

University Curriculum Development for Decentralized Wastewater Management

Hydraulics IV: Groundwater and Onsite Module Text

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Groundwater and Onsite

A. Overview

Ultimately there are very few options for the dispersal of partially treated effluent from an individual residential system or a larger but still decentralized system serving more than one residence. The wastewater can be:

Released to flow over ground to the nearest surface drainage feature

Retained in a lined Evapo-Transpiration (ET) until it dissipates as gas into the atmosphere or

Discharged to a subsurface feature designed to allow the infiltration of the effluent.

Discharges to surface drainage features are strictly controlled by the National Pollution Discharge Elimination System (NPDES) permitting system and generally are difficult to obtain except for large scale facilities with constant monitoring and certified system operators. Discharges to the atmosphere are generally legally limited only to the extent that surface and groundwater needs to be protected from possible overflows and leaks from the Evapo-Transpiration system. Therefore regulatory agencies mandated to protect surface and groundwater may carefully review plans for ET beds studying the potential evaporation, precipitation, water usage and available storage in the ET bed's media.

By far the most common method of dispersal of treated effluent back into the environment is through infiltration into the local soils. It is therefore important the onsite professional understand the basic principles affecting the introduction and movement of water through soils.

B. Principles of soil water flow (Dave Gustofson)

LEARNING OBJECTIVE: Know the factors that directly impact soil water movement.

Water is the main carrier for the transport of pollutants through soils. Therefore, knowledge of water flow pattern and magnitude is essential in designing land-based waste disposal systems, as well as for conducting environmental assessments. We can look at water movement in an aquifer or in a partially wet soil (i.e., under saturated or unsaturated condition) at macroscopic (water movement through the bulk of soil) or microscopic scale (water movement through individual pores and at points in the soil). For practical applications, we look at water flow in soils and aquifers at macroscopic levels.

Water moves through the soil as a result of forces acting on it. The major forces that act on water in the soil are those related to the gravity (gravitational potential), attraction of solid particles for water (matric potential), capillary rise of water in narrow pores (matric potential), water pressure within the saturated zone (submergence potential) (and in some cases pressure due to the mass of soil above the point of interest, overburden pressure), attraction of salts for water (osmotic potential), and air pressure (pneumatic potential). To assess soil water movement, the cumulative effect of these forces must be considered. As

mentioned earlier, although both kinetic energy (related to motion) and potential energy (related to internal conditions or position within a force field) are important in water movement, only the potential energy is directly considered for analyzing soil water movement under most conditions. This is because soil water movement is relatively slow, and its kinetic energy, which is related to velocity squared, is considered to be negligible.

Water can move through the soil in all directions, but (between two points) it always moves from a point at higher potential (i.e., higher total hydraulic head) to a point at lower potential. At equilibrium, when there is no soil water movement, the total soil water potential or total hydraulic head (H) at every location within the soil volume under consideration is a constant. The difference between the soil water potential or total hydraulic head (H) between two points determines the direction of water flow, and the rate of change of the hydraulic head along the flow path between the points, referred to as gradient (H/L , where L is the length of the flow path), determines the rate of movement of water between them. The rate of water movement between the two points also depends on the hydraulic conductivity of the medium. Hydraulic conductivity can be defined as a measure of the ability of soil (or any porous medium) to transport water.

1. Darcy's law

LEARNING OBJECTIVE: know Darcy's law and its components

In his work with sand filters for purifying water over a century ago, a French engineer named Henry Darcy observed that increasing the head of water over a sand column of a given length increases the rate of water flow through the column. Figure 1 Schematic diagram of sand columns showing the effect of total hydraulic on water flow through the column, illustrates the fundamental concept. After a number of experiments, he determined that the quantity of water (Q) flowing through water-saturated sand filters of length L and cross sectional area A during a time period t was proportionally related to the hydraulic gradient $\Delta H/L$ by:

