SECTION VI SITE HYDRAULIC CAPACITY ANALYSIS

TABLE OF CONTENTS

SECTION VI SITE HYDRAULIC CAPACITY ANALYSIS

TABLE OF CONTENTS

FIGURES

TABLES

SECTION VI   SITE HYDRAULIC CAPACITY ANALYSIS

A.   Introduction

This section discusses methods that can be used to determine the hydraulic conductivity of soils and methods available for analysis of the hydraulic capacity of a potential site of a subsurface wastewater absorption system (SWAS).

As discussed in Section II, the hydraulic capacity of a site is the first of the two basic "capacity factors" that determine the soil's ability to accept and renovate pretreated wastewater discharged to a SWAS. The second factor is the soil's renovative capacity; its ability to remove or largely attenuate the contaminants found in domestic wastewater. The hydraulic capacity factor deals with the soil's ability to accept, contain and transport the wastewater percolating from a SWAS in such a manner that renovation will occur before the percolate reaches a point of public health or environmental concern.

In the past, the terms "permeability", or "coefficient of permeability" were often used to define the soil characteristic used in the Darcy equation to determine the hydraulic capacity of a soil. Hydraulic Conductivity (K) is the current designation of a soil's ability to transmit water (NRCS -1998). It is a mathematical coefficient that relates the rate of water movement to the hydraulic gradient and is one of the terms in Darcy's law on flow through a porous medium.

To confound this issue of terminology, hydraulic conductivity is determined in the laboratory by the use of a "permeameter", and ASTM uses the terms "Permeability" in the title of one Standard Test Method (D2434) and "Hydraulic Conductivity" in the title of another method (D5084) to determine the same soil property. Also, a soil that has very little to no capacity to transmit water is described as "impermeable". Further, the former Soil Conservation Service (SCS) published soil surveys for each of the eight counties in Connecticut that provided data for soil permeabilities that were actually values for hydraulic conductivities. (However, the information in these soil surveys is presently being updated by the NCRS and now refers to the former SCS soil permeability data as "Hydraulic Conductivity" data.)

The original "Seepage and Pollutant Renovation" document issued by the Department (Healy and May - 1982, rev. 1997) also used the term permeability as synonymous with hydraulic conductivity. This section is primarily an update of the original section on "Permeability Testing" and "Seepage Analysis" in the Healy and May document that has been in use for over two decades by many SWAS designers in Connecticut. In this update, the words "permeability" and "permeabilities" has been replaced with "hydraulic conductivity" and "hydraulic conductivities". Further, the term "seepage analysis" has been replaced with the term "hydraulic capacity analysis".

B. **Hydraulic Conductivity Testing**

The determination of soil hydraulic conductivity is critical to the evaluation of how a SWAS will work. Determining the hydraulic conductivity, or K, is a difficult task involving testing and measurement, tempered by reasonable judgment. Most engineers are familiar with methods of laboratory testing for hydraulic conductivity. These methods are often applied and valid when an earthen material is to be excavated, transported, and re-compacted to a known density. Examples of this are construction of a road base or earthen dam. The problem in discharging pretreated wastewater to subsurface soils is quite different. The effort here is to determine a range of hydraulic conductivities for in-place soils that will not be substantially altered by man. The testing culminates in a judgment process about what values are reasonable and what numbers should be utilized in design.

The process of deciding what, where, and how to test, then being confronted by a wide range of values, causes many engineers great difficulty. There will always be uncertainty arising from a best effort to understand natural phenomena and deal with them. It should be remembered that with experience, and after forming a reasonable mental picture of how a site handles water, confidence in hydraulic conductivity results will be forthcoming.

For application to the Department it is preferable to attempt to determine hydraulic conductivity by three different tests, observation, or measurements. All methods and measurements should be contained in an engineering report and clearly explained.

Inevitably during any discussion of hydraulic conductivity testing, confusion occurs over the issue of whether a percolation (perc) test is a hydraulic conductivity test. A perc test is not a hydraulic conductivity test and in fact measures no parameter of soil. Percolation tests are run with unknown boundary conditions and are greatly affected by soil moisture and capillarity. Hydraulic conductivity tests are run with defined boundaries under saturated conditions. These two tests should never be confused. The perc test is a procedure that has, by trial and error, been given an empirical relationship to the amount of wastewater that can be applied at a SWAS interface. It has limited relation to the hydraulic capacity of a site. The Department requires that site hydraulic capacity for large-scale OWRS be based on reasonable values for the soil's saturated hydraulic conductivity.

C. **Procedures for Hydraulic Conductivity Tests**

The test procedures discussed in this section are simple, practical, require low capital expenditure, and are well within the capability of any professional engineer. These procedures yield hydraulic conductivity values of suitable accuracy for the intended use. The saturated hydraulic conductivity of a soil is measured by filling the voids with water and measuring the steady rate of water flow through a soil sample of known dimensions under known hydraulic conditions.

There are some field situations under which water flows through unsaturated soil, such as when rain falls on a soil at a slower rate than the soil can absorb it, or during intermittent application of effluent in a disposal system when slugs of water passing through the soil are separated by unsaturated zones. Steady state saturated flow does not simulate either one of these situations; however, flow rates calculated assuming saturated steady state flow are the maximum flow rates possible for given hydraulic conditions, and can be used for design.

The rate of saturated water flow (Q = Volume/Time) through soil depends on the hydraulic gradient (i,) the cross sectional area (A,) and the saturated hydraulic conductivity (K) of the soil (Q = K i A). The hydraulic gradient is equal to the change of elevation between the upstream and downstream water surface, divided by the length of flow between these surfaces. Figure 1(a) shows these measurements for flows down hill through a layer of soil, and Figure 1(b) shows these measurements for the flow through a cylinder of soil used to measure the hydraulic conductivity.

FIGURE No. 1 a.

FLOW DOWN HILL THROUGH A LAYER OF SOIL

$$Q = K \times \frac{H}{L} \times A$$
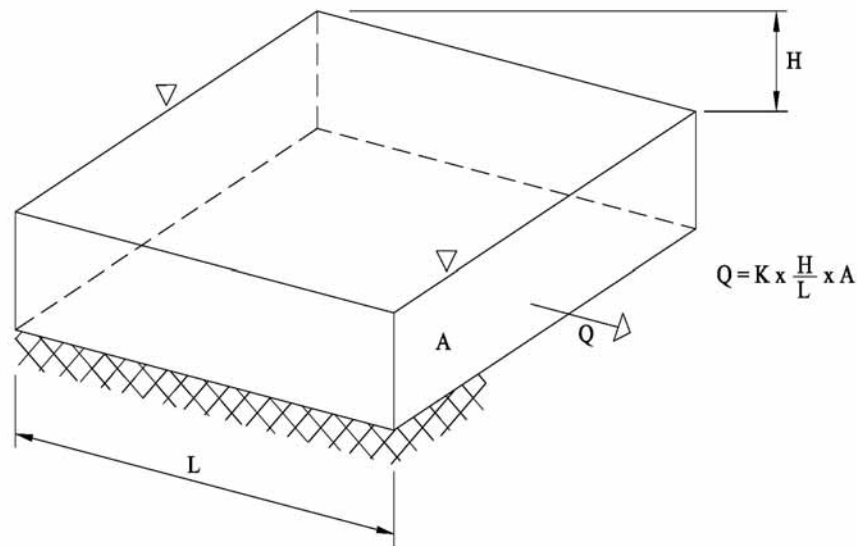
In Figure No. 1a and 1b, the hydraulic gradient, $i = H/L$

Q

Permeameter Tube

A

$$Q = K \times \frac{H}{L} \times A$$

H

Soil Section

Q

L

Overflow Pan
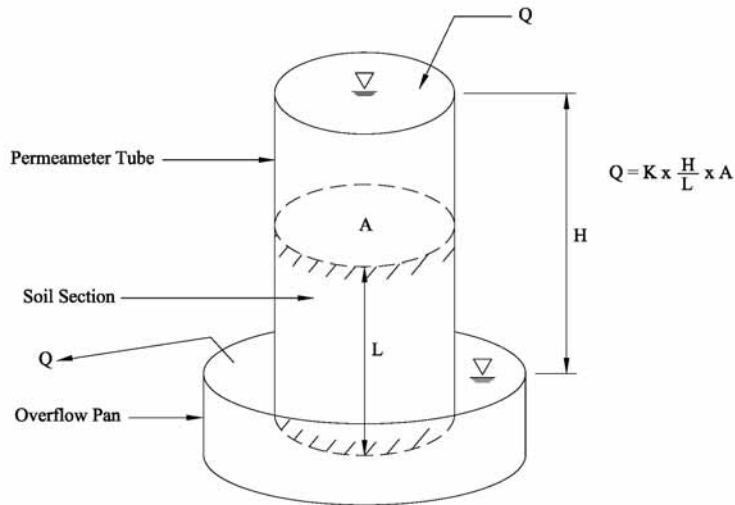
D.   Factors Determining Hydraulic Conductivity

The properties of the soil that determine the hydraulic conductivity are primarily the grain size distribution, how tightly the grains are packed, and the arrangement (layering) of the grains. Table No. 1 gives typical hydraulic conductivity values for various types of soil.

Table No. 1.

Typical Hydraulic Conductivity Values for Various Types of Soils

Soil Type	Hydraulic Conductivity K, ft/day	Cross-Section Area, in Sq. Ft. required for same flow as 2" Pipe
$1^{1}/_{2}$ " Stone	100,000.00	1.20
3/8" to No. 4 Gravel	7,000.00	2.00
Coarse Sand	1,000.00	180.00
Fine Sand	10.00	18,000.00
Silt	0.01	18,000,000.00

A small amount of silt in sand can reduce the hydraulic conductivity dramatically. Table No. 2 gives the hydraulic conductivity of a clean sand vs. different percentages of material passing the #100 sieve.

Table No. 2

Hydraulic Conductivity of Sand vs. % Passing No. 100 Sieve

% Passing No. 100 Sieve	Hydraulic Conductivity, ft/day
0.0	300.0
4.0	15.0
7.0	1.5

The tightness of packing tends to have a greater effect on fine-grained soils then on coarse-grained soils and gravels as shown in Table No. 3.

Table No. 3

Effect of Packing on Hydraulic Conductivity

Soil	Hydraulic Conductivity, ft/day	
	Loose	Dense
Silty Sand	0.30	0.03
Sandy Gravel	30.00	5.00

Many deposits of soils, particularly in New England, contain lenses and pockets of different soil, and the soil may contain fissures or cracks. The effect of these discontinuities can be very large and is destroyed by remolding (recompacting) as is shown in Table #4.

Table No, 4

Effect of Remolding on Hydraulic Conductivity

Soil	Hydraulic Conductivity, ft/day	
	Undisturbed	Remolded
Glacial Till	1.50	0.0015
Silty Sand	30.00	0.03

A relationship between hydraulic conductivity and the d_{10} grain size is shown in Figure No.2. [The d_{10} size of a soil is defined under E 1 - Grain Size Distribution]

FIGURE No. 2

K vs. D_{10}- SANDS & GRAVELS
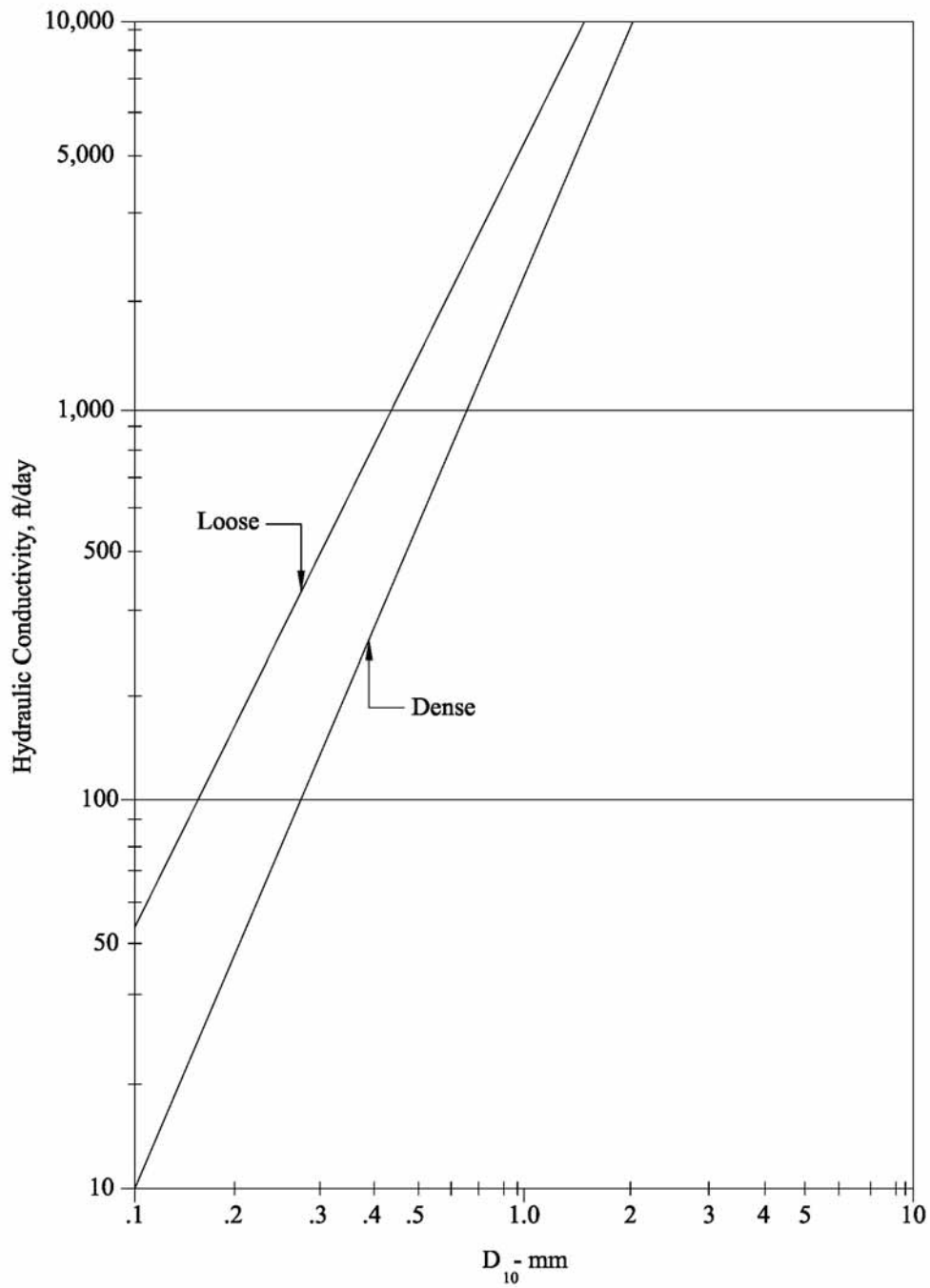
E.   Methods of Measuring Saturated Hydraulic Conductivity

There are four general methods of measuring the saturated hydraulic conductivity of a soil deposit: (1) inference from grain size distribution, (2) tests on disturbed samples, (3) tests on undisturbed samples, and (4) In Place (field) tests. Methods (3) and (4) are more appropriate for use in selecting values of K for design of a SWAS.

1.   Grain Size Distribution

Estimating the hydraulic conductivity from the grain size distribution ignores the effect of compaction or discontinuities and may give very misleading results if the percentage of fine grains is not measured carefully. The method is most accurate with soils that don't contain any silt or clay. Figure (2) gives the hydraulic conductivity of sands and gravel in ft/day versus the d_{10} size. The d_{10} size of a soil is that sieve size that only 10% by weight of the grains are smaller than. A chart such as shown provides accuracies of K within a factor of 2 or 3.

2.   Tests on Disturbed Samples

A disturbed sample of soil is re-compacted into a tube to approximately the field density, and a falling or constant head permeability test is run as shown in Figures 3a and 3b. (Tests on re-compacted soil samples also ignore the effect of soil structure and generally will give a lower hydraulic conductivity than exists in the field.) In structure-less soils, such as clean sand or gravel, this test will indicate a hydraulic conductivity that is within a factor of 2 or 3 of the field hydraulic conductivity. Note that warm water that has been cooled to 20 degrees C should be used to reduce the accumulation of air bubbles in the soil.

3.   Tests on Undisturbed Samples

Undisturbed samples must be used for hydraulic conductivity testing of all but sand or gravel due to the structure that may be present in the finer grain soils. If the water appears muddy when mixed with the soil, undisturbed samples should be used. There are a variety of undisturbed testing methods as described below.

3.a   Tube Tests

Un-cemented soil with little gravel can be sampled with a sharp edge tube 1-4" in diameter. If excessive force is needed to push the sampler into the soil, block samples should be taken. The tube that the soil is sampled with can be used as the permeameter for a falling or constant head test as shown in Figure 3a and b.

Tube tests of undisturbed samples can be useful in cohesive sands, sandy loams, and loose till where the sample can be easily taken and will not crack or fall from the tube.

The material or facilities required are as follows:

   1) A sink, or place where water can overflow.

2) A shallow pan, such as a baking dish, kitty litter pan, etc.

3) 1 1/2 - 1 1/4" thin wall tubes 6" - 12" long; plated sink drain tubes are readily available and inexpensive.

4) A bag of Ottawa testing sand or other uniform clean sand, and pieces of filter fabric that have a hydraulic conductivity considerably greater than the soil sample so that the permeability of the fabric will not affect the test results.

5) Scale or ruler.

6) Grease for tubes.

7) A supply of hot water that has been cooled to 20 degrees C. and utensils for pouring it. (Water that has been heated will have been essentially de-aerated. Care should be taken not to agitate the water after cooling to a degree that would cause its reaeration. Water containing air bubbles will interfere with the test if the air bubbles become trapped in the soil pores.)

8) A vacuum pump or aspirator if air blockage of the soil pores is suspected.

FIGURE No. 3 a

FALLING HEAD TEST

$$K = \frac{(H_1 - H_2) L}{t \; x \; \frac{(H_1 - H_2)}{2}}$$
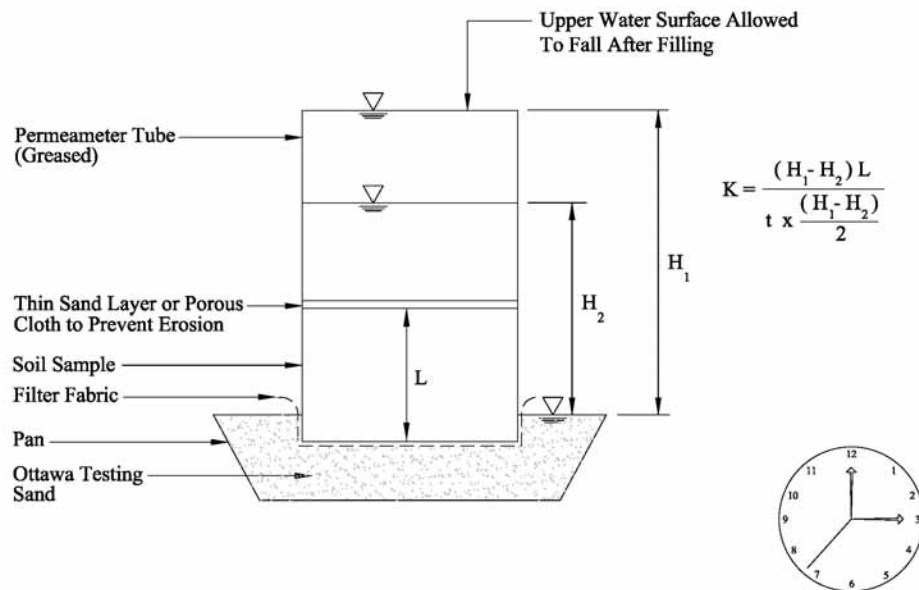
Procedure For Falling Head Test

Select the suitable soil strata for testing and collect a number tube samples from each stratum (soil horizon), the more the better. The number of samples will depend to some extent on the size of the SWAS and the nature of the soil deposits. It is recommended that the number and location of samples to be taken be discussed with Department staff before proceeding with the field sampling. Tubes should be greased internally and pushed into the soil to remove a 3 - 6" plug of soil. Tubes can then be placed upright on a bed of Ottawa sand in the shallow pan for transport. Tubes can be taken vertically or horizontally to reflect the appropriate K value for the flow regime. Back at the office, measure the length (L) of each sample and record it. Replace the tubes upright and put 1/2" of Ottawa sand on the surface of the sample. Saturate the sample and base sand with water until the shallow pan overflows and a head exists in the tube. Continue until you are sure the sample is saturated and a steady state of flow exists.

It is often difficult to determine if the sample is thoroughly saturated so that no air is blocking any of the pores in the sample. Air blockage will result in the test K value being less than the actual K value for the soil in the tube. A more valid procedure is to apply a vacuum to the tube, as given in the following procedure. Fill the shallow pan with water until the Ottawa sand is saturated and the pan overflows. Then place the samples under a vacuum of 20 inches of Mercury for a minimum of 15 minutes to evacuate all of the air in the sample pores. Continue applying the vacuum until water has been drawn up into the headspace above the sample in the tube to a height sufficient to run the test. (In cases where the sample tube is completely filled with the soil sample, it will be necessary to add an extension to the sampling tube). Then release the vacuum and measure the drop (H_1 - H_2) and the time of drop. Use the formula in Figure No. 3a to calculate K. This calculation uses an average approach velocity and an average head loss but is more than accurate enough, and more exact calculations are not necessary.

Procedure for Constant Head Test

Constant head tests (figure 3b) may be used for samples of soils of high hydraulic conductivity where the drop in head in a falling head test is too rapid to record accurately. They are run much like the falling head test but more apparatus is needed. A valving system must supply water at low constant rates (Q in) to maintain H. Alternately the overflow from the tube must be collected and not allowed to enter the pan overflow. Pan overflow (Q out) must be collected and accurately measured in a graduated cylinder.

FIGURE No. 3 b

CONSTANT HEAD TEST

$$K = \frac{Q}{\frac{H}{L} \times A}$$
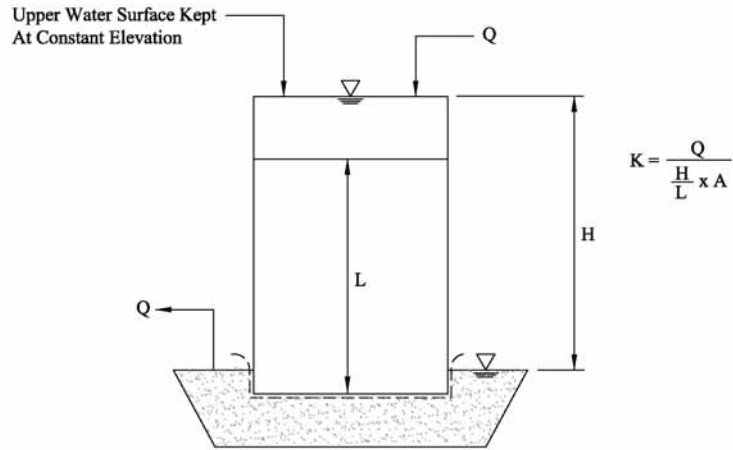
3.b Undisturbed Block Samples

It is often possible to use a shovel or knife to cut out undisturbed blocks of cemented sand and gravel or soils containing silt or clay. The blocks are trimmed with a knife to measurable dimensions and placed in the permeameter and tested as shown in Figure No. 4. This test is very useful in compact tills and can measure vertical or horizontal K.

FIGURE No. 4

UNDISTURBED BLOCK SAMPLES TAKEN FROM TEST PIT.
(GLACIAL TILL OR CEMENTED SAND)
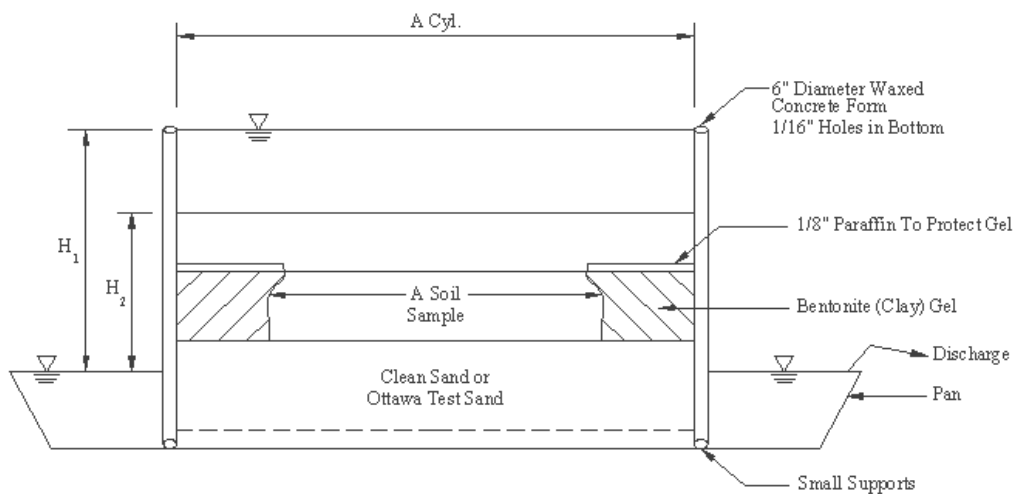
Materials and Procedures for Undisturbed Block Sample Testing.

1. Block of soil taken from a soil horizon is trimmed to measurable dimensions and bedded 1/8" on clean well-graded concrete sand.
2. Concrete or Ottawa sand must be coarse enough so that K is 20-50 times the anticipated K of the soil, and fine enough so that gel won't penetrate it.
3. Molten paraffin is poured over gel to protect if from erosion by water above.
4. Test is run in a similar manner to the tube test in figures 3a and 3b.

$$K = \frac{Q}{H/L \times A \text{ soil}} = \frac{\text{Surf. Velocity} \times A \text{ cylinder} \times L \text{ soil}}{H \text{ avg.} \times \text{Area soil}} , \text{ where:}$$

$$\text{Surface Velocity} = \frac{\text{drop of water surface}}{\text{Time}}$$

$$H \text{ avg.} = \text{Average head during test period} = \underline{H_1 + H_2} \text{ time}$$

4. In-Place Field Tests

The primary advantage of in-place tests is that the mass of soil being tested is undisturbed and generally large. In order for this test to be meaningful, however, the dimensions of the strata in which the test is run must be accurately measured, utilizing test pits and in some cases borings.

(a) Pit Bailing Tests

The test makes use of the test pit that is dug to determine the depth to the water table and the position of the various soil strata. The water table must be within 8-10 feet of the ground surface to carry out the test, and the hydraulic conductivity of only the soil below the water table is measured. The test procedure is based on gravity well theory and is illustrated in figure No. 5 on the following page. The radius of drawdown can be assumed equal to four times the effective radius of the test pit without significant error. The depth to impermeable strata should be estimated, although in sites where the depth to an impermeable strata is not well defined it can be assumed equal to the depth of the pit, and the hydraulic conductivity will be slightly over estimated. There are two ways to carry out this test. The first entails measuring the rate of water level rise in the pit when it is first dug and returning a day or two later to establish the equilibrium water level. The second entails digging the test pit, waiting several days for the water level to stabilize, lowering the water level by bailing, and measuring the rate at which it rises. The water level should be lowered at least one foot for reliable measurements. [NOTE: When this test is run in sandy soils and the test pit barely penetrates the water table, the results will be inaccurate.]

Figure No. 5

PIT BAILING TESTS
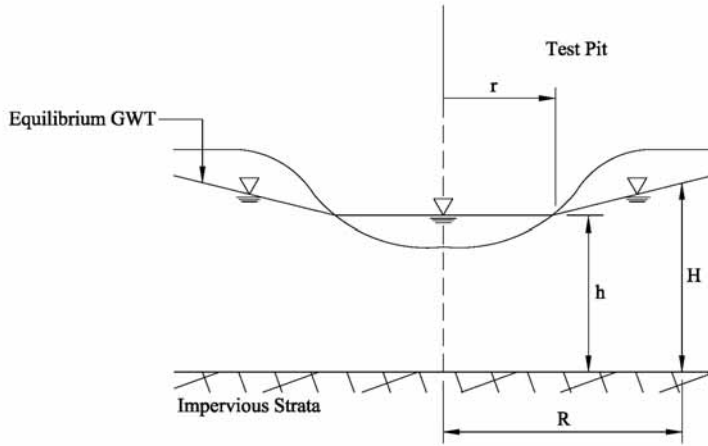
$$Q = \frac{\pi K (H^2 - h^2)}{\ln R/r};$$

Q = Rate of water level rise in pit x water surface area in pit.

H and h can be taken from bottom of pit.

ln R/r can be assumed = 1.4

Procedure

1) Excavate test pits providing a suitable step or sloped end to facilitate measurement.
2) Establish a fixed point to measure depth to equilibrium water table, time related water table and pit bottom.
3) Measure depth to pit bottom and equilibrium water table (highest seep or stable level).
4) Measure width of pit.
5) At suitable intervals measure and log:
    a) Time
    b) Depth of water surface
    c) Length and width of water surface.

For example, assume:

Pit depth = 56.5 inches
Equilibrium water table is 17.25 inches below ground surface.

Time	Depth to water level	Area of water surface
4:10	55.5"	1.5 x 3 = 4.5 ft^2
4:20	54.5"	1.5 x 4 = 6 ft^2
10 min.	1 inch	Avg. area = 5.25 ft^2

$$Q = 5.25 \text{ ft}^2 \text{ x } \frac{1}{10 \text{ min}} \text{ x } \frac{1}{12} \text{ ft} = 0.044 \text{ ft}^3/\text{min.}$$

h = 56.5 - 54.5" = 2 inches = 0.17 ft
H = 56.5 - 17.25" = 39.25 inches = 3.25 ft . (H^2-h^2) = 10.5 ft^2

$$K = \frac{1.4 \, Q}{\pi \, (H^2-h^2)} = \frac{1.4 \text{ x } 0.044 \text{ ft}^3/\text{min}}{\pi \text{ x } 10.5 \text{ft}^2} = 2.7 \frac{\text{ft.}}{\text{day}}$$

(b) Auger Hole Bailing

The auger hole bailing test was developed by agricultural engineers to determine the field hydraulic conductivity of soil where the water is near the ground surface. The test is an accurate test and can be done by one person with a 4-6" auger, and a bailing apparatus that will fit down the hole. The hole should be dug to a depth below the water table of at least five hole diameters. The water level in the hole is allowed to rise to the equilibrium elevation. The water level is then lowered several hole diameters by bailing, and the rate of water level rise is measured. This test, because of the hole shape, measures primarily the horizontal hydraulic conductivity, which is generally what is desired for hydraulic capacity analysis, as the subsurface flow away from the SWAS is predominately in a horizontal direction.. The method of computing the hydraulic conductivity is illustrated in Figures 6a and 6b.
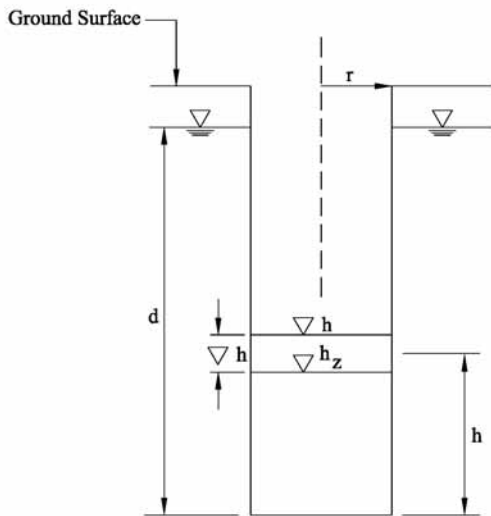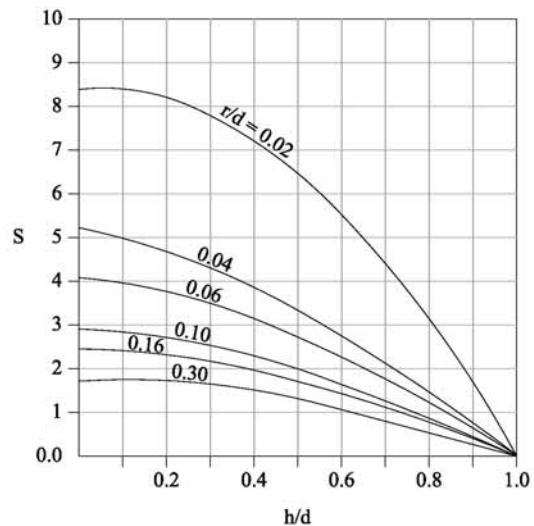
AUGER HOLE BAILING

Figure No. 6 a   Figure 6 b.

K = [888 r/(S x d)] x (ΔH/Δ t)

Where;

K – Hydraulic Conductivity, ft/day
r - ft
d - ft
Δh - ft
Δt - minutes
S - a factor read from Figure 6b

(c)   Well Pumping Tests

These tests are more complex, normally involving pumping wells at a constant pumping rate until steady state, or near steady state conditions are achieved and deriving K from the results. These tests should be conducted under direction of someone who has the requisite hydrogeologic training and experience. The Department should be consulted before proceeding with such tests.

(d)   Slug Tests

Because of their simplicity relative to well pumping tests, a number of slug tests can usually be performed with less effort and expense than a single well pumping test. The well pumping test, if properly performed and results properly analyzed, may provide a higher degree of accuracy in determining K while testing a larger aquifer volume as compared to a slug test. However, simple techniques yielding reasonable results are often preferable to more complicated techniques given the heterogeneous nature of soil.

A slug test consists of measuring the recovery of head vs. time in a well after a near-instantaneous change in head (water level) at that well. The sudden change in head can be accomplished by rapidly introducing a solid object (the slug) into the well or rapidly removing it from the well, causing an abrupt increase (decrease) in the water level in the well. Following this sudden change, the water level in the well returns to its original level in response to the gradient imposed by the sudden change in head. Timed measurements of the water level (head) changes, which are termed the response data, can be used to estimate the hydraulic conductivity of the soil formation through comparisons with theoretical models of test responses (Cedergren - 1989). It is assumed that the soil medium in which the slug test is conducted is homogeneous and isotropic.

Slug tests are normally used to estimate the value of K where the strata that will convey the treated wastewater away from the SWAS extends both beneath and above the seasonal high ground water table. The value of K determined for the soil below the water table will thus be applicable to determining the flow in the soil above the seasonal high water table that will occur due to the rise in the water table (mounding) resulting from the discharge of the wastewater to the subsurface.

Slug tests will also provide a means of estimating the hydraulic conductivity of the soil when pit bailing, auger hole bailing, or collection of tube or block samples is impracticable. This is usually the case in relatively deep aquifers in glacial stratified drift or outwash deposits. These tests are only useful if the monitoring well extends below the water table.

Slug-in tests consist of rapidly introducing the slugs into the groundwater monitoring well. Slug-out tests consist of rapidly removing the slugs from the groundwater monitoring well. It is recommended that a minimum of two slug-in tests and two slug-out tests be performed for each groundwater monitoring well location with three of each test being preferable. If possible, response data should be reviewed in the field to determine if smooth slug introduction or removal was achieved (no oscillation in water level). Figure No. 7 depicts the conditions for a slug-out test.

While it is possible to conduct depth to water level measurements in low hydraulic conductivity environments (glacial till or soils with a high fines content) with a typical well probe, high hydraulic conductivity environments (stratified drift and outwash deposits) typically require the use of a fixed transducer and water level data recorder. Most water level data recorders record data in linear or logarithmic intervals with logarithmic data recording being preferable in very high permeability environments (coarse grained stratified drift), due to the small time increments involved.

FIGURE No. 7

SLUG-OUT TEST CONDITIONS
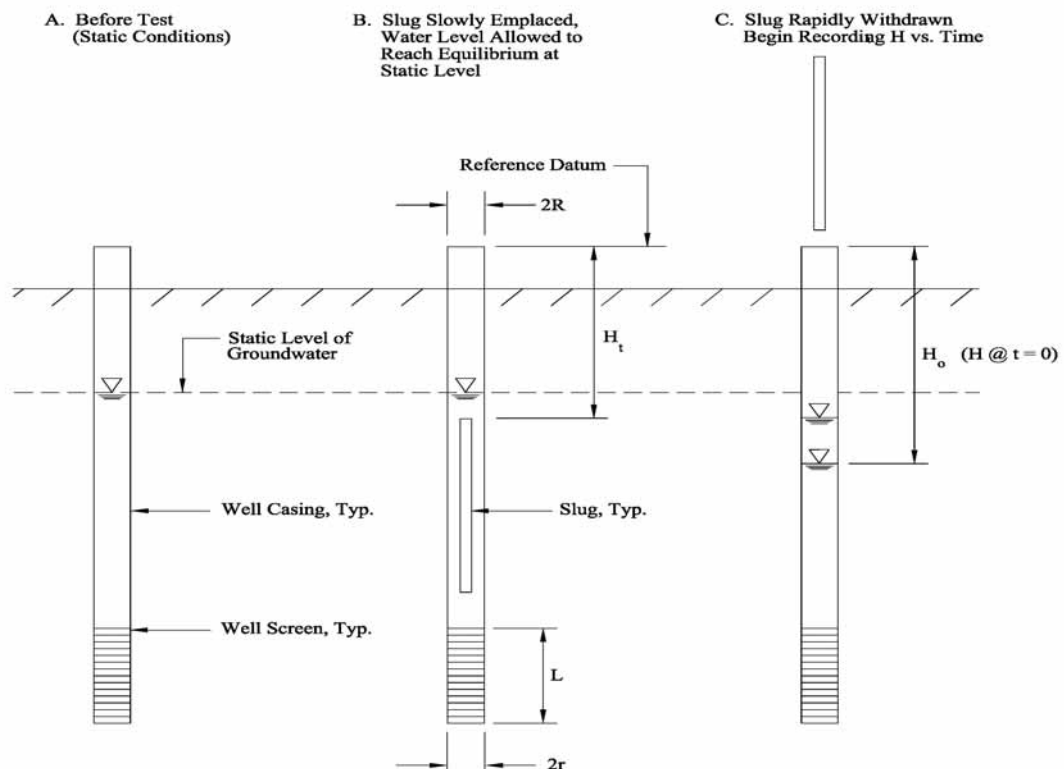
The groundwater monitoring wells should be properly developed prior to conducting slug tests by surging the screen interval with a surge rod to ensure that any skin effects are removed. (The "skin" is the altered soil conditions in the immediate vicinity of the well screen, such as can be caused by the mobilization of near-well fine materials during construction of the well. For example, auger rotation and retrieval can smear fine materials on the wall of the auger hole.) Surge rods can be constructed of threaded steel stock with an assortment of rubber, nylon, brass and stainless steel washers and the groundwater monitoring wells are developed by repeatedly raising and lowering the surge rod attached to a slug.

While numerous slug test equations and references exist, groundwater monitoring well geometry relative to formation changes (i.e. confined formation, fully penetrating screen, partially penetrating screen, unconfined formation) and nature of formation dictate the correct equation to utilize in estimation of hydraulic conductivity. The U.S. Department of the Navy, Naval Facility Engineering Command (1982) provides tables and figures for hydraulic conductivity determination based on borehole condition, formation shape factor (geometry of the groundwater monitoring well relative to the formation position), and applicability. Some additional reference publications include Cedergren (1989), Butler (1997) and ASTM (1999). A Site Assessment CD in multi-media format, primarily developed to investigate fuel releases at underground storage tank sites, is available from the Department at modest cost that contains spreadsheets that can be used to analyze slug test data obtained under various soil and well geometry conditions to determine hydraulic conductivity. Other computer programs useful for processing slug test data are available from commercial sources.

For simplicity in analysis it is preferable to construct the groundwater monitoring well such that the induced recharge (slug-in) or drawdown (slug-out) results in the water surface always being within the solid case interval. Butler (1997) recommends that the primary direction of flow during a series of slug tests should be from the formation into the well (Slug-out test). Flow from the well into the aquifer formation during a slug-in test will often lead to decreased estimates of hydraulic conductivity as a result of mobilized fine material being lodged deeper in the formation. Thus, where both slug-in and slug-out tests are performed on the same well, the slug-out tests should be performed first. It is important to note that the method is dependent on the response data being unaffected by the storage properties of the well-formation configuration. Therefore, it is important that the normalized semi-log response data (H/Ho) vs. time is linear.

Aquifer slug tests are valuable tools in determining an estimate of hydraulic conductivity. Slug tests, when properly performed and data properly analyzed, allows for more accurate determination of in-situ formation hydraulic conductivity than tube tests, since the volume of soil for which K is determined is much greater in slug tests. However, the test results are only representative of the average hydraulic conductivity of the portion of the aquifer adjacent to the open interval of the well. Therefore, slug tests should be conducted in a number of ground water observation wells to obtain representative site-specific data. The results obtained from slug tests should be checked against several other methods used to estimate hydraulic conductivity.

Similar to well pumping tests, slug tests should be conducted under the direction of someone who has the requisite hydrogeologic training and experience. It is recommended that the Department be consulted before proceeding with such tests if the results are to be submitted to the Department as part of a permit application.

5.   Determination of Hydraulic Conductivity from Site Performance.

Observations of ground water elevations, slope and boundary conditions can be utilized to determine hydraulic conductivity. In this type of testing the response of a site to a known recharge can be utilized to estimate hydraulic conductivity. Two examples of this determination are included in Subsection F - Basic Site Hydraulic Capacity Analysis, Case 2. A and Case 2. B.

6.   Verifying Hydraulic Conductivity

Various soils texts provide tables giving ranges of hydraulic conductivity for various typical soils groups. The soils map prepared by the United States Department of Agriculture Soils Survey list engineering properties, including hydraulic conductivity ranges for the A, B and C horizons of mapped soils. This information can and should be utilized as a check to see if the values you have determined are reasonable. The final check on a hydraulic conductivity value comes in its application in a hydraulic capacity analysis. If a reasonable job has been done identifying boundary conditions and hydraulic gradients, then the K value obtained should project the actual predevelopment site conditions. For example, if you have tested during high ground water and found it 6' below grade then a predevelopment hydraulic capacity analysis should come close to that condition. If your analysis shows ground water should be at 6" below grade than an error has been made in the hydraulic conductivity value or assumed boundary conditions.

7.   Number of Tests Required

The number of tests that should be made in order to estimate K values will depend to a great extent upon the test methods. Because of the heterogeneity of soils in general, a significant number of tests should be made when estimating K from results of tube tests, block tests or grain size analyses, since the soil samples used in these tests are representative of only a very small portion of the soils on a site. On the other hand, a fewer number may suffice when field tests, such as auger hole tests, slug tests, etc. are utilized or when observing the behavior of a site before, during and after a significant rainfall event, as such tests are usually representative of a much greater volume of soil. In general, the rule should be the more the better. If the results of initial testing are inconclusive, additional sampling and testing should be performed.

8.   Estimation of Mean Hydraulic Conductivity

It has been shown that, for a random distribution of K values, the "true" mean lies between the arithmetic and harmonic means of the individual K-values. The geometric mean also lies between the arithmetic and harmonic means. The geometric mean is used when the distribution of values of the sample population is skewed from the normal distribution, and prevents one or two high values from giving overestimates of the mean value.

The geometric mean (G) is determined as follows:

G = Nth root $(X_1X_2X_3..X_N)$ which is the Nth root of a product of N values of X.

For example, the geometric mean of the numbers $3.20 + 5.10 + 8.30 = (3.20 \times 5.10 \times 8.30)^{1/3} = (135.4)^{1/3} = 5.135$

The geometric mean K of a number of individual K values can also be obtained using the following procedure:

Log X = [$\sum$ (log x)]/N, where X is the geometric mean, x is each individual value, and N is the number of values.

Thus, the logarithm of each value of K is determined, the resulting logarithms of all of the values are summed, and this sum is divided by the number of values to yield the logarithm of the geometric mean K. The antilog of the logarithm of the geometric mean is the geometric mean of all of the values. Either Log_{10} or Ln (natural log) can be used to calculate the geometric mean.

A more detailed explanation of determining the geometric mean is given in Appendix C - Selecting Hydraulic Conductivity Values for Design.

9.   Utilization of Hydraulic Conductivity Values

After testing with several methods the engineer is confronted with a range of hydraulic conductivity values. After checking values and due consideration, "outliers", extremely high or low values, can normally be discarded. This leaves the designer with a reasonable range of values for use in three areas of design. These areas are:

1)   Determination of system size by the long-term acceptance rate method.

2)   Determination of the capacity of the site to handle the discharge volume (hydraulic capacity analysis).

3)   Calculation of the capacity to renovate wastewater (definition of the zone of influence).

For example, when performing a hydraulic capacity analysis or selecting a value for long term acceptance rate, or LTAR, (see Section X), a conservative value in the lower 50 percentile of the range of values should be used and certainly should not exceed the geometric mean. On the other hand, when computing the zone of influence (travel time from the SWAS to the closest points of concern), a conservative value from the high end of the range of K values should be used. It is recommended that the Department be consulted with respect to the values of K to be used for hydraulic capacity analysis, LTAR, and travel time after the results of testing for K by several methods have been determined. (See Section X and the Design Standards for additional information on selection of K values for design.)

All of the hydraulic conductivity tests described measure the saturated hydraulic conductivity of the soil that allows, in conjunction with hydraulic capacity analysis, a calculation of the maximum rate that water can be carried by the soil strata. As with any testing program, the more time that is spent the better the results, but the investigator should never have such complete faith in the results of a few tests that he or she neglects to observe other bits of information that are available. This is particularly true of hydraulic capacity testing. Observation of the reaction of a site to rain may provide a more complete picture of the hydraulic conductivity of a soil deposit than a hundred hydraulic conductivity tests.

F.   Basic Site Hydraulic Capacity Analysis

1.   Introduction

The hydraulic capacity of a site depends on the hydraulic conductivity and dimensions of the soil, the slope and position of the ground water table beneath and near the site, and the position of any impermeable boundaries beneath or around the site. When designing a SWAS, it should be determined that the soil deposit has adequate hydraulic capacity to carry the septic tank effluent below ground surface for a sufficient period of time, and for a sufficient distance, to bring the pretreated wastewater into compliance with the required ground water quality standards of the Department before it reaches a point of concern. This hydraulic capacity should be available under all conditions, so that any hydraulic capacity analysis of a potential SWAS site must consider all water that flows into the site, including ground water, surface water, precipitation, and the pretreated effluent. Water must be able to flow away underground faster than it enters the area of the SWAS. While the examples contained in this section do not address reserve hydraulic capacity, adequate reserve capacity shall be provided in the system design. This may be done either by applying an additional factor of safety to the design flow in the hydraulic capacity analysis or by provision of additional area for installation of a SWAS elsewhere on-site.

The hydraulic capacity analyses that follow are based on two important assumptions. First, it is assumed that the SWAS trenches, galleries or chambers will be ponded to a depth sufficient to push the effluent through a layer of reduced hydraulic conductivity at the soil interface (the biomat) at some known rate.

Second, it is assumed that there will be saturated flow at some depth below the SWAS during the wet season when the ground water table is highest. These two assumptions are illustrated in Figure No. 8. [Note: While the examples discussed herein are generally based on a SWAS for a single-family dwelling, the principles applied are also applicable to large-scale systems.]

FIGURE No. 8

SECTION THROUGH SUBSURFACE WASTEWATER ABSORPTION SYSTEM
(Section shows typical trench type of SWAS)
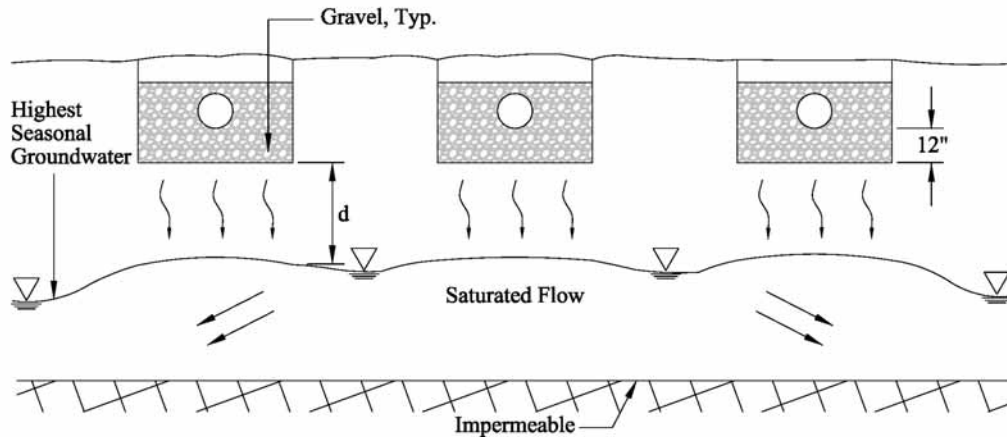
Assumptions for analysis

1. Effluent ponded in trenches, galleries or chambers

2. Saturated flow at depth d below trenches, galleries
    or chambers.

Neither of these are desirable operating conditions; however, based on these assumptions, the maximum capacity of the SWAS under the worst condition (high GWT) can be calculated. If the SWAS is loaded at some fraction of this maximum capacity, the soil immediately below the system will not be saturated even during the wet season, and the effluent will not pond to one foot in the trenches, galleries or chambers.

The choice of that fraction of the maximum capacity that can be used for normal operation will depend on the consequences of failure, which generally occur in the form of toilet backup or surface flooding. Failing systems can contaminate surface or ground water, create nuisance conditions, present a source of infectious material, and can have a severe financial impact.

2.   Analysis Based on Ground Water Levels During Wet Season

A valuable and simple hydraulic capacity analysis of a site involves observation of the ground water levels in standpipes or pits during the wet season. During this period, ground and surface water inflow is a maximum and there is minimum evapotranspiration. If the ground water table remains well below the ground surface during the wet season, the site can probably absorb the additional water from a SWAS.

Calculations of the hydraulic capacity of a site can be made based on ground water levels during wet weather conditions, if test pits are dug to identify various soil strata and measurements are made of the rate of ground water table drop between rainfalls.

Following are examples in which observations of the ground water levels and depths to impermeable strata allow calculation of the soil hydraulic conductivity, site hydraulic capacity, and the effectiveness of curtain drains.

Case A - Figure No. 9 shows a 3-D view of a one-acre hillside site with 11 feet of soil overlying an impermeable stratum on a 6% slope.

FIGURE No. 9

3-D VIEW OF HILLSIDE SITE
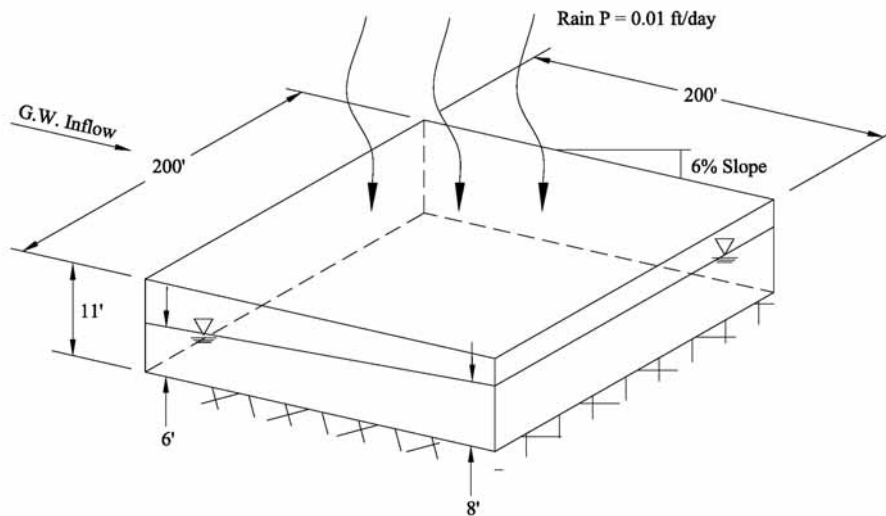
Test pits show that the GWT is 6 feet above the impervious strata at the upper end of the site, and 8 feet above the impervious strata at the lower end. The increase in the depth of flowing water due to rainfall (8'-6') allows calculation of the hydraulic conductivity of the soil, which in turn allows calculation of the increased depth of the flowing water that would occur due to the discharge from a house. Knowledge of soil hydraulic conductivity, slope and depth of flowing water allow the calculations of the effect of a curtain drain at the upper boundary of the site.

a) Assume the inflow from rain averages 0.01* ft/day. The total volume of inflow = 200 ft. x 200 ft. x 0.01* ft/day = 400 ft^3/day

The hydraulic gradient = 0.06 ft/ft. The increase in flow area due to the rainfall = 2 ft (depth) x 200 ft.(width) = 400 sq. ft.

Thus, Hydraulic Conductivity, $K = \dfrac{Q}{I \times A} = \dfrac{400 \text{ ft}^3/\text{day}}{0.06 \text{ ft/ft} \times 400 \text{ sq. ft.}} = 17$ ft/day

* In actual design, the inflow from rainfall should reflect the amount of rainfall that actually infiltrates and reaches the ground water.

b) Assume a discharge from 3 bedroom house = 40 ft³/day and that this flow is discharged to a SWAS situated across the full width of the lot. The increase in depth of the flowing water is calculated as follows:

$$A = \frac{Q}{K \times i} = \frac{40 \text{ ft}^3/\text{day}}{17 \text{ ft/day} \times 0.06 \text{ ft/ft}} = 39 \text{ ft}^2$$

$$\text{Depth Increase} = \frac{39 \text{ ft}^2}{200 \text{ ft}} = 0.2 \text{ ft} \ \ (\text{Area/width})$$

c) Ground water inflow from uphill = K x i x A = 17 ft/day x 0.06 ft/ft x 6 ft x 200 ft = 1224 ft³/day. Intercepting all incoming ground water with curtain drains and bypassing it would increase site capacity by 1224 ft³/day.

In considering this example you must think about the figure of 0.01 ft of rain/day. This figure represents annual average rainfall divided by 365. In this case you have tested the site during the spring high ground water period when evapotranspiration is at a minimum but frost is gone. In this case the figure is probably reasonable. At other times of the year when evapotranspiration occurs, the analysis can be made more conservative by using a lower or more refined rainfall infiltration rate (A more detailed discussion of the infiltration of precipitation into the subsurface is given in G.2 of Section X)

Case B - Figure No. 10 shows the cross section of a level site. Observation of the water level in standpipes indicates that the highest seasonal GWT is 4 ft from the ground surface and that between rains in the spring, the ground water table drops 2 inches per day. Although there may be no visual indication of how the water is leaving the site, downward or laterally, since evapotranspiration is near zero, the water must be flowing away underground.

The rate water leaves the site can be calculated by multiplying the rate of water table drop by the area involved, times the drainable porosity. Drainable porosity is that volume of water that drains by gravity from the soil divided by the total volume of the soil.
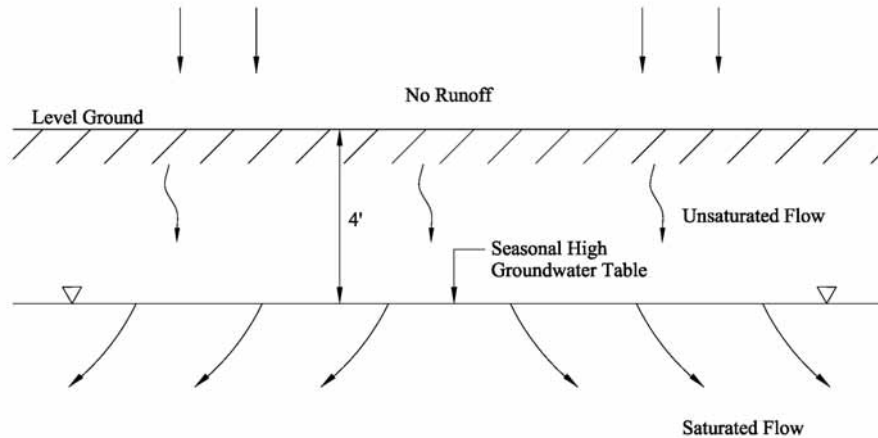
TABLE No. 5

APPROXIMATE VALUES FOR DRAINABLE POROSITIES

coarse sand - 0.25
fine sand   - 0.15
silt   - 0.10
clay   - 0.05

FIGURE No. 10
SECTION THROUGH LEVEL SITE
RAIN = 0.01* ft/day

* As noted previously, this value should be adjusted on a case-by-case basis as necessary to reflect actual infiltration.

Procedure

Between rains in spring, GWT drops at rate of 2 inches/day.
If drainable porosity = 0.1,
A drop of 2 in./day = 0.2 in. of water/day = $\frac{0.2 \text{ inches}}{12 \text{ inches/ft}}$ = 0.017 ft/day.

This is the rate at which water is leaving site. Thus, effluent discharge plus rain infiltration must be not greater than 0.017 ft/day.

Assuming an average daily rainfall infiltration of 0.01 ft/day, the average effluent discharge rate can be not greater than 0.017 ft/day - 0.01 ft /day = 0.007 ft/day.

The area needed for a wastewater discharge of 40 ft^3/day = $\frac{40 \text{ ft}^3/\text{day}}{0.007 \text{ ft/day}}$ = 5700 ft^2.

This could be provided, for example, by a SWAS footprint = 57' x 100'.

The measurement or estimate of the drainable porosity of the soil permits a calculation of how fast water leaves the site.

Effluent can be discharged at a rate such that the combined inflow of effluent and rain does not exceed the rate that water can leave the site. This analysis cannot be done in a vacuum however. If under certain intense rainfall conditions the site is saturated to grade or saturated within a reasonable distance from the system then effluent will break out during that period and at that point. The analysis is valid if the site has been observed for a time during high ground water and the maximum groundwater table elevation for a 2-week period is several feet below grade.

Both of these cases illustrate how, by careful observations over a period of time, the hydraulic capacity of a site can be determined without any sophisticated measurements or calculations. This type of analysis, if possible, provides the most reliable results because it considers all the hydrologic conditions of the site. However, such analyses should be carefully made and include all variables in order for their results to be reliable, and the results should be carefully checked with other methods presented in this section.

3.   Analysis Based on Soil Hydraulic Conductivity and Topography

If it is not possible to monitor a site during the wet season, it is still possible to deduce the hydraulic capacity based on ground water levels inferred from mottling, soil strata identified in test pits, and hydraulic conductivity measurements.

As with any analysis, the inflow from all sources must be considered. Using approximate two-dimensional and three-dimensional hydraulic capacity analysis, and hydraulic conductivity data, it is possible to predict how a site will react to the inflow of effluent during the wet season. The greatest possible error is not in the actual calculations, but in the hydraulic conductivity values used and boundary conditions assumed. The Northeast is noted for the non-homogeneity of its soil deposits and this is particularly true of hydraulic conductivity. The various soil deposits both in and around the site must be accurately identified and evaluated, and the highest elevation of the ground water must be estimated.

The following two situations show how two and three-dimensional hydraulic capacity analysis can be used.

FIGURE No. 11 - (Case C)

ANALYSIS BASED ON HYDRAULIC CONDUCTIVITY AND TOPOGRAPHY
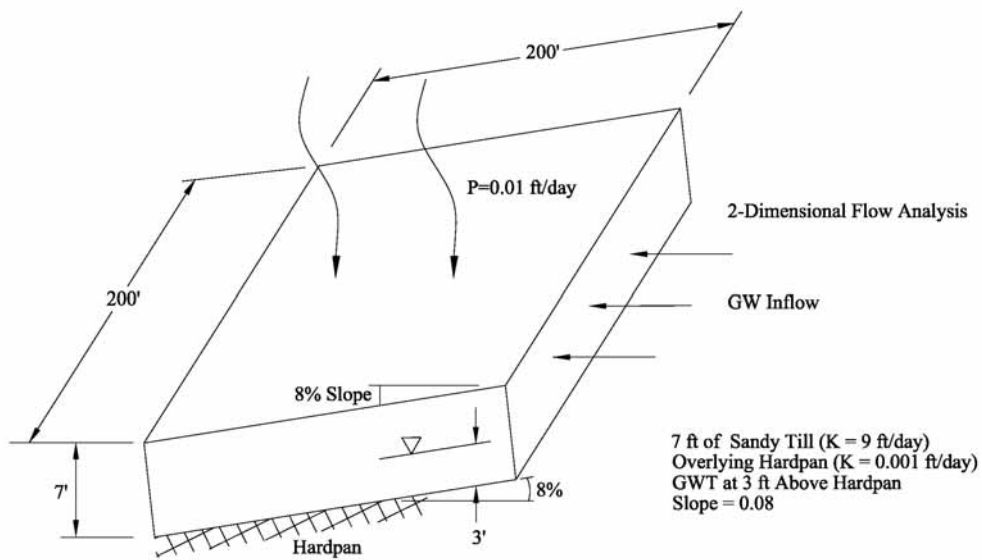
Note: Hardpan is a word often used to describe dense till.

Procedure – Case C

Ground water inflow (K x i x A) = 9 ft/day x .08 ft/ft x 3 ft x 200 ft = 432 ft^3/day (Q inflow). Rain inflow = 200 ft x 200 ft x 0.01 ft/day = 400 ft^3/day (Q rain)

Outflow = Inflow = 432 ft^3/day + 400 ft^3/day= 832 ft^3/day.

GWT Down Hill

$$\text{Flow area required} = \frac{Q}{K \times i} = \frac{832 \text{ ft}^3/\text{day}}{9 \text{ ft/day} \times .08 \text{ ft/ft}} = 1155 \text{ ft}^2$$

$$\text{Depth of Flow} = \frac{1155 \text{ ft}^2}{200 \text{ ft}} = 5.8 \text{ ft above hardpan.}$$

An additional 40 ft^3/day from a house discharged into a single 200-ft. trench across the lot would raise GWT by:

$$\frac{40 \text{ ft}^3/\text{day}}{9 \text{ ft/day} \times .08 \text{ ft/ft} \times 200 \text{ ft}} = 0.28 \text{ ft. Therefore; the water table would be}$$

0.28 ft + 5.8 ft = 6.1 ft above hardpan, or 0.9 ft below ground surface.

If the SWAS consisted of four 50-ft trenches side by side, GWT would rise:

$$\frac{40 \text{ ft}^3/\text{day}}{9 \text{ ft/day} \times 0.08 \text{ ft/ft} \times 50 \text{ ft}} = 1.1 \text{ ft}$$

1.1 ft + 5.8 ft = 6.9 ft above hardpan; thus the water table would be essentially at the ground surface.

FIGURE No. 12 (CASE D-1)

PROCEDURE FOR 3-DIMENSIONAL HYDRAULIC CAPACITY ANALYSIS
USING WELL RECHARGE EQUATION

(Note: See page 35 for definitions of terms in Well Recharge Equation)

Well Recharge Equation

$$Q = \frac{\pi K (H^2 - h^2)}{\ln R/r}$$
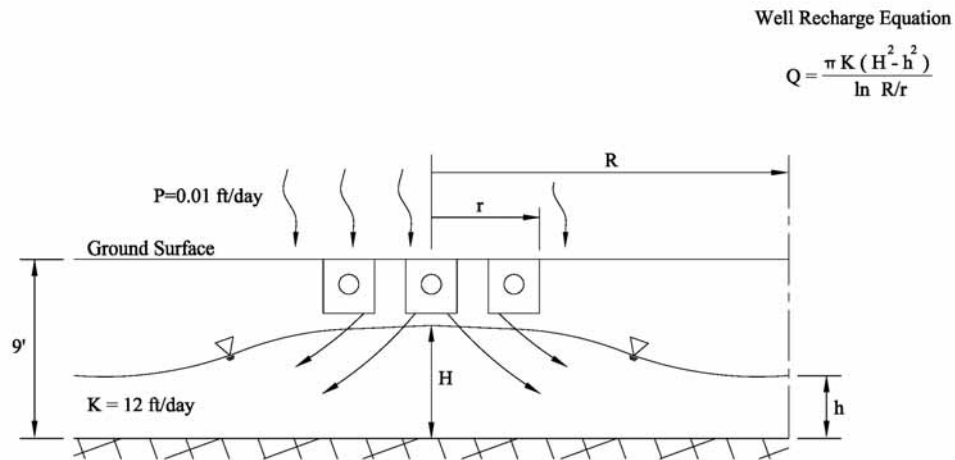
Case D-1

R = 100', SWAS r = 50' (This is the radius of a circular area having the same bottom area as the actual SWAS.)

Rain = 200 ft x 200 ft x 0.01 ft/day = 400 ft^3/day = Q rain

ln R/r = 0.7, h = 4 ft, What is H?

$$(H^2 - h^2) = \frac{Q \times \ln R/r}{\pi K} = \frac{400 \text{ ft}^3/\text{day} \times 0.7}{\pi \times 12 \text{ ft/day}} = 7.4 \text{ ft}^2$$

$(H^2 - 4^2) = 7.4 \text{ ft}^2$; $H^2 = 7.4 \text{ ft}^2 + 16 \text{ ft}^2 = 23.4 \text{ ft}^2$; H = 4.8 ft

GWT due to rain will be 9 ft - 4.8 ft = 4.2 ft below ground surface.

FIGURE No. 13 (CASE D-2)

PROCEDURE FOR 3-DIMENSIONAL HYDRAULIC CAPACITY ANALYSIS
USING WELL RECHARGE EQUATION
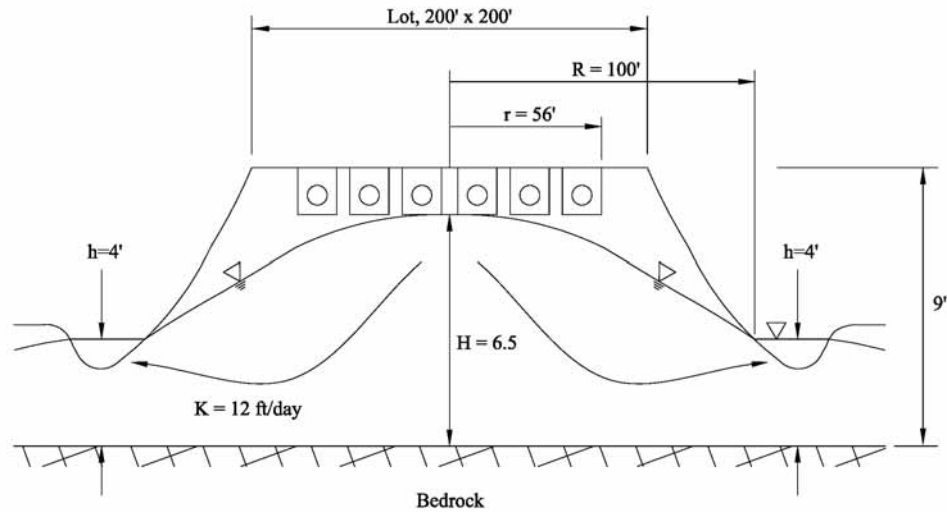
Case D-2

What is the capacity of a 100' x 100' SWAS that is 2.5' deep and has an area equivalent to a circle with a radius of 56 feet, if the GWT is allowed to mound up to bottom of SWAS? (H = 6.5')

$$Q = \frac{\pi K (6.5^2 - 4^2)}{\ln 100/56} = \frac{\pi \times 12 \times (42.25 - 16)}{0.58} = 1,690 \text{ ft}^3/\text{day}$$

(Rain)

Capacity of SWAS = 1690 ft^3/day - 400 ft^3/day = 1,290 ft^3/day

Capacity of SWAS by bottom interface area required = 0.6 GPD/ft^2 x 100 x 100 = 6,000 GPD = 800 ft^3/day

Case C in Figure No. 11 shows how to calculate the effect of a house discharge on the position of the water table downhill, and the effect of the shape of the SWAS on the ground water table, using two-dimensional flow analysis. Cases D-1 and D-2 (Figures 13 and 14) show the use of a three-dimensional radial well recharge equation in calculating the hydraulic capacity of a level site.

Again, it should be pointed out that the greatest source of error in these analyses is not in the theory or calculations, but in the assumed values of depth of strata, hydraulic conductivity, and ground water elevation.

4.   Site Drainage

Many regulatory agencies do not recognize engineering predictions of the effect of subsurface draining systems on the hydraulic capacity of a site because of poor analyses in the past. The prevalent attitude is to "install the drainage system and see what effects it has during the wet season". In the absence of good analyses, this is the logical attitude; however, it is possible to predict the effect of surface and subsurface drainage systems if accurate site investigations are made.

Drainage systems are generally used to increase the hydraulic capacity of a site or to reduce the natural inflow of surface or subsurface water and make room for the effluent that is to be treated. Installing the drainage system "downhill" can increase the site hydraulic capacity, so that water can leave the site faster. Intercepting the inflow from uphill and bypassing it around the site through pipes or ditches can reduce natural inflow. It is not possible to bypass the precipitation although the surface can be graded to prevent ponding or water during very heavy rainstorms or when the ground is frozen.

The most reliable type of drainage system is a ditch that will intercept both surface and subsurface water; however, a ditch requires slopes of 1:3 or flatter and a deep ditch therefore requires considerable land space. The alternative is a buried pipe. Subsurface drains often consist of perforated pipe surrounded by a processed stone filter in a trench. This type of filter may allow the migration of fine particles into the pipe resulting in clogging of the pipe. In recent years filter fabrics have proven to be more cost-effective and more effective than processed stone filters. Figure No. 14 shows cross sections of typical curtain drains that operate satisfactorily.

FIGURE No.14

TYPICAL CURTAIN DRAINS
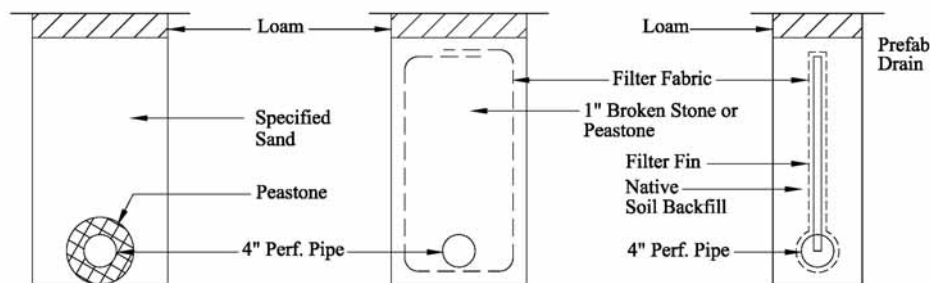
In order for a subsurface drain to function properly it must (1) let the water into the drain as fast as it can leave the soil, and (2) carry the water away as fast as it enters the drain. A proper filter constructed of either aggregate or fabric will fulfill the first requirement. The pipe must be large enough to fulfill the second requirement.

The analysis of the effects of a subsurface drain is most easily done assuming steady state flow conditions. The ground water and precipitation are assumed to be entering the site at a constant rate. This obviously doesn't occur often, but the effect of the drains can be determined for the worst condition, a long heavy rain for instance. During the operation of a SWAS, having the ground water table up high for a few days during an exceptionally rainy period is not detrimental, but it should not be high for longer than this. For this reason, in designing drains it is appropriate to assume a 4-5 day rain storm of about 0.1 ft/day.

It should be pointed out that many curtain drains are installed on sites where the ground water is high during the spring. A ground water table that rises in the spring indicates that water is entering faster than it can leave, which implies that lateral drainage is restricted by soil strata of low hydraulic conductivity. In such a situation the water entering a site as ground water may be only a fraction of the water entering as precipitation or surface water, and an uphill curtain drain may not lower the ground water table appreciably. The influence of a subsurface drain is very dependent on the hydraulic conductivity of the soil.

The following cases illustrate the calculations used to predict the effect of curtain drains.

Case A. Figure No 15, shows a level site of 6 feet of sandy glacial till underlain by hardpan which is assumed to have zero hydraulic conductivity.

The general theory for drain operation on level ground underlain by an impermeable stratum is shown in Figure No. 15. All the water is assumed to come from rain that percolates downward to the ground water table and then moves laterally to the drains. It is assumed that the rain occurs at a constant rate long enough to develop a steady state flow. The desired position of the ground water table is assumed and the drain spacing necessary to cause this condition is calculated. The calculation can involve a series of flow nets, mathematical analyses, or an approximation as shown. All these methods give about the same answer.

FIGURE No. 15

EFFECTS OF CURTAIN DRAINS, CASE A
Rain $= P = 0.1$ ft/day
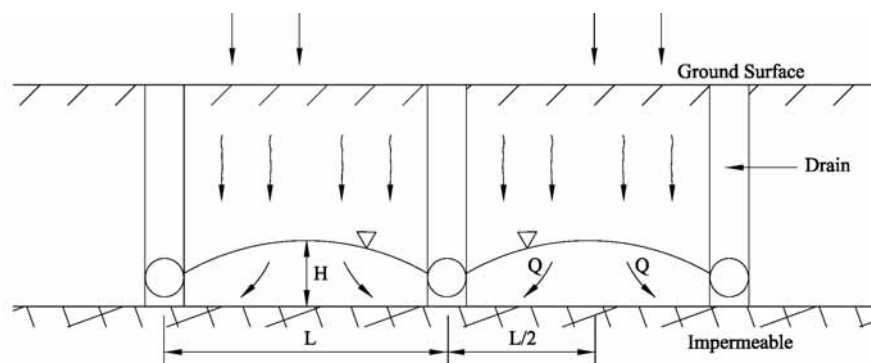
Calculation of Drain Spacing

$$Q = P \times \frac{L}{2}$$

$$Q = KiA = K \times \frac{H}{L/2} \times \frac{H}{2} = K\frac{H^2}{L}$$

$$K\frac{H^2}{L} = P \times \frac{L}{2}$$

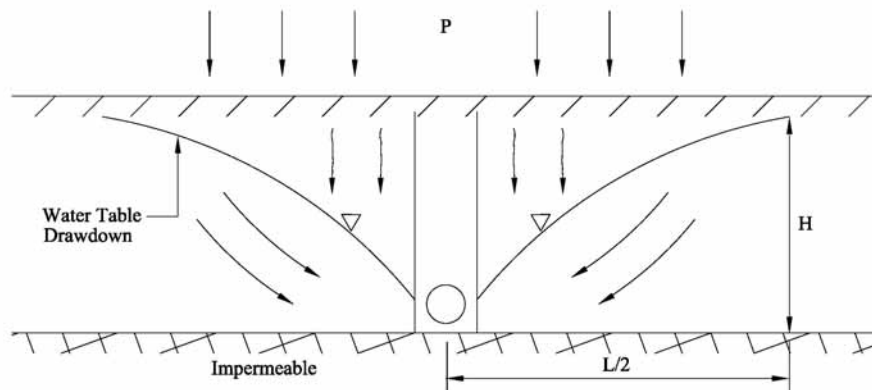$$\frac{L^2}{H^2} = \frac{2K}{P}$$

$$\frac{L}{H} = 1.4\sqrt{\frac{K}{P}}$$

Case B. If a single drain only is installed in an area with the ground water table at the ground surface, the effects of the drain will be felt at a distance slightly less than L/2, as shown in Figure 16:

$$\frac{L}{2} = \frac{1.4}{2}H\sqrt{\frac{K}{P}} = 0.7H\sqrt{\frac{K}{P}}$$

FIGURE No. 16

EFFECTS OF CURTAIN DRAINS, CASE B

In both the preceding cases, the water collected by the drains came from rain falling on adjacent ground. In Case C (Figure No. 17), a drain is installed on a slope to intercept ground water and lower the ground water downhill from the drain. If the drain is constructed down to the impermeable strata it will intercept all the flow from uphill. The effect of slopes up to 10% is minimal and the zone of influence is only slightly greater than on level ground.

FIGURE No. 17

EFFECTS OF CURTAIN DRAINS, CASE C
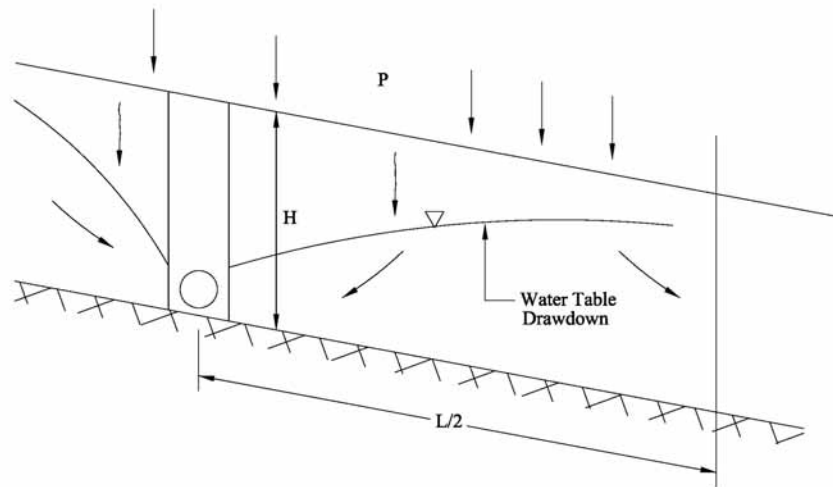
It should be pointed out that groundwater is drawn to the drain from a distance of about

$$0.75H\sqrt{\frac{K}{P}}.$$

If the soil had a hydraulic conductivity of 4 ft/day and the drain was 3 feet deep, for a rain of 0.1 ft/day, the drain would influence the water table only 0.75 x 3 x $(4/0.1)^{0.5}$ = 14 feet downhill from the drain.

One very important conclusion that can be drawn from these calculations is that on a site in which rain is sufficient to raise the water table in the spring, any drainage system that will lower the water table in the area of a SWAS will generally collect effluent also. This is not a health hazard if the effluent travels through enough soil for treatment, and if there is sufficient dilution by rainwater.

An example of what happens if a curtain drain is installed around a SWAS is given in Figure No. 18.

FIGURE 18

EFFECTS OF CURTAIN DRAINS AROUND A SWAS
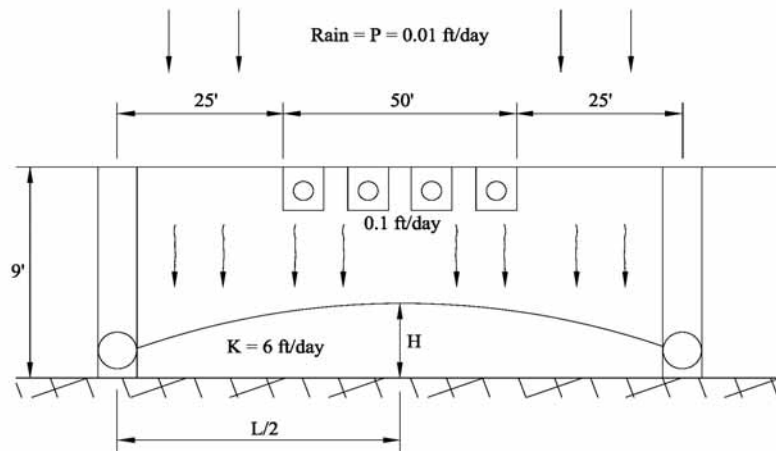
Input from SWAS = 0.1 ft/day x 50 ft = 5 ft^3/day/ft of SWAS

Equivalent P $= \dfrac{5 \text{ ft}^3/\text{day}}{100 \text{ ft}^2} = 0.05$ ft/day.

Rainfall $= 0.01$ ft/day

$\sum$P +P $_{equiv.}$ = 0.01ft. +0.05 ft = 0.06 ft/day.
K = 6 ft/day.
L = 100'

Procedure:

$$\frac{L}{2} = 1.4H\sqrt{\frac{K}{P}} = 50\,ft$$

and,

$$H = \frac{L}{1.4}\sqrt{\frac{P}{K}} = \frac{100}{1.4}\sqrt{\frac{0.06}{6}} = 7.1\,ft$$

Therefore, the ground water table will be approximately (9-7.1) or 1.9' ft below ground surface in center of field. In this example, it is likely that the effluent will reach the underdrains before it is fully renovated in the soil.

5.   Regulatory Constraints on Drains

The use of a drain to increase site hydraulic capacity presents some significant regulatory problems. If a drain is located on a property with a SWAS, and is down gradient from that system, the drain will collect treated effluent and discharge it to a surface water body. Unless the surface water body is designated as Class B by the Water Quality Standards adopted pursuant to Section 22a-426 of the Connecticut General Statutes, the system is in violation of these standards. Class AA, or A waters are not allowed as receiving water bodies for point source discharge of treated effluent. Assuming the drain as described discharged to a class B water course then a federal N.P.D.E.S. permit, administered by the Connecticut Department of Environmental Protection, would be required.

It is possible to design a drain at a distance that will provide adequate renovation of effluent to a drinking water standard prior to discharge. The problem is that the calculations supporting such an application are approximations dealing with complex site data. A regulatory agency that is charged with adopting and enforcing water quality standards must take a very conservative attitude toward such a proposal. The Department's policy with respect to acceptable use of drains for interception of ground water is provided in the Design Standards.

6.   Fill

A satisfactory SWAS can be constructed with fill placed on bare impermeable bedrock. However, it should be noted that impermeable bedrock conditions are rarely encountered in CT, as virtually all of the bedrock in the state is fractured. Since it is very difficult to determine travel time in fractured bedrock, it would be necessary to place a hydraulic barrier (compacted clay liner, synthetic membrane, etc.) between the fractured bedrock and the fill so as to ensure that the travel time in the vertical direction through the barrier met the travel time requirements of the Department before any flow entered the fractured bedrock. The site preparation would consist of installing the hydraulic barrier and placing and compacting carefully selected soil to an adequate depth and lateral extent. The soil should contain enough sand and fines to provide adequate treatment of the effluent, and should have sufficient hydraulic conductivity to allow the water to move away laterally faster than it is applied. The soil should be of sufficient depth to allow a depth of unsaturated flow beneath the bottom of the system that meets the Department's requirements, and should be of sufficient lateral extent so that the effluent travels through the soil for a period of time that complies with the travel time requirements of the Department before it reaches a point of concern.

Such a SWAS, utilizing entirely imported soil, would be very expensive, would be prohibited by most health codes, and is not normally built for residences. However, marginal sites can be significantly improved by the placement of select fill. This improvement can be in the form of improved treatment by providing additional soil for the effluent to flow through before reaching wells or the ground surface, or in the form of increasing the hydraulic capacity of the site by providing soil with greater hydraulic conductivity or hydraulic gradient by elevating the system. Systems utilizing imported soil have been approved by the Department for large-scale on-site systems. Further discussion of fill systems is given Section X of this document.

7.   Submission of Hydraulic Capacity Analysis in the DEP Permit Process

The presentation of a hydraulic capacity analysis should be logical and clear if it is to support a permit application. The following procedure is recommended:

Step 1. Assemble site-testing data, including soils, boundaries, topography, sensitive areas, hydraulic conductivity and estimated discharge volume.

Step 2. Calculate the size of SWAS per the Department's Long Term Acceptance Rate Criteria. This gives linear feet of SWAS; its configuration is determined by the balance of analysis.

Step 3. From this data, consider and develop a model for the predevelopment hydrologic conditions on the site. In other words, how does the site handle existing recharges, and where is effluent likely to go?

Step 4. Lay out a trial SWAS superimposed on the model.

Step 5   Perform hydraulic capacity analysis, check by alternate method if feasible.

Step 6   Demonstrate that under the worst case conditions the site can transmit the wastewater discharge with a reasonable safety factor.

Step 7   Define the plume direction, velocity and area to facilitate definition of the zone of influence of the system. This topic is covered in Section X of this document.

G.   Other Methods For Hydraulic Capacity Analysis Under Low Hydraulic Gradient Conditions

1.   General

Under low hydraulic gradient conditions (where the water table is essentially in a horizontal plane), the effects on the height of a ground water table caused by discharging pretreated wastewater into an unconfined aquifer are similar to the effects of an artificial ground water recharge basin. When water is discharged into an unconfined low gradient aquifer, the water table rises under and in the immediate vicinity of the discharge area, forming a ground water mound. Therefore, the height of the ground water mound that will develop beneath a subsurface wastewater absorption system must be superimposed on the pre-existing seasonal high water table when determining the vertical separating distance between the bottom of the system and the seasonally high water table. Ground water mound heights will be greatest in shallow aquifers of low hydraulic conductivity.

A detailed analysis of the growth characteristics of ground water mounds is a sophisticated mathematical process, involving complex equations that are difficult to evaluate unless certain simplifying assumptions are made. Many researchers have devised both analytical and numerical solutions to the equations describing pressure and flow in porous media with certain boundary and initial conditions. Many of these solutions have been incorporated into computer models.

Because of the significant data requirements and considerable expertise required for use of the complex numerical models, they should not be used as part of the submittal for a Discharge Permit without first receiving approval of the Department.

The Department recommends that initial analysis of the effect of ground water mounding be made using the methods presented in this subsection. If the results indicate that the hydrogeologic conditions appear suitable to accept the proposed design discharge without causing the resulting mound to impinge on the unsaturated zone (vertical separating distance) required by the Department, the results can then be checked using one of the computer based analytical models discussed below. Such models may also be used where the results obtained from the initial analysis indicate the vertical separating distance is problematic, ("too close for comfort") and a more detailed approach may be warranted.

Analytical models contain a closed-form or analytical solution of the field equations subject to specified initial and boundary conditions. Because of the complex nature of ground-water problems, the analytical solutions generally are available for problems that entail a simplifying nature of the ground-water system, its geometry and external stresses (U. S. EPA-1993).

The areal extent and height of a ground water mound are dependent upon the size and shape of the discharge area, the duration and rate of discharge, the stratigraphic configuration of the subsurface formations, the saturated and unsaturated hydraulic properties of the geologic materials, and the aquifer boundary conditions. Under constant recharge conditions, and in an aquifer of infinite extent, the height of a ground water mound will continue to increase over time, although at ever reducing rates.

Under constant recharge to an aquifer whose extent is limited by boundary conditions, a ground water mound will continue to grow until some control, potential or lateral, provides a limit. Lateral controls can consist of an impermeable boundary (no flow condition), a ground water divide, or the presence of a stream, lake or other surface water body (constant head condition). In the latter case, for a large horizontal extent of these types of boundary conditions, the ground water mound approaches equilibrium with a constant recharge rate. Potential controls occur when the ground water mound builds up to the recharge surface; with a fixed maximum height the ground water mound gradient and hence the recharge rate must decrease with time (Todd - 1980).

The analysis of ground water mounding beneath a subsurface wastewater absorption system (SWAS) using an analytical model can be carried out by assuming that the SWAS is equivalent to a recharge basin of similar dimensions and that the recharge rate is constant with time. While the rate of discharge into a SWAS actually is intermittent, the assumption of a uniform recharge rate is conservative. In addition to the proposed discharge, the recharge rate should include the effect of infiltration of precipitation, as previously discussed.

2.   Well Discharge Equation

It should be noted that the equation used in the Pit Bailing Test method for determining hydraulic conductivity (Subsection E 4), and in the example given for 3-Dimensional Hydraulic Capacity Analysis (Subsection F 3) is an analytical model known as the "simple well formula" (Cedergren- 1989). While developed to estimate the yield of a well under steady-state conditions, it can also be used to estimate the local rise (mounding) of a water table in response to a recharge well operating at steady-state conditions in an unconfined aquifer. This formula is based on the following assumptions (Cedergren, ibid.):

    a. The pumping well penetrates the full thickness of the water-bearing formation.
    b. A steady-state flow (equilibrium) condition exists,
    c. The water bearing formation is homogeneous and isotrophic and extends an infinite distance in all directions.
    d. The Dupuit assumption is valid [i.e.: the hydraulic gradient at any point is constant from top to bottom of the water-bearing layer and is equal to the slope of the water surface].

If the water table is slightly sloping, the bottom boundary of the water bearing strata may be drawn parallel to the original water table and the computation carried out the same as for a level water table (Cedergren, ibid.).

Since the simple well formula is based on radial flow, it is basically applicable to a circular recharge basin. However, it has also been found suitable for a square recharge basin, where the area of the square basin is equivalent to the area of a circular basin of radius R. The formula is less suitable for rectangular basins where the basin length is much greater than the basin width, which is most often the case with subsurface soil absorption systems.

The simple well formula, (as given in Subsection F) is:

$$Q = (\pi K (H^2 - h^2))/\ln (R/r)$$

where k = saturated hydraulic conductivity, H = depth from top of mounded ground water table to an impermeable lower boundary, h = original saturated thickness of aquifer, R = radial distance from center of recharge basin to an aquifer boundary or an assumed outer limit of the mound, and r = the radius of the recharge basin. This equation can be used as a first approximation of the ground water mound height development beneath a SWAS.

Analytical solutions of the governing equations for saturated flow have been made for certain shapes of recharge basins (square, rectangular, and circular). These solutions are based on assuming the aquifer to be homogeneous, isotropic (hydraulic conductivity being independent of direction of measurement), of infinite areal extent, and having an initially horizontal water table and impermeable bottom. They can be adapted to "real world" conditions by making other reasonable assumptions and using certain superposition techniques.

In unconfined aquifers with low hydraulic gradients, the hydraulic properties of unconfined sediments that control their ability to accept water are the hydraulic conductivity, saturated thickness, and specific yield of these deposits. The specific yield of an aquifer is a dimensionless parameter that is defined as the volume of water that an unconfined aquifer releases from storage per unit surface area of aquifer per unit decline in the water table (Freeze and Cherry - 1979). It is also sometimes referred to as "drainable porosity", the ratio of the volume of water that would be released from the soil pores under gravity drainage conditions to the total volume of the pores. Thus, this parameter is related to the porosity of the soil, although some soils having high porosities (very fine sandy silts and clays) may have low specific yields. It is also related to the "Available Water Capacity" parameter utilized by soil scientists and agronomists.

3.   Computer Models

There are several computer models available for analytical solutions of ground water mounding. Among these are the following:

Flow From Wells and Recharge Pits, Version 2.0 is a proprietary program available from the Groundwater Program, Colorado State University, Ft. Collins, CO, or from the IGWMC, which lists the program as CSUPAW (Colorado State University Pit and Well). The program runs under DOS on desktop computers.

MOUNDHT, a proprietary program originally developed in 1992-93 by Professor E. John Finnemore, Department of Civil Engineering, Santa Clara University, Santa Clara, CA 95053. The program runs under DOS on desktop computers.

Estimation of Groundwater Mounding Beneath Septic Drain Fields. This program is an adaptation of the MOUNDHT program to a WINDOWS environment. It was developed by the Center for Resources Studies, Technical University of Nova Scotia (Mooers-1994).

GRAMP, (Groundwater Recharge and Mounding Program) is an interactive program designed to predict ground water mounding below rectangular basins. This program is based on the Hantush equation (Hantush-1967) equation for ground water flow for recharge from a rectangular basin. The program runs under Windows 95 and 98.

Hantush's Rectangular Recharge Basin Analysis, a proprietary program based on the Hantush method of solving the governing differential equations for ground water flow for recharge from a rectangular basin (Hantush-1967), it is an interactive program that predicts the growth and decay of ground water mounds in response to uniform percolation. The program runs under DOS on most desktop computers.

It should be noted that the Hantush method of solving the governing differential equations for saturated ground water flow to calculate ground water mounding resulting from recharge from a rectangular basin (Hantush-1967) may also be solved manually (Cantor and Knox - 1985; Walton - 1970).

However, the use of computer programs for solving the Hantush equations makes the computations quicker, easier and less subject to error and can provide more information than is readily available using the manual solution. The use of such computer programs also facilitates performing sensitivity analyses by allowing rapid "what if" iterations by changing input data values.

All of the analytical models described above have limitations with respect to accounting for boundary conditions. Accordingly, it is sometimes necessary to modify the results obtained from these analytical models by use of methods of superposition using the theory of images.

When an aquifer is bounded on one side by an impermeable boundary and on the other side by a stream (an equipotential, or "constant head" boundary), the ground water mound profile resulting from operation of a recharge basin between the two boundaries will be higher and flatter on the impermeable boundary side and lower and steeper on the stream side of the basin. To predict the height of the ground water mound above the normal ground water level in such aquifer systems, the principal of theory of images (image well theory) can be used to simulate the effects of such boundaries (Ferris, et al.- 1962; Lohman -1972; McWhorter and Sunada - 1977; Freeze and Cherry -1979).

Finally, while the data needs of these analytical models are modest and use of the models is deceptively simple, sound judgement must be used in selecting the data to ensure that it is reasonably representative of the hydrogeologic conditions at the project site. As stated by Finnemore in the documentation for GROUND WATER MOUNDHT, "Such prediction methods need to be used with judgement by experienced engineers who are aware of their limitations."

4.   Hydraulic Load Testing

A technique sometimes used for determining the hydraulic capacity of a proposed SWAS site is a hydraulic load test that simulates the ground water mounding that will result from the proposed discharge. Demonstration of hydraulic capacity is accomplished by hydraulic loading of trenches or basins while monitoring of water levels in ground water monitoring wells located immediately adjacent, and proximal to, the hydraulic loading area(s).

It is recommended that the results of hydraulic load tests should not be used as part of the submittal for a Discharge Permit without first receiving approval of the Department with respect to methods and procedures for conducting such tests in order to avoid the Department's refusal to accept such results.

The technical feasibility of conducting hydraulic load tests will depend upon the availability of an adequate supply of water that will permit conducting the test at the hydraulic loading rate anticipated for the proposed SWAS over a sufficient length of time to establish a relatively "stable" ground water mound. This in turn requires that an adequate water supply source is available within a reasonable transport distance and that a suitable means of transporting the water from the source to the test site is available or can be constructed.

Where the quantity of water required for the test is greater than 50,000 gallons per day, a diversion permit will need to be obtained from the CT DEP. In addition, permission to use the water will need to be obtained from the owner of the water supply source, and temporary easements may be required for constructing a temporary water transmission line from the source to the test site. A permit from the local Inland Wetlands and Watercourses Agency is also likely to be required. A determination will also have to be made whether the quantity of water to be taken from the source will be such as to cause detrimental effects to the source or will adversely effect the supply available to nearby developed areas that depend on that source for their water supply. If such adverse effects will occur and cannot be mitigated, hydraulic load testing should not be done.

5.   Tracer tests

Tracer tests can be used to demonstrate the area proposed to be utilized for an SWAS has adequate aquifer hydraulic characteristics to provide the minimum travel time to the nearest property line, or point of surface breakout in accordance with the pathogen travel time criteria of the Department. Tracer tests can also be used to delineate the zone of influence of the proposed discharge. Demonstration of travel time and zone of influence can be accomplished by sampling and testing of water from selected ground water wells located immediately adjacent to, and downgradient of the point of tracer injection for the presence and concentration of the tracer.

Tracer tests are often conducted in conjunction with hydraulic load tests. The ground water monitoring wells to be sampled can be determined from phreatic surface contour maps generated from ground water monitoring data (water table elevations) developed prior to and during a hydraulic load test. The zone of influence can be determined from the presence of the tracer in the various monitoring wells.

The tracer method of estimating ground water travel times and the zone of influence of a discharge to the ground water has disadvantages. These include the long periods of time normally required for tracers to move significant distances because of low ground water velocities, and the need for numerous monitoring wells and sample analyses. Each monitoring well may have to contain a nest of sampling tubes designed to sample the ground water at different elevations below the water table in order to be reasonably certain of intercepting the tracer plume. Thus, a properly designed tracer test is apt to be time consuming and costly.

As in the case of a hydraulic load test, the results of tracer tests should not be included as part of the submittal for a Discharge Permit without first receiving approval of the Department.

H.   References

ASTM 1999 ASTM Standards on Determining Subsurface Hydraulic Properties and Ground Water Modeling. 2nd Edition. ASTM D 4044-96 (for field procedures) and ASTM D4104 (for analytical procedures). American Society for Testing and Materials. West Conshohocken, PA

Bouwer, H. and R.C. Rice. 1976. A Slug Test for Determining Hydraulic Conductivity of Unconfined Aquifers with Completely or Partially Penetrating Wells. Water Resources Research, Vol. 12, No. 3.

Bouwer, H. 1978. Ground Water Hydrology. McGraw-Hill, Inc. New York, NY.

Bouwer, H. 1989. The Bouwer and Rice Slug Test - An Update. Ground Water, V. 27, No. 3. pp. 304-309.

Bouwer, H. 1989 Discussion of "The Bouwer and Rice Slug Test - An Update." Ground Water, V. 27, No 5 . pg. 715

Butler, J.J. Jr. 1997. The Design, Performance and Analysis of Slug Tests. Lewis Publishers, New York, NY.

Cantor, L. W., and R.C. Knox. 1985. Septic Tank System Effects on Ground Water Quality. Lewis Publishers, Inc. Chelsea, MI.

Cedergren, H R. 1989 Seepage, Drainage and Flow Nets. Third Edition. John Wiley & Sons, NY, NY.

Ferris, J. G., D. B. Knowles, R. H. Brown and R.W. Stallman. 1962. Theory of Aquifer Tests. Geological Survey Water Supply Paper 1536-E. U.S. Geological Survey, Washington, D.C.

Finnemore, J.E. 1993. Estimation of Ground-Water Ground Water Mounding Beneath Septic Tank Drain Fields. Ground Water, V. 31, No. 6; pp 884-889

Finnemore, J.E. 1995. A Program to Calculate Ground-Water Ground water mound Heights. Computer Notes. Ground Water, V. 33, No. 1; pp 1139-143.

Freeze, R. A. and J.A. Cherry. 1979 Groundwater. Prentice-Hall Inc., Englewood Cliffs, NJ.

Hantush, M.S. 1967. Growth and Decay of Groundwater Ground water mounds in Response to Uniform Percolation. Water Res. Research, V. 3, No. 1, pp 227-234.

Healy, K.A. and R. Laak. 1974. Site Evaluation and Design of Seepage Fields. Jour. EE Div. ASCE. Vol. 100, No. 5.

I. References, Continued.

Lohman, S.W. 1972. Ground-Water Hydraulics. Geological Survey Professional Paper 708. U.S. Geological Survey, Washington, D.C.McWhorter, D.B. and D.K. Sunada. 1977. Ground-Water Hydrology and Hydraulics. Water Resources Publications, Fort Collins, CO.

Molden, D., D.K. Sunada and J.W. Warner. 1984. Microcomputer Model of Artificial Recharge Using Glover's Solutions. Ground Water. Vol. 22, No. 1.

Mooers, J.D. and D.H. Waller 1996. Vertical Separation and Groundwater Mounding Beneath Septic Systems-Appendix 1: Mound Calculation Program. In: On-Site Wastewater Research Program – Final Report, Phase III 1994-1996. Report to Province of Nova Scotia.   Center for Water Resources Study, Technical University of Nova Scotia.

Todd, David K.   1980 Groundwater Hydrology, Second Edition. John Wiley & Sons New York, NY. 1980.

U.S. EPA 1993 Compilation of Ground-Water Models. EPA/600/R-93/118. U. S. Environmental Protection Agency, Office of Research and Development. Washington, D.C.

U.S. Department of the Navy, Naval Facilities Engineering Command.   1982. Soil Mechanics. NAVFAC Design Manual 7.1.

Walton, W.C. 1970   Groundwater Resource Evaluation. McGraw-Hill Inc. New York, NY.