is represented by,

$$\frac{x - \alpha t}{H_o - \alpha T} = 1$$

or by means of equation (10),

$$z - x = \alpha T = \text{constant} \tag{16}$$

That is, the difference between the actual ground-water pressure and that indicated by the piezometer is constant and equal to αT during the steady state. The difference between the pressures corresponding to the transient and steady states of the piezometer curve

$$H' = y - x \tag{17}$$

may be called the transient pressure differential. For the conditions shown in Fig. 4-C, this differential is negative. With

$$x = H_o - \alpha T + \alpha t \quad \text{and} \quad H'_o = \alpha T - H_o$$

equation (17) can be written,

$$H' = (y - \alpha t) + H'_o$$

and by means of equation (12)

$$H' = H'_o \, e^{-\frac{t}{T}} \tag{18}$$

This equation is identical with equation (6) for constant ground-water pressure; that is, the transient pressure differential can be determined as if the line representing the steady state were a constant piezometric pressure level. As will be seen in Fig. 4-C and also the diagram in Fig. 3-C, the steady state may for practical purposes be considered attained at a time after a change in piezometer level, or start of a change in the rate α, equal to three to four times the basic time lag.

When the piezometer level increases or decreases linearly with time, it may be concluded that the steady state is attained and that the rate of change, α, is equal to that for the ground-water pressure. If the piezometer level now is raised or lowered by the amount H'_o, and the transient pressure differentials are observed, then the basic time lag may be determined by means of a semi-logarithmic plot of the ratios H'/H'_o and the time, t, as in Fig. 3-D; that is, the basic time lag is the time corresponding to $H'/H'_o = 0.37$. To complete the analogy with constant ground-water pressures, the transient pressure differential may be observed at equal time intervals, t, and the basic time lag determined by,

$$\frac{t}{T} = \ln \frac{H'_o}{H'_1} = \ln \frac{H'_1}{H'_2}, \text{ etc.}$$

or by

$$\frac{t}{T} = \ln \frac{h'_1}{h'_2} = \ln \frac{h'_2}{h'_3}, \text{ etc.}$$

where h'_1, h'_2, h'_3 are the increment pressure differentials. However, it is generally advisable to use the ratios H'/H'_o and a diagram of the type shown in Fig. 3-D. Having thus determined the basic time lag, the difference between the piezometer and ground-water levels, αT, can be computed.

Applications for Sinusoidal Fluctuating Pressures

Periodic fluctuations of the ground-water pressure, in form approaching a sinusoidal wave, may be produced by tidal variations of the water level of nearby open waters, Fig. 5-A. Such fluctuations of the ground-water pressure may be represented by the equation

$$z = z_a \sin \frac{2\pi t}{T_w} \tag{19}$$

where z_a is the amplitude and T_w the period of the wave. By means of the basic differential equation (4) the following equation for the fluctuations of the piezometer level is obtained,

$$\frac{dy}{dt} = \frac{1}{T} \left(z_a \sin \frac{2\pi t}{T_w} - y\right) \tag{20}$$

Through the temporary substitution of a new variable v and $y = v e^{-\frac{t}{T}}$, setting $\frac{2\pi T}{T_w} = \tan \frac{2\pi t_s}{T_w}$, and with $y = y_o$ for $t = 0$, the following solution of the equation is obtained,

$$y = z_a \cos \frac{2\pi t_s}{T_w} \sin \frac{2\pi}{T_w} (t - t_s) + \left[y_o + z_a \cos \frac{2\pi t_s}{T_w} \sin \frac{2\pi t_s}{T_w}\right] e^{-\frac{t}{T}}$$

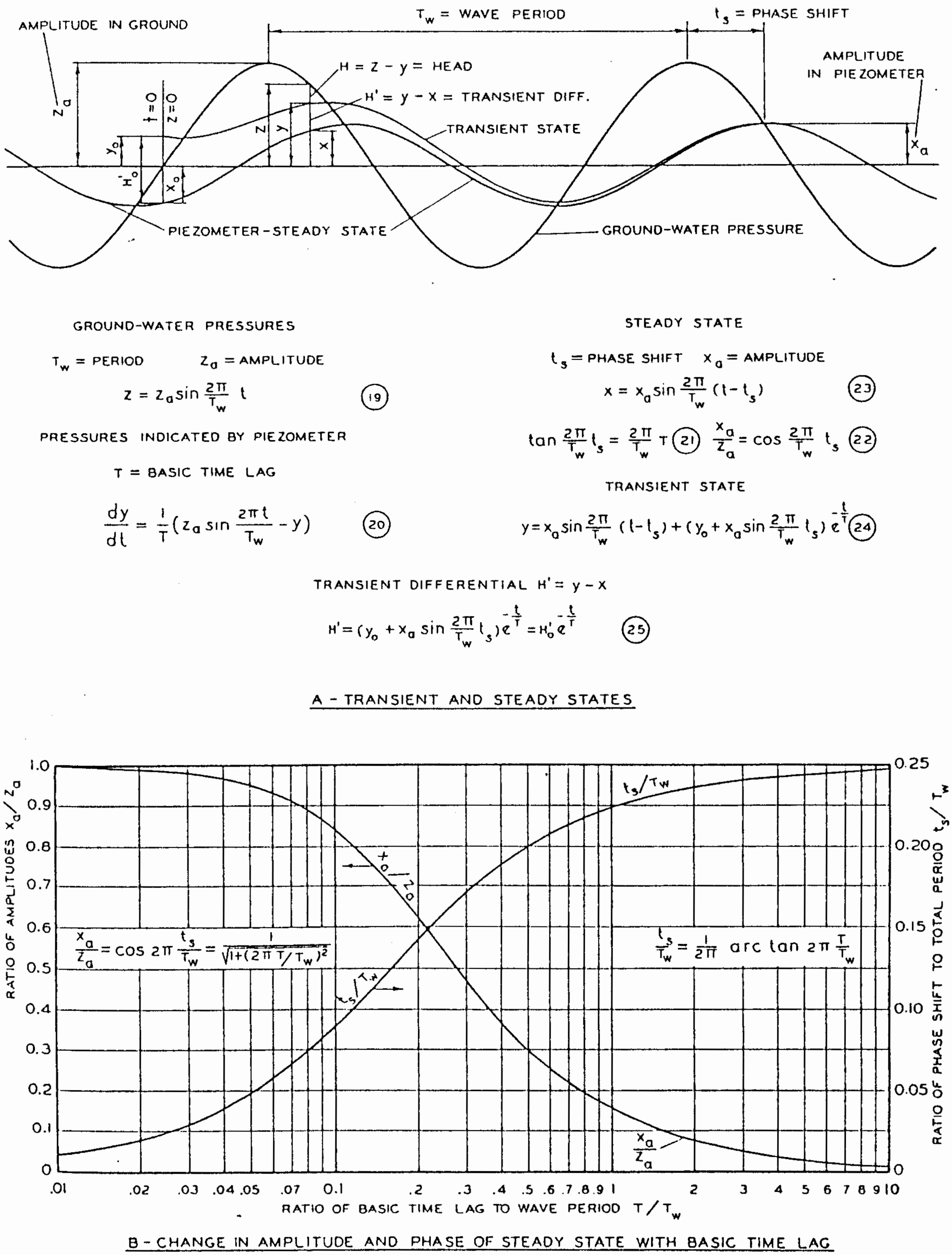

Fig. 5. Sinusoidal fluctuating ground-water pressure

For large values of t, $e^{-\frac{t}{T}}$ becomes very small and is zero for the steady state, for which the following equation applies, substituting x for y,

$$x = z_a \cos \frac{2\pi t_s}{T_w} \sin \frac{2\pi}{T_w} (t - t_s)$$

This equation represents a sinusoidal wave with the phase shift t_s, determined by,

$$\tan \frac{2\pi t_s}{T_w} = \frac{2\pi T}{T_w} \tag{21}$$

and the amplitude

$$x_a = z_a \cos \frac{2\pi t_s}{T_w} = \frac{z_a}{\sqrt{1 + (2\pi T/T_w)^2}} \tag{22}$$

The equation for the steady state can then be written,

$$x = x_a \sin \frac{2\pi}{T_w} (t - t_s) \tag{23}$$

and the equation for the transient state,

$$y = x_a \sin \frac{2\pi}{T_w} (t - t_s) + (y_o + x_a \sin \frac{2\pi t_s}{T_w}) e^{-\frac{t}{T}} \tag{24}$$

The transient pressure differential, $H' = y - x$, is determined by

$$H' = (y_o + x_a \sin \frac{2\pi t_s}{T_w}) e^{-\frac{t}{T}} = H_o' e^{-\frac{t}{T}} \tag{25}$$

where H_o' is the transient differential for $t = 0$. Equation (25) is identical with equations (6) and (18), and the transient pressure differential can also in this case be computed as if the steady state were a constant pressure level. H' may be determined as a function of H_o' by means of the diagram shown in Fig. 3-C, and it will be seen that for practical purposes the steady state is reached after elapse of a time equal to three to four times the basic time lag.

Equations (22) and (23) are represented by the diagram in Fig. 5-B, by means of which the phase shift and the decrease of amplitude in the piezometer can easily be determined. If the fluctuations of the piezometer level have reached the steady state and the wave period, T_w, and the phase shift, t_s, can be observed in the field, it is theoretically possible to determine the basic time lag by means of the diagram in Fig. 5-B. However, it is difficult to determine the phase shift by direct observation, since it cannot be assumed that the pressure fluctuations in the ground water are in phase with those of the surface waters. When the fluctuations in the ground-water pressure are caused by load and stress changes without material seepage and volume changes of the soil, it is possible that the phase shift in pore-water pressures, with respect to the surface water, may be insignificant even though a material decrease in amplitude occurs. On the other hand, when pressure changes in the pore water in part are caused by infiltration or are accompanied by changes in water content of the soil, then it is possible that there also will be a material shift in phase of the pressure fluctuations. The basic time lag may be determined during the steady state by raising or lowering the piezometer pressure, observing the transient pressure differentials, and plotting the ratios H'/H'_o and the elapsed time in a diagram similar to that shown in Fig. 3-D.

Corrections for Influence of the Hydrostatic Time Lag

The characteristics of an installation for determination of ground-water levels and pressures may change with time because of sedimentation, clogging, and accumulation of gases in the system or in the soil near the intake. When observations of such levels and pressures are to be corrected for influence of the hydrostatic time lag, the first task is to determine the basic time lag and verify that the assumptions, on which the general theory is based, are satisfied. This is best accomplished during periods when the ground-water pressure is constant, but as shown in the foregoing sections, the verification may also be performed during the steady state of linear and sinusoidal variations in the ground-water and piezometer levels.

Verification by means of transient pressure differentials can be used irrespective of the form of the curve representing the steady state of pressure variations. The pressure variations may be represented by the following general equations, $z = F(t)$ for the ground-water pressure; $x = f(t)$ for the steady state of the piezometer pressure; and $y = g(t)$ for the transient state or after the piezometer pressure has been raised or lowered by an arbitrary amount H'_o. The transient pressure differential is the $H' = y - x$, and according to equation (4), which applies to all conditions,

$$\frac{dy}{z - y} = \frac{dt}{T} = \frac{dx}{z - x} = \frac{dy - dx}{x - y} = \frac{dH'}{H'}$$

or

$$\ln H' = -\frac{t}{T} + C$$

and with $H' = H'_o$ for $t = 0$

$$\frac{t}{T} = \ln \frac{H'_o}{H'}$$

which is identical with equation (5). Therefore, when the piezometer pressure varies in such a manner that the pressures can be predicted with sufficient accuracy for a future period of reasonable length, the basic time lag may be determined by raising or lowering the piezometer pressure by an arbitrary amount, H'_o, observing the transient pressure differentials, H', and plotting the ratios H'/H'_o as a function of time as shown in Fig. 3-D. Application of the basic equation (4) requires that the points in the semi-logarithmic plot fall on a straight line through the origin of the diagram.

Having determined the basic time lag and verified that the assumptions are satisfied, corrections for influence of the time lag in case of linear or sinusoidal variations may be determined as shown in Figs. 4 and 5. In case of irregular fluctuations, it should first be noted that when the piezometer curve passes through a maximum or minimum, the pressure indicated by the piezometer must be equal to that of the ground water. In this connection it is again emphasized that the fluctuations of the ground-water pressure are not necessarily in phase with those of the water level of nearby surface waters. The maxima or minima of the piezometer variations may be used as starting points for the corrections, which may be determined by assuming either an equivalent constant value or, alternatively, an equivalent constant rate of change of the ground-water pressure during each time interval.

The first of these methods is shown in Fig. 6-A. The difference, H_c, between the equivalent constant ground-water pressure and the piezometer pressure at the start of the time interval may be determined by equation (7) and substituting H_c for H_o and h for y; that is,

$$H_c = \frac{h}{E} \tag{26}$$

where h is the change in piezometer pressure and E is the equalization ratio for the time interval, t, or time lag ratio t/T; see Fig. 3-C. It is now assumed that the actual ground-water pressure in the middle of the time interval is equal to the equivalent constant pressure during the interval.

In applying the second method of correction, Fig. 6-B, it is assumed that the pressure difference at the beginning of the time interval, H_o, has been determined, for example by starting the operations at a maximum or minimum of the piezometer curve. Designating the equivalent uniform rate of change in ground-water pressure by α, the total change during the time interval, $H_t = \alpha t$, can be computed by means of equation (12), or when solving for αt and introducing the equalization ratio E,

$$H_t = \frac{h - E H_o}{1 - E \frac{T}{t}} \tag{27}$$

This method will usually give more accurate results than the method of equivalent constant pressure, but the latter method is easier to apply. The results obtained by the two methods are compared in Fig. 6-C, and it will be seen from the equations and the diagram that the difference in results is only a few per cent when the initial pressure difference is large and the time interval is small, in which case the easier method of equivalent constant pressures may be used. On the other hand, there is considerable difference in results and the method of equivalent constant rate of change should be used when the initial pressure difference is small and the time lag ratio is large.

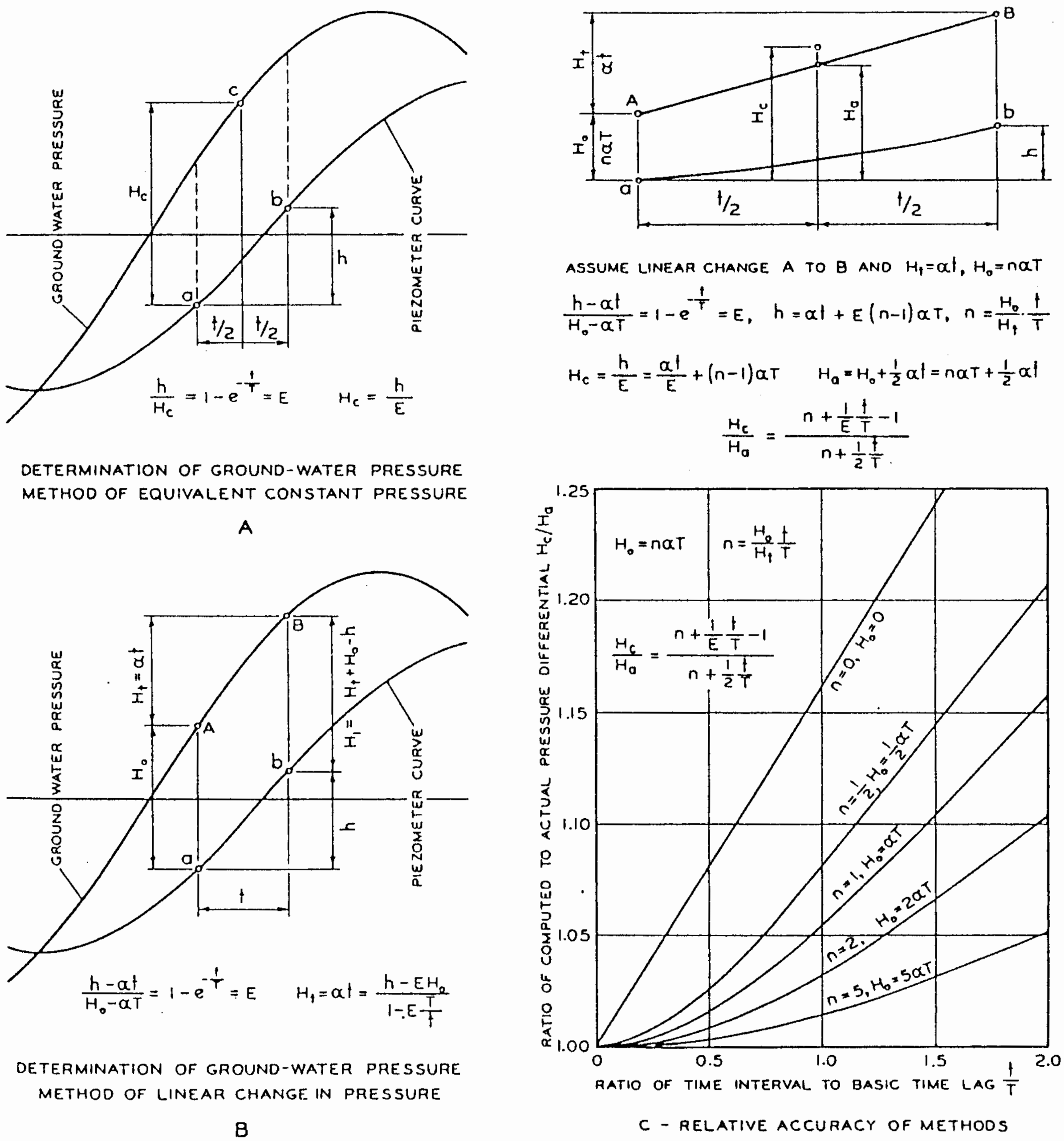

Fig. 6. Corrections for influence of hydrostatic time lag

Influence of the Stress Adjustment Time Lag

In absence of detailed theoretical and experimental investigations of the stress adjustment time lag and its influence on pressure observations, the following discussion is tentative in character, and its principal object is to call attention to the problems encountered.