$$Q/(At) = v = K_{sat} \times \Delta H/L \quad \text{Equation 1}$$

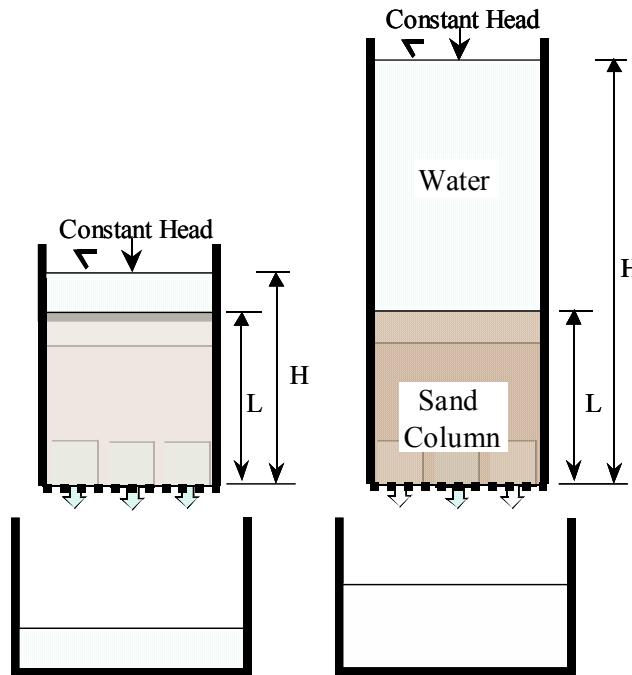
where K_{sat} is the saturated hydraulic conductivity of the medium and v is called flux density or flux. The above equation, known as Darcy's law, was later modified and presented in a differential form to describe water flow through both saturated and unsaturated soils. For one-dimensional flow, Equation 1 can be written as:

$$v = -K(h) dH/dx \quad \text{Equation 2}$$

where $K(h)$ is the soil hydraulic conductivity as a function of soil water pressure head in the unsaturated zone (i.e., matric potential) and dH/dx is the gradient. For saturated soils, (i.e., when $h = 0$), $K(h)$ is equal to K_{sat} . [NOTE: In this equation the flux v is a vector and it has a magnitude and direction (similar to the gradient dH/dx). For our purposes,

we take the value of v as positive at all times, and determine the direction of the flow independently from the values of the total hydraulic head at points of interest.] Darcy's law has been accepted as the physical law governing soil water movement and applies to both saturated and unsaturated flow under steady state or transient conditions. This law simply states that the rate of movement of water through a soil is proportionally related to the hydraulic gradient (i.e., the driving force acting on water) and the conductivity of the medium (i.e., a measure of the ability of soil to transmit water). Darcy's law (or any version of it) is applied to soil at a macroscopic level (i.e., it does not directly deal with the movement of water through individual pores), and it is an integral part of any model describing water movement through porous media. This law, however, has certain limitations and fails when water flow through the pores of the porous medium under consideration is turbulent (i.e., it requires the flow through the soil to be laminar).

Figure 1 Schematic diagram of sand columns showing the effect of total hydraulic on water flow through the column



2. Water flow through soils

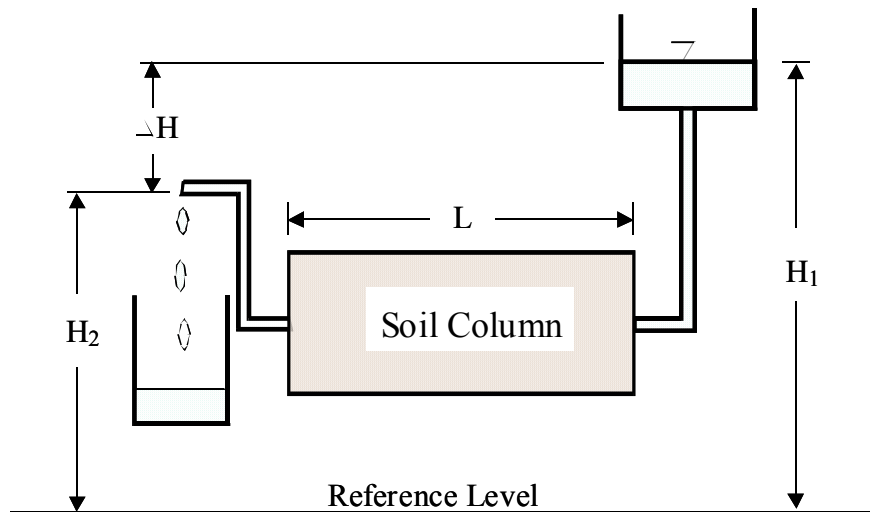
LEARNING OBJECTIVE:

1. Know various zones in the soil water profile during/after infiltration
2. Know the impacts of initial soil water content on the initial infiltration rate
3. Know the impacts of final infiltration rate on runoff
4. Know some examples of various types of water flow

As stated above, the direction of the flow depends on the direction of the gradient, and the rate of water movement depends on the magnitude of the hydraulic gradient and

hydraulic conductivity of the medium. For example, consider that water is applied to one end of a horizontal soil column of known cross sectional area under a constant head, H_1 , and exits the other end of the column under a constant head, H_2 (see Figure 2, A schematic diagram of a soil column showing the total hydraulic head at the inlet (H_1) and at the outlet (H_2)). Knowing the saturated hydraulic conductivity of the column and applying Darcy's law (Eq. [1]), one can determine the quantity of water that can pass through the column during a specified time period. Conversely, knowing the flux and gradient, one can calculate a saturated hydraulic conductivity based on the flow rate through the column.

Figure 2, A schematic diagram of a soil column showing the total hydraulic head at the inlet (H_1) and at the outlet (H_2)



Soil condition(s), the source of water, direction of the flow, and the nature of the flow must be considered when assessing water movement. Following is a list of examples for these:

- * Saturated flow -- Unsaturated flow
- * Macropore flow -- Matrix flow
- * Steady-state flow -- Transient (unsteady-state) flow
- * Vertical flow (gravitational potential) -- Horizontal flow (no gravitational potential)
- * Homogeneity -- Heterogeneity
- * Isotropic -- Anisotropic
- * Layered soil -- Non-layered soil
- * Source -- Sink
- * Inlet(s) -- Outlet(s) (boundary conditions)
- * Direction of flow -- one-, two-, three-dimensional flow

a. Saturated flow

Generally speaking, large pores and fractures in a soil conduct water at a faster rate than smaller diameter pores or tight fractures. When the soil is saturated, most of the water passes through large tubular pores created by roots and animals or the planar voids between soil peds (often referred to as macropores). Although the interparticle pores or pores inside soil peds (matrix pores) may be filled up with water, the rate of water movement is relatively slow in these pores as compared to the macropores. Under saturated conditions the hydraulic conductivity (i.e., K_{sat}) of the soil is a constant value at any given time for any given point within the soil body. Within a soil volume, K_{sat} varies from location to location (spatial variability) as well as in different directions (directional conductivity due to anisotropic nature of soils). In addition to spatial and directional variabilities, K_{sat} may change with time (temporal variability) due to changes in temperature and other factors. Unless we are studying a soil at a microscopic scale, any measured K_{sat} value represents an average conductivity of the volume of the soil under consideration.

b. Unsaturated flow

During the drainage process of a saturated soil volume with no water input, water initially moves out of the larger pores due to gravity. Subsequently, water moves out of the smaller pores due to the reduction in soil water pressure head resulting from vertical drainage, plant root uptake, evaporation from the surface, or other factors. As water moves out of this volume of soil, the soil water content (represented by θ) decreases and the tendency of the soil for holding to water increases (i.e., pressure head, h , decreases). Consequently, the flow path for water becomes narrower and the ability of the soil to transmit water (represented by conductivity) decreases as the soil water content decreases. Mathematically speaking, the unsaturated hydraulic conductivity (K_{unsat}) depends on the soil water content or soil water pressure head, and is represented by $K(\theta)$ or $K(h)$. If the relationship between soil water content (θ) and pressure head (h), also known as soil water characteristic curve, is known, then Darcy's law can be written in terms of soil water content in place of pressure head.

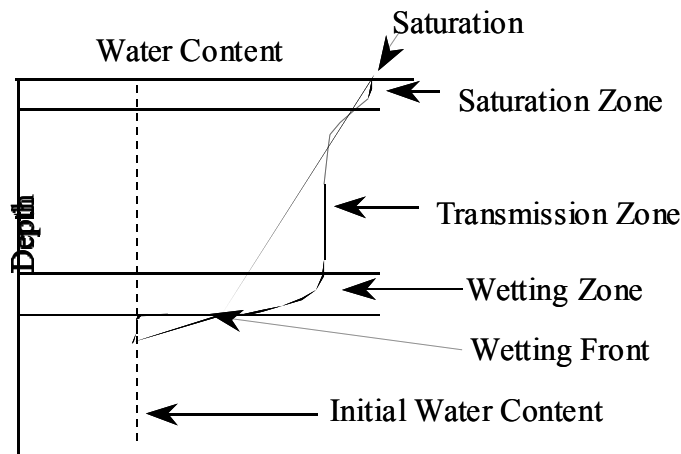
c. Steady-state and transient flow

The rate of water movement through a volume of a soil under a steady-state condition is constant (i.e., does not change with time), whereas under transient conditions the rate of flow depends on time. For example, the ground water flow is considered to be under steady state if it flows toward a natural drainage system at a constant rate with no change in the total hydraulic head or volume of saturation. Near a well pumped intermittently, on the other hand, the flow is under transient conditions. Another example is the transient condition in the upper part of the vadose zone immediately after a major rainfall event.

d. Infiltration and soil water profile

Water may enter the soil under an array of conditions. Under a rainfall event or a spray irrigation system, water may approach the soil surface fairly uniformly in a relatively large area. In an infiltration gallery, water enters the soil in a relatively small area. Water applied to a trench enters the soil through the bottom and sidewalls of the trench. Under any of these conditions, a general pattern for the soil water distribution with depth (referred to as soil water profile) can be established. When the rate of water application to the soil equals or exceeds the maximum rate of infiltration into the soil the zone immediately below the surface where water is applied becomes saturated. This zone is called saturation zone (Figure 3 A schematic diagram showing the soil water distribution with depth during the infiltration of water into an initially dry soil). The volume of the soil below this saturation zone is referred to as the transmission zone. In this unsaturated zone soil water content is fairly uniform (for a uniform soil). At the other end of the transmission zone is the wetting front zone where water content decreases rapidly. At the edge of the wetting front zone (referred to as wetting front) the hydraulic gradient is relatively high and the wetted zone advances into the dry soil despite the low hydraulic conductivity.

Figure 3 A schematic diagram showing the soil water distribution with depth during the infiltration of water into an initially dry soil



e. Infiltration rate and cumulative infiltration

The rate of water entry into the soil from a surface (e.g., surface of the land, bottom or side of a trench) is referred to as infiltration rate. The rate of infiltration into a dry soil decreases with time (Figure 4 Infiltration rate (A) and cumulative infiltration (B) as a function of time) due to an increase in soil water content and a decrease in hydraulic gradient. If infiltration continues, the rate of water entry into the soil reaches a constant value (steady-state condition) that is considered to be the same as the saturated hydraulic conductivity of the soil volume near the infiltrative surface. The total amount of water entering the soil since the start of infiltration into the soil is referred to as the cumulative

infiltration. The slope of the flat portion of the cumulative infiltration curve at large time is the final infiltration rate.

f. Effect of initial soil water content on infiltration rate

The rate of water entry into the soil (referred to as infiltration rate) depends on the amount of water in the soil (Fig. 28). The wetter the soil, the lower is the rate of water infiltration into the soil. This is illustrated below.

Figure 4 Infiltration rate (A) and cumulative infiltration (B) as a function of time

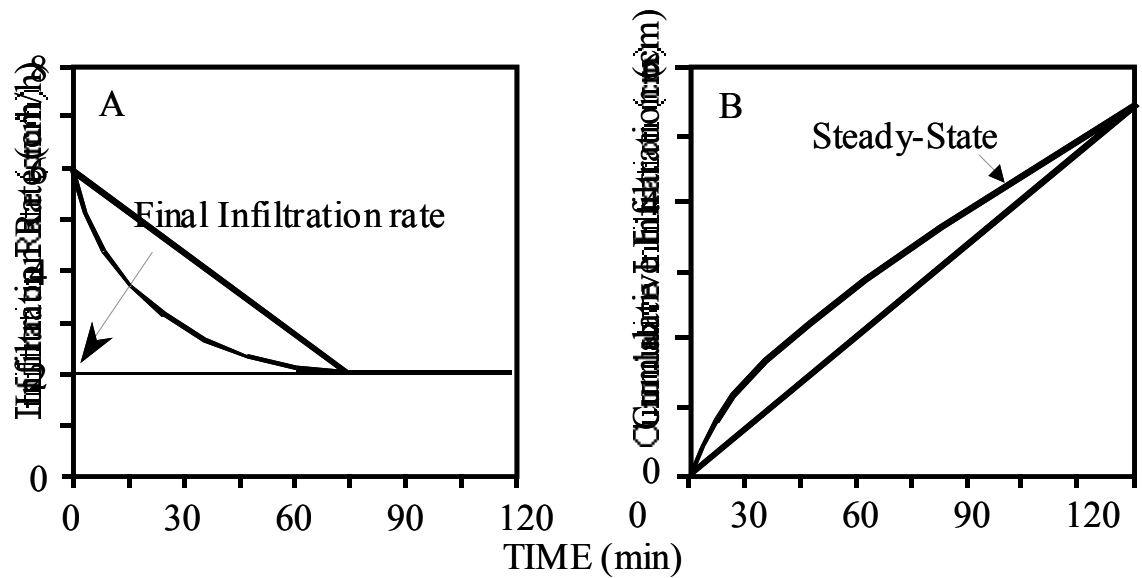
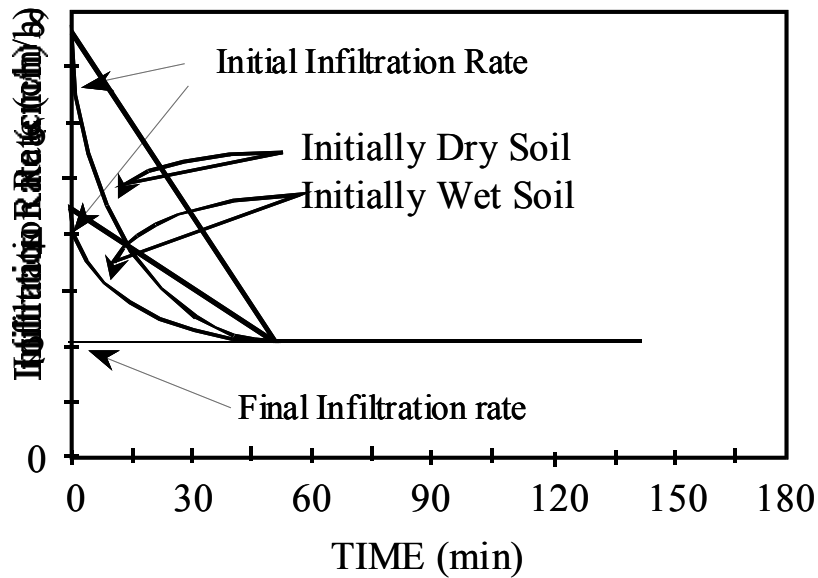


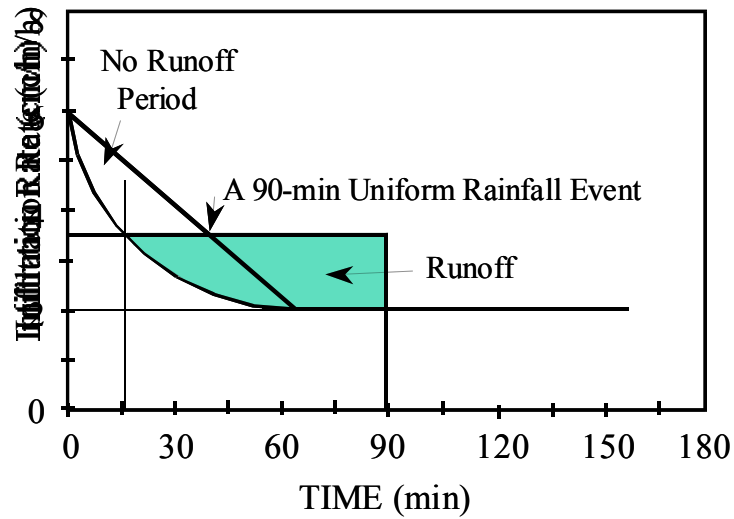
Figure 5 The rate of infiltration into a soil under wet and dry initial conditions



g. Runoff occurrence

Runoff occurs when the rate of rainfall exceeds the final infiltration rate of a soil. Using the infiltration curve for a given initial soil water content, the amount of runoff can be estimated based on the rate and duration of the rainfall event Figure 6 The infiltration rate and the amount of runoff generated at the site for a given rainfall event illustrates this effect.

Figure 6 The infiltration rate and the amount of runoff generated at the site for a given rainfall event



C. Application of Groundwater Movement to Onsite Systems

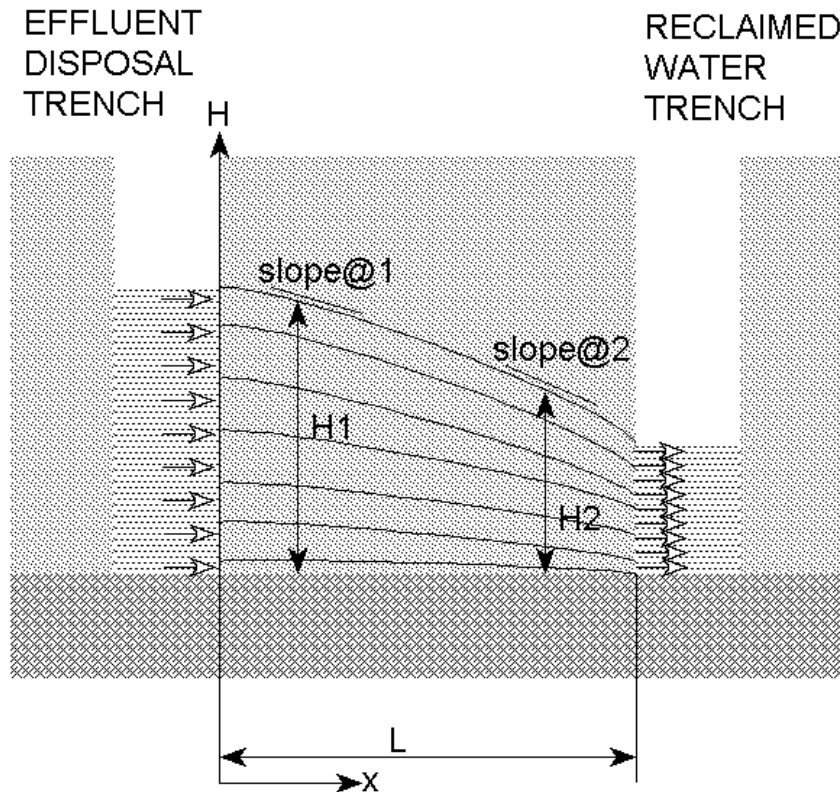
LEARNING OBJECTIVE: apply groundwater movement principles to onsite systems

As discussed above, the movement of saturated flows within a conducting layer of soil is generally analyzed by the Darcy equation which estimates the flow of water through a cross section of soil by multiplying the cross sectional area by the slope of the saturated water surface and the appropriate value of the conductivity.

Issues related to saturated vs. unsaturated flow; hydraulic conductivity that may be direction depended, steady state vs. non steady state (transient) and variations in soil storage capabilities all complicate the analysis of water being dispersed into a simple onsite disposal trench.

Consider a simple situation illustrated below in Figure 7 One Dimensional Flow in which a discharge from a dispersal trench is flowing underground (saturated and steady state) over an impervious surface from the dispersal trench to another trench (possibly a French drain or a water reuse recovery trench) through a homogeneous material. We will quickly see that Darcy's equation cannot be used to immediately determine the flow of water from one trench to the other.

Figure 7 One Dimensional Flow



As discussed above Darcy's equation for one-dimensional flow is:

$$v = - K_{\text{sat}} \frac{dH}{dx} \quad \text{Equation 3}$$

The total discharge, which is often referred to as Q is equivalent to the velocity v multiplied by the area being considered. In the situation illustrated above the area is the vertical depth of flow (H) multiplied by the total width of the trench. Considering only the flow per unit width of the trench allows the equation to be rewritten as:

$$Q = - K_{\text{sat}} \frac{dH}{dx} * H \quad \text{Equation 4}$$

The dH/dx term represents the slope of the water surface at any point along the flow path but the slope of the top of the saturated band of water leaving the dispersal trench and approaching the recovery trench requires careful analysis since it is changing at every point as is the depth, H . Theoretically, the depth of saturated flow cannot be known until the flow velocity (or discharge) is determined and the flow velocity cannot be determined unless the slope of the water surface can be determined. Ultimately both issues must be dealt with simultaneously realizing that the total discharge ultimately passes through the total area regardless of where the boundary is being drawn for the analysis (assuming

there are no losses either up to evaporation or down into the layer taken to be impervious).

Calculus can be used to develop another equation (the Dupuit equation) that will enable the determination of the flow between these two trenches:

$$Q = - K_{sat} dH/dx * H \quad \text{Equation 5}$$

Separating variables

$$\frac{Q}{K_{sat}} dx = H * dