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FIELD TESTING OF EXFILTRATION SYSTEMS

SOUTH FLORIDA WATER MANAGEMENT DISTRICT

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by

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Water Resources Division
Resource Planning Department
South Florida Water Management District

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EXECUTIVE SUMMARY

Water quality research indicates that the initial half inch of stormwater runoff known as the 'first flush' carries the highest concentration of surface pollutants occurring during a storm event. It has been found that most of these pollutants are substantially removed as the stormwater flows over or percolates through unsaturated soil. Exfiltration trenches are commonly used in commercial developments to retain the 'first flush' of stormwater runoff. The retained water is allowed to percolate through the trench gravel into the surrounding soil as a means of improving the quality of the water.

The linear dimensions of the trench are determined by the volume of water to be treated by retention and percolation. A procedure for design of exfiltration trenches is recommended by the South Florida Water Management District (District). This trench design methodology was developed using an entirely theoretical approach which, until this study, had not been tested in the field. The purpose of this study was to evaluate the ability of exfiltration trenches, designed according to recommended procedures, to treat the required volume of stormwater and to improve upon the design procedures if necessary.

Two test exfiltration trenches were constructed at the District headquarters in West Palm Beach. A number of field tests were conducted to measure the average exfiltration rates from the trenches during the design period (one hour). The measured exfiltration rates were compared with computed exfiltration rates obtained using the recommended design method. The computed exfiltration rates were found to be greater than observed rates by 32%-158%.

The measured exfiltration rates were also compared with computed rates obtained using an alternate design procedure. The alternate method was found to

give fairly accurate estimates of trench exfiltration rates (within 13% of measured rates).

The alternate method can be extended to predict the impact of exfiltration from trenches on groundwater elevations close to the trench (mounding). This can be very useful in determining the most efficient layout for trenches. The available data is not sufficient to determine the accuracy of the mounding predictions of this method and further investigation is recommended.

Field test results indicate that the alternate method gives better predictions of exfiltration rates than the currently recommended procedure. Although further testing is appropriate, it is recommended that the alternate procedure be adopted for future exfiltration trench designs.

ACKNOWLEDGEMENTS

This report is the result of the combined efforts of a number of District employees. The authors would like to acknowledge Nettie Winnograd and Marjorie Coley for doing the typing and retyping of the manuscript and Barbara Brown for doing the graphics. The authors would like to express appreciation to the following persons for their constructive comments and advice: Alan Hall, Jorge Marban, Richard Gregg, Mike Cullum, Ed Yaun, Roberto L. Sanchez, Richard Bouwer, Mary-Jo Shine and Nicole Duplaix.

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ABSTRACT

Exfiltration trenches are used in many commercial developments to retain the 'first flush' of stormwater runoff, for water quality purposes. A field testing program was conducted to evaluate the performance of the exfiltration trench design methodology currently employed by the South Florida Water Management District (District). Comparison between measured trench exfiltration rates and rates predicted by the current design method indicates that this design procedure significantly overestimates the exfiltration capabilities of trenches. An alternate exfiltration trench design procedure was proposed and tested. Field test results indicate close agreement between measured and design exfiltration rates when the alternate method is used. The alternate method is analytical and can be extended to predict changes in the water table elevation (mounding) around an exfiltration trench during operation. Further testing is recommended; however, field tests indicate that the alternate method is superior to the current exfiltration trench design method.

Key Words: Hydraulic Conductivity, Exfiltration Trench, French Drain, Line Source

INTRODUCTION

The initial half inch of stormwater runoff known as the 'first flush' contains the highest concentration of surface pollutants during a storm event. Research indicates that a substantial portion of these pollutants are removed as the stormwater flows over or through unsaturated soil. Exfiltration systems are employed throughout the District as a means of removing the 'first flush' of stormwater from surface runoff. A typical trench system comprises an inlet structure which leads to a horizontal perforated pipe, surrounded by gravel. Designs generally include a weir structure at the outfall end which retains the stormwater and surcharges the perforated pipe and trench to induce exfiltration into the soil.

The volume of water to be treated by retention and exfiltration under the District water quality criteria is 50% of the greater of the first inch of runoff from the total project or 2.5 inches of runoff from the project's impervious areas. The trench systems are designed such that the volume of storage and exfiltration for one hour is equal to the design volume. The recommended procedure for design of exfiltration trenches is described in <u>Permit Information Manual</u>, Volume IV (1984) (referred to as Volume IV).

The trench design methodology described in the manual (referred to as Volume IV Methodology) is based on a purely theoretical approach which, until this study, had not been tested in the field.

The purpose of this study was to evaluate the ability of exfiltration systems to function as designed under the Volume IV Methodology and to improve upon the design procedures if necessary.

SCOPE

The Volume IV design procedure can, for convenience, be considered as two independent procedures. The first is the determination of hydraulic conductivity of the soil in which the trench is to operate. The second is the computation of minimum trench dimensions for a given hydraulic conductivity and required trench capacity. Trench capacity refers to the total volume of water which can be stored in and exfiltrated from the trench in one hour.

The scope of the study included the following:

- Field measurement of soil hydraulic conductivity.
- Construction of two exfiltration trenches and measurement of trench capacities under different flow conditions.
- Comparison of measured exfiltration trench capacities with computed values
- Assessment of the Volume IV design procedures and recommendations for future designs

FIELD TESTING

Field tests were conducted to measure soil hydraulic conductivity and exfiltration trench capacities. Testing procedures and results are presented in this section. The test area is located in the northwest portion of the District property in West Palm Beach, approximately 300 feet south of the C-51 canal. Because of the large distance between the canal and the test site and the relatively small volumes of flow involved in the field tests, the impact of the canal on test results was ignored.

Hydraulic Conductivity Tests

Two types of hydraulic conductivity tests are described in Volume IV, the falling head percolation test and the constant head percolation test. In the falling head test, the water level in a borehole is raised above the water table, by adding water, and then allowed to subside. Hydraulic conductivity is computed from the rate of fall of water level in the borehole. In areas of relatively high soil hydraulic conductivities, the rate of fall is so rapid that accurate measurement is difficult. In the constant head test, hydraulic conductivity is computed from Q_p, the rate of inflow of water required to maintain a constant head above the water table in a borehole. Because of the greater likelihood of personal error in the falling head test, only constant head tests were performed.

A total of seven constant head percolation tests were performed in 6" augered cased boreholes. Figure 1 shows the layout of the boreholes.

Procedure

Using a metered water supply, the borehole was filled to the test elevation (usually the top of the ground) indicated in Figure 2. The inflow rate was then adjusted 'as required' to keep the water level in the hole at test elevation. The rate of inflow was recorded for 10 minutes after the flow rate became constant. Q_p is the final, constant flow rate measured during the test.

Figure 1 BOREHOLE LAYOUT, PERCOLATION TESTS

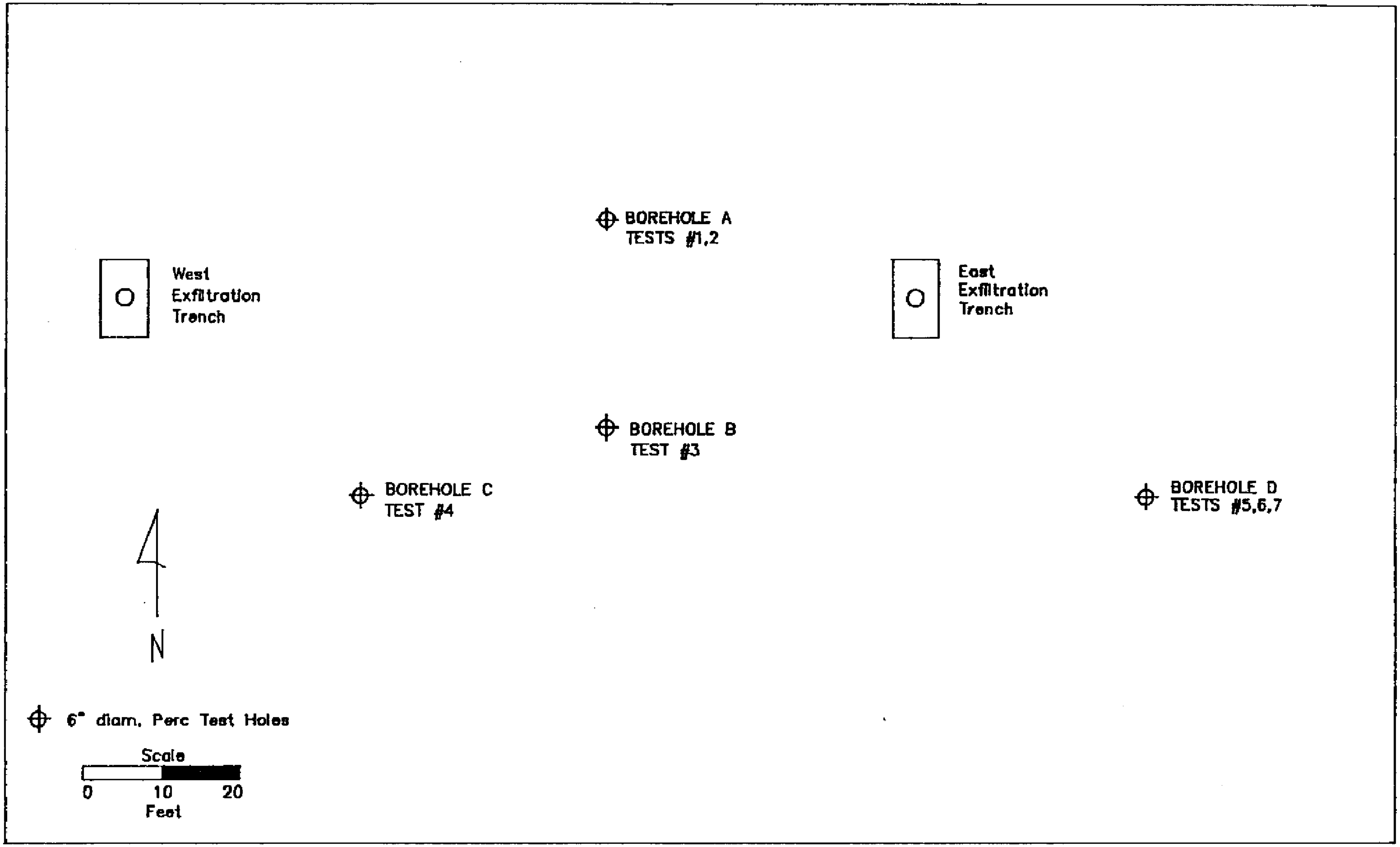


Figure 2 PERCOLATION TEST BOREHOLES

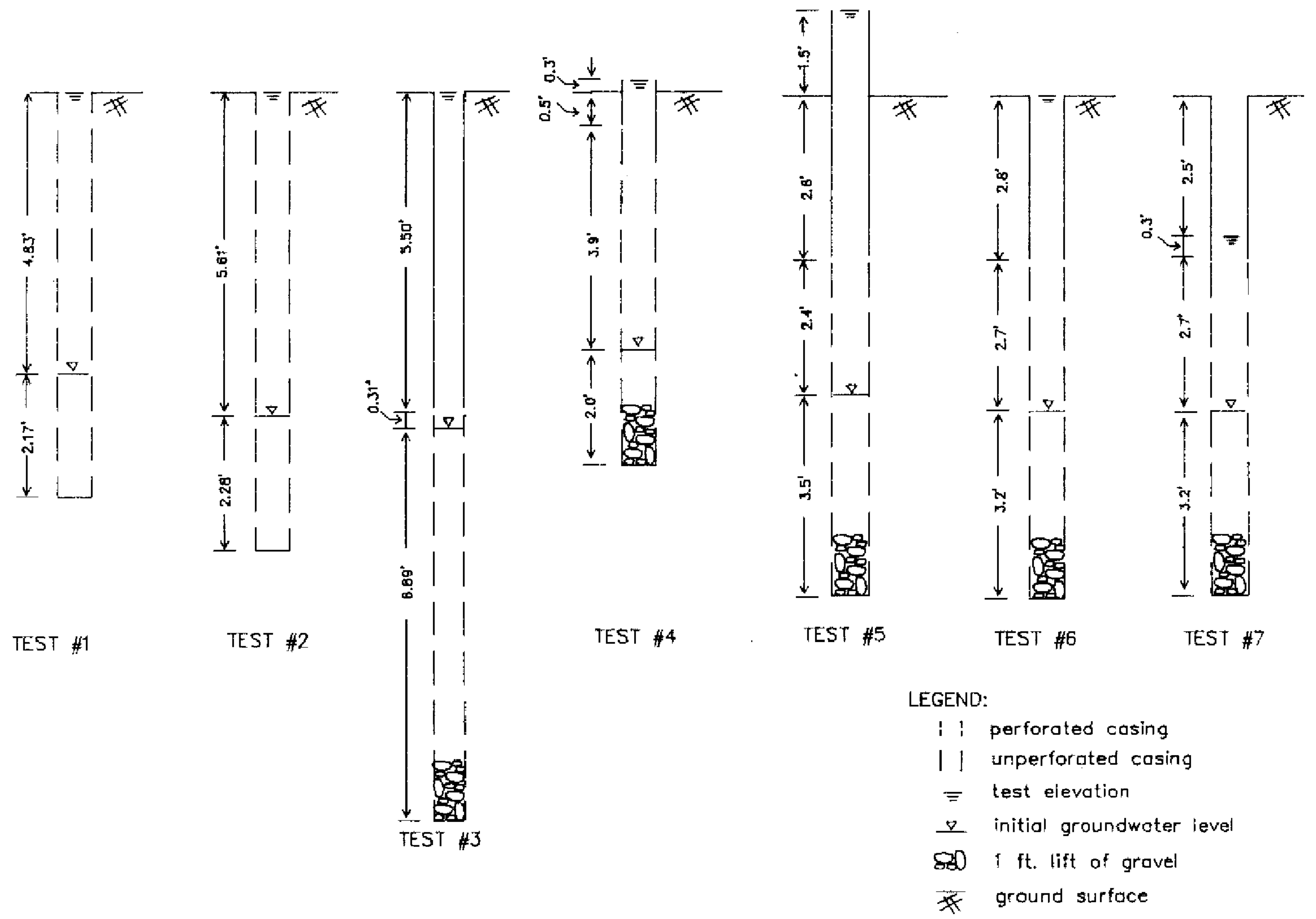


Figure 2 shows the dimensions of the boreholes and casing and the location of the water table at the beginning of each test.

Tests 2 and 3 were performed several months after their respective boreholes were augered. In the interim, the sides of the holes had slumped in and the boreholes were cleaned out by jetting with water prior to the tests.

Results

Results of the hydraulic conductivity tests are presented below (Table 1.)

TABLE 1

Hydraulic Conductivity Test Results

Test#	Borehole	L (ft)	D_s (ft)	H_2(ft)	Q_p (10^{-3}cfs)
1	A	7.0	2.17	4.83	8.2
2	A	7.0	2.28	5.61	42.4
3	B	7.0	6.69	5.81	22.7
4	C	5.9	2.00	4.70	8.9
5	D	5.9	3.50	6.70	6.6
6	D	5.9	3.20	5.50	10.0
7	D	5.9	3.20	3.00	3.6

D_s - length of borehole below the water table
H_2- head in borehole above water table
L - permeable length of borehole
Q_p - steady volume rate of discharge of water into the borehole

The steady rate of inflow of water, Q_p, was much higher in tests 2 and 3 than in the other tests. Test 2 and 3 were conducted in boreholes which were cleaned out by jetting prior to the test. It is likely that when the hole was jetted out, the cross section of the hole and the arrangement of the soil particles were altered, leading to increased percolation rates. The inflow rate in test 7 was much lower than in the other tests. This is probably a result of the lower head maintained in the borehole during test 7.

Analysis

Hydraulic conductivity is defined in terms of Darcy's Law which describes laminar flow through soil. Several formulae exist in the literature which relate Q_p to hydraulic conductivity. One commonly used formula is Thiem's equation (Equation 1) for unconfined aquifers which describes saturated flow to and from a well (Bouwer (1978)).

$$K = \frac{Q_p \log_e (r_e / r_w)}{\pi (h_w^2 - h_e^2)} \qquad (1)$$

Q_p is the steady volume rate of flow required to maintain a constant head in the testhole

h_e is the height of the initial water table elevation above the bottom of the borehole

h_w is the height of water in the borehole during the test

r_w is the radius of the borehole

r_e is the radius of influence of the borehole

This equation is based on steady state Darcian flow in uniform aquifers and assumes that flow is predominantly horizontal.

Two formulae for computing 'hydraulic conductivity' from constant head percolation tests are recommended in Volume IV, the Usual Open Hole formula and the DOT standard test formula. For reasons discussed later on in this report, these formulae are not consistent with Darcy's law and the computed coefficients are therefore not typical soil hydraulic conductivities. They will be referred to as

Volume IV discharge coefficients, K_{IV}. The Usual Open Hole formula (Equation 2) applies to cased or uncased boreholes which allow percolation along the entire length of the hole. Tests 1,2,4 and 7 were Open Hole type tests.

$$K_{IV} = \frac{4Q_p}{\pi d(2H_2^2 + 4H_2D_s + H_2d)} \qquad (2)$$

K_{IV} = Volume IV discharge coefficient (c f s /f t.^{2} - f t. Head)

Q_p = "Stablized" Flow Rate (c f s)

d = Diameter of Test Hole (Feet)

H_2 = Depth from test elevation to Water Table (Feet)

D_s = Depth of borehole below the Water Table (Feet)

The DOT standard test formula (Equation 3) applies to cased boreholes which allow percolation along the lower portion of the borehole only. Tests 3,5 and 6 were DOT type tests.

$$K_{IV} = \frac{4Q_p}{\pi(20.25H_2 - H_2^2 - 9)} \qquad (3)$$

K_{IV}, Q_p, H_2 and D_s are defined above.

Hydraulic conductivities and K_{IV} values were computed using measured values of Q_p (Table 1) and Equations 1-3. These are listed in Table 2. The units of K_{IV} are 10^{-4} ft/sec per ft head. The units of K are 10^{-4} ft/sec.

TABLE 2

Valves of K and K_{IV} Computed from Hydraulic

Conductivity Tests

Test#	Type	K_{IV}	K
1	open	2.3	2.8
2	open	9.2	11.2
3	DOT	3.5	2.8
4	open	2.7	3.3
5	DOT	1.0	1.6
6	DOT	1.8	2.3
7	open	1.6	2.2
Range*		1.7	1.7
Average*		1.9	2.4

. 'open' indicates the Usual Open Hole type test
.'DOT' indicates the DOT standard type test

* values for tests 2 and 3 not included

The K and K_{IV} values computed for tests 2 and 3 are, in general, much higher than those obtained for the other tests. Because of the uncertainty of these results, K and K_{IV} values for tests 2 and 3 were disregarded. Test 2 and 3 results were not included in the computation of the range and average values of K and K_{IV}.

Trench Test

Procedure

Two test exfiltration trenches were constructed together with an array of thirty-four, 2" diameter monitoring wells. See Figure 3 for general layout.

Both trenches were constructed in accordance with the schematic design shown in Figure 4. The trenches were rectangular in cross section and consisted principally of horizontal perforated cylindrical pipe surrounded by coarse gravel. The trenches were lined with filter fabric to prevent clogging of the gravel layer with the finer surrounding soil. The ends of the pipe were capped so that the water exited the pipe through the perforations only. Water entered the systems via 12" diameter vertical inlet pipes connected to the horizontal perforated pipes.

The west trench was constructed with an impervious (plywood) bottom to eliminate trench bottom exfiltration. The eastern trench was constructed without the plywood bottom to permit bottom exfiltration. This was done in order to determine the importance of bottom exfiltration in the trenches.

Monitoring wells were installed generally 10' and 20' from the trench centers. The wells were 13' deep and screened from 3' to 13' below ground surface.

The trenches were tested by discharging metered flow into them via a 3" diameter pipe connected to a fire hydrant. First the trenches were filled at a steady inflow rate which was controlled using a valve in the inlet pipe. Once the design stage was reached, the inflow rate was reduced, as required, to maintain the design stage in the trench. Once full, further discharges into the trench represent exfiltration rates. Discharge rates were measured using a stop watch and flow meter.

Figure 3 GENERAL LAYOUT OF TEST TRENCHES AND OBSERVATION WELLS

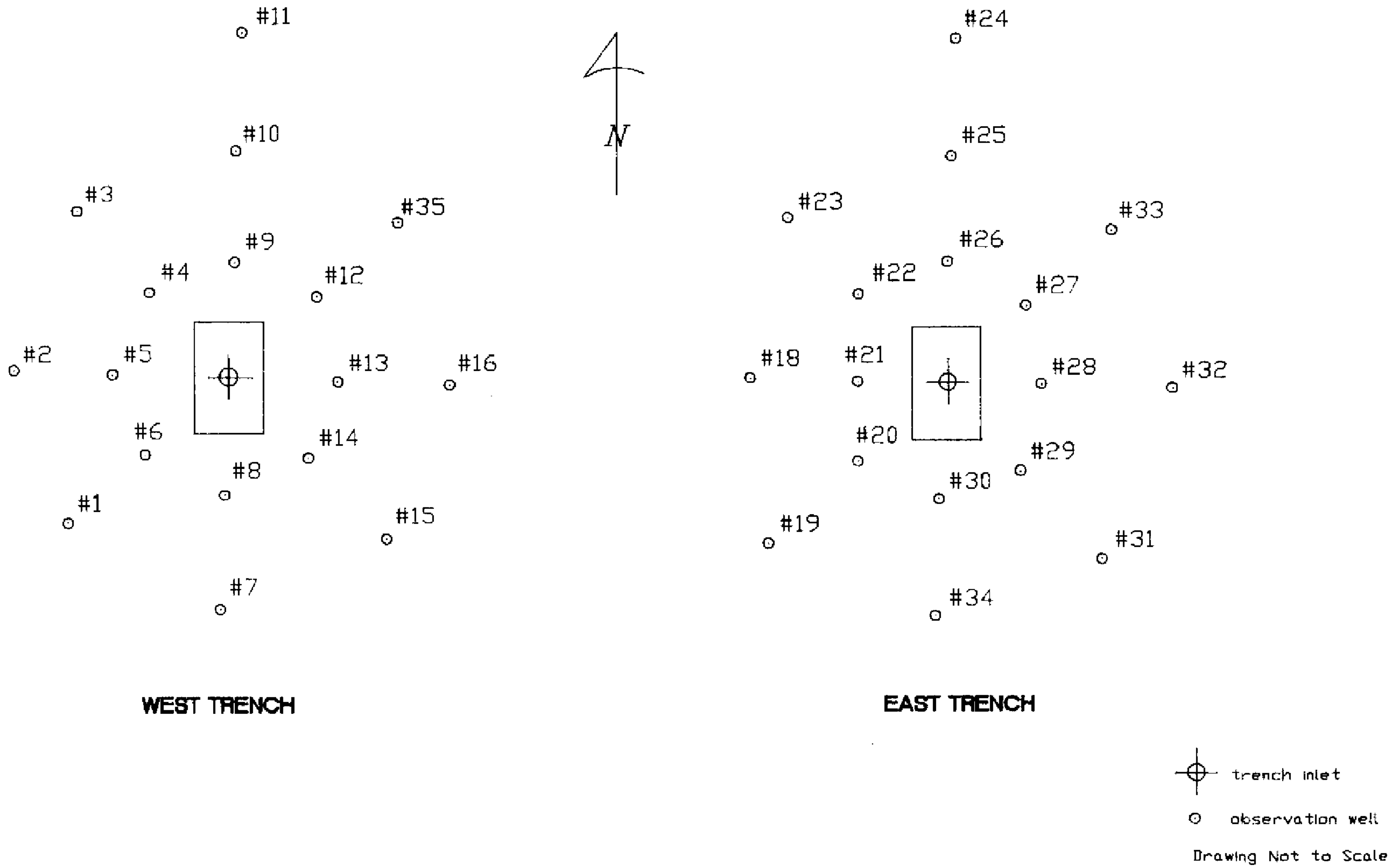
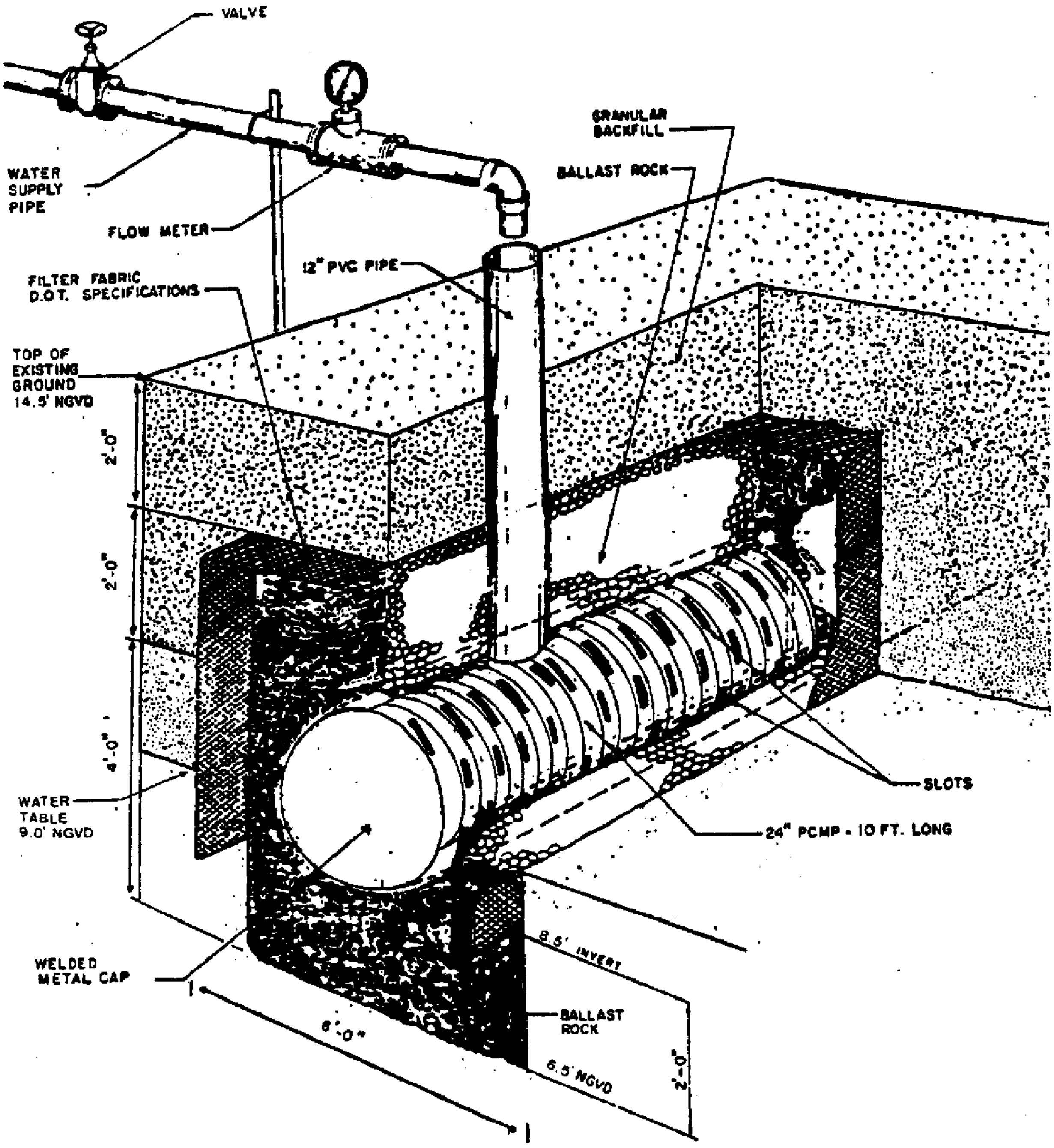


Figure 4 SCHEMATIC DESIGN OF EXFILTRATION TRENCH



TOTAL LENGTH OF TRENCH - 10 FEET

Preliminary tests (numbered 1-4) were run to determine the filling time of trench (the time required to achieve the design stage, given a steady inflow rate) and to estimate the total volume of water the trenches could handle in 1 hour (the design period). The main objectives of the preliminary tests were to check the workability of test procedures, to measure trench exfiltration rates and to determine the need for further testing. The design stage during the preliminary tests was the top of the ground.

Subsequent tests (numbers 5-7) were run with a more sophisticated monitoring system in which groundwater elevations in the wells were recorded throughout the test. Groundwater elevations were recorded using either insitu water level monitors or weighted tapes. Since the trenches exfiltrate best when they are full (i.e.,with the design head), during the final tests, the trenches were filled as quickly as possible. The design stage for tests 5 and 6 was the top of the ground. For test 7 the design stage was the top of the trench.

The primary purpose of these final tests was to provide the necessary information to evaluate current and alternative design procedures.

Results and Discussion

The results of the trench tests are given in Table 3. Initial groundwater levels relative to trench elevations are shown in Figure 5.

Figure 5 INITIAL CONDITIONS, TRENCH TESTS

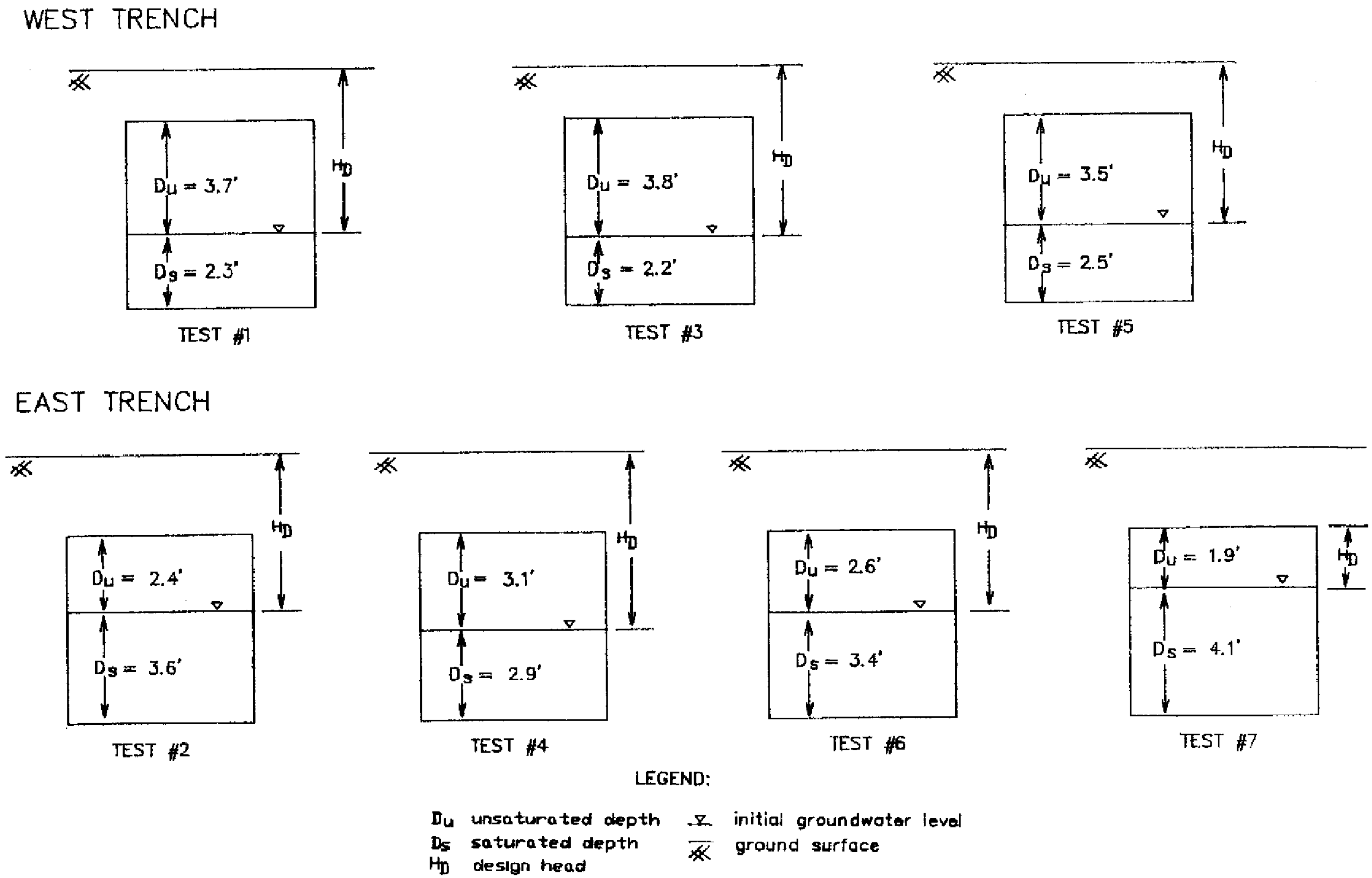


TABLE 3

Results of Trench Tests

Test #	Trench	H_D (ft.)	Fill Time (min.)	Average Measured Disch (gpm) (first 60 mins. of test)
1	West	5.4	16	63 [49]
2	East	5.0	14	60 [51]
3	West	5.5	29	55 [41]
4	East	5.7	43	57 [45]
5	West	5.2	10	68 [55]
6	East	5.2	13	64 [54]
7	East	1.9	8	24 [17]

. values in [ ] are exfiltration rates not including storage (Q_1)
. an effective porosity of 0.5 was assumed in the trench
. H_D is the height in ft. of the design stage in the trench above the initial water table.

The values in square brackets (Q_1) represent pure exfiltration rates (averaged) during the first 60 minutes of the test (ie., Q_1 = (total discharge - initial storage volume)/60 mins.). Tests 1,2,5 and 6 showed higher exfiltration rates than tests 3 and 4. The fill times of tests 3 and 4 were 2-3 times longer than in tests 1, 2, 5 and 6. Hence, full design head was maintained for longer times in tests 1, 2, 5 and 6, leading to higher averaged exfiltration rates.

The average exfiltration rate for test 7 was the lowest of all the tests. The driving head, H_D, for this test was lower than H_D for the other tests by 3-4 ft., resulting in a lower exfiltration rate.

The observed discharge rates presented in Table 3 are averaged over the first 60 minutes of the test. These rates do not represent pure exfiltration since the volume of water stored in the trench is included in the total inflow volume. Pure exfiltration rates for the first 60 minutes of the test (Q_1) include exfiltration before the trench was completely full. Average discharge rates during the first 60 minutes after filling (Q_2) are presented in Table 4 for tests 4-7 (tests 1-3 were terminated 60 minutes after the beginning of the test). Q_2 represents pure exfiltration rates under full design head.

Q_2 is 1 gpm higher than Q_1 for tests 4 and 7. This is not surprising, since Q_1 includes exfiltration while the trench is partially full and the driving head is less than design head. Q_2 is lower than Q_1 for tests 5 and 6. This is probably due to a reduction in exfiltration rates resulting from a progressively increasing groundwater mound. Changes in groundwater elevations with time are discussed later in this section.

Test 5 (west trench) and test 6 (east trench) were conducted with the same head above the water table. The exfiltration rate in test 5 is 1 gpm lower than in test 6. The difference in exfiltration rates is probably because the east trench allows bottom exfiltration while the west trench does not. This suggests that bottom exfiltration accounts for about 2% of the total trench exfiltration and is therefore not significant. The fill time and design head appear to be the dominant influences on exfiltration rates.

TABLE 4

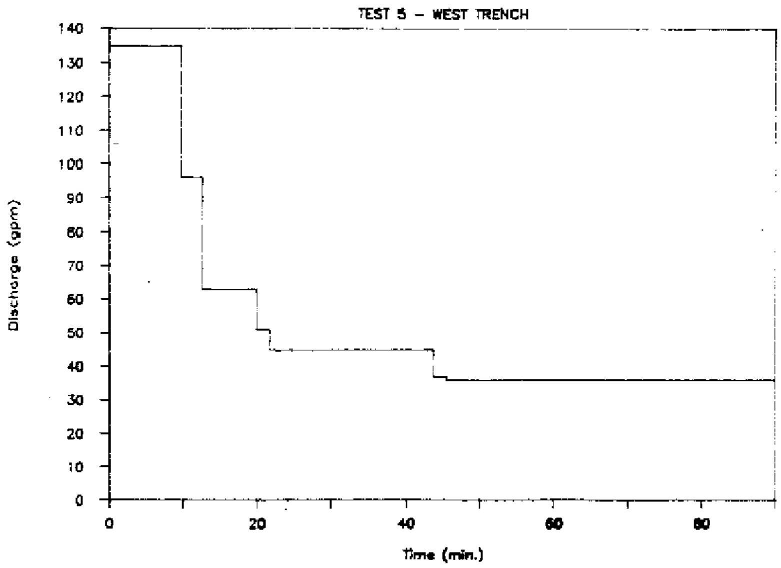
Trench Exfiltration Rates

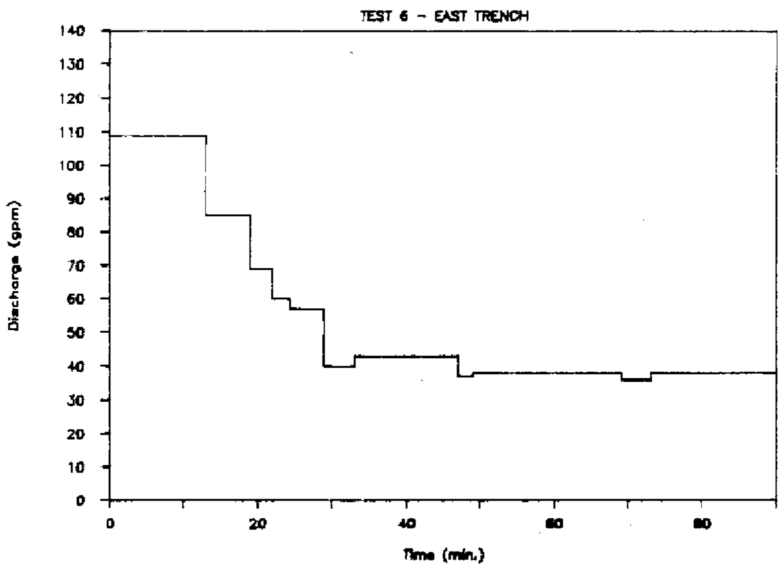
Test #	Trench	H_D (ft)	Avg. Measured Disch (gpm) 60 mins. after filling trench (Q_2)
4	East	5.7	46
5	West	5.2	46
6	East	5.2	47
7	East	1.9	18

Plots of discharge rate vs time for tests 5 through 7 are given in Figure 6 Filling times are shown. Plots for tests 5 and 6 show discharge rates (exfiltration rates) decreasing rapidly within the first half hour of the test and remaining relatively stable after that. Discharge rates for test 7 were smaller than for tests 5 and 6 and decreases during the course of the test were less dramatic.

Figures 7-12 give groundwater profiles at various times during the tests. The vertical scale of these plots is exaggerated for clarity. These plots show changes in groundwater elevations with time and space. The initial profiles show groundwater elevations decreasing towards the trench. One possible reason for this, is the capillary effects in sand. The water level rise, typically 5"-14" in medium-grained sand (ref. Harr (1962)), diminished in the vicinity of the gravel filled trench where capillary action is negligible.

Figure 6 DISCHARGE RATE VS TIME





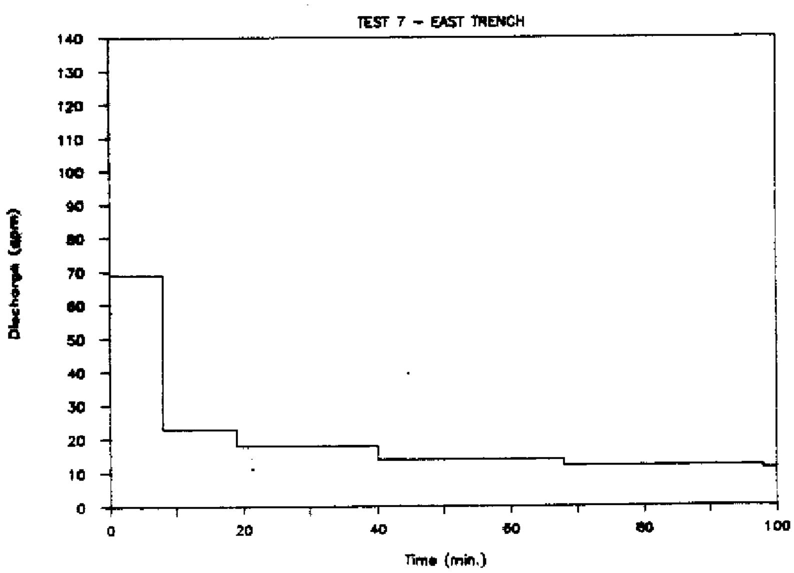


Figure 7 NORTH-SOUTH GROUNDWATER PROFILES, TEST 5 (WEST TRENCH)

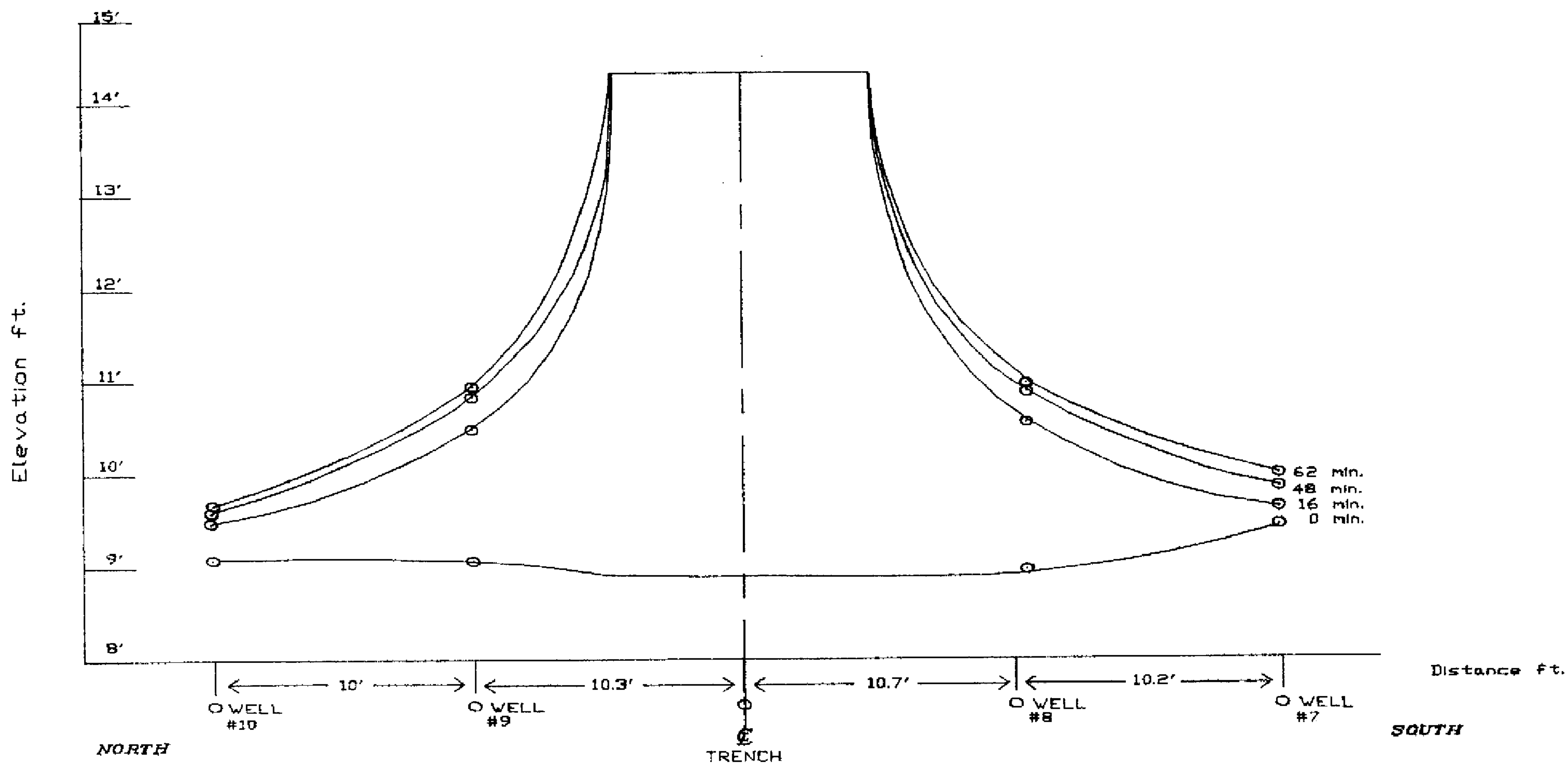


Figure 8 EAST-WEST GROUNDWATER PROFILES, TEST 5 (WEST TRENCH)

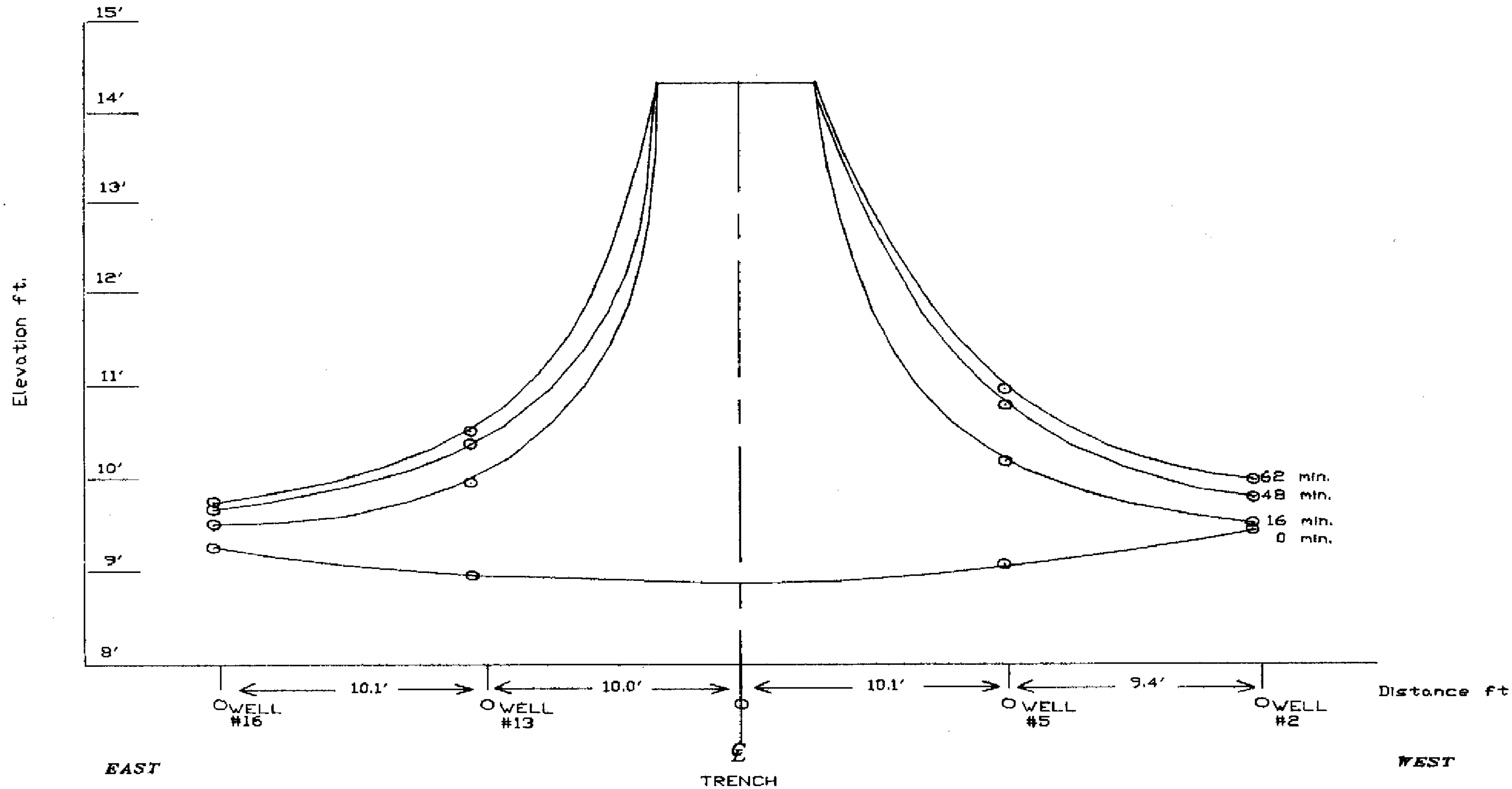


Figure 9 NORTH-SOUTH GROUNDWATER PROFILES TEST 6 (EAST TRENCH)

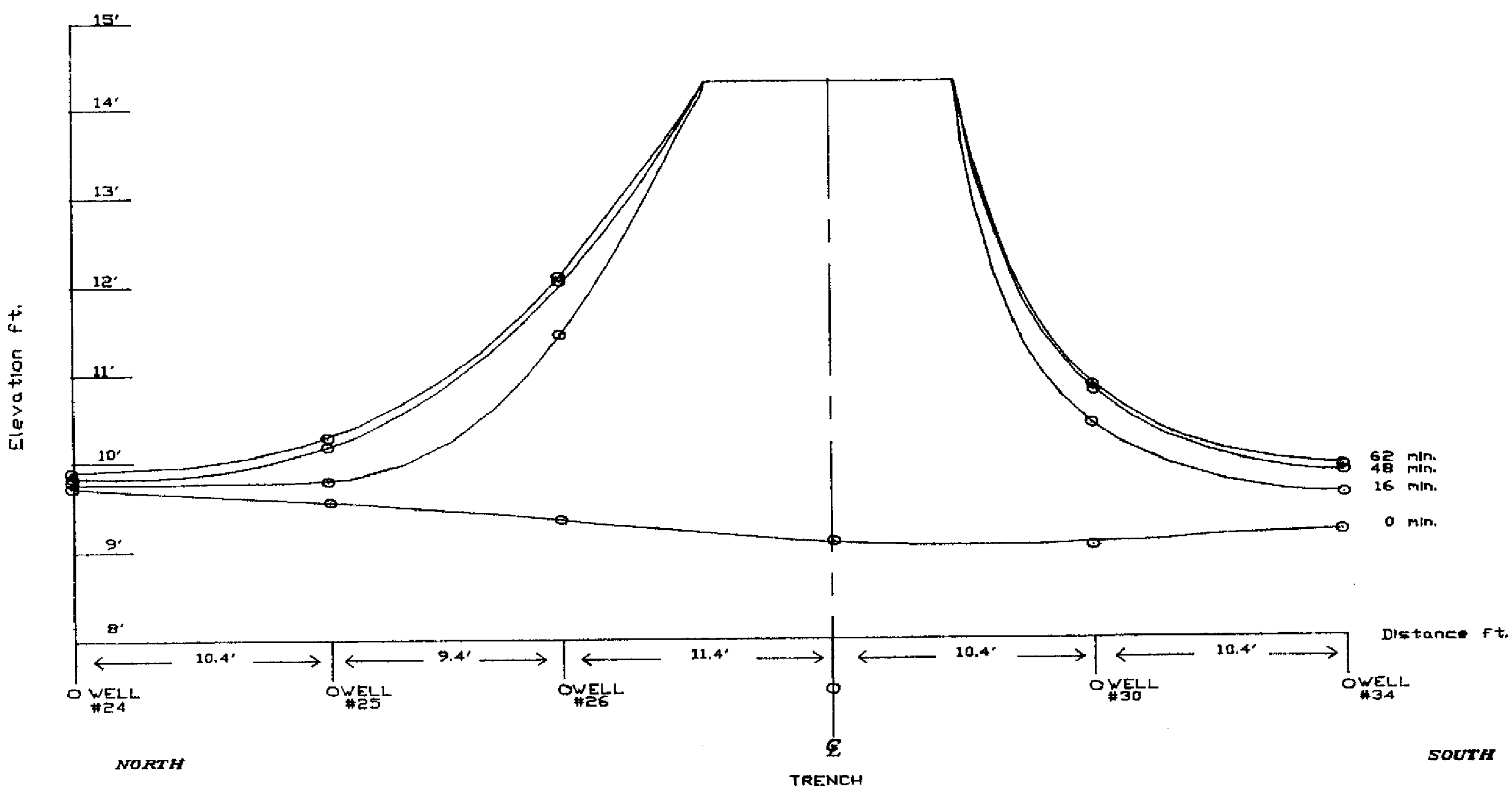


Figure 10 EAST-WEST GROUNDWATER PROFILES, TEST 6 (EAST TRENCH)

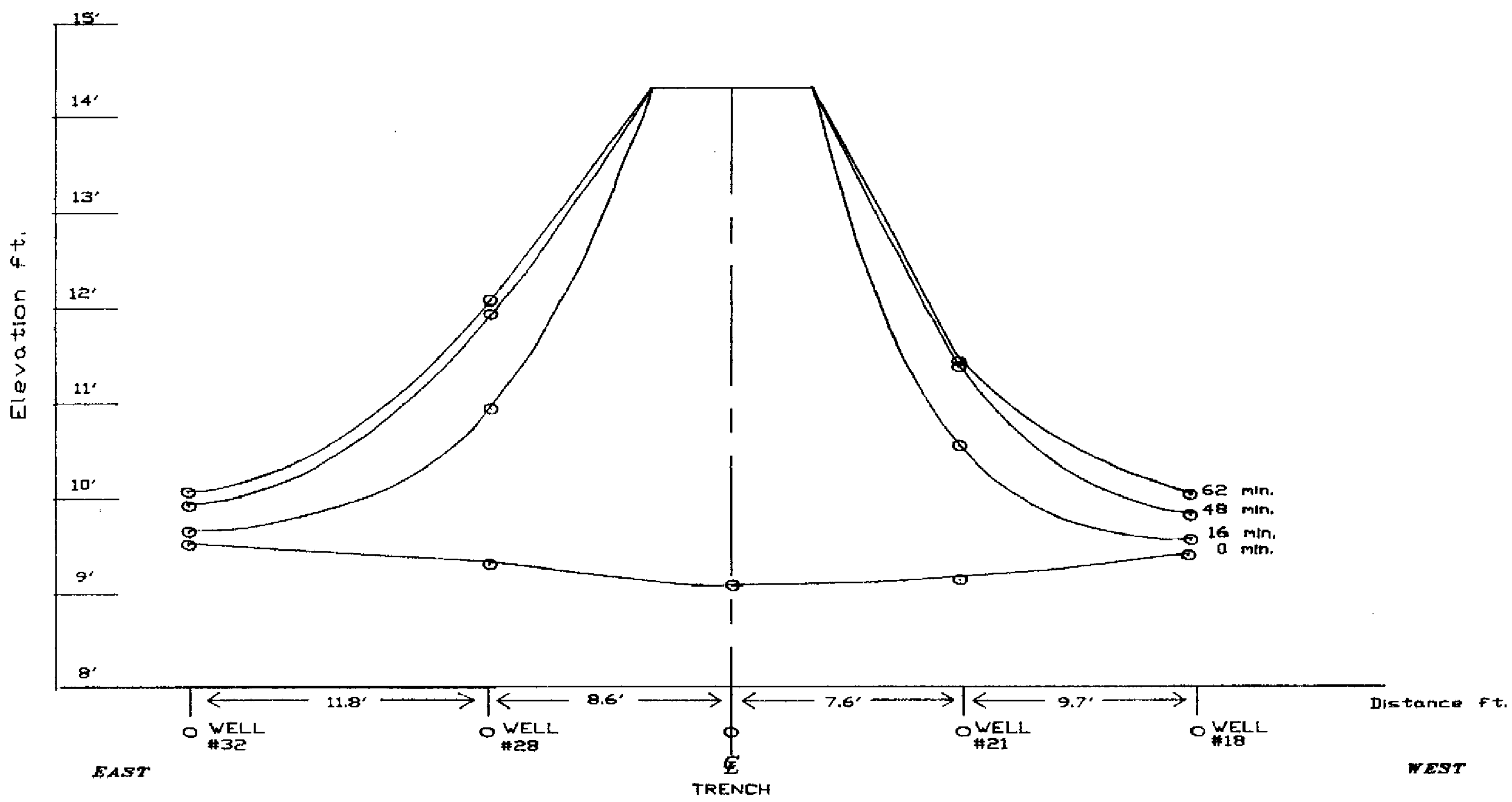


Figure 11 NORTH-SOUTH GROUNDWATER PROFILES, TEST 7 (EAST TRENCH)

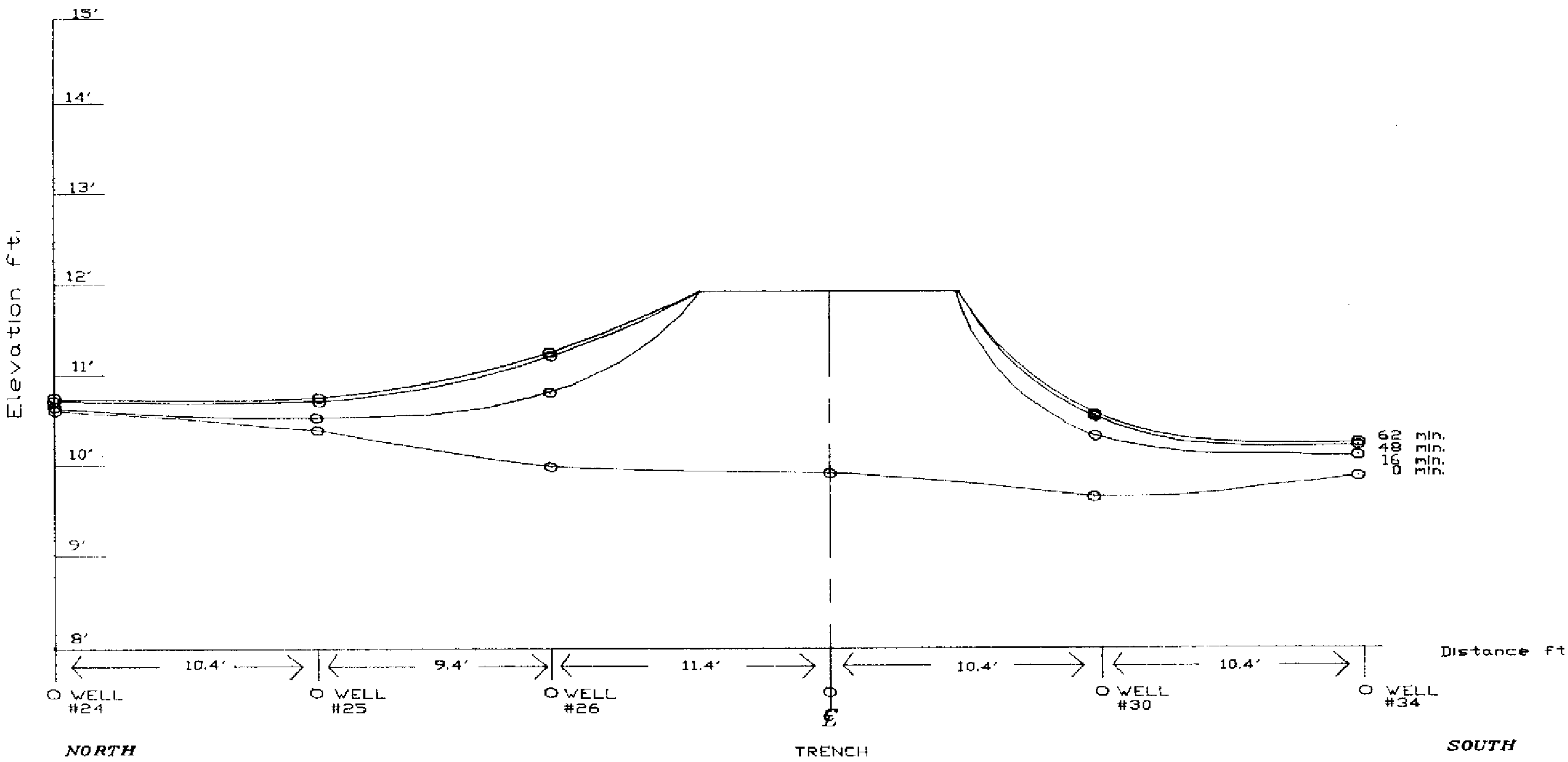
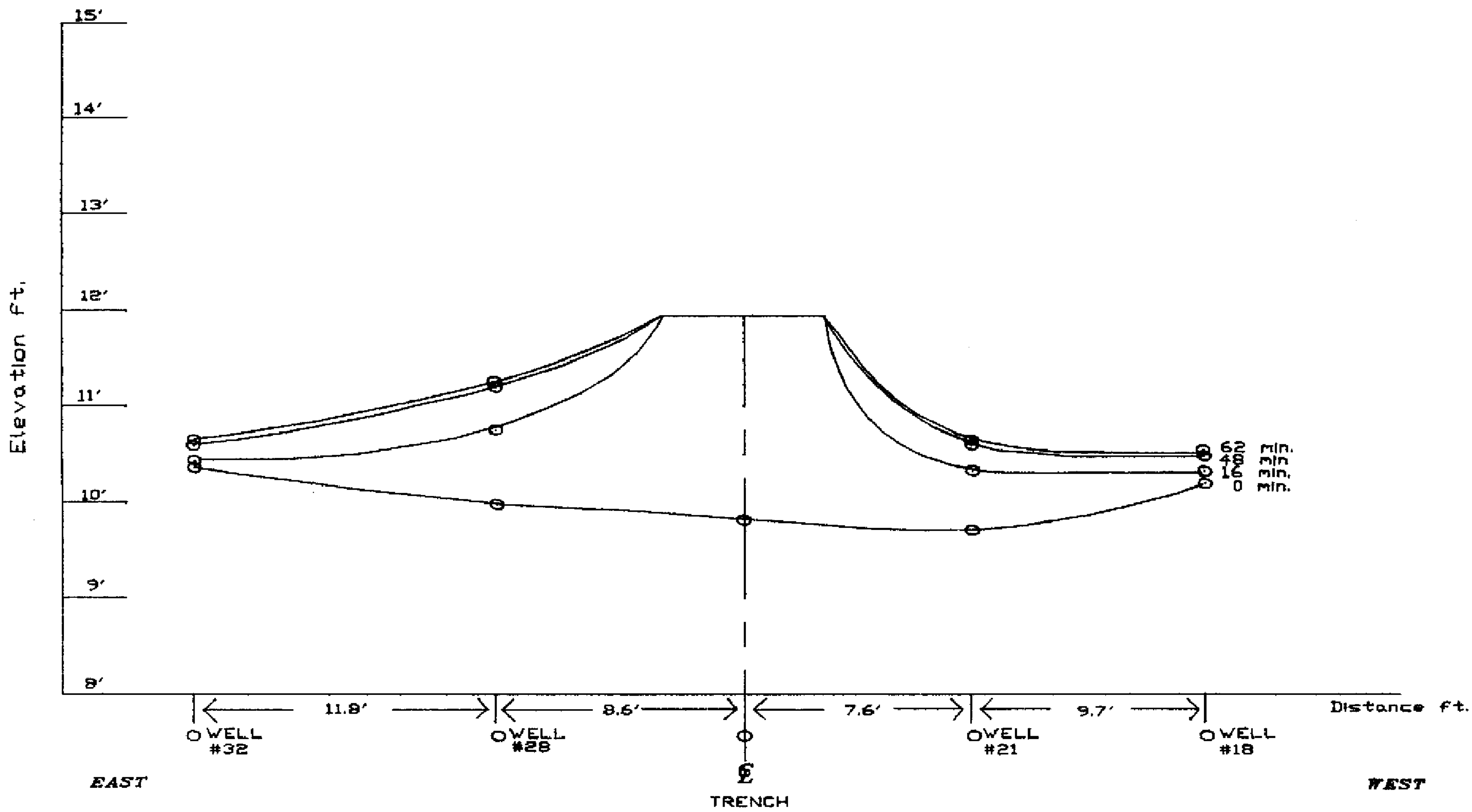


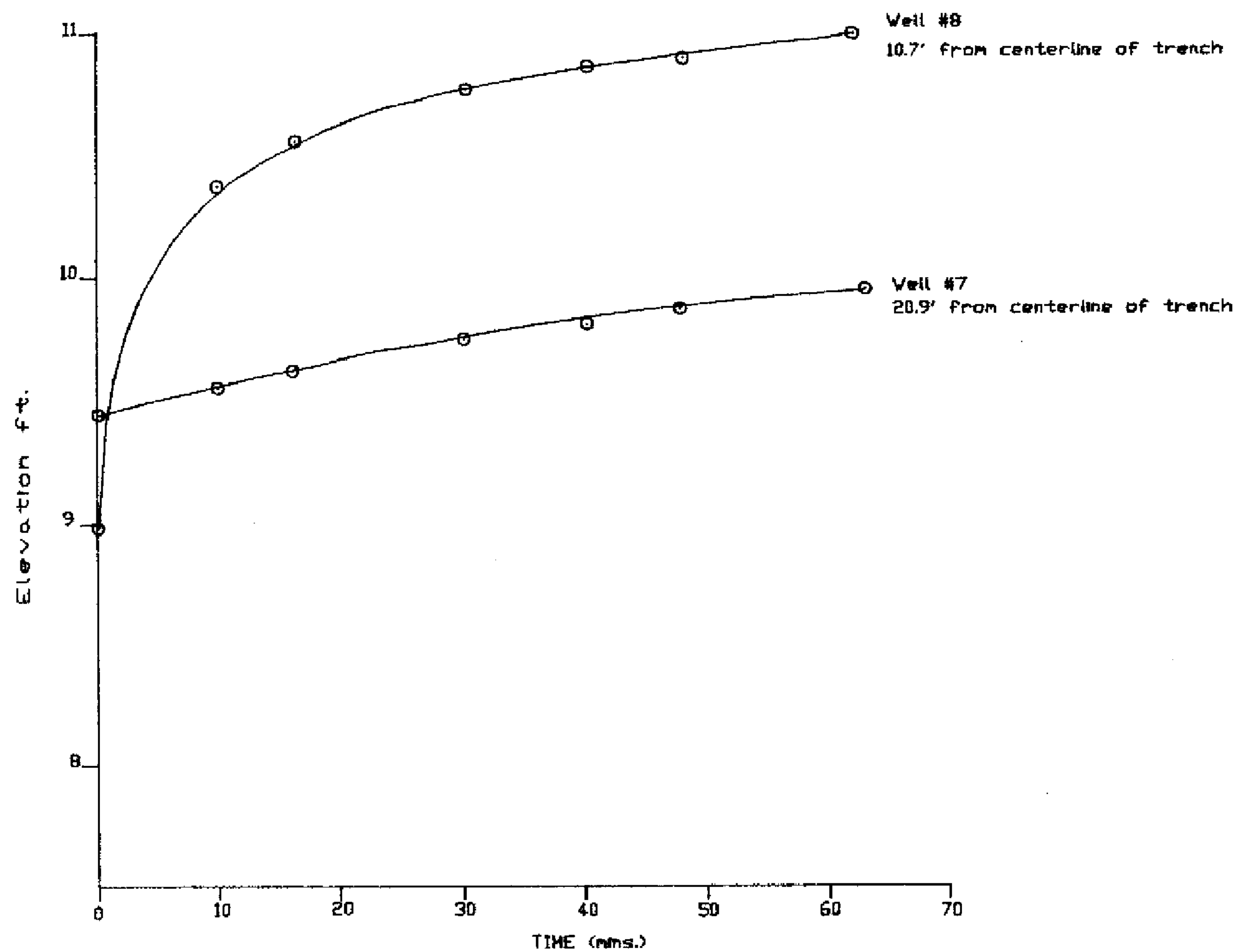
Figure 12 EAST-WEST GROUNDWATER PROFILES, TEST 7 (EAST TRENCH)



The inflow to the trenches during the tests resulted in increases to the groundwater levels in a mounding fashion. The wells closer to the trench (nominal 10 feet) show more pronounced and variable increases than those further away (nominal 20 feet).

Figure 13 shows time varying plots of water elevations in wells #7 and #8 during test 5. Well #8 is 10.7' from the trench center. In well #8 groundwater elevations increase rapidly in the first 15 minutes of the test (75% of the total increase in 15 minutes) and then taper off to a more gradual increase with time (25% of total increase in 45 minutes). Groundwater elevation increases in well #7 (20.9' from trench center) were smaller in magnitude and relatively steady with time.

Figure 13 GROUNDWATER ELEVATIONS VS. TIME FOR WELLS #7 AND #8



COMPARISON BETWEEN COMPUTED AND OBSERVED DISCHARGES

In order to test the accuracy of the Volume IV trench design methodology, design discharge rates computed using Volume IV formulae were compared with observed discharge rates in tests 1-7. An average Volume IV discharge coefficient (K_{IV}) of 1.9 x 10^{-4} sec^{-1} was obtained using results from two types of Volume IV recommended constant head percolation tests, the DOT standard test and the Usual Open Hole test.(See Table 2). This K_{IV} value, together with the test trench dimensions, was substituted in the Volume IV trench design formula and design discharge rates were obtained.

End wall exfiltration, usually neglected because of the large length to width ratio in most trenches, was included in these computations because of the relatively short lengths of the test trenches. Bottom exfiltration was excluded from the computations for the west trench, since it has an impervious bottom.

Computed discharge rates obtained using Volume IV design formula are shown in Table 5 together with averaged discharge rates measured during the trench tests.

The computed discharge rates obtained from the Volume IV method are greater than observed discharge rates by 32-158%. It is interesting to note that for the west trench (trench bottom exfiltration excluded) there was better agreement between computed and observed discharge rates.

TABLE 5

Computed Discharge Rates,
Vol. IV Method Compared with Observations

Test #	Fill Time (min)	Observed Average Discharge Rates First 60 min of Test (gpm)	Computed Discharge Rates Volume IV Method (gpm)
1 W	16	63	93 (48%)*
2 E	14	60	130 (117%)
3 W	29	55	94 (71%)*
4 E	43	57	147 (158%)
5 W	10	68	90 (32%)*
6 E	13	64	134 (109%)
7 E	9	24	54 (125%)

. $K_{IV} = 0.00019\ sec^{-1}$
. values in ( ) are the percent error between computed and observed discharge rates
. W denotes west trench
. E denotes east trench
* bottom exfiltration excluded from west trench computations.

REVIEW OF VOLUME IV DESIGN METHOD

Hydraulic Conductivity

The hydraulic conductivity of a soil is defined in terms of Darcy's Law which states that the velocity of discharge (Darcian velocity) of water through soil (v), is directly proportional to the hydraulic gradient. This relation is expressed by the equation

$$v = K\, i \tag{4}$$

where, i is the hydraulic gradient.

K is the constant of proportionality known as hydraulic conductivity.

Hydraulic conductivity represents the ability of a soil to transmit water under a given hydraulic head and is expressed in units of velocity.

From equation (4), we get,

$$K = \frac{v}{i} = \frac{Q}{Ai} \tag{5}$$

where, Q is the volume rate of seepage of water

A is the area over which seepage takes place

Hence, knowing the discharge rate corresponding to a measured hydraulic gradient during a percolation test, we can obtain the hydraulic conductivity of the soil.

The Volume IV formulae for computing 'hydraulic conductivity' from percolation test results assume that the volume rate of seepage, measured over a given time, divided by the integral product of the exfiltrating area and hydraulic head is equal to the hydraulic conductivity of the soil.

$$K_{IV} = \frac{Q}{\Sigma(A \cdot \ head)} \tag{6}$$

K_{IV}, as defined in equation (6), is independent of the hydraulic gradient and is instead, a function of the hydraulic head in the percolation test hole. Equation (6) ignores the effects of hydraulic gradients in the soil on the seepage rate,Q, and is more like a membrane conductance than a soil hydraulic conductivity. The units of K_{IV} are in ft/sec.ft head while units of hydraulic conductivity are in ft/sec.

Trench Design Formula

The trench design formula assumes that the volume rate of exfiltration from the trench is equal to the product of K_{IV}, the head in the trench and the surface area of the trench. ie.,

$$Q = K_{IV} \cdot \ \Sigma(A \cdot \ head) \tag{7}$$

Equation (7) is the same as equation (6) and K_{IV} is again applied like a membrane conductance and no account is taken of the difference in flow geometry between percolation test hole and exfiltration trench. The influence of hydraulic gradients on the exfiltration rate are also not taken into consideration.

Equation (7) implies that Q is independent of time. The steady state assumption is not valid for exfiltration trenches since exfiltration trenches operate under transient conditions. This was illustrated in Figures 6-13.

The Volume IV method assumes the design stage for exfiltration trench design to be the top of the inlet grate. Where no overflow exists, this may be an appropriate choice, but in the typical exfiltration system designs permitted by the District, overflow weirs are used to surcharge the systems. In this case, the design stage should be the weir crest elevation.

The Volume IV method assumes that the rate of exfiltration per unit area from the bottom of the trench is the same as the exfiltration rate per unit area from the side of the trench under the same design head. The computed bottom exfiltration would then account for 30-32% of the total exfiltration from the test trenches. The results presented in Table 4 suggest that trench bottom exfiltration accounts for only about 2% of the total exfiltration from the test trenches and is therefore negligible compared with exfiltration from the side walls.

The Volume IV trench design equation does not include exfiltration from the end walls. For very long trenches, end wall exfiltration comprises a relatively small portion of the total exfiltration volume and may be ignored.

ALTERNATE PROCEDURE

The Volume IV Methodology overestimates trench exfiltration rates which leads to an underestimation of trench lengths. An alternate design procedure is presented and tested against field measurements of tests 1 - 7. This procedure treats the trench as a line source and will be referred to as the "line-source" method.

The line-source method is a two part design procedure involving

1. the determination of soil hydraulic conductivity, K
2. the determination of minimum trench dimensions to provide the required trench capacity.

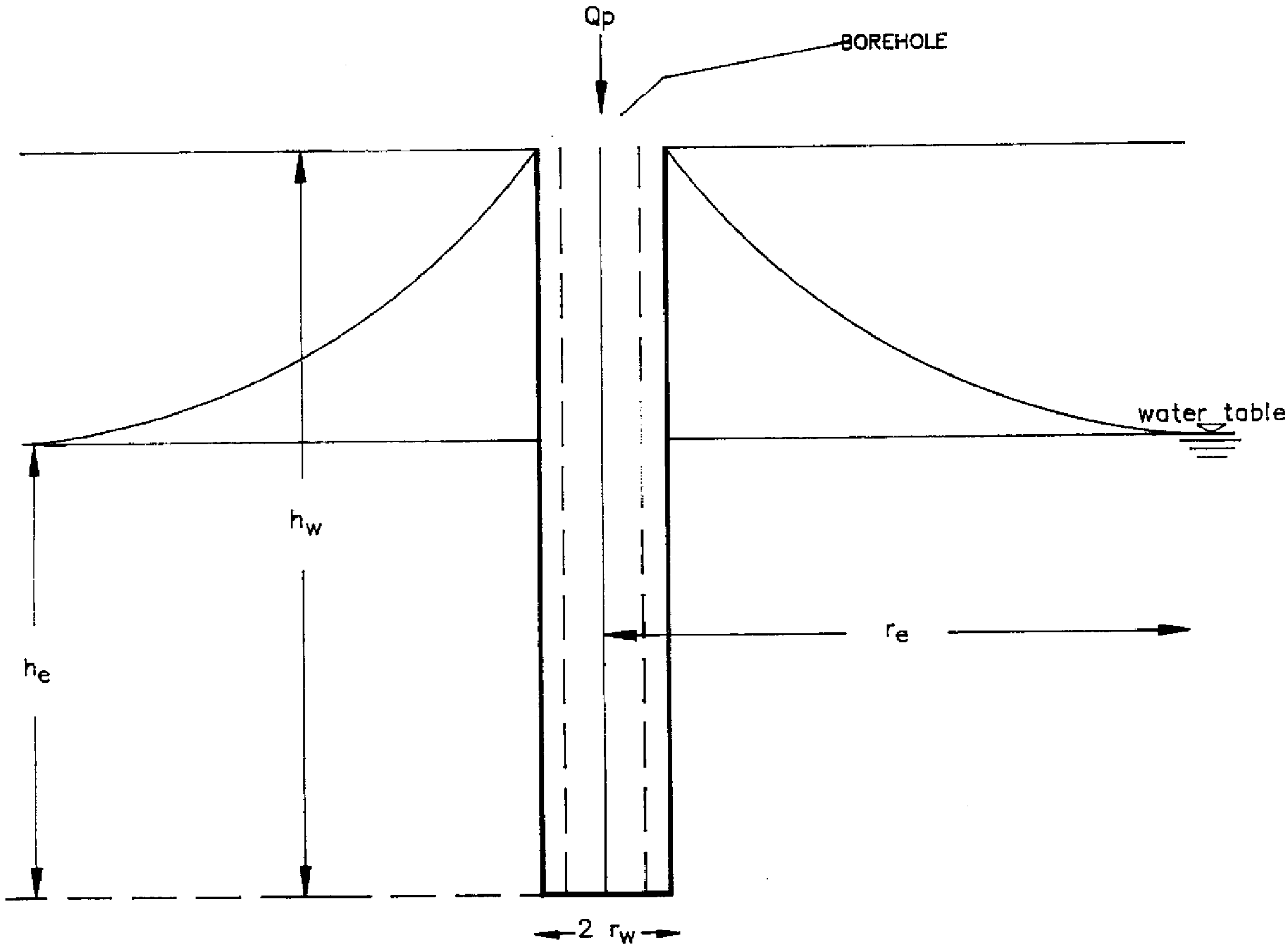
Determination of K

The proposed alternate method determines hydraulic conductivity, K, using a constant head percolation test. The field procedure for the percolation test is the same as the Usual Open Hole test described in Volume IV. Q_p, the steady rate of discharge into the borehole and h_w, the initial water table elevation are measured. K is then computed using Thiem's equation (Illustrated in Figure 14).

In order to apply Thiem's equation, the radius of influence, r_e, of the borehole must be determined. This can be done using observation wells placed at varying distances from the borehole. The minimum distance from the borehole at which no change in groundwater elevation is observed during the test is the radius of influence, r_e.

Observation wells are expensive to install and if an array of observation wells is required for each percolation test, the cost of field testing would increase significantly. Even when observation wells are installed, the chances of placing an observation well at a distance equal to r_e from the borehole are relatively small. The

Figure 14 PERCOLATION TEST, LINE-SOURCE METHOD



$$K = \frac{Qp\ \ln(r_e / r_w)}{\pi\ (h_w^2 - h_e^2)}$$

Qp = steady inflow rate to borehole (cfs)

K = Hydraulic conductivity (ft/sec)

r_e = radius of influence of borehole (ft)

r_w = radius of borehole (ft)

h_e = depth of borehole below water table (ft)

h_w = total depth of borehole (ft)

choice of r_e depends on the judgement of the designer and the value chosen is, therefore, approximate. Because of this, it is acceptable to determine a typical r_e from observation wells during one percolation test on the site and apply it to all other percolation tests conducted on site. Alternatively, typical values of r_e may be specified for use in various areas in the District.

Trench Length

The length of trench required to handle the design flow can be computed using the following equations:

$$L = \frac{(Q_D - q_e W)}{q_e + q_v} \tag{8}$$

$$q_e = \frac{1}{T} \int_0^T q_t \cdot dt = \frac{\sqrt{\Pi} \cdot 2KD H_D}{(\alpha T)^{\frac{1}{2}}} \tag{9}$$

$$q_t = \frac{\sqrt{\Pi} \cdot KD H_D}{(\alpha t)^{\frac{1}{2}}} \tag{10}$$

$$q_v = \frac{N_g W (H - D_s)}{T} \tag{11}$$

$$\alpha = \frac{KD}{N_a} \tag{12}$$

where,

L = required length of trench (feet)

Q_D = required trench capacity (cfs)

W = trench width (feet)

q_t = exfiltration rate per linear foot of trench (cfs/ft)

q_e = time integrated exfiltration rate per linear foot of trench (cfs /ft)

q_v = storage rate per linear foot of trench (cfs /ft)

α = aquifer diffusivity (sq ft /sec)

D = effective aquifer depth (assumed equal to two times the trench height)

N_a = effective porosity of aquifer i.e., the ratio of drainable or fillable voids to the total volume (cu ft/cu ft)

N_g = effective porosity of the trench, assumed to be 0.5

t = time in seconds from the beginning of exfiltration

T = design period in seconds (3,600 seconds)

H_D = height of weir crest above wet season water table

H = height of trench (feet)

K = hydraulic conductivity (feet/sec)

D_s = saturated trench depth (feet)

N_a, the effective porosity of the aquifer is determined using field measurements as described in Appendix A.

The exfiltration rate per linear foot (q_t) is time dependent (inversely proportional to the square root of time). The shape of discharge vs time plots given in Figure 6 confirms this relation. The line-source method, therefore, takes into account the time varying hydraulic gradient produced by the developing groundwater mound above the water table. Equations (9) and (10) were developed by Glover (1966) for transient flow from a line source in an unconfined aquifer. Equation (11) expresses q_v as the total volume of fillable voids per unit length of trench divided by 1 hour.

Mounding

Equation (10) is a limiting case of the following equation, which describes groundwater profiles around a line-source at any given time. See Glover (1974).

$$h = \frac{q_e r}{2\pi KD} \cdot \sqrt{\pi} \int_{\frac{r}{2\sqrt{(\alpha t)}}}^{\infty} \frac{e^{-u^2}}{u^2} \, du \tag{13}$$

where, h is the height of the mound built up at time

r is the distance from the trench

The integral in Equation (13) can be expressed in terms of the error function which can be solved readily.

The ability to predict changes in groundwater levels (mounding) during exfiltration is an attractive feature of the line-source method. It can be used to check the impact of proposed exfiltration trenches on adjacent structures and to establish spacing criteria for trenches. Preliminary checks indicate that predictions of groundwater elevations using Equation (13) match observations fairly well close to the test trench. However, at distances greater than 17' from the trench, mounding predictions are generally much lower than observed. The lack of agreement between computed and observed mounding is not surprising since Equation (10) applies to line sources while the test trenches are rectangular rather than line sources. Further investigation is recommended using data from full-sized, prototype trenches.

Method of Testing

The accuracy of the line-source method was tested by checking the accuracy with which equations (1) and (8) could be used to predict exfiltration for the test trenches.

Exfiltration rates were computed for the test trenches, using the experimental conditions of tests 1 through 7. The average K value of 2.4×10^{-4} ft/sec obtained using the Thiem's equation was used as the hydraulic conductivity in equations (9) and (12). Q_D was then obtained from equation (8). Computed and measured exfiltration rates are presented for comparison in Table 6.

TABLE 6

Computed Disch. Rates, Line Source Method, Compared with Observed Discharge Rates

Test #	Filling Time (mins)	Measured Discharge First 60 Minutes After Start of Test (gpm)	Computed Trench Capacity (exfil. & storage) for 1 Hour After Start of Exfiltration (gpm)
1W	16	63	63 (+0%)
2E	14	60	53 (-12%)
3W	29	55	60 (+9%)
4E	43	57	54 (-5%)
5W	10	68	64 (- 6%)
6E	13	64	56 (- 13%)
7E	8	24	27 (+13%)
Average		56	54 (-4%)

.Percentage error shown in parentheses
.$K = 2.4 \times 10^{-4}$ ft/sec
.$N_a = 0.23$
.$N_g = 0.5$

An aquifer effective porosity of 0.23 was used. (See Appendix A.) The effective depth of aquifer, D, was assumed equal to 12', i.e., twice the height the trench. It was assumed that exfiltration under full design head started after the trench was half full.

The results given in Table 6 show that, the line-source method estimated discharge rates within 13% of the observed rates. The variation in accuracy can be attributed to experimental variations between tests.

The line-source method gives fairly accurate estimates of trench exfiltration rates and could be an acceptable alternative to the Volume IV procedure. Application of the line source method is very similar to the Volume IV method (Design example given in Appendix B) and changing from one procedure to the next should not present any problems.

There are a number of potential drawbacks to the line-source method which should be pointed out. Values of effective porosity of aquifer, N_a, effective aquifer depth, D, and radius of influence of the percolation test borehole, r_e, must be determined in order to use the line-source method. Field determination of aquifer effective porosity would lead to additional costs to the developer but this is not expected to be significant. An alternative would be to use typical values of N_a for ranges of soil types or particle size distributions (if known).

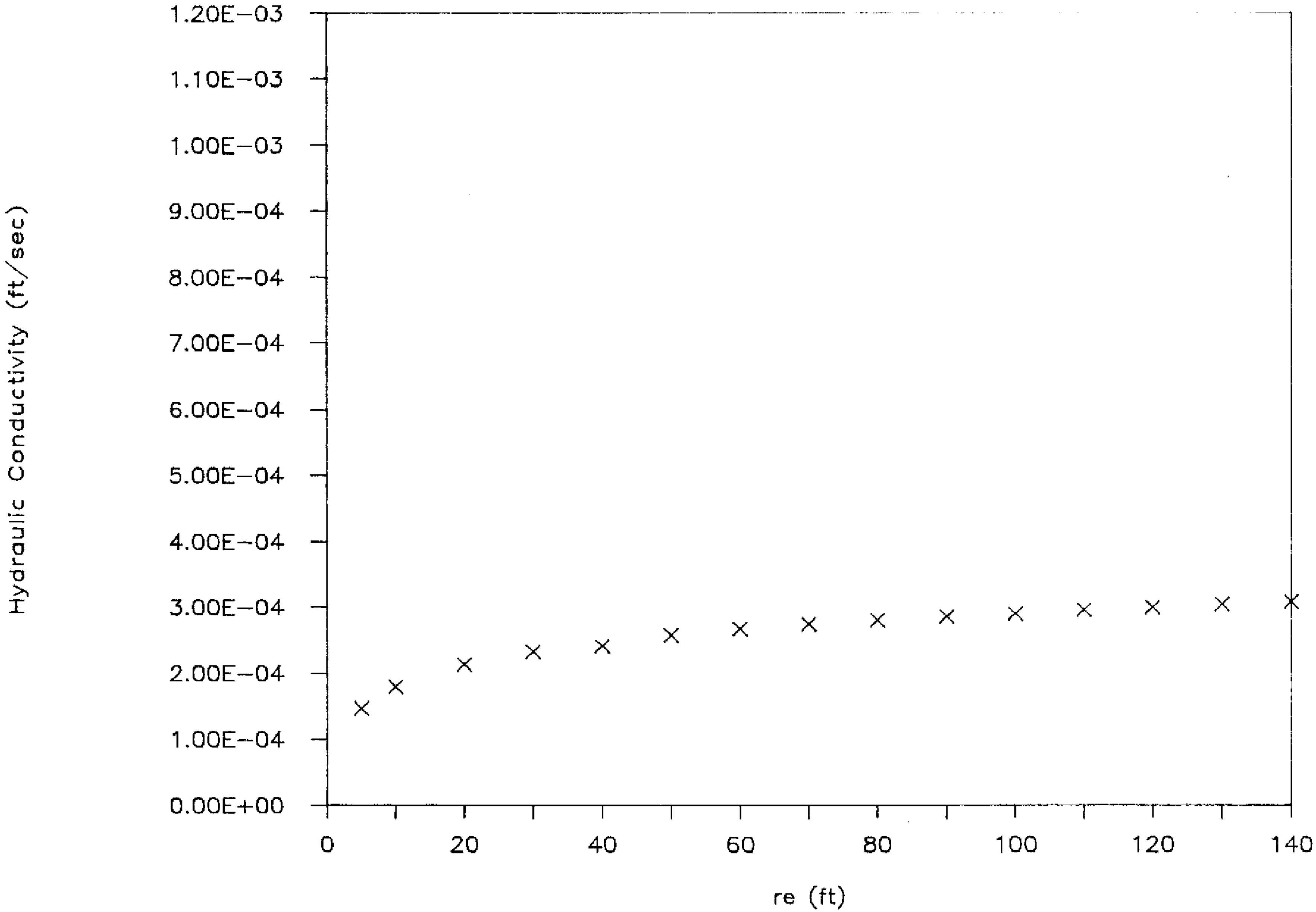
The application of a typical radius of influence, r_e, (determined from one percolation test) to all percolation tests performed on the site is an approximation which introduces some uncertainty in the computed hydraulic conductivity, K. Figure 15 shows a plot of K vs. r_e for percolation test 6. (This plot is typical of all the percolation tests which were performed.) The values of K increase as r_e increases.

When r_e is greater than 20', K does not appear to be very sensitive to the changes in r_e. Errors in K due to uncertainties in the value of r_e are therefore not critical.

The effective aquifer depth, D, is the depth of aquifer below the initial water table which is affected by the exfiltration trench in one hour. This is usually unknown and D was used as a calibration factor in our tests. The closest agreement between observed and computed trench capacities was obtained when D was set equal to 12', twice the height of the trench. There is no theoretical justification for this and further investigation of this aspect of the line-source method is recommended.

Figure 3 HYDRAULIC CONDUCTIVITY VS RADIUS OF INFLUENCE

THIEM'S EQUATION (TEST #4)



SOURCES OF ERROR

Discharge rates were obtained by recording the change of flow meter reading over a given time. This method is subject to personal error especially during times of large flow.

In general, the equations which were used to estimate exfiltration rates apply to trenches which are much longer than they are wide. These equations were applied to test trenches which are almost as wide (6') as they are long (10'). The shape of the trench is expected to influence the hydraulic gradients around the trench which will in turn impact the exfiltration rates. It would be instructive to check the current and proposed design formulae using data from a full scale exfiltration trench.

CONCLUSIONS AND RECOMMENDATIONS

Comparison between computed and measured exfiltration rates showed that the Volume IV Method substantially overestimates exfiltration trench capacities. Thus, shorter trenches than actually required to meet retention criteria are being designed and constructed using this method.

Examination of the derivation of the K_{IV} formula and the application of K_{IV} to the trench design formula indicates that the flow through a unit area of the trench wall is assumed to be equal to the flow through a unit area of the percolation borehole wall under the same head condition, without regard for the effects of hydraulic gradients or flow geometries. Accordingly, in cases where the gradient becomes important to the exfiltration rate, the formula will over-estimate discharge capacity.

An alternate design procedure, the line-source method, was examined. Using this method, exfiltration trench capacities were predicted to within 13% of measured trench capacities. This represents a significant improvement over the Volume IV method, which over estimates trench capacities by 32 to 158%. In addition, this method may be extended to predict the impact of exfiltration on groundwater elevations (mounding) which can be very useful in determining the most efficient layout for trenches. Although some aspects of this procedure may warrant further investigation, it is recommended that the line-source method be adopted for future exfiltration trench designs.

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APPENDIX A

Field Measurement of Effective Porosity

Effective porosity, n, is defined as the ratio of the total volume of fillable voids in a soil sample to the total volume of the sample. In terms of the soil phase diagram in Figure A-1,

$$N = V_A / V_T$$

Effective porosity was computed using field measurements of

1. the wet weight (M_w) of an undisturbed soil sample of known volume
2. the moisture content of the soil (mc)

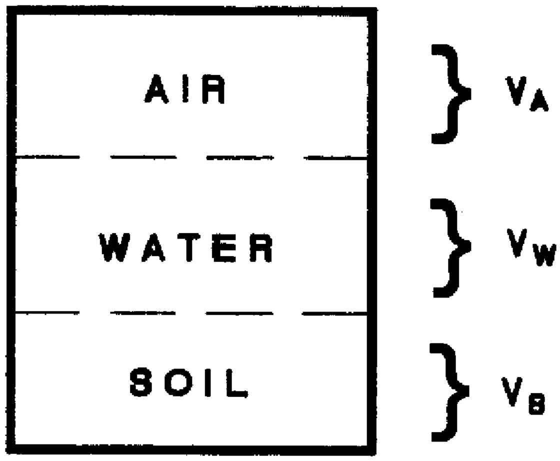
Undisturbed samples were obtained by hammering an open ended metal sleeve of known weight into the soil until the sleeve was completely filled with soil. The soil filled sleeve was weighed.

$$M_w = M_s - \text{weight of empty sleeve}$$

where, M_s is the weight of the soil and sleeve together.

The moisture content of the soil (mc) was measured. M_w and mc were then used to compute the aquifer effective porosity, N, as shown in Tables A-1 and A-2.

Figure A-1 SOIL PHASE DIAGRAM



Total Volume of Sample = $V_T = V_A + V_W + V_S$

Total Volume of Voids = $V_V = V_A + V_W$

Effective Porosity, $N = \dfrac{V_A}{V_T}$

Table A-1

Field Measurements

Depth of Test (ft)	M_S Total Wt of Sample & Sleeve (gm)	M_W Wet Wt of Sample (gm)	mc Moisture Content (%)	M_D DryWeight of Sample (gm)
0-1	1052.6	593.4	9.1	543.9
1-2	997.8	538.6	5.7	509.5
2-3	985.1	525.9	3.6	507.6

Weight of sleeve = 459.2 gm

Volume of sample = 294.5 cm^3

$$M_W = M_S - wt.\ of\ sleeve$$

$$M_D = \frac{M_W}{1 + mc}$$

Table A-2

Calculation of Effective Porosity

Depth (ft)	Dry Wt of Sample M_T (gm)	Volume of Sample V_T (cm^3)	Volume of Solids $V_S = M_T/\rho_S$ (cm^3)	Volume of Voids $V_V = V_T - V_S$ (cm^3)	Volume of Water in the Voids $V_W = mc \times M_T/\rho_W$	Effective Porosity $N = (V_V - V_w)/V_T$ (%)
0-1	543.9	294.5	202.95	91.55	49.49	14
1-2	509.5	294.5	190.11	104.39	29.04	26
2-3	507.6	294.5	189.40	105.10	18.27	29
					Average	23

ρ_S - assumed density of sand = 2680 kg/m^3

ρ_W - assumed density of water = 1000 kg/m^3

APPENDIX B

Design Example

The line source trench design equation may be simplified to the following equation for a one hour design period if an effective aquifer depth equal to twice the trench height is assumed, and if the effective porosity of the trench is set equal to 0.5.

$$L = \frac{1.01\,V - 0.084\,D_u\,W\sqrt{(KHN_a)}}{0.084\,D_u\,\sqrt{(KHN_a)} + 1.39 \times 10^{-4}\,WD_u}$$

where	L	length of trench (ft)
	V	design trench capacity (acre-inches)
	D_u	unsaturated depth of trench (ft)
	K	hydraulic conductivity (ft/sec)
	H	height of trench (ft)
	N_a	effective porosity of aquifer
	W	width (ft)

<u>Design Example</u>

An exfiltration system is to be designed for an 8 acre shopping complex with 7.5 acres impervious. Percolation tests indicate a hydraulic conductivity of 2.4×10^{-4} ft/sec. Soil tests indicate an effective porosity of 0.23. Average ground elevation is 14.0' NGVD. Mean wet season water table elevation is 9.0' NGVD.

Field Test Results

$N_a = 0.23$

$K = 2.4 \times 10^{-4}$

SFWMD volume retention criterion specifies a design runoff of 2.5" times the percentage of imperviousness; i.e.,

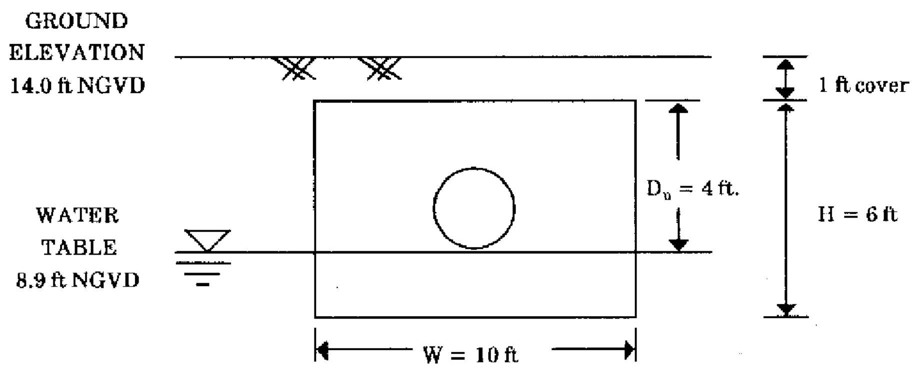
$$R = 2.5 \times \frac{7.5}{8.0} = 2.34$$

Total design volume = R $\times$ area of site (acres)

$$= 2.34 \times 8.0 = 18.75 \text{ acre-inches}$$

Assume

Trench height	H	= 6'
Trench width	W	= 10'
	D_u	= 4'



Design Sketch

$$L = \frac{1.01 \times 18.75 - 0.084 \times 4 \times 10 \times \sqrt{(2.4 \times 10^{-4} \times 6 \times 0.23)}}{0.084 \times 4 \times \sqrt{(2.4 \times 10^{-4} \times 6 \times 0.23)} + 1.39 \times 10^{-4} \times 10 \times 4}$$

$$= 1617.15' \simeq 1620'$$

Required length of exfiltration trench is 1620 ft.