As mentioned in discussing Fig. 1, the stress adjustment time lag is the time required for changes in water content of the soil in the vicinity of the intake or well point as a result of changes in the stress conditions. A distinction must be made between the initial stress changes and adjustments, which occur only during and immediately after installation of a pressure measuring device, and the transient but repetitive changes which occur each time water flows to or from the intake or well point during subsequent pressure observations.

Initial disturbance and stress changes

When a boring is advanced by removal of soil, the stresses in the vicinity of its bottom or section below the casing will be decreased with a consequent initial decrease in pore-water pressure and tendency to swelling of the soil. A flow of water from the boring to the soil will increase the rate of swelling, and the combined initial hydrostatic and stress adjustment time lags will probably be decreased when the initial hydrostatic pressure inside the boring or well point is slightly above the normal ground-water pressure, Fig. 7-A.

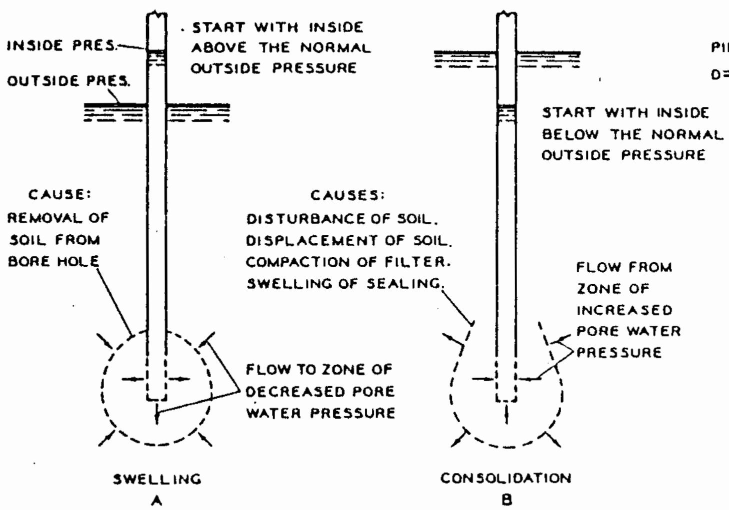

Fig. 7. Initial disturbance and stress changes

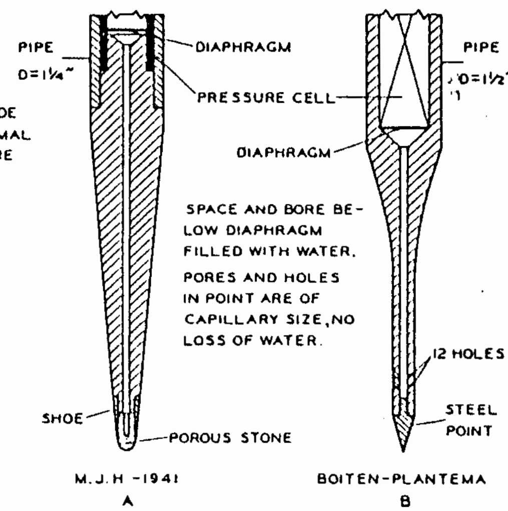

Fig. 8. Points for pressure sounding rod

A zone with increased pore-water pressures and a tendency to consolidation of the soil may be caused by disturbance and displacement of soil during the driving of a well point and by compaction of a sand filter or a seal above a well point or pressure cell installed in an oversize bore hole, Fig. 7-B. Subsequent swelling of the sealing material may also cause consolidation of the surrounding soil, but its effect on the pore-water pressures in the vicinity of the well point is uncertain. A flow of water from the soil to the well point will increase the rate of consolidation, and when the basic time lag of the installation is large, the combined initial hydrostatic and stress adjustment time lags will probably be decreased when the initial hydrostatic pressure inside the well point is below the normal ground-water pressure.

The initial stress adjustment time lag depends on the dimensions of the zone of stress changes and on the permeability, sensitivity to disturbance, and consolidation characteristics of the soil. The initial stress adjustment time lag will be small compared to the hydrostatic time lag when the total volume change of the soil is small compared to the required increase or decrease of the volume of water in the pressure measuring device, as in case of a boring or observation well in coarse-grained soils. On the other hand, the stress adjustment time lag may be very large compared to the hydrostatic time lag for a pressure cell installed in fine-grained and highly compressible soils.

The initial stress adjustment time lag can be reduced by decreasing the dimensions of the well point and/or filter, but this will increase the hydrostatic time lag. When the ground-water observations are to be extended over a considerable period of time, the hydrostatic time lag is usually governing and the well point should be large. On the other hand, when it is desired to make only a single or a few measurements at each location and depth, and when a sensitive pressure measuring device is used, then the well point should be small in order to reduce the zone of disturbance and the initial stress adjustment time lag. Even then there is an optimum size, and when the dimensions of the well point are made smaller than that size, the consequent decrease in the initial stress adjustment time lag may be more than offset by an increase in the hydrostatic time lag.

Examples of points for pressure measuring devices, similar to sounding rods and intended for reconnaissance exploration of ground-water conditions in soft or loose soils, are shown in Fig. 8. The one to the left, designed by the writer (14, 15), has a larger intake area than the one shown to the right and designed by BOITEN and PLANTEMA (1), but the latter is sturdier and will probably cause less disturbance of the soil in the immediate vicinity of the point.

Transient consolidation or swelling of soil

When water is flowing to or from a pressure measuring device, the pore-water pressures, the effective stresses in, and the void ratio of the soil in the vicinity of the well point or intake will be subject to changes. As a consequence, the rate of flow of water to or from the intake will be increased or decreased, and this will influence the shape of the equalization diagrams. The above mentioned changes are more or less transient, and with decreasing difference between the piezometer and ground-water pressures, the stress conditions and void ratios will approach those corresponding to the pore-water pressures in the soil mass as a whole. The probable sequence of consolidation and swelling of the soil around a rigid well point when the piezometer level is lowered or raised is shown in Fig. 9.

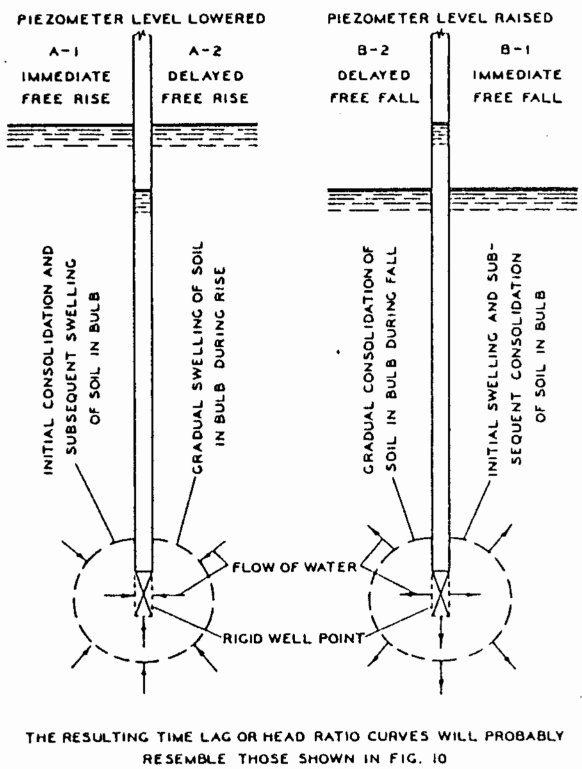

Fig. 9. Transient changes in void ratio

It is difficult by theory or experiment to determine the changes in void ratio and water content around a well point, but similar changes occur during soil permeability tests with a rising or falling head permeameter, and observations made immediately after the head is applied in such a permeameter usually furnish too high values for the coefficient of permeability and are discarded as unreliable. Although the stress conditions around a rigid well point are more complicated than in a soil test specimen in a permeameter, the results of permeability tests, which are extended until practical equalization of the water levels is attained, will furnish an indication of the magnitude of the transient consolidation and swelling and on the resulting shape of equalization diagrams for a rigid well point*. A series of such tests were performed with Atlantic muck, a soft organic clay, and the testing arrangement and some test results are shown in Fig. 10. The volume changes during these permeability tests were very small since the test specimens were overconsolidated in order to obtain nearly equal consolidation and swelling characteristics.

When the water level in the standpipe, Fig. 10, is raised and immediately thereafter allowed to fall -- corresponding to Case B-1 in Fig. 9 -- an initial swelling of the soil takes place, since the total vertical stresses remain constant whereas the pore-water pressure has been increased and the effective stresses tend to decrease. As a consequence, the rate of flow from the standpipe to the soil sample is increased and the initial slope of the equalization diagram becomes steeper. As the swelling progresses and the water level in the standpipe falls, the rate of excess flow decreases; the equalization diagram acquires a concave curvature, and a condition will be reached where the void ratio of the soil corresponds to the pore-water

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* The relatively simple conditions shown in Fig. 9, and a comparison with the conditions in a permeameter, may not apply in case of an open bore hole, when the well point or intake is not rigid, and when the pressure in Case B is so great that the soil is deflected and a clearance is created between the well point and the soil.

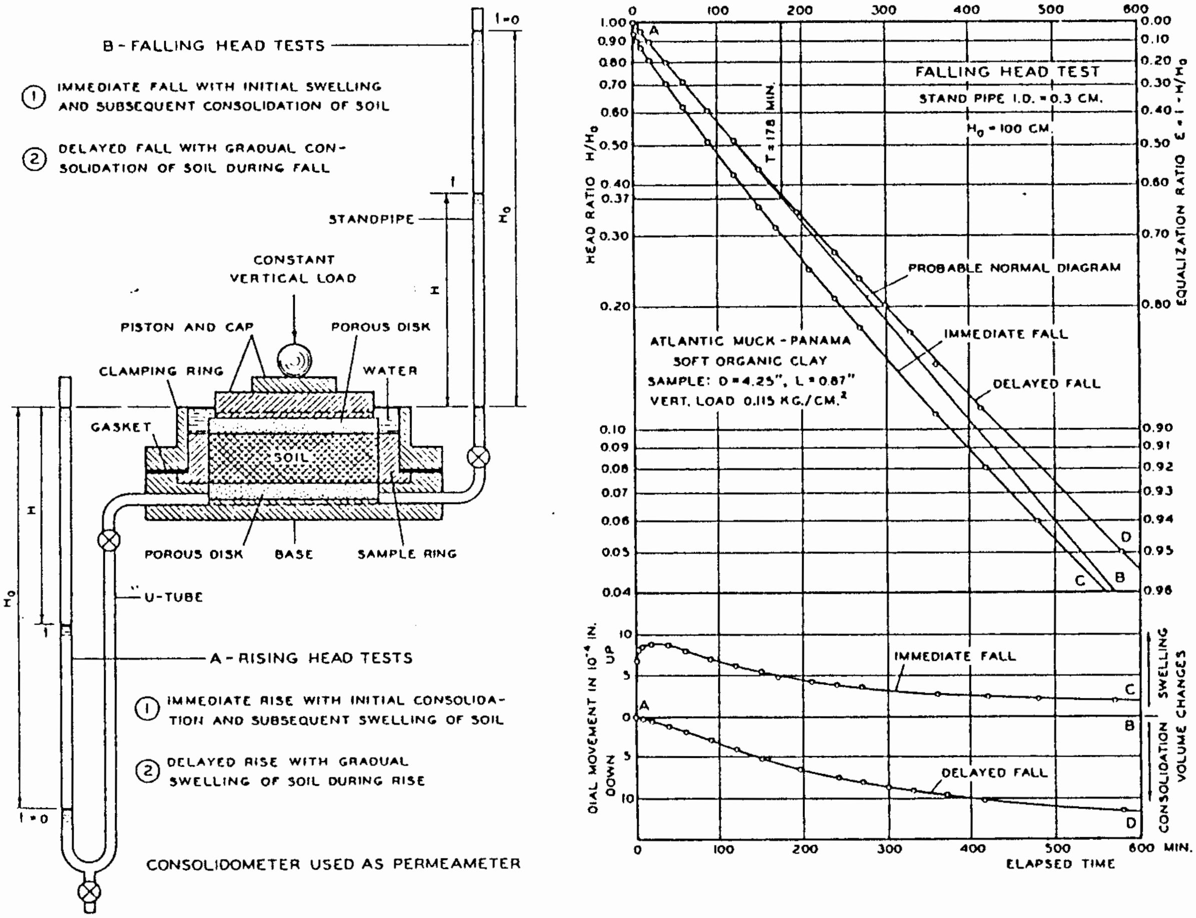


Fig. 10. Volume changes during laboratory permeability tests

pressure indicated by the standpipe level. With further fall in this level and decrease in pore-water pressures, a reconsolidation of the soil takes place with a consequent deficiency in rate of flow from the standpipe. The curvature of the equalization diagram decreases; the diagram becomes fairly straight and may even acquire a slight convex curvature as it approaches the normal diagram, obtained when there is no change in void ratio of the soil. However, the ultimate shape and slope of the diagram could not be determined from the results of tests so far performed, since these results were influenced by very small temperature changes in the laboratory.

When the water level in the standpipe is raised and maintained in its upper position until the initial swelling of the soil sample is completed and then allowed to fall -- Cases B-2 in Figs. 9 and 10 -- a gradual re-consolidation of the soil takes place during the actual test, and an equalization diagram which lies above the normal diagram is obtained, but its lower part is more or less parallel to the lower part of the diagram for immediate fall.

Similar diagrams were obtained by rising head tests. When the water level in the U-tube is lowered and immediately thereafter allowed to rise, Case A-1 in Figs. 9 and 10, the soil will be subjected to an initial consolidation with a consequent increase in rate of flow to the U-tube, but this volume decrease of the soil will later be eliminated by a swelling and a corresponding deficiency in rate of flow to the U-tube. The resulting equalization diagram has a concave curvature and lies below the normal diagram. When the water level in the U-tube is maintained in its lower position until the initial consolidation is completed and then allowed to rise, a gradual swelling of the soil takes place; the rate of flow to the U-tube is decreased, and the equalization diagram lies above the normal diagram.

All the above mentioned tests were repeated several times with both undisturbed and remolded soil, and the results obtained were all similar to those shown in Fig. 10. A slight sudden drop in head ratio in case of immediate fall -- or rise -- is probably due to a small amount of air in the system. As already indicated, the shape of the lower part of the diagrams was influenced by small amounts of leakage and evaporation and by temperature changes. The temperature in the laboratory did not vary more than 1.5° F from the mean temperature, but even such small variations are sufficient to cause conspicuous irregularities in the test results when the active head is small. However, it is believed that the results are adequate for demonstration of the consolidation and swelling of the soil during permeability tests and of the resulting general shape of the equalization diagrams.

Volume changes of gas in soil

The influence of gas bubbles in an open or closed pressure measuring system is summarized in Fig. 1 and discussed briefly on pages 6 and 7. Whereas such gas bubbles may cause a change in both the ultimate indicated pressure and the time lag or slope of the equalization diagram, they will not materially influence the shape of the latter, since changes in pressure and volume of the gas bubbles occur nearly simultaneously with the changes in hydrostatic pressure within the system. On the other hand, when the gas bubbles are in the soil surrounding the well point and their volume and the water content of the soil are changed, there will be a time lag between changes in hydrostatic pressure in the system and corresponding changes in pressure and volume of the gas bubbles, and this time lag will cause a change in both slope and shape of the equalization diagrams. The general effect of the gas bubbles is an increase in the apparent compressibility of the soil, and the equalization diagrams should be similar to those shown in Fig. 10.

The change in volume of the gas bubbles, when the piezometer level is lowered or raised, and probable resulting equalization diagrams are shown in Fig. 11. This figure and the following discussion are essentially a tentative interpretation of the results of the laboratory permeability tests and the field observations shown in Figs. 10 and 17.

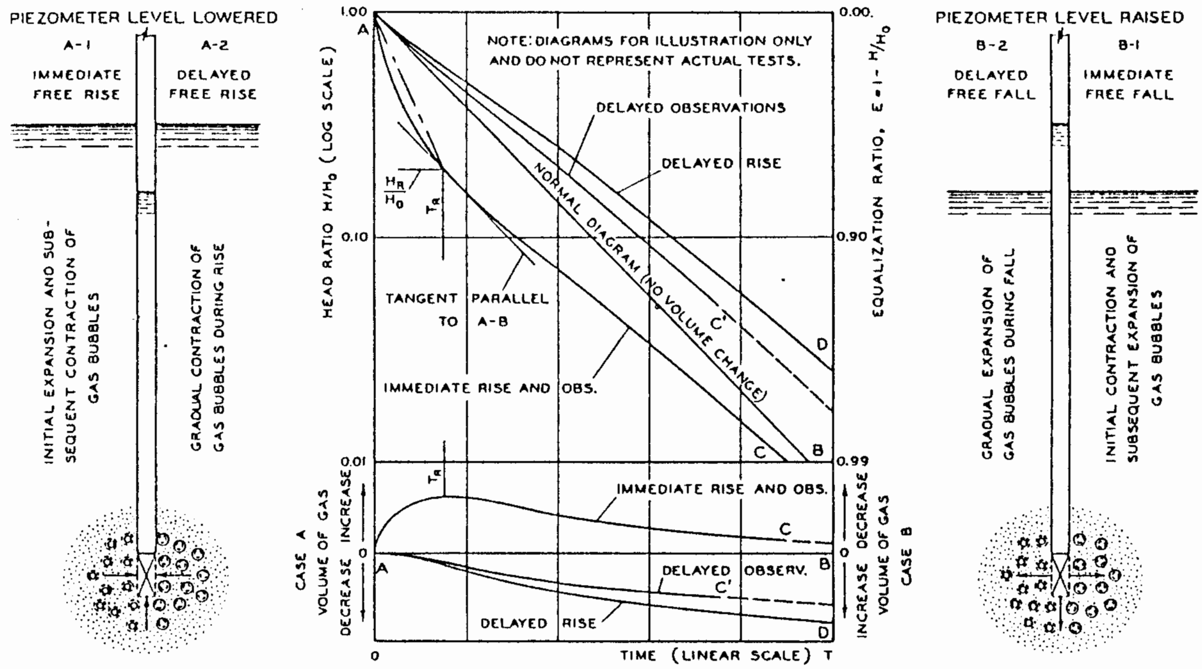


Fig. 11. Influence of volume changes of gas in soil

When the piezometer level suddenly is lowered and immediately thereafter allowed to rise, Case A-1, the pressure in the pore water is decreased, and the gas bubbles tend to expand and force an excess amount of water into the well point; that is, the initial rate of rise of the piezometer level will be increased and the equalization diagram, A-C, will have a steeper slope than the normal diagram, A-B, and a concave curvature. It is emphasized that the normal diagram, A-B, corresponds to the condition of no volume change of the gas bubbles and not to complete absence of gas bubbles in the soil. Even when the volume of the gas bubbles does not change, the presence of these bubbles will decrease the effective permeability of the soil and increase the time lag of the piezometer. As the piezometer level rises, the difference between the pressures in the gas bubbles and the surrounding pore water decreases. At the time T_r these pressures are equalized, and the rate of excess inflow ceases; that is, the tangent to the equalization diagram, A-C, at the time T_r should be parallel to the normal diagram, A-B. With a further rise in piezometer level, the pore-water pressure around the well point increases; the volume of the gas bubbles decreases, and there will be a deficiency in inflow of water. The curvature of the equalization diagram decreases and may eventually become zero or, perhaps, even change to a slight convex curvature as the volume of the gas bubbles approaches its original value.

If the observations were started at the time of reversal of the volume changes, T_r, the volume of the gas bubbles would decrease throughout the observations; there would be a deficiency in the rate of inflow, and the equalization diagram, A-C', would be above the normal diagram. A similar but higher-lying diagram, A-D, would be obtained if the piezometer level is not allowed to rise immediately after lowering but is maintained in its lower position until the initial swelling of the gas bubbles is completed, Case A-2. The two diagrams A-C and A-D should ultimately become parallel, and the normal diagram is a straight line between these limiting diagrams and is tangent at "A" to diagrams A-C' and A-D.

When the piezometer level suddenly is raised and immediately thereafter is allowed to fall, Case B-1, the volume of the gas bubbles at first decreases with a consequent excess outflow of water from the piezometer. Later on the gas bubbles expand until their original volume is attained, and during this period there will be a corresponding deficiency in rate of outflow. The resulting equalization diagram is similar in form to A-C for Case A-1. When the piezometer level is maintained in its upper position until the initial contraction of the gas bubbles is completed and then is allowed to fall, an equalization diagram similar to A-D is obtained.

Normal operating conditions

The discussions in the foregoing sections concern mainly time lag tests during which the piezometer level suddenly is changed whereas the general ground-water level or pore-water pressure remains constant. In normal operation the ground-water pressure changes first, and the piezometer level follows these changes with a certain pressure difference or time lag. When the ground-water level or pore-water pressure changes, the void ratio of the soil and the volume of gas bubbles below the ground-water level also tend to change, but the rate of such changes generally decreases in the immediate vicinity of a well point or intake for a pressure measuring installation on account of the pressure difference and time lag. However, all changes progress in the same direction and there is no initial increase in void ratio and water content followed by a decrease -- or vice versa -- as in the case of time lag tests.

In general, normal operating conditions resemble in most cases those of delayed fall or rise, or rather delayed observations, shown in Figs. 10 and 11. *It is probable that the time lag during normal operating conditions corresponds to an equalization diagram which, for practical purposes, may be represented by a straight line through the origin of the diagram and parallel to the lower portions of the diagrams obtained in time lag tests.* However, sufficient experimental data for verification of the suggested approximation -- especially comparative tests during rapidly changing ground-water pressures and with several pressure measuring installations having widely different basic time lags -- are not yet available.

As indicated by permeability tests of the type shown in Fig. 10, it is probable that the influence of swelling or consolidation of the soil is very small or negligible when observation wells or open piezometers are used in ground-water observations, but it is also possible that such changes in void ratio may cause appreciable distortion of the equalization diagrams and increase in actual time lag when pressure gages or cells with a small basic time lag are used and the soil is relatively compressible. On the other hand, gas bubbles in the soil around a well point may cause considerable distortion of the equalization diagrams and increase in actual time lag even for open piezometers; see Fig. 17. Accumulation of gas in the pressure measuring system causes no curvature of the equalization diagram but materially decreases its slope and increases the effective time lag under normal operating conditions.

PART III -- DATA FOR PRACTICAL DETERMINATION AND USE OF TIME LAG

Flow through Intakes and Well Points

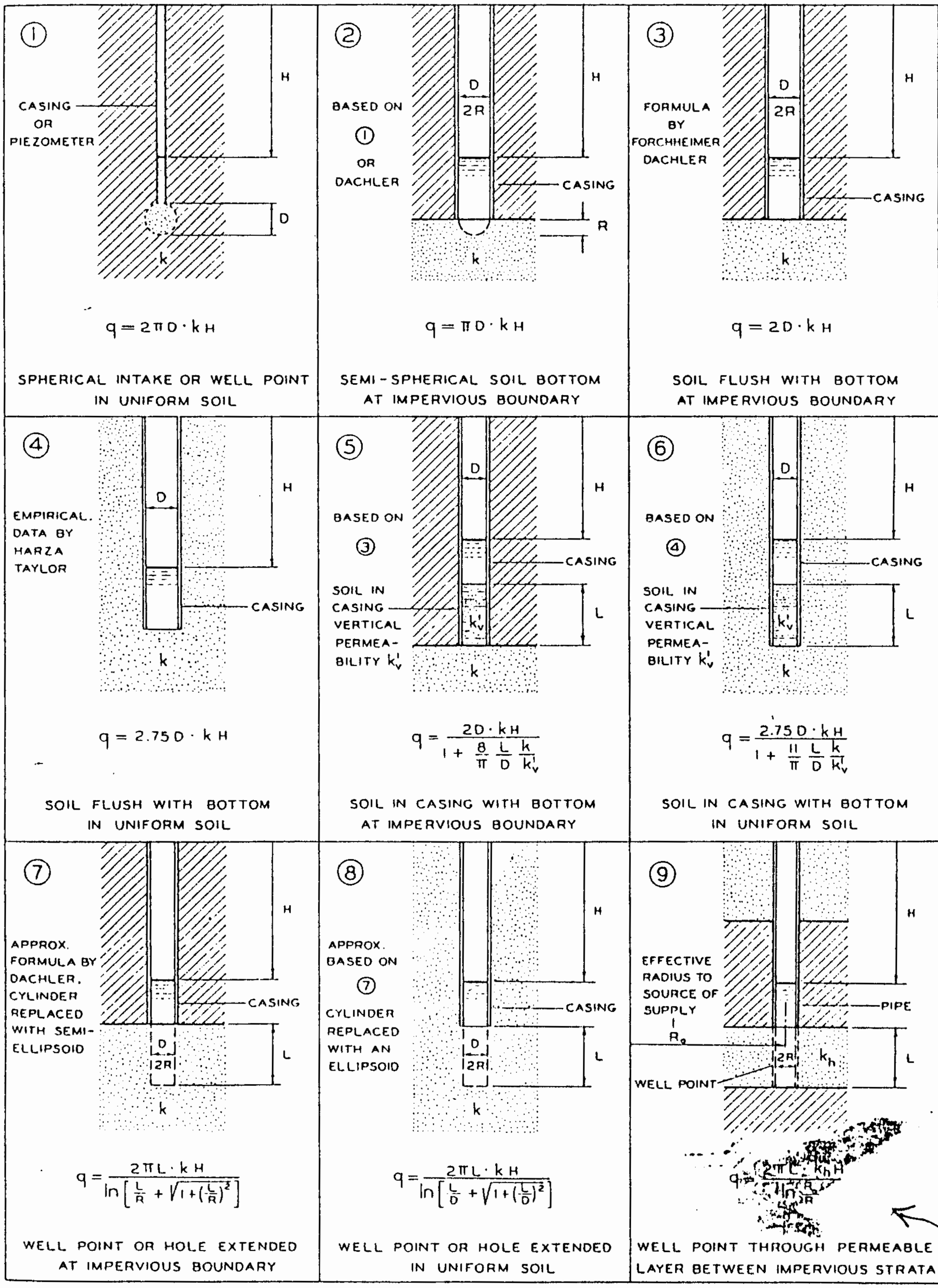
For the purpose of designing or selecting the proper type of pressure measuring installation for specific soil and ground-water conditions, the basic time lag may be computed by means of equation(3). In order to facilitate such computations, formulas for flow through various types or shapes of intakes or well points are assembled in Fig. 12. These formulas are all derived on the assumption that the soil stratum in which the well point is placed is of infinite thickness and that artesian conditions prevail, or that the inflow or outflow is so small that it does not cause any appreciable change in the ground-water level or pressure. Except when otherwise noted by subscripts, as in k_v and k_h, it is also assumed that the permeability of the soil, k, is uniform throughout the stratum and equal in all directions.

The formula for Case 1 is that for a point source, and by reasons of symmetry the flow in Case 2 is half as great, but the formula for this case has also been derived directly by DACHLER (6). Derivation of the formula for Case 3 is given in the books by FORCHHEIMER (9) and DACHLER (6). A simple formal mathematical solution for Case 4 is not known to the writer, and the formula shown in Fig. 12 is empirical and based on experiments by HARZA (12) and a graphical solution through radial flow nets by TAYLOR (28). The formulas for Cases 5 and 6 are derived by addition of the losses in piezometric pressure head outside the casing -- Cases 3 and 4 -- and in the soil inside the casing. The formulas are only approximately correct since it is assumed that the velocity of flow is uniformly distributed over the length and cross section of the soil plug. It is taken into consideration that for soil within the casing the vertical permeability is governing and may be different from that of the soil below the casing on account of soil disturbance and sedimentation.

The formula given for Case 7 is derived by DACHLER (6) on basis of flow from a line source for which the equipotential surfaces are semi-ellipsoids. Therefore, and as emphasized by DACHLER, the formula can provide only approximate results when it is applied to a cylindrical intake or well point. In Case 8 it is assumed that the flow lines are symmetrical with respect to a horizontal plane through the center of the intake, and the formula for Case 7 is then applied to the upper and lower halves of the intake. The accuracy of these formulas probably decreases with decreasing values of L/R and L/D. When these ratios are equal to unity, Cases 7 and 8 correspond to Cases 2 and 1, respectively, but furnish 13.4 per cent greater values for the flow. For large values of L/R and L/D the following simplified formulas may be used,

$$\text{CASE 7.} \qquad q = \frac{2\pi L k H}{\ln (2L/R)}$$

$$\text{CASE 8.} \qquad q = \frac{2\pi L k H}{\ln (2L/D)}$$



q = RATE OF FLOW IN cm^3/SEC, H = HEAD IN cm, k = COEF. OF PERMEABILITY IN cm/SEC, ln = $\log_e$, DIMENSIONS IN cm.

CASES 1 TO 8: UNIFORM PERMEABILITY AND INFINITE DEPTH OF PERVIOUS STRATUM ASSUMED

FORMULAS FOR ANISOTROPIC PERMEABILITY GIVEN IN TEXT

$$\frac{2\pi L \cdot k_h H}{\ln \frac{R_o}{R}}$$

Fig. 12. Inflow and shape factors

In this form the formulas were derived earlier by SAMSIOE (26). When L/R or L/D is greater than four, the error resulting from use of the simplified formulas is less than one per cent. In Case 9 the flow lines are horizontal and the coefficient of horizontal permeability, k_h, is governing. The effective radius, R_o, depends on the distance to the source of supply and to some extent on the compressibility of the soil, MUSKAT (22) and JACOB (17, 18). It may be noted that the simplified formula for Case 7 is identical with the formula for Case 9 when $R_o = 2L$. For flow through wells with only partial penetration of the pervious stratum, reference is made to MUSKAT (22) and the paper by MIDDLEBROOKS and JERVIS (21).

The assumptions, on which the derivation of the formulas in Fig. 12 are based, are seldom fully satisfied under practical conditions. It is especially to be noted that the horizontal permeability of soil strata generally is much larger than the vertical permeability. Correction of the formulas for the effect of anisotropic permeability is discussed in the following section. Even when such corrections are made, the formulas should be expected only to yield approximate results, since the soil strata are not infinite in extent and are rarely uniform in character. However and taking into consideration that the permeability characteristics of the soil strata seldom are accurately known in advance, the formulas are generally adequate for the purpose of preliminary design or selection of the proper type of pressure measuring installation, but the basic time lag obtained by the formulas should always be verified and corrected by means of field experiments.

Influence of Anisotropic Permeability

As first demonstrated by SAMSIOE (26) and later by DACHLER (6) for two-dimensional or plane problems of flow through soils, the influence of a difference between the coefficients of vertical and horizontal permeability of the soil, k_v and k_h, may be taken into consideration by multiplying all horizontal dimensions by the factor $\sqrt{k_v/k_h}$ and using the mean permeability $k_m = \sqrt{k_v \cdot k_h}$, whereafter formulas or flow nets for isotropic conditions may be used.

A general solution for three-dimensional problems and different but constant coefficients of permeability k_x, k_y, and k_z in direction of the coordinate axes is given by VREEDENBURG (31) and MUSKAT (22). With k_o an arbitrarily selected coefficient the following transformation is made,

$$x' = x\sqrt{k_o/k_x} \qquad y' = y\sqrt{k_o/k_y} \qquad z' = z\sqrt{k_o/k_z} \tag{28}$$

and when an equivalent coefficient of permeability

$$k_e = k_o\sqrt{\frac{k_x}{k_o} \cdot \frac{k_y}{k_o} \cdot \frac{k_z}{k_o}} \tag{29}$$

is used, then the problem may be treated as if the conditions were isotropic. In applying these transformations to problems of flow through intakes or well points in soil with horizontal isotropic permeability, k_h, and vertical permeability k_v, it is convenient to use the following substitutions,

$$k_o = k_z = k_v \qquad k_x = k_y = k_h \qquad \text{and} \qquad m = \sqrt{k_h/k_v} \tag{30}$$

whereby the transformations assume the following form,

$$x' = x/m \qquad y' = y/m \qquad \text{or} \qquad r' = r/m \qquad \text{and} \qquad z' = z \tag{31}$$

$$k_e = k_v \sqrt{m^2 \cdot m^2} = k_v \cdot m^2 = k_h \tag{32}$$

That is, the problems can be treated as for isotropic conditions when the horizontal dimensions are divided by the square root of the ratio between the horizontal and vertical coefficients of permeability and the flow through the transformed well points is computed for a coefficient of permeability equal to k_h. When these transformations are applied to Cases 1 and 2 in Fig. 12, the sphere and semi-sphere become an ellipsoid, respectively a semi-ellipsoid, and formulas corresponding to those for Cases 7 and 8 should then be used. In Cases 5 and 6 the transformations should be applied only to flow through soil below the casing and not to soil within the casing. With introduction of the mean coefficient of permeability,

$$k_m = \sqrt{k_v \cdot k_h} = m \cdot k_v = k_h/m \tag{33}$$

the flow through the intakes and well points shown in Fig. 12 can be expressed as follows:

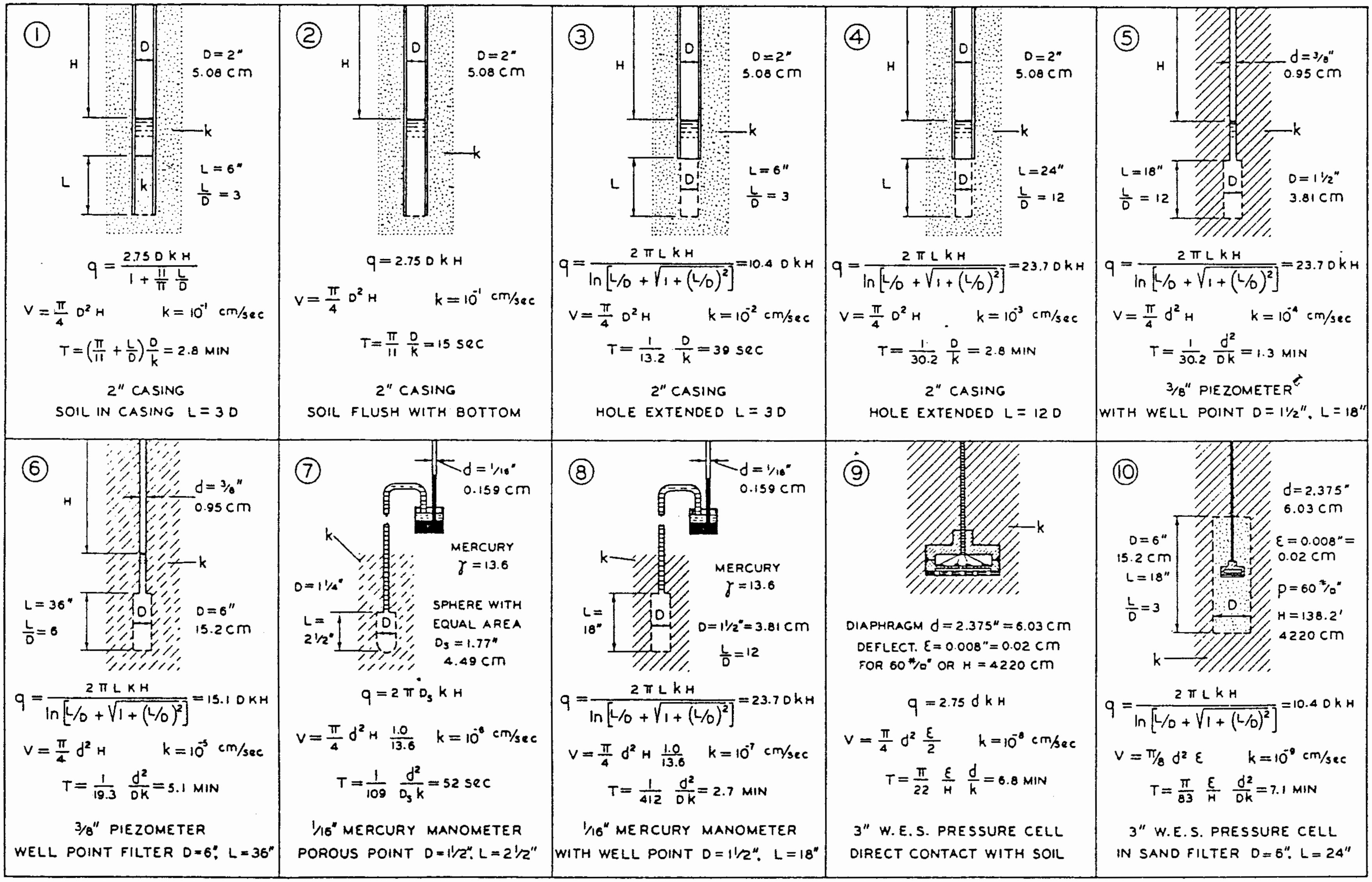
CASE 1. $$q = \frac{2\pi D k_h H}{\ln\left(m + \sqrt{1 + m^2}\right)}$$

CASE 2. $$q = \frac{\pi D k_h H}{\ln\left(m + \sqrt{1 + m^2}\right)}$$

CASE 3. $$q = 2 D k_m H$$

CASE 4. $$q = 2.75 D k_m H$$

CASE 5. $$q = \frac{2 D k_m H}{1 + \frac{8}{\pi} \frac{L}{D} \frac{k_m}{k'_v}}$$



q = RATE OF FLOW FOR HEAD H, V = TOTAL VOLUME OF FLOW TO EQUALIZE PRESSURES, T = V/q = BASIC TIME LAG, ISOTROPIC SOIL CONDITIONS ASSUMED

Fig. 13. Examples of computation of basic time lag

$$\text{CASE 6.} \qquad q = \frac{2.75\, D\, k_m H}{1 + \frac{11}{\pi}\frac{L}{D}\frac{k_m}{k_v'}}$$

$$\text{CASE 7.} \qquad q = \frac{2\pi L\, k_h H}{\ln\left(mL/R + \sqrt{1 + (mL/R)^2}\right)}$$

$$\text{CASE 8.} \qquad q = \frac{2\pi L\, k_h H}{\ln\left(mL/D + \sqrt{1 + (mL/D)^2}\right)}$$

The formula for Case 9 in Fig. 12 is already expressed in terms of the horizontal permeability and is not affected by the transformation. The modified formulas for Cases 1 and 2 should be considered as being only approximately correct, and for isotropic conditions or $m = 1$ they yield 13.4 per cent greater values of flow than obtained by the basic formulas in Fig. 12. In Cases 7 and 8 and for large values of mL/R or mL/D the denominators may be replaced with $\ln(2mL/R)$, respectively $\ln(2mL/D)$.

Computation of Time Lag for Design Purposes

Examples of computation of the basic time lag, using the flow formulas in Fig. 12, are shown in Fig. 13. In all cases it is assumed that the soil is uniform and the permeability equal in all directions; this applies also to soil in the casing as shown in Case 1. The porous cup point in Case 7 is replaced with a sphere of equal surface area and the flow computed as through a spherical well point. This transformation furnishes a time lag which is slightly too small, since flow through a spherical well point is greater than through a point of any other shape and equal surface area. The pressure cell shown in Cases 9 and 10 is similar to the one described in a report by the WATERWAYS EXPERIMENT STATION (33). It may be noted that hydrostatic pressure cells with a diaphragm diameter of only 3/4 in. have been built and used successfully by the Waterways Experiment Station, and that a pressure cell with a diaphragm diameter of about one inch is described in a paper by BOITEN and PLANTEMA (1); see also Fig. 8-B. It is emphasized that the basic time lags for Cases 9 and 10 are computed on the assumption that there is no accumulation of gases below the diaphragm or in the sand filter; see discussion on pages 7 and 8.

A few general rules may be deduced from the examples shown in Fig. 13. In all cases the basic time lag is inversely proportional to the coefficient of permeability. When the ratio between the effective length and the diameter of the intake,

TIME LAGS		FOR 90 PERCENT EQUALIZATION = T_{90}										BASIC TIME LAG T
APPROXIMATE SOIL TYPE		SAND			SILT			CLAY				
COEFFICIENT OF PERMEABILITY IN cm/sec		10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}	10^{-8}	10^{-9}	10^{-10}	10^{-6}
1	2" CASING - SOIL IN CASING, L = 3D = 6"	6^m	1^h	10^h	4.2^d							193^d
2	2" CASING - SOIL FLUSH BOTTOM CASING	0.6^m	6^m	1^h	10^h	4.2^d						17^d
3	2" CASING - HOLE EXTENDED, L = 3D = 6"		1.5^m	15^m	2.5^h	25^h	10^d					4.5^d
4	2" CASING - HOLE EXTENDED, L = 12D = 24"			6^m	1^h	10^h	4.2^d	42^d				47^h
5	3/8" PIEZOMETER WITH WELL POINT DIAMETER 1 1/2", LENGTH 18"				3^m	30^m	5^h	50^h	21^d			130^m
6	3/8" PIEZOMETER WITH WELL POINT AND SAND FILTER, D = 6", L = 36"					12^m	2^h	20^h	8.3^d	83^d		51^m
7	1/16" MERCURY MANOMETER, SINGLE TUBE WITH POROUS CUP POINT, D = 1 1/4", L = 2 1/2"		ONE-HALF OF VALUES FOR 1/16" MERCURY U-TUBE MANOMETER OR 4 1/2" BOURDON GAGE.				2^m	20^m	3.3^h	33^h	14^d	52^s
8	1/16" MERCURY MANOMETER, SINGLE TUBE WITH WELL POINT, D = 1 1/2", L = 18"							6^m	1^h	10^h	4.2^d	16^s
9	3" W. E. S. HYDROSTATIC PRESSURE CELL IN DIRECT CONTACT WITH SOIL								16^m	2.6^h	26^h	4^s
10	3" W. E. S. HYDROSTATIC PRESSURE CELL IN SAND FILTER, D = 6", L = 18"									16^m	2.6^h	0.4^s

SYMBOLS: s = SECONDS, m = MINUTES, h = HOURS, d = DAYS – ASSUMPTIONS: CONSTANT GROUND-WATER PRESSURE AND INTAKE SHAPE FACTOR, ISOTROPIC SOIL, NO GAS, STRESS ADJUSTMENT TIME LAG NEGLIGIBLE. THE COMPUTED TIME LAGS HAVE BEEN ROUNDED OFF TO CONVENIENT VALUES

Fig. 14. Approximate hydrostatic time lags

L/D, remains constant, the basic time lag is inversely proportional to the diameter of the intake and directly proportional to the cross-sectional area or the square of the diameter of the piezometer or manometer tube. When furthermore the diameters of the intake and piezometer are equal, Cases 1 to 4, the basic time lag is directly proportional to the diameter.

The results of the examples in Fig. 13 are summarized in a slightly different form in the last column in Fig. 14. The basic time lags are here given for a coefficient of permeability $k = 10^{-6}$ cm/sec., and these time lags may be used as a rating of the response to pressure changes for the various types of installations. For the examples shown in Figs. 13 and 14 this rating time lag varies from 193 days for a 2-in. boring with 6 in. of soil in the casing to 0.4 seconds for a 3-in. pressure cell placed in a 6-in. by 18-in. sand filter.

In the central part of Fig. 14 the basic time lags for various coefficients of permeability have been multiplied by 2.3 and indicate the time lags for 90 per cent equalization of the original pressure difference, which approximately is the time lag to be considered in practical operations. As mentioned on page 12, the time lag for 99 per cent equalization is twice as great as for 90 per cent equalization. According to data furnished the writer by Dr. A. WARLAM, the volume change of a 4-1/2-in. Bourdon pressure gage is 0.5 to 1.0 cm^3 for 1.0 kg/cm^2 change in pressure, or approximately half of that for a 1/16-in., single-tube, mercury manometer. Therefore, when the standpipe in Cases 7 and 8 is connected to a 4-1/2-in. Bourdon gage or to a double-tube mercury manometer with 1/16-in. inside diameter, the time lags will be about one-half those shown for a 1/16-in., single-tube mercury manometer. It is possible that the above mentioned volume change for a Bourdon pressure gage includes deformations of pliable rubber or plastic tube connections used in the experiments, and that the volume changes and corresponding time lags are smaller when rigid connections are used.

In all cases the computed time lags should be considered as being only approximate values, and they have been rounded off to convenient figures. The actual time lags may be influenced by several factors not taken into consideration in the above mentioned computations, such as stress adjustment and volume changes of soil and gases in the soil or pressure measuring system, sedimentation or clogging of the well point, filter, or surrounding soil, etc. The actual time lags may therefore be considerably greater or smaller than those indicated in Figs. 13 and 14, and special attention is called to the fact that the horizontal permeability of the soil, because of stratifications, often is many times greater than the vertical permeability as generally determined by laboratory tests and often used as a measure of the permeability of the soil stratum as a whole. Nevertheless, the examples shown in Figs. 13 and 14 will furnish some indication of the relative responsiveness of the various types of installations and permit a preliminary selection of the type suited for specific conditions and purposes.

Examples of Field Observations and Their Evaluation

Logan International Airport, Boston

Observations of pore-water pressures in the foundation soil of Logan International Airport at Boston are described in papers by CASAGRANDE (3) and GOULD (10). Most of the piezometers used were of the Casagrande type, shown diagrammatically in Fig. 15-A. The results of a series of time lag tests for piezometer C are

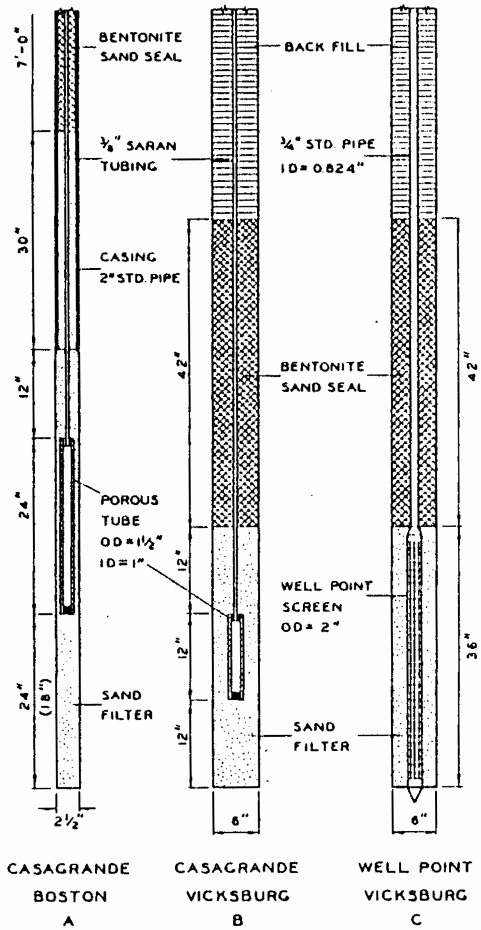


Fig. 15. Piezometers used in tests

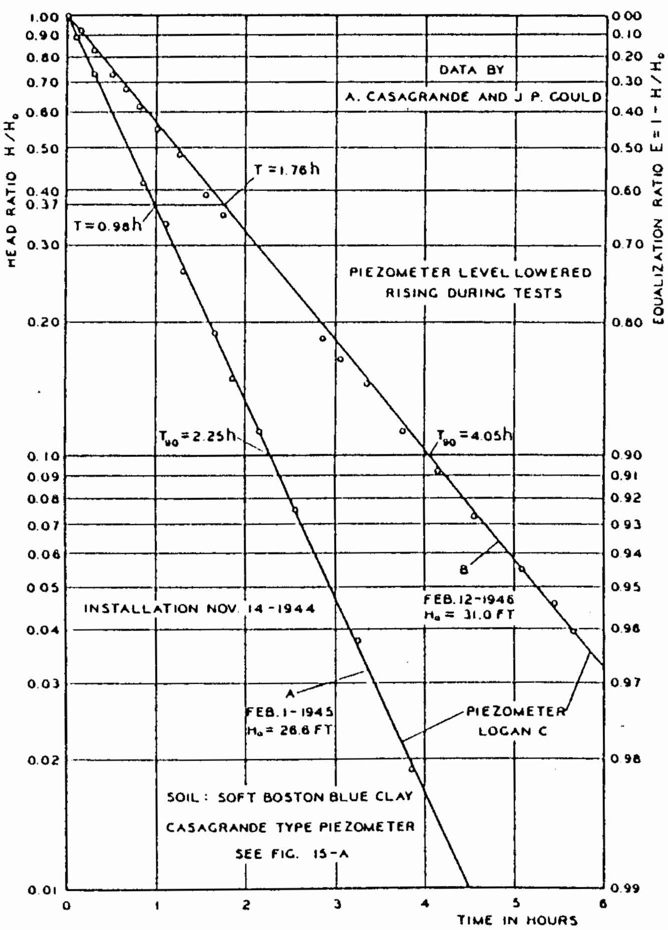


Fig. 16. Time lag tests at Logan Airport, Boston

summarized in the paper by GOULD and further details were placed at the writer's disposal by CASAGRANDE. The filter or intake for this piezometer is installed in soft Boston Blue clay at a depth of 47 ft below the finished grade of fill.

The equalization diagrams obtained in two of the above mentioned tests, performed a year apart, are shown in Fig. 16. The first of these diagrams is straight, thereby indicating that the influence of transient stress adjustments or volume changes of the soil and gas in the voids is negligible; the basic time lag determined by this diagram is 0.98 hours. The equalization diagram obtained a year later shows a slight curvature and a basic time lag of 1.76 hours. Since the curvature is very small, the increase in time lag is probably caused by clogging of the porous tube or point and the filter. Estimates of the coefficients of permeability of the soil were obtained by means of new methods of settlement analysis, GOULD (10), and it was found that k_v varies between (28 and 35) x 10^{-9} cm/sec and k_h between (940 and 1410) x 10^{-9} cm/sec. Using the average values $k_v = 31.5 \times 10^{-9}$ cm/sec and $k_h = 1175 \times 10^{-9}$ cm/sec, the transformation ratio, m, is then

$$m = \sqrt{k_h/k_v} = \sqrt{37.5} = 6.1$$

The dimensions of the installation as given in the paper by GOULD are: diameter of filter D = 2.5 in. = 6.35 cm; length of filter L = 54 in. = 137.2 cm; inside diameter of piezometer d = 0.375 in. = 0.95 cm. The rate of flow for the active head H is obtained by the simplified formula for Case 8 on page 35

$$q = \frac{2\pi L k_h H}{\ln (2mL/D)}$$

and the total volume of flow required for equalization is,

$$V = \frac{\pi}{4} d^2 H$$

The basic time lag as determined by equation 3 is then,

$$T = \frac{V}{q} = \frac{d^2 \ln (2mL/D)}{8 L k_h} = \frac{0.95^2 \ln (263.6)}{8 \cdot 137.2 \cdot 1175} 10^9 = 3910 \text{ sec} = 1.09 \text{ hours} \quad (34)$$

which agrees closely with the actual time lag, T = 0.98 hours.

Vicinity of Vicksburg, Mississippi

A preliminary series of comparative tests with various types of observation wells and piezometers has been performed by the WATERWAYS EXPERIMENT STATION (34). The wells and piezometers were installed behind the Mississippi River levees at two locations, Willow Point and Reid Bedford Bend. Time lag tests were made one to eight months after installation, and some of the equalization diagrams obtained in these tests are shown in Fig. 17. All the diagrams show a distinct initial curvature, and the period of observations was often too short, covering only the first and curved part of the diagrams. It was observed that gas emerged from some of the piezometers, and it is probable that the initial curvature of the equalization diagrams is caused by transient volume changes of gas bubbles accumulated in the

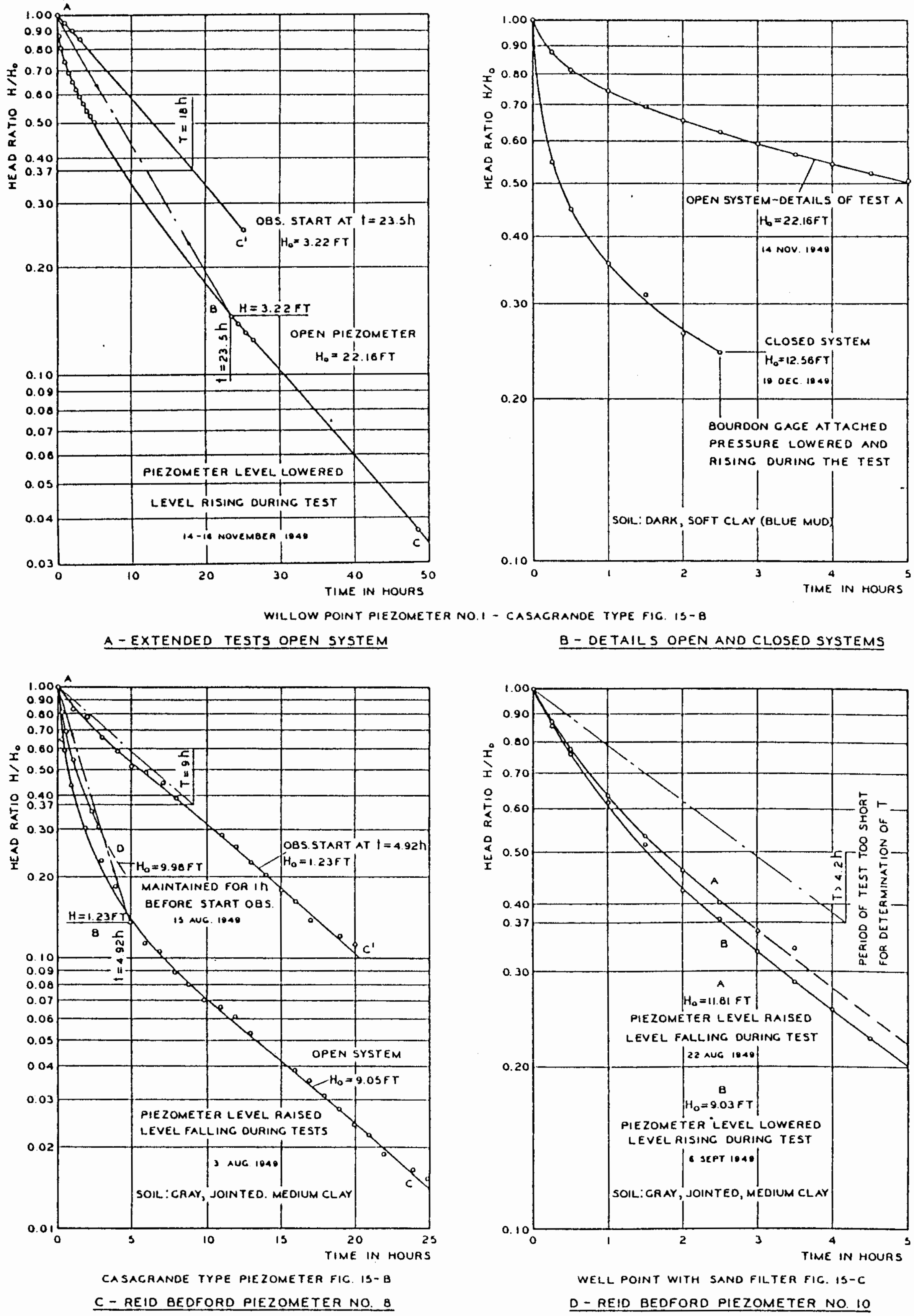


Fig. 17. Time lag tests by the Waterways Experiment Station, Vicksburg

soil near the well points or filters. The individual piezometers in the two groups are only 15 ft apart, and it is possible that time lag tests on a piezometer to a minor extent were influenced by flow to or from neighboring piezometers.

Laboratory tests on soil samples from the vicinity of the intakes for these installations indicate that the coefficients of vertical permeability vary between (10 and 150) x 10^{-9} cm/sec. Data on the coefficients of horizontal permeability are not available, and the soils at Reid Bedford Bend were jointed. Therefore, reliable estimates of the theoretical basic time lags cannot be made, but the basic time lags obtained by means of the equalization diagrams fall between those computed on basis of isotropic conditions and coefficients of permeability equal to the above mentioned upper and lower limits of the coefficients of vertical permeability.

Piezometer No. 1 at Willow Point is of the modified Casagrande type, Fig. 15-B, and is installed 92.5 ft below ground surface in a soft dark clay, locally known as "blue mud." The first part of the equalization diagram, Fig. 17-A, is curved but the lower part is fairly straight, possibly with a slight reverse curvature. If the observations are started 23.5 hours after the piezometer level was lowered, the diagram A-C' would be obtained; this diagram is parallel to the lower part, B-C, of the main diagram. As indicated on page 28, it is probable that the effective equalization diagram for the piezometer under normal operating conditions may be represented by a straight line through the origin and parallel to the lower and fairly straight part of the diagram obtained in a time lag test. By drawing such a line in Fig. 17-A, an effective basic time lag $T = 18$ hours is obtained.

In a second time lag test a Bourdon pressure gage was attached to the piezometer so that a closed system was formed. The pressure in the system was lowered by bleeding off a small amount of water, but the piezometric pressure level was above the gage level throughout the test. The equalization diagram obtained by observing the subsequent rise in pressure, Fig. 17-B, is lower and has considerably greater curvature than the one for an open system, which can be explained by the fact that the total amount of flow required for pressure equalization in the closed system is materially decreased, and the influence of volume changes of the gas bubbles and the soil consequently is greater.

Piezometer No. 8 at Reid Bedford is also of the modified Casagrande type and is installed 30 ft below ground surface in a gray, jointed, medium clay. The irregular, closely spaced joints in this clay are probably caused by previous drying, and the surfaces of some of the joints are covered with a thin layer of silt, but the joints at the depth of the piezometer intake are probably closed. The equalization diagram, A-B-C in Fig. 17-C, shows a pronounced initial curvature, but the lower part of the diagram is fairly straight. A straight line through the origin and parallel to the lower part of the diagram indicates an effective basic time lag $T = 9$ hours. In a second test the head -- $H_0 = 9.98$ ft -- was maintained for one hour before the piezometer level was allowed to fall and the observations were started. The resulting equalization diagram, A-D, is above the first diagram and not so strongly curved. If the full head had been maintained for at least 24 hours, it is probable that a diagram similar to A-C' or the lower portion, B-C, of the main diagram would have been obtained.

Piezometer No. 10 at Reid Bedford is installed 15 ft from piezometer No. 8 and at the same depth. The sand filter has the same dimensions as for No. 8, but the porous tube is replaced with a well point screen extending through the whole length of the filter, and the piezometer proper is a 3/4-in. standard pipe; Fig. 15-C. Equalization diagrams were obtained for both falling and rising piezometer levels and are shown in Fig. 17-D. The periods of observation are too short for definite determination of the effective basic time lag, which is greater than 4.2 hours but probably smaller than the 9 hours obtained for piezometer No. 8. The initial curvature of the diagrams is considerably less than that of the diagrams for piezometer No. 8, which may be explained by the fact that the cross-sectional area of the piezometer pipe is $(0.824/0.375)^2 = 4.8$ times as great and that the influence of volume changes of soil and gas bubbles consequently is smaller. However, the basic time lag should then also be 4.8 times as great, since the dimensions of the sand filters for piezometers 8 and 10 are identical, but the equalization diagrams indicate a smaller time lag. This inconsistency may be due to local joints and other irregularities in soil conditions, but it is also probable that the well point screen is less subject to clogging than a porous tube, and that gases can escape more easily since the screen extends to the top of the sand filter.

Piezometer No. 11 at Reid Bedford consists of a 3/4-in. standard pipe with its lower end in the center of a sand filter at the same depth and with the same dimensions as the filters for piezometers 8 and 10. The time lag observations for piezometer No. 11 are incomplete but indicate that the effective basic time lag is at least 25 hours. It is probable that this increase in time lag, in comparison with piezometers 8 and 10, is caused by clogging of the sand in the immediate vicinity of the end of the pipe and of sand which may have entered the lower part of the pipe. Cleaning of the pipe and subsequent careful surging would undoubtedly decrease the time lag, but it is probable that clogging would re-occur in time.

Piezometer No. 15 at Reid Bedford is a 3/4-in. standard pipe with a solid drive point and a 4-in.-long, perforated section above the point. The pipe was driven to the same depth as the other piezometers and then withdrawn one foot. In a time lag test the piezometer level was raised 7.48 ft, and in 22.7 hours it fell only 0.12 ft. The lower part of the equalization diagram, during which the piezometer level fell from 7.45 ft to 7.36 ft in 17 hours, is fairly straight. For such a small drop in piezometer level it is better to compute the effective basic time lag by means of equation (5) than to determine it graphically; that is,

$$T = \frac{t}{\ln (H_o/H)} = \frac{17}{\ln (7.45/7.36)} = 1730 \text{ hours} = 72 \text{ days} \tag{35}$$

Because of the solid drive point, it is doubtful that withdrawal of the pipe for one foot materially affects flow to or from the perforated section, and the effective length of the latter would then be less than 4 in., even when the perforations remain open. However, it is possible that the perforations have been filled with molded soil during the driving, that a smear layer of remolded soil is formed around the pipe, and that this layer has covered the joints in the clay and decreased its effective permeability.

Determination of Permeability of Soil in Situ

Basic formulas

When the dimensions or shape factor, F, of a pressure measuring installation are known, it is theoretically possible to determine the coefficients of permeability of the soil in situ by field observations.

For constant head, H_c, and rate of flow, q, equation (1) yields,

$$k = \frac{q}{F H_c} \tag{36}$$

For variable head but constant ground-water level or pressure, the heads H_1 and H_2 corresponding to the times t_1 and t_2, and $A = \frac{\pi}{4} d^2$ the cross-sectional area of the standpipe, the following expression is obtained by means of equation (5),

$$t_2 - t_1 = T \left(\ln \frac{H_o}{H_2} - \ln \frac{H_o}{H_1}\right) = \frac{A}{F k} \ln \frac{H_1}{H_2}$$

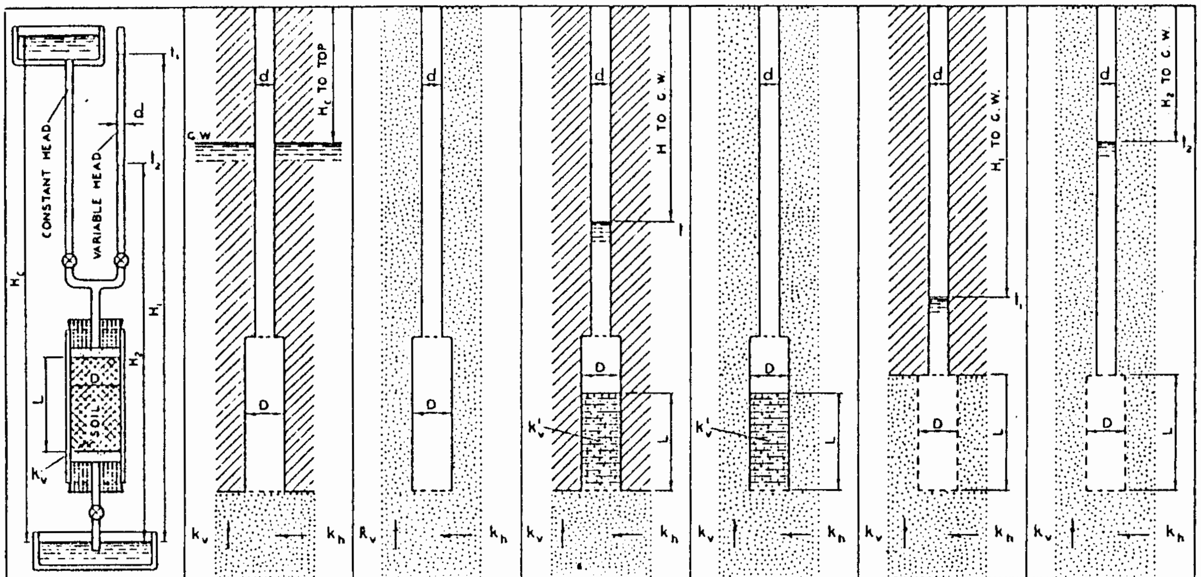
$$k = \frac{A}{F (t_2 - t_1)} \ln \frac{H_1}{H_2} \tag{37}$$

This is also the formula commonly used for determination of coefficients of permeability in the laboratory by means of a variable head permeameter.

The simplest expression for the coefficient of permeability is obtained by determination of the basic time lag, T, of the installation and use of equation (3); that is,

$$k = \frac{A}{F T} \tag{38}$$

The shape factors, F, for various types of observation wells and piezometers may be obtained from the formulas in Fig. 12 and on pages 33 and 35 by eliminating the factors ($k H$), respectively ($k_m H$) or ($k_h H$), from the right side of the



LABORATORY PERMEAMETER (CONSOLIDOMETER)	FLUSH BOTTOM AT IMPERVIOUS BOUNDARY	FLUSH BOTTOM IN UNIFORM SOIL	SOIL IN CASING AT IMPERVIOUS BOUNDARY	SOIL IN CASING IN UNIFORM SOIL	WELL POINT-FILTER AT IMPERVIOUS BOUNDARY	WELL POINT-FILTER IN UNIFORM SOIL
A	B	C	D	E	F	G

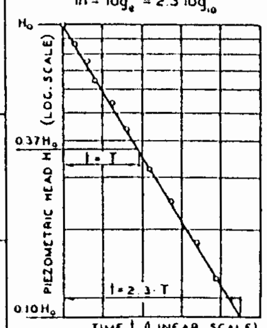
CASE	CONSTANT HEAD	VARIABLE HEAD	BASIC TIME LAG
A	$k_v = \frac{4\cdot q\cdot L}{\pi\cdot D^2\cdot H_c}$	$k_v = \frac{d^2\cdot L}{D^2\cdot(t_2-t_1)}\ln\frac{H_1}{H_2}$ $k_v = \frac{L}{t_2-t_1}\ln\frac{H_1}{H_2}$ FOR $d = D$	$k_v = \frac{d^2\cdot L}{D^2\cdot T}$ $k_v = \frac{L}{T}$ FOR $d = D$
B	$k_m = \frac{q}{2\cdot D\cdot H_c}$	$k_m = \frac{\pi\cdot d^2}{8\cdot D\cdot(t_2-t_1)}\ln\frac{H_1}{H_2}$ $k_m = \frac{\pi\cdot D}{8\cdot(t_2-t_1)}\ln\frac{H_1}{H_2}$ FOR $d = D$	$k_m = \frac{\pi d^2}{8\cdot D\cdot T}$ $k_m = \frac{\pi\cdot D}{8\cdot T}$ FOR $d = D$
C	$k_m = \frac{q}{2.75\cdot D\cdot H_c}$	$k_m = \frac{\pi\cdot d^2}{11\cdot D\cdot(t_2-t_1)}\ln\frac{H_1}{H_2}$ $k_m = \frac{\pi\cdot D}{11\cdot(t_2-t_1)}\ln\frac{H_1}{H_2}$ FOR $d = D$	$k_m = \frac{\pi\cdot d^2}{11\cdot D\cdot T}$ $k_m = \frac{\pi\cdot D}{11\cdot T}$ FOR $d = D$
D	$k_v' = \frac{4\cdot q\cdot\left[\frac{\pi}{8}\cdot\frac{k_v'}{k_v}\cdot\frac{D}{m}+L\right]}{\pi\cdot D^2\cdot H_c}$	$k_v' = \frac{d^2\cdot\left[\frac{\pi}{8}\cdot\frac{k_v'}{k_v}\cdot\frac{D}{m}+L\right]}{D^2\cdot(t_2-t_1)}\ln\frac{H_1}{H_2}$ $k_v = \frac{\frac{\pi}{8}\cdot\frac{D}{m}+L}{t_2-t_1}\ln\frac{H_1}{H_2}$ FOR $k_v' = k_v$, $d = D$	$k_v' = \frac{d^2\cdot\left[\frac{\pi}{8}\cdot\frac{k_v'}{k_v}\cdot\frac{D}{m}\right]+L}{D^2\cdot T}$ $k_v = \frac{\frac{\pi}{8}\cdot\frac{D}{m}+L}{T}$ FOR $k_v' = k_v$, $d = D$
E	$k_v' = \frac{4\cdot q\cdot\left[\frac{\pi}{11}\cdot\frac{k_v'}{k_v}\cdot\frac{D}{m}+L\right]}{\pi\cdot D^2\cdot H_c}$	$k_v' = \frac{d^2\left[\frac{\pi}{11}\cdot\frac{k_v'}{k_v}\cdot\frac{D}{m}+L\right]}{D^2\cdot(t_2-t_1)}\ln\frac{H_1}{H_2}$ $k_v = \frac{\frac{\pi}{11}\cdot\frac{D}{m}+L}{t_2-t_1}\ln\frac{H_1}{H_2}$ FOR $k_v' = k_v$, $d = D$	$k_v' = \frac{d^2\cdot\left[\frac{\pi}{11}\cdot\frac{k_v'}{k_v}\cdot\frac{D}{m}+L\right]}{D^2\cdot T}$ $k_v' = \frac{\frac{\pi}{11}\cdot\frac{D}{m}+L}{T}$ FOR $k_v' = k_v$, $d = D$
F	$k_h = \frac{q\cdot\ln\left[\frac{2mL}{D}+\sqrt{1+\left(\frac{2mL}{D}\right)^2}\right]}{2\cdot\pi\cdot L\cdot H_c}$	$k_h = \frac{d^2\cdot\ln\left[\frac{2mL}{D}+\sqrt{1+\left(\frac{2mL}{D}\right)^2}\right]}{8\cdot L\cdot(t_2-t_1)}\ln\frac{H_1}{H_2}$ $k_h = \frac{d^2\cdot\ln\left(\frac{4mL}{D}\right)}{8\cdot L\cdot(t_2-t_1)}\ln\frac{H_1}{H_2}$ FOR $\frac{2mL}{D} > 4$	$k_h = \frac{d^2\cdot\ln\left[\frac{2mL}{D}+\sqrt{1+\left(\frac{2mL}{D}\right)^2}\right]}{8\cdot L\cdot T}$ $k_h = \frac{d^2\cdot\ln\left(\frac{4mL}{D}\right)}{8\cdot L\cdot T}$ FOR $\frac{2mL}{D} > 4$
G	$k_h = \frac{q\cdot\ln\left[\frac{mL}{D}+\sqrt{1+\left(\frac{mL}{D}\right)^2}\right]}{2\cdot\pi\cdot L\cdot H_c}$	$k_h = \frac{d^2\cdot\ln\left[\frac{mL}{D}+\sqrt{1+\left(\frac{mL}{D}\right)^2}\right]}{8\cdot L\cdot(t_2-t_1)}\ln\frac{H_1}{H_2}$ $k_h = \frac{d^2\cdot\ln\left(\frac{2mL}{D}\right)}{8\cdot L\cdot(t_2-t_1)}\ln\frac{H_1}{H_2}$ FOR $\frac{mL}{D} > 4$	$k_h = \frac{d^2\cdot\ln\left[\frac{mL}{D}+\sqrt{1+\left(\frac{mL}{D}\right)^2}\right]}{8\cdot L\cdot T}$ $k_h = \frac{d^2\cdot\ln\left(\frac{2mL}{D}\right)}{8\cdot L\cdot T}$ FOR $\frac{mL}{D} > 4$

NOTATION

D = DIAM. INTAKE, SAMPLE, CM
d = DIAMETER, STANDPIPE, CM
L = LENGTH, INTAKE, SAMPLE, CM
H_c = CONSTANT PIEZ. HEAD, CM
H_1 = PIEZ. HEAD FOR $t = t_1$, CM
H_2 = PIEZ. HEAD FOR $t = t_2$, CM
q = FLOW OF WATER, CM^3/SEC.
t = TIME, SEC.
T = BASIC TIME LAG, SEC.
k_v' = VERT. PERM. CASING, CM/SEC.
k_v = VERT. PERM. GROUND, CM/SEC.
k_h = HORZ. PERM. GROUND, CM/SEC.
k_m = MEAN COEFF. PERM., CM/SEC.
m = TRANSFORMATION RATIO

$k_m = \sqrt{k_h\cdot k_v} \qquad m = \sqrt{k_h/k_v}$

$\ln = \log_e = 2.3\log_{10}$



DETERMINATION BASIC TIME LAG T

ASSUMPTIONS

SOIL AT INTAKE, INFINITE DEPTH AND DIRECTIONAL ISOTROPY (k_v AND k_h CONSTANT) - NO DISTURBANCE, SEGREGATION, SWELLING OR CONSOLIDATION OF SOIL - NO SEDIMENTATION OR LEAKAGE - NO AIR OR GAS IN SOIL, WELL POINT, OR PIPE - HYDRAULIC LOSSES IN PIPES, WELL POINT OR FILTER NEGLIGIBLE

Fig. 18. Formulas for determination of permeability

equations. Explicit formulas for determination of coefficients of permeability by constant head, variable head, and basic time lag tests with permeameters and various types of borings and piezometers are summarized in Fig. 18. For a permeameter, Case A, the rate of flow for the head H is $q = \frac{\pi}{4} D^2 kH/L$, or $F = \frac{\pi}{4} D^2/L$. In cases D and E the coefficient of vertical permeability of soil in the casing is usually governing, and the equations have been solved for this coefficient and appear in a form slightly different from that corresponding to Cases (5) and (6) in Fig. 12 and on pages 33 and 35. Simplified formulas for $d = D$, $k_v' = k_v$, and the ratio (mL/D) greater than 2 or 4, are given below the main formulas in each case.

The basic time lag is easily determined by means of an equalization diagram -- or a semilogarithmic plot of time versus head -- as the time T corresponding to $H = 0.37H_o$; i.e., $\ln(H_o/H) = 1$. The work involved in plotting the diagram is offset by simpler formulas for computing the coefficient of permeability, compared to the formulas for variable head, and the diagram has the great advantage that it reveals irregularities caused by volume changes or stress adjustment time lag and permits easy advance adjustment of the results of the tests. It is emphasized that the above mentioned methods and formulas are applicable only when the basic assumptions for the theory of time lag, page 9, are substantially correct.

Examples of applications

The following dimensions apply to the permeability tests on Atlantic muck, Fig. 10: D = 4.25 in. = 10.8 cm; L = 0.87 in. = 2.21 cm; d = 0.30 cm. The basic time lag obtained from the probable normal diagram in Fig. 10 is T = 178 minutes, and hence

$$k_v = \frac{d^2 L}{D^2 T} = \frac{0.30^2 \cdot 2.21}{10.8^2 \cdot 178 \cdot 60} = 159 \times 10^{-9} \text{ cm/sec.}$$

The slope of the lower parts of the equalization diagrams corresponds to a basic time lag T = 210 min and $k_v = 135 \times 10^{-9}$ cm/sec. Larger basic time lags and correspondingly smaller values of the coefficients of permeability were obtained in similar tests with other undisturbed samples of Atlantic muck.

The first test with piezometer C at Logan International Airport, Fig. 16, gave a basic time lag T = 0.98 hours = 3530 seconds. With $k_v = 31.5 \times 10^{-9}$ cm/sec and the dimensions given on page 39, the coefficient of horizontal permeability of Boston Blue clay may be determined as follows:

$$k_h = \frac{d^2 \ln(2mL/D)}{8 \cdot L \cdot T} = \frac{0.95^2 \ln(m \cdot 43.2)}{8 \cdot 137.2 \cdot 3530} = 233.5 \cdot 10^{-9} \cdot \ln(m \cdot 43.2)$$

This equation may be solved by estimating the value of $m = \sqrt{k_h/k_v}$ and successive corrections, which yield

$$k_h = 1310 \times 10^{-9} \text{ cm/sec} \quad \text{and} \quad k_h/k_v = 1310/31.5 = 41.6$$

These values lie within the limits obtained by other methods, GOULD (10), and discussed on page 39.

The second time lag test with piezometer C gave T = 1.76 hours and indicated thereby that clogging of the porous tube had taken place. Therefore, reliable values of the coefficient of permeability can no longer be obtained by means of this installation. This applies also to the installations at Willow Point and Reid Bedford, Fig. 17, since the strong initial curvature of the equalization diagrams indicates large transient volume changes and probably accumulation of gas bubbles in the sand filters and surrounding soil with a consequent decrease in permeability of this soil and increase in time lag.

Advantages and limitations

Observation of the basic time lag for borings and piezometers provides theoretically a very simple method for determination of the permeability of soil in situ, even for anisotropic conditions. However, many difficulties are encountered in the practical execution of such permeability tests and evaluation of the results obtained, since the latter are subject to the same sources of error as those of pressure observations discussed in Part I, and since methods of correction for the influence of some of these sources of error have not yet been devised.

The shape factor of the installation must be computed, but some of the formulas in Figs. 12 and 18 are empirical or only approximately correct, and they are all based on the assumption of infinite thickness of the soil layer in which the well point or intake is installed. When sand filters are used, the dimensions must be determined with greater accuracy than is required for pressure observations. The greatest part of the hydraulic friction losses occur near the intake, and the results of a test consequently indicate the permeability of the soil in the immediate vicinity of the intake. Misleading results are obtained when the permeability of this soil is changed by disturbance of the soil during advance of a bore hole or installation of filters or well points. Leakage, clogging of the intake or removal of fine-grained particles from the surrounding soil, and accumulation of gases near the intake or within the pressure measuring system may render the installation wholly unreliable as a means of determining the permeability of the undisturbed soil. Gas bubbles in the soil near the intake will decrease the permeability, cause curvature of the equalization diagram, and increase the effective basic time lag. Gas bubbles in a coarse-grained filter or within the pressure measuring system will not cause any appreciable curvature of the equalization diagram but will materially decrease the slope of the diagram and increase the basic time lag so that too small values of the coefficients of permeability are obtained.

Many of the above mentioned sources of error are avoided in the commonly used pumping tests, during which the shape of the draw-down curve is determined for a given rate of flow, but such tests are expensive and time consuming. Determination of the permeability of soil in situ by means of the time lag of observation wells and piezometers has so many potential advantages that it is to be hoped that systematic research will be undertaken in an effort to develop reliable methods of calibration or experimental determination of shape factors, and also of methods for detection, correction, or elimination of the various sources of error in the observations. Until such research is successfully completed, it is advisable to exert great caution in the practical application of the results obtained by the method.

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DEWATERING AND GROUNDWATER CONTROL FOR DEEP EXCAVATIONS

DEPARTMENTS OF THE ARMY, THE NAVY, AND THE AIR FORCE

APRIL 1971

TECHNICAL MANUAL
No. 5-818-5
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AIR FORCE MANUAL
No. 88-5, Chapter 6

DEPARTMENTS OF THE ARMY,
THE NAVY, AND THE AIR FORCE

Washington, D. C., 6 April 1971

DEWATERING AND GROUNDWATER CONTROL FOR DEEP EXCAVATIONS

APPENDIX I
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APPENDIX IV

METHODS OF ANALYSIS FOR GROUNDWATER FLOW TO A DEWATERING OR DRAINAGE SYSTEM

SECTION 1. MATHEMATICAL ANALYSES

1. INTRODUCTION.

a. Design of a dewatering system requires determination of the number, size, spacing, and penetration of wells or wellpoints and the rate at which water must be removed from the pervious strata to achieve the required groundwater lowering or pressure relief. The size and capacity of pumps and collectors also depend on the required discharge and drawdown. This appendix presents fundamental relations between well and wellpoint discharge and corresponding drawdown. The equations presented assume that (a) laminar flow exists, (b) the pervious stratum is homogeneous and isotropic, (c) water draining into the system is pumped out at a constant rate, and (d) flow conditions have stabilized. Procedures for transforming an anisotropic aquifer, with respect to permeability, to an isotropic section are presented in appendix V.

b. The equations in this appendix are in two groups: (1) drawdown for flow to slots and (2) drawdown for flow to wells. Equations for slots are applicable to flow to trenches, French drains, and similar drainage systems. They may also be used where the drainage system consists of closely spaced wells or wellpoints. Assuming a well system equivalent to a slot usually simplifies the analysis; however, corrections must be made to consider that the drainage system consists of wells or wellpoints rather than the more efficient slot. These corrections are given with the well formulas discussed in paragraph 3 of this appendix. When the well system cannot be simulated with a slot, well equations must be used. The figures in which these equations appear are listed in table IV-1. The equations for slots and wells do not consider the effects of hydraulic head losses H_w in wells or wellpoints; procedures for accounting for these effects are presented separately.

2. FLOW TO A SLOT.

a. Line Slots. Equations presented in figures IV-1 through IV-5 can be used to compute flow and drawdown produced by pumping either a single or a double continuous slot of infinite length. These equations assume that the source of seepage and the drainage slot are infinite in length and parallel, and that seepage enters the pervious stratum from a vertical line source. In actuality, the slot will be of finite length and the flow at the ends of the slot for a distance of about L/2 (where L equals distance between slot and source) will be greater, and the drawdown less than for the central portion of the slot. Flow to the central portion of a long slot will be approximately that computed for an assumed infinite length. Flow to the ends of a fully penetrating slot can be estimated, if necessary, from flow-net analyses subsequently presented.

b. Circular and Rectangular Slots. Equations for flow and drawdown produced by circular and rectangular slots supplied by a circular seepage source are given in figures IV-6 through IV-9. Equations for flow from a circular seepage source assume that the slot is located in the center of an island of radius R . However, for many dewatering projects R is the radius of influence rather than the radius of an island, and procedures for determining the value of R are discussed in paragraph 4 of this appendix. Dewatering systems of relatively short length are considered to have a circular source when they are far removed from a line source such as a river or reservoir.

3. FLOW TO WELLS.

a. Flow to Wells from a Circular Source.

(1) Equations for flow and drawdown produced by a single well supplied by a circular source are given in figures IV-10 through IV-12. It is apparent from figure IV-11 that considerable computation is required to determine the height of the phreatic surface and resulting drawdown in the immediate vicinity of a gravity well (r/h less than 0.3). The drawdown in this zone usually is not of special interest in dewatering systems and seldom needs to be computed. However, it is always necessary to compute the water level in the well for selection and design of the pumping equipment.

(2) The general equations for flow and drawdown produced by pumping a group of wells supplied by a circular source are given in figure IV-13. These equations are based on the fact that drawdown at any point is the summation of drawdowns produced at that point by each well in the system [31, 34]. The drawdown factors, F , to be substituted into the general equations in figure IV-13 appear in the equations for both artesian and gravity flow conditions. Consequently, the factors given in figure IV-14 for commonly used well arrays are applicable for either condition.

(3) Flow and drawdown for circular well arrays can also be computed, in a relatively simple manner, by first considering the well system to be a slot, as shown in figure IV-15 or IV-16. However, the piezometric head in the vicinity of the wells (or wellpoints) will not correspond exactly to that determined for the slot due to conveyance of flow to the wells. As discussed by

Engelund [32], the piezometric head in the vicinity of the well is a function of (a) well flow, Q_w ; (b) well spacing, a ; (c) well penetration, W ; (d) effective well radius, r_w ; (e) aquifer thickness, d , or gravity head, H ; and (f) aquifer permeability, k . The equations given in figures IV-15 and IV-16 consider these variables.

b. Flow to Wells from a Line Source.

(1) Equations given in figures IV-17 through IV-19 for flow and drawdown produced by pumping a single well or group of fully penetrating wells supplied from an infinite line source were developed using the method of image wells. The image well (a recharge well) is located as the mirror image of the real well with respect to the line source, and supplies the pervious stratum with the same quantity of water as that being pumped from the real well.

(2) The equations given in figures IV-18 and IV-19 for multiple-well systems supplied by a circular source are based on the fact that the drawdown at any point is the summation of the drawdown produced at the point by each well in the system. Consequently, the drawdown at a point is the sum of the drawdown produced by the real wells and the negative drawdown produced by the image or recharge wells.

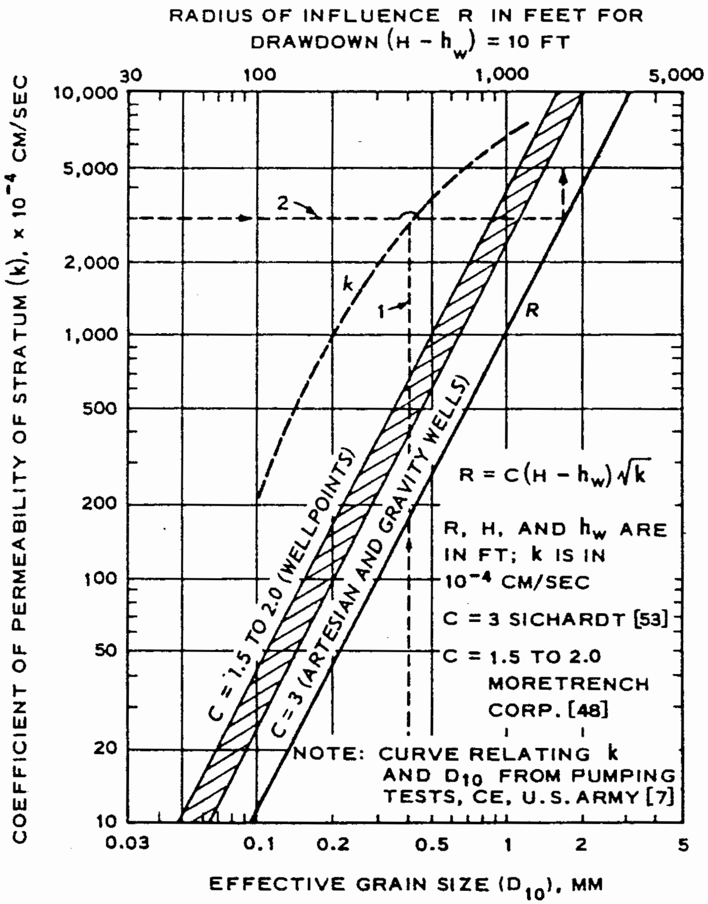
(3) Equations are given in figures IV-20 through IV-22 for flow and drawdown produced by pumping an infinite line at wells supplied by a line source. The equations are based on the equivalent slot assumption. As noted in figure IV-17, the source is to be considered circular when the radius of influence, R (fig. IV-23), of the real well or wells is less than twice the distance between the source and well ($2L \geqq R$).

4. RADIUS OF INFLUENCE R . Equations for flow to drainage systems from a circular seepage source are based on the assumption that the system is centered on an island of radius R . Generally R is the radius of influence which is defined as the radius of a circle beyond which pumping of a dewatering system has no significant effect on the original groundwater level or piezometric surface. The value of R can be estimated from the equation and plots in figure IV-23. Where there is little or no recharge to an aquifer, the radius of influence will become greater with pumping time and with increased drawdown in the area being dewatered. Generally R is greater for coarse, vary pervious sands than for finer soils. If the value of R is large relative to the size of the excavation, a reasonably good approximation of R will serve adequately for design because flow and drawdown for such a condition are not especially sensitive to the actual value of R . As it is usually impossible to determine R accurately, the value should be selected conservatively from pumping test data or, if necessary, from figure IV-23.

5. HYDRAULIC HEAD LOSS H_w

a. The equations in figures IV-1 through IV-22 do not consider hydraulic head losses that occur in the filter, screen, collector pipes, etc. These losses cannot be neglected, however, and must be accounted for separately. The hydraulic head loss in a well and wellpoint system can be estimated from figures IV-24 and IV-25, respectively.

b. Well screen and filter entrance losses, H_e, for designed and installed wells are generally small and can be estimated from figure IV-24a. Figure IV-24a was developed from data from a field pumping test of a 16-in.-diameter well with a 100-sq-in. screen of 5/32-in. slots and a 6-in.-thick filter. Entrance losses through other types of screens are discussed by Peterson, Rohwer, and Albertson [22]. Head losses in the screened section of well, H_s, are calculated from figure IV-24b. This head loss is based on equal inflow per unit of screen surface and turbulent flow inside the well, and is equivalent to the entire well flow passing through one-half the screen length. Other head losses can be determined directly from figure IV-24. Hydraulic head loss within a wellpoint system can be estimated from figure IV-25. Figure IV-26 gives the equivalent length of straight pipe for various fittings for use in computing head loss in the fittings.



1. R DETERMINED WHEN ONLY D_{10} IS KNOWN.
2. R DETERMINED WHEN k IS KNOWN.

RADIUS OF INFLUENCE, R, CAN BE ESTIMATED FOR BOTH ARTESIAN AND GRAVITY FLOWS BY

$$R = C\,(H - h_w)\,\sqrt{k} \qquad \text{(IV-89)}$$

WHERE R, H, AND h_w ARE DEFINED PREVIOUSLY AND EXPRESSED IN FEET. COEFFICIENT OF PERMEABILITY, k, IS EXPRESSED IN 10^{-4} CM/SEC.

AND C = 3 FOR ARTESIAN AND GRAVITY FLOWS TO A WELL.

C = 1.5 TO 2.0 FOR A SINGLE LINE OF WELLPOINTS.

THE VALUE OF R FOR $(H - h_w) = 10$ FT CAN BE DETERMINED FROM THE PLOT HEREIN WHEN EITHER THE D_{10} SIZE OR PERMEABILITY OF THE MATERIAL IS KNOWN. THE VALUE OF R WHEN $(H - h_w) \neq 10$ CAN BE DETERMINED BY MULTIPLYING THE R VALUE OBTAINED FROM THE PLOT BY THE RATIO OF THE ACTUAL VALUE OF $(H - h_w)$ TO 10 FT.

A DISCUSSION ON THE DETERMINATION OF R FROM EQ IV-89 AND PUMPING TESTS IS CONTAINED IN SECTION 3 OF THE TEXT.

(Courtesy of McGraw-Hill Book Co.)

Figure IV-23. Determination of the radius of influence R [modified from ref 45]

Construction Dewatering

A GUIDE TO THEORY AND PRACTICE

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Vice President-Engineering
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Contents

excavation of length x. The wells are staggered at a distance r_s from the center of the trench. The northward and southward flow from the line sources at distance L can be approximated from the trench equation 6.9. However, equation 6.9 assumes a drainage trench of infinite length. Since the length of the actual system is finite, the end effects must be considered. This can be done by assuming that at each end of the system, there is a flow equal to one half the flow to a circular well of radius r_s. The total flow to the system may be approximated by adding equations 6.3 and 6.9.

$$Q = \frac{\pi K \,(H^2 - h^2)}{\ln R_0/r_w} + 2\left(\frac{xK\,(H^2 - h^2)}{2L}\right) \tag{6.12}$$

While the total Q from this model is usually a reliable approximation, it is obvious that the wells at the ends will pump more than those in the center, if spacing is constant. In practice, such systems are leapfrogged as the trench excavation continuously progresses, so a given well will at times be anywhere in the system. It is good practice therefore to design each well and its pump for the high capacity it will yield when near the end of the system.

6.6 *Radius of Influence* R_0

The equivalent radius of influence R_0 that appears in equations 6.1–6.5 is a mathematical convenience. As discussed in Section 5.3, the sum of the recharge to the aquifer is assumed to create an effect similar to that of a constant source on a vertical cylindrical surface at R_0. Thus the concept is to a degree nebulous. Because R_0 appears as a log function in equations 6.1–6.4, precision in estimating it is not necessary. However, the author has seen R_0 vary from 100 to 100,000 ft (30 to 30,000 m) on various projects. The literature cites instances of even greater magnitude. So the possibility of gross error exists.

The most reliable means of estimating R_0 is by Jacob analysis of a pumping test, as described in Chapter 8. Only this method will reveal recharge from other aquifers, and the degree of connection with surface water bodies. It is necessary also to extrapolate from the conditions existing during the pumping test to others that may occur within the life of the dewatering system. We have seen the Q of a dewatering system increase by 20, 40, or even 100% during high river stages, particularly when accompanied by inundation of large surface areas (Section 5.3).

Lacking a pumping test, it is necessary to make rough approximations of R_0 from topography and areal geology, or from estimated aquifer parameters. In an ideal aquifer, without recharge, R_0 is a function of the transmissibility, the storage coefficient and the duration of pumping. By adapting the Jacob formula (equation 4.5), we can estimate the order of magnitude of R_0, without recharge as follows:

$$R_0 = r_w + \sqrt{\frac{Tt}{C_4 C_s}} \tag{6.13}$$

Units to be used in this equation are given in Table 4.2. The value for pumping time t is selected from schedule or cost considerations regarding the time available to accomplish the result.

The value computed for R_0 by equation 6.13 should be adjusted downward on the basis of judgments as to possible recharge. Equation 6.13 is valid only for confined aquifers, but results obtained for water table aquifers are reasonable, provided the drawdown $H - h$ is not a large percentage of the original saturated thickness H. It is apparent from equation 6.11 that R_0 computed for a typical confined aquifer ($C_s = 0.001$) will be some 14 times greater than that in a typical water table aquifer ($C_s = 0.2$), with the same transmissibility, pumped for the same time. Experience confirms that very large values for R_0 are typical of confined aquifers.

An empirical relationship developed by Sichart (43) gives R_0 as a function of drawdown $H - h$ and K:

$$R_0 = 3\,(H - h)\,\sqrt{K} \tag{6.14}$$

where $H - h$ is in feet and K is in microns per second. Theoretically R_0 is independent of drawdown, and is related to pumping time, which does not appear in the Sichart relationship. Nevertheless, the formula has produced reasonable values in some situations.

In many problems, the source of water is conveniently approximated by a vertical line source at distance L from the center of the system, rather than the vertical cylindrical source at R_0. A line source will produce the same flow to a well as a circular source at twice the distance. For use in equilibrium equations 6.1 and 6.3,

$$R_0 = 2L \tag{6.15}$$

Chapter 9 discusses estimates of the distance L to a line source.

6.7 *Permeability* K *and Transmissibility* T

The equilibrium formulas assume an isotropic homogeneous aquifer. When transmissibility T is determined by Jacob analysis of a pump test, it is an *equivalent isotropic transmissibility* T_i, or the transmissibility of an isotropic therefore on the net Q_w. Sichart has suggested that r_w should be such that the radial velocity at the cylindrical surface of the well bore does not exceed a critical value, related to the permeability.

Permeability K

It is evident that Q_w is a function of the permeability K of the sands which the well contacts. If the filter pack made perfectly unobstructed contact with the natural sand, it is possible that Q_w could approach a value such that the gradient at the contact is theoretically almost unity, Terzaghi's critical gradient. This concept can be written in terms of D'Arcy's law:

$$\frac{Q_w}{l_w} < 2\pi\, r_w\, K \tag{6.23}$$

or

$$\frac{Q_w}{A} < K \tag{6.24}$$

where A is the cylindrical surface of the well bore. Theoretically, if this value of Q_w/A were exceeded, the well would be subject to sand packing or piping. In an actual well, however, perfect contact between filter and aquifer cannot be achieved, and if equation 6.24 were used to predict Q_w/A, unrealistically high values would be indicated.

Sichart's empirical relationship (43) is useful in predicting Q_w. He suggests that a practical value of Q_w/A is a function of the square root of permeability. It can be expressed as follows:

$$Q_w = 0.035\, l_w r_w \sqrt{K} \qquad \text{(U.S.)} \tag{6.25}$$

where Q_w is in gallons per minute, l_w in feet, r_w in inches, and K in gallons per day per square foot.

$$Q_w = 0.0247\, l_w\, r_w \sqrt{K} \qquad \text{(metric)} \tag{6.26}$$

where Q is in l/min, l_w in meters, r_w in millimeters, and K in microns per second.

The Sichart relationship has given conservative values for predicting Q_w in wells that have been constructed and completed in accordance with good practice, as discussed in Chapter 16. Other formulas have been suggested. Minster (34) states that in the Soviet Union Q_w/A is predicted as a multiple of the cube root of permeability.

Normally, r_w is selected on the basis of drilling method, difficulty in penetration, type of wellscreen available and other factors. The radius ranges from 4 in. (100 mm) for wells constructed by jetting, or small rotary drills, up to 21 in. (525 mm) for wells constructed by bucket augers or reverse circulation drilling.

One procedure of predicting Q_w for the purposes of preliminary design is as follows:

1. r_w is selected at a reasonable value based on drilling method and difficulty.
2. A value of Q_w/l_w is estimated from equation 6.25, or read from the curves of Fig. 6.15.
3. A value of Q_w is assumed, and the necessary length of wetted screen for this Q_w is calculated.
4. An analysis is made of the available l_w under the predicted job conditions to check the assumed Q_w.

   (a) In a confined aquifer, l_w can be assumed equal to the thickness B, unless it is desired to use partial penetration, either to reduce the total flow, the cost of drilling or for some other reason.

   (b) In a water table analysis, an approximate estimate of l_w in the dewatered condition can be made using a plot of the type in Fig. 6.4. Knowing K, H, and R_0, and with the assumed Q_w, a value of $H^2 - h^2$ at the well can be estimated by cumulation, and l_w calculated. For more accurate work the Borelli correction should be used.

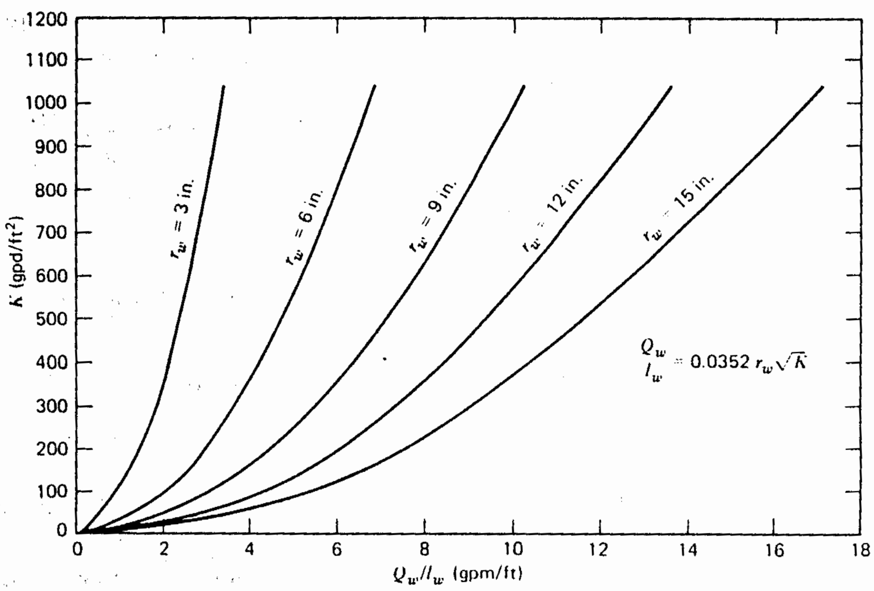

Fig. 6.15 Sichart plot of Q_w/l_w versus K.

A precautionary note is in order. Since Q_w is critical to the design, and the cost of executing the dewatering program, appropriate safety factors should be used. The most reliable method of predicting Q_w is to conduct a step drawdown test during the pumping test prior to design (Chapter 8). An estimated Q_w in the dewatered condition can be extrapolated from the results of the step drawdown test.

6.14 Flow Net Analysis

For aquifer situations which are of irregular geometry, the simple mathematical models described previously are suitable for only rough approximations. For more precise analysis, the flow net method has been used effectively. The construction of flow nets and the use of the method in dewatering analysis has been discussed in detail by Cedergren (16) and Mansur & Kaufman (32).

Figure 6.16 shows a plan flow net of a rectangular system of wells to dewater a trench excavation for the circulating water lines for a power house. Because the ratio of length to width of the rectangular system of wells is large, and because the distance L to the line source is small, the use of a simplified mathematical model would result in serious error. Because the source is close, the cumulative drawdown method is unsuitable, since it

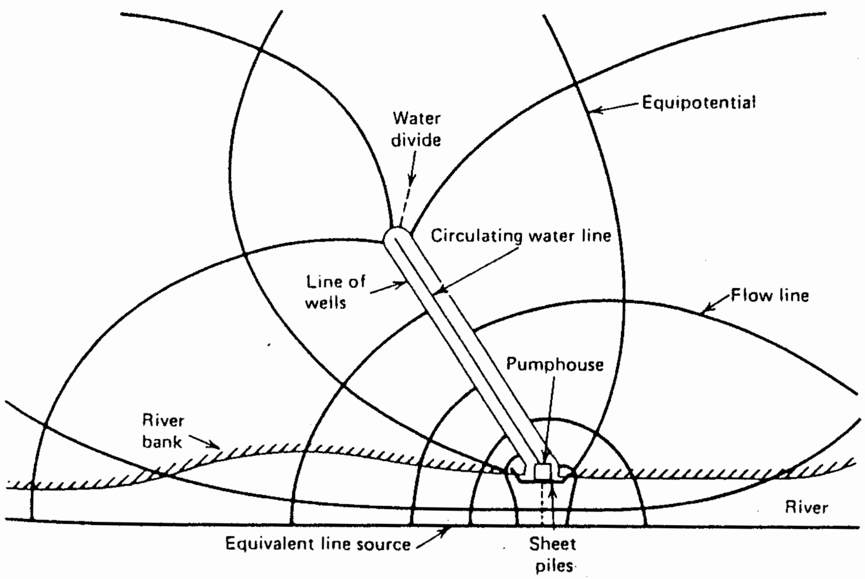

Fig. 6.16 Plan flow net analysis.

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	Steel—Schedu	
Discharge (gpm)	v (ft/sec)	$v^2/2g$ (ft)
¾ Inch Nominal		
1.0	0.602	0.00563
1.5	0.903	0.0127
2.0	1.20	0.0225
2.5	1.50	0.0352
3.0	1.81	0.0506
3.5	2.11	0.0689
4.0	2.41	0.0900
4.5	2.71	0.114
5.0	3.01	0.141
6.0	3.61	0.203
7.0	4.21	0.276
8.0	4.81	0.360
9.0	5.42	0.456
10	6.02	0.563
11	6.62	0.681

APPENDIX 4

SAMPLE CALCULATIONS

SAMPLE CALCULATIONS

UPPER FLOW ZONE

A. Calculation of In Situ Field Permeability, K

1. Using the pump test drawdown values measured after the pump was shut off, the permeability may be calculated with:

$$K_h = \frac{d^2 \ln\left(\frac{2mL}{D}\right)}{8\ L\ (t_2 - t_1)} \ln \frac{H_1}{H_2} \qquad \text{(Hvorslev,1951)*}$$

* Case G, well point-filter in uniform sand, for variable head tests with the condition m L/D > 4 (see Figure 1).

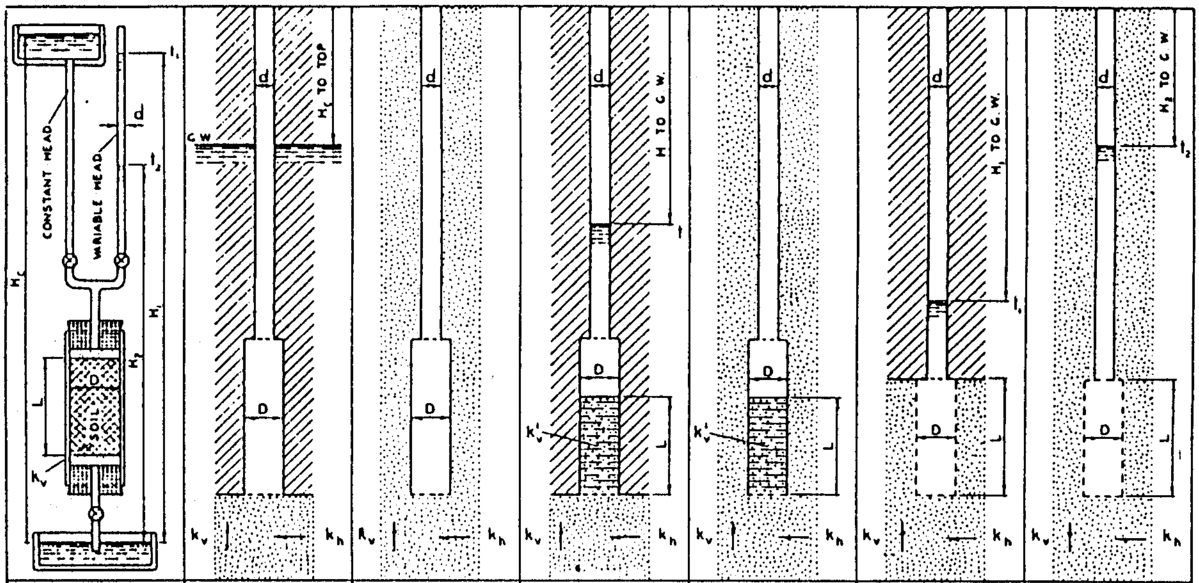
WHERE :
K_h = Horizontal Coefficient of Permeability
K_v = Vertical Coefficient of Permeability
m = Transformation Ratio = $\sqrt{K_h / K_v}$
d = Diameter, standpipe
D = Diameter, intake pipe
L = Length of intake
t = time
H_1 = Drawdown at time t_1
H_2 = Drawdown at time t_2

2. An example of the calculations for Recovery Well PW-1 follows: Using these parameters for Recovery Well PW-1,

m = 3 (approximated)
d = 25.4 cm
D = 25.4 cm
L = 304.8 cm
mL/D = 36 > 4.

Hvorslev's equation reduces to

$$K_h = \frac{1.13}{t_2 - t_1} \ln (H_1/H_2)$$



A	B	C	D	E	F	G
LABORATORY PERMEAMETER (CONSOLIDOMETER)	FLUSH BOTTOM AT IMPERVIOUS BOUNDARY	FLUSH BOTTOM IN UNIFORM SOIL	SOIL IN CASING AT IMPERVIOUS BOUNDARY	SOIL IN CASING IN UNIFORM SOIL	WELL POINT-FILTER AT IMPERVIOUS BOUNDARY	WELL POINT-FILTER IN UNIFORM SOIL

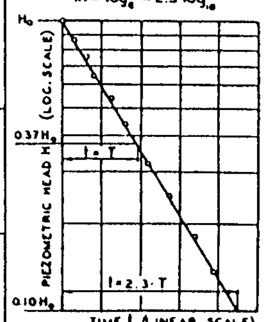
CASE	CONSTANT HEAD	VARIABLE HEAD	BASIC TIME LAG
A	$k_v = \frac{4 \cdot q \cdot L}{\pi \cdot D^2 \cdot H_c}$	$k_v = \frac{d^2 \cdot L}{D^2 \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_v = \frac{L}{t_2 - t_1} \ln \frac{H_1}{H_2}$ FOR $d = D$	$k_v = \frac{d^2 \cdot L}{D^2 \cdot T}$ $k_v = \frac{L}{T}$ FOR $d = D$
B	$k_m = \frac{q}{2\, D \cdot H_c}$	$k_m = \frac{\pi \cdot d^2}{8 \cdot D \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_m = \frac{\pi \cdot D}{8 \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ FOR $d = D$	$k_m = \frac{\pi d^2}{8 \cdot D \cdot T}$ $k_m = \frac{\pi \cdot D}{8 \cdot T}$ FOR $d = D$
C	$k_m = \frac{q}{2.75 \cdot D \cdot H_c}$	$k_m = \frac{\pi \cdot d^2}{11 \cdot D \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_m = \frac{\pi \cdot D}{11 \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ FOR $d = D$	$k_m = \frac{\pi \cdot d^2}{11 \cdot D \cdot T}$ $k_m = \frac{\pi \cdot D}{11 \cdot T}$ FOR $d = D$
D	$k_v' = \frac{4 \cdot q \cdot \left[\frac{\pi}{8} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m} + L\right]}{\pi \cdot D^2\, H_c}$	$k_v' = \frac{d^2 \cdot \left[\frac{\pi}{8} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m} + L\right]}{D^2 \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_v = \frac{\frac{\pi}{8} \cdot \frac{D}{m} + L}{t_2 - t_1} \ln \frac{H_1}{H_2}$ FOR $k_v' = k_v$, $d = D$	$k_v' = \frac{d^2 \cdot \left[\frac{\pi}{8} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m}\right] + L}{D^2 \cdot T}$ $k_v = \frac{\frac{\pi}{8} \cdot \frac{D}{m} + L}{T}$ FOR $k_v' = k_v$, $d = D$
E	$k_v' = \frac{4 \cdot q \cdot \left[\frac{\pi}{11} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m} + L\right]}{\pi \cdot D^2 \cdot H_c}$	$k_v' = \frac{d^2 \left[\frac{\pi}{11} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m} + L\right]}{D^2 \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_v = \frac{\frac{\pi}{11} \cdot \frac{D}{m} + L}{t_2 - t_1} \ln \frac{H_1}{H_2}$ FOR $k_v' = k_v$, $d = D$	$k_v' = \frac{d^2 \cdot \left[\frac{\pi}{11} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m} + L\right]}{D^2 \cdot T}$ $k_v' = \frac{\frac{\pi}{11} \cdot \frac{D}{m} + L}{T}$ FOR $k_v' = k_v$, $d = D$
F	$k_h = \frac{q \cdot \ln\left[\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right]}{2 \cdot \pi \cdot L \cdot H_c}$	$k_h = \frac{d^2 \cdot \ln\left[\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right]}{8 \cdot L \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_h = \frac{d^2 \cdot \ln\left(\frac{4mL}{D}\right)}{8 \cdot L \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ FOR $\frac{2mL}{D} > 4$	$k_h = \frac{d^2 \cdot \ln\left[\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right]}{8 \cdot L \cdot T}$ $k_h = \frac{d^2 \cdot \ln\left(\frac{4mL}{D}\right)}{8 \cdot L \cdot T}$ FOR $\frac{2mL}{D} > 4$
G	$k_h = \frac{q \cdot \ln\left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{2\, \pi \cdot L \cdot H_c}$	$k_h = \frac{d^2 \cdot \ln\left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{8 \cdot L \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_h = \frac{d^2 \cdot \ln\left(\frac{2mL}{D}\right)}{8 \cdot L \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ FOR $\frac{mL}{D} > 4$	$k_h = \frac{d^2 \cdot \ln\left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{8 \cdot L \cdot T}$ $k_h = \frac{d^2 \cdot \ln\left(\frac{2mL}{D}\right)}{8 \cdot L \cdot T}$ FOR $\frac{mL}{D} > 4$

NOTATION

D = DIAM. INTAKE, SAMPLE, CM
d = DIAMETER, STANDPIPE, CM
L = LENGTH, INTAKE, SAMPLE, CM
H_c = CONSTANT PIEZ. HEAD, CM
H_1 = PIEZ. HEAD FOR $t = t_1$, CM
H_2 = PIEZ. HEAD FOR $t = t_2$, CM
q = FLOW OF WATER, CM³/SEC.
t = TIME, SEC.
T = BASIC TIME LAG, SEC.
k_v' = VERT. PERM. CASING, CM/SEC.
k_v = VERT. PERM. GROUND, CM/SEC.
k_h = HORZ. PERM. GROUND, CM/SEC.
k_m = MEAN COEFF. PERM., CM/SEC.
m = TRANSFORMATION RATIO

$k_m = \sqrt{k_h \cdot k_v} \qquad m = \sqrt{k_h / k_v}$

$\ln = \log_e = 2.3 \log_{10}$



DETERMINATION BASIC TIME LAG T

ASSUMPTIONS

SOIL AT INTAKE, INFINITE DEPTH AND DIRECTIONAL ISOTROPY (k_v AND k_h CONSTANT) - NO DISTURBANCE, SEGREGATION, SWELLING OR CONSOLIDATION OF SOIL - NO SEDIMENTATION OR LEAKAGE - NO AIR OR GAS IN SOIL, WELL POINT, OR PIPE - HYDRAULIC LOSSES IN PIPES, WELL POINT OR FILTER NEGLIGIBLE

Formulas for determination of permeability

Figure 1.

The permeability for various values of H_1, H_2, t_1, t_2 was calculated, then averaged for a reported value as shown in Table 1 below.

Table 1

H_1 (ft)	t_1 (sec)	H_2 (ft)	t_2 (sec)	$\ln(H_1/H_2)$	t_2-t_1(sec)	K_h(cm/sec)
1.98	0	1.90	120	0.041	120	3.86×10^{-4}
1.94	45	1.86	210	0.042	165	2.88×10^{-4}
1.88	150	1.73	360	0.083	210	4.47×10^{-4}
1.80	300	1.66	600	0.081	300	3.05×10^{-4}
1.63	720	1.50	1080	0.083	360	2.61×10^{-4}
1.50	1080	0.93	3000	0.478	1920	2.81×10^{-4}
0.93	3000	0.42	6000	0.795	3000	2.99×10^{-4}

average K_h: 3.24×10^{-4} cm/sec

Table 2 provides a summary of field permeabilities for all eight Recovery Wells and MW-16.

Table 2

Well No.	In situ field permeabilities (cm/sec)
PW-1	3.24×10^{-4}
MW-16	2.39×10^{-4}
MW-18	3.46×10^{-4}
MW-23	2.53×10^{-3}
MW-24	4.36×10^{-4}
MW-25	4.50×10^{-4}
MW-26	3.56×10^{-4}
MW-27	2.90×10^{-3}
MW-28	2.91×10^{-5}

B. Calculation of Radius of Influence, r_o

1. Using the permeabilities calculated with Hvorslev's equation and a well drawdown equal to the upper flow zone saturated thickness, the Radius of Influence (r_o) at each well location may be calculated with the following equation:

$$r_o = C\,(H - h_w)\,\sqrt{K}$$

(Sichardt's method, U.S. Department of the Army, 1971)

WHERE:
- r_o = Radius of Influence, ft
- C = Empirical Relation of K vs. r
- H = Height of water table (saturated thickness), ft
- h_w = Head of water in well, ft
- K = Coefficient of Permeability, microns/sec

2. An example of the calculations for Recovery Well PW-1 follows:

$$C = 3 \text{ (for a single well)}$$
$$K_h = 3.24 \times 10^{-4} \text{ cm/sec} = 3.24 \text{ microns/sec}$$
$$H - h_w = 10 \text{ ft}$$
$$r_o = 3\,(10 \text{ ft})\,(\sqrt{3.24})$$
$$\mathbf{r_o = 54 \text{ ft}}$$

Table 3 provides a summary of calculated Radii of Influence for all eight Recovery Wells and MW-16.

Table 3

Well No.	Calculated Radius of Influence (ft)
PW-1	54
MW-16	46
MW-18	56
MW-23	136
MW-24	63
MW-25	93
MW-26	57
MW-27	162
MW-28	35

C. <u>Calculation of Transmissivity, T</u>

1. Using the permeability values calculated with Hvorslev's equation and an upper flow zone saturated thickness of 10 feet, the Transmissivity, T, for each well location may be calculated with the following equation:

$$T = k\,b$$

WHERE:

T = Transmissivity
k = Permeability
b = saturated thickness

2. An example of the calculations for Recovery Well PW-1 follows:

$k = 3.24\text{x}10^{-4}$ cm/sec = 0.28 m/day
b = 10 ft = 3.05 m

T = (0.28 m/day) (3.05 m) (80.5 gal/day/ft per m^2/day)
T = 68.7 gal/day/ft

Table 4 provides a summary of Transmissivity values for all eight Recovery Wells and MW-16.

Table 4

Well No.	Transmissivity, T (gal/day/ft)
PW-1	68.7
MW-16	50.7
MW-18	73.7
MW-23	536.4
MW-24	92.5
MW-25	95.5
MW-26	75.5
MW-27	615.0
MW-28	6.2

D. <u>Calculation of Storage Coefficient, S</u>

1. Using the Transmissivity, T, and Radius of Influence, r_o, values previously calculated, as well as the elapsed time from pump test start to finish, the Storage Coefficient, S, for each well location may be calculated with:

$$S = 2.25\ T \left(\frac{t}{r^2}\right)_0 \qquad \text{(Lohman, 1979)}$$

WHERE:
- S = Storage Coefficient
- T = Transmissivity
- t = time
- r_o = Radius of Influence

2. An example of the calculation for Recovery Well PW-1 follows:

$$T = 68.7 \text{ gal/day/ft} = 0.84 \text{ m}^2\text{/day}$$
$$t = 4332 \text{ min} = 3.0 \text{ days}$$
$$r_o = 54 \text{ ft} = 16.5 \text{ m}$$

$$S = \frac{2.25\ (0.84 \text{ m}^2\text{/day})\ (3.0 \text{ days})}{(16.5 \text{ m})^2}$$

$$\mathbf{S = 0.0205}$$

Table 5 provides a summary of Storage Coefficient values for all eight Recovery Wells and MW-16.

Table 5

Well No.	Storage Coefficient, S
PW-1	0.0205
MW-16	0.0217
MW-18	0.0144
MW-23	0.0261
MW-24	0.0214
MW-25	0.0095
MW-26	0.0207
MW-27	0.0206
MW-28	0.0045

PERMEABILITY CALCULATIONS FOR EACH WELL BASED ON REPORTED VALUES OF H AND t.

PW-1						
H1 (ft)	t1 (sec)	H2 (ft)	t2 (sec)	ln(H1/H2)	t2-t1 (sec)	Kh(cm/sec)
1.98	0	1.90	120	0.041	120	3.88E-04
1.94	45	1.86	210	0.042	165	2.88E-04
1.88	150	1.73	360	0.083	210	4.47E-04
1.80	300	1.66	600	0.081	300	3.05E-04
1.63	720	1.50	1080	0.083	360	2.61E-04
1.50	1080	0.93	3000	0.478	1920	2.81E-04
0.93	3000	0.42	6000	0.795	3000	2.99E-04

average Kh: 3.24E-04 cm/sec

MW-16						
H1 (ft)	t1 (sec)	H2 (ft)	t2 (sec)	ln(H1/H2)	t2-t1 (sec)	Kh(cm/sec)
2.38	0	0.83	450	1.053	450	2.58E-04
2.38	0	0.70	504	1.224	504	2.67E-04
0.83	450	0.65	534	0.244	84	3.20E-04
0.70	504	0.42	720	0.511	216	2.60E-04
0.55	600	0.21	1080	0.963	480	2.21E-04
0.24	960	0.03	3000	2.080	2040	1.12E-04

average Kh: 2.40E-04 cm/sec

MW-18						
H1 (ft)	t1 (sec)	H2 (ft)	t2 (sec)	ln(H1/H2)	t2-t1 (sec)	Kh(cm/sec)
5.02	0	4.09	150	0.205	150	3.00E-04
4.90	15	3.57	240	0.317	225	3.10E-04
4.63	60	2.57	420	0.587	360	3.60E-04
3.57	240	1.79	600	0.690	360	4.22E-04
2.89	360	0.20	2100	2.671	1740	3.38E-04

average Kh: 3.46E-04 cm/sec

MW-23						
H1 (ft)	t1 (sec)	H2 (ft)	t2 (sec)	ln(H1/H2)	t2-t1 (sec)	Kh(cm/sec)
2.46	0	1.39	15	0.571	15	4.19E-03
1.39	15	0.68	45	0.715	30	2.62E-03
0.83	30	0.19	90	1.474	60	2.70E-03
0.19	90	0.07	270	0.998	180	6.10E-04

average Kh: 2.53E-03 cm/sec

MW-24						
H1 (ft)	t1 (sec)	H2 (ft)	t2 (sec)	ln(H1/H2)	t2-t1 (sec)	Kh(cm/sec)
3.26	0	0.85	300	1.344	300	4.93E-04
3.26	0	0.57	480	1.744	480	4.00E-04
0.85	300	0.57	480	0.399	180	2.44E-04
0.64	450	0.42	540	0.421	90	5.15E-04
0.48	510	0.32	600	0.405	90	4.96E-04
0.38	570	0.25	660	0.419	90	5.12E-04
0.38	570	0.13	870	1.073	300	3.93E-04

average Kh: 4.36E-04 cm/sec

MW-25						
H1 (ft)	t1 (sec)	H2 (ft)	t2 (sec)	ln(H1/H2)	t2-t1 (sec)	Kh(cm/sec)
1.367	45	0.933	90	0.382	45	9.34E-04
0.933	90	0.615	150	0.417	60	7.64E-04
0.763	120	0.466	240	0.493	120	4.52E-04
0.466	240	0.281	1200	0.506	960	5.80E-05
0.371	420	0.281	1200	0.278	780	3.92E-05

average Kh: 4.49E-04 cm/sec

MW-26						
H1 (ft)	t1 (sec)	H2 (ft)	t2 (sec)	ln(H1/H2)	t2-t1 (sec)	Kh(cm/sec)
2.53	0	1.77	90	0.357	90	4.37E-04
2.38	15	1.39	150	0.538	135	4.38E-04
2.13	45	1.29	180	0.501	135	4.09E-04
1.77	90	1.10	240	0.476	150	3.49E-04
1.39	150	0.53	540	0.964	390	2.72E-04
1.00	270	0.23	960	1.470	690	2.34E-04

average Kh: 3.56E-04 cm/sec

MW-27						
H1 (ft)	t1 (sec)	H2 (ft)	t2 (sec)	ln(H1/H2)	t2-t1 (sec)	Kh(cm/sec)
1.88	0	1.10	15	0.536	15	3.93E-03
1.88	0	0.73	30	0.946	30	3.47E-03
1.10	15	0.73	30	0.41	15	3.01E-03
1.10	15	0.47	45	0.85	30	3.12E-03
0.73	30	0.11	120	1.893	90	2.31E-03
0.47	45	0.07	180	1.904	135	1.55E-03

average Kh: 2.90E-03 cm/sec

MW-28						
H1 (ft)	t1 (sec)	H2 (ft)	t2 (sec)	ln(H1/H2)	t2-t1 (sec)	Kh(cm/sec)
2.15	45	2.08	150	0.033	105	3.47E-05
2.06	180	2.01	270	0.025	90	3.00E-05
1.97	360	1.87	600	0.052	240	2.39E-05
1.77	840	1.33	2100	0.285	1260	2.50E-05
1.26	2400	0.20	9600	1.841	7200	2.81E-05

average Kh: 2.83E-05 cm/sec

RECOVERY WELL CAPTURE ZONE DIMENSIONS

$x_0 = -Q/(2\pi Kbi)$ (Fetter, 1994)

$y_{max} = \pm Q/(2Kbi)$ (Fetter, 1994)

Well No.	K (cm/sec)	Q (gal/hr)	b	AVG GRADIENT, i	x (ft)	y max +/- (ft)
PW-1	3.24E-04	3.7	10	0.00817	-25.19	79.14
MW-18	3.46E-04	10.0	10	0.00911	-57.15	179.56
MW-23	2.53E-03	21.3	10	0.00651	-23.29	73.16
MW-24	4.36E-04	1.0	10	0.00391	-10.56	33.17
MW-25	4.50E-04	1.8	10	0.00407	-17.69	55.57
MW-26	3.56E-04	2.0	10	0.01032	-9.81	30.82
MW-27	2.90E-03	13.4	10	0.01806	-4.61	14.48
MW-28	2.91E-05	2.9 (7.3)	10	0.00344	-522.02	1,639.96

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RECOVERY WELL CAPTURE ZONE- x DIMENSIONS AS A FUNCTION OF y

$x = \frac{-y}{\tan(2\pi Kbiy/Q)}$ (Fetter, 1994)

y	PW1	MW 18	MW 23	MW 24	MW 25	MW 26	MW 27	MW 28
0.001	-25.1909	-57.1544	-23.2882	-10.5583	-17.6873	-9.81076	-4.61028	-522.017
2	-25.1379	-57.131	-23.2309	-10.4317	-17.6119	-9.67448	-4.31737	-522.014
3	-25.0716	-57.1019	-23.1592	-10.2726	-17.5174	-9.50305	-3.94041	-522.011
4	-24.9788	-57.061	-23.0587	-10.0482	-17.3848	-9.26102	-3.39088	-522.007
5	-24.8592	-57.0085	-22.9292	-9.75695	-17.2136	-8.94627	-2.64298	-522.001
6	-24.7127	-56.9442	-22.7706	-9.39648	-17.0036	-8.55599	-1.65639	-521.994
7	-24.5391	-56.8683	-22.5825	-8.96399	-16.7541	-8.08654	-0.36748	-521.985
8	-24.3382	-56.7806	-22.3648	-8.45593	-16.4644	-7.53335	1.327652	-521.976
9	-24.1098	-56.6812	-22.1171	-7.86793	-16.1338	-6.89077	3.608961	-521.965
10	-23.8535	-56.5699	-21.8389	-7.19468	-15.7613	-6.15182	6.816018	-521.953
11	-23.569	-56.4469	-21.5299	-6.42973	-15.3459	-5.30789	11.67547	-521.939
12	-23.2559	-56.312	-21.1896	-5.56526	-14.8864	-4.34841	20.07764	-521.925
14	-22.5423	-56.0067	-20.4127	-3.49768	-13.8294	-2.02708	132.9728	-521.892
20	-19.6613	-54.8022	-17.2599	6.70449	-9.41666	10.10355		-521.761
25	-16.3211	-53.4619	-13.5697	25.58763	-3.96666	37.0673		-521.618
30	-11.9785	-51.8064	-8.71077	96.90608	3.779829	357.4711		-521.442
33	-8.80688	-50.6573	-5.11471	2051.614	10.02567			-521.321
40	0.68333	-47.5033	5.915122		33.06126			-520.995
55	38.64851	-38.317	55.61055		1717.057			-520.084
65	103.3798	-30.0882	177.8056					-519.316
73	293.5756	-22.0669	10497.81					-518.609
79	14275.89	-15.0765						-518.025
100		18.07833						-515.616
110		40.62953						-514.267
120		70.1137						-512.789
130		110.3326						-511.18
140		168.909						-509.441
150		263.7385						-507.57
160		449.2305						-505.567
170		1007.307						-503.431
179		18409.39						-501.395
200								-496.221
300								-463.241
400								-415.612
500								-351.676
600								-268.957
700								-163.785
800								-30.6378
900								139.0484
1000								359.2043
1100								654.0246
1200								1069.544
1300								1705.633
1400								2827.95
1500								5459.766
1600								20858.61
1639								887500.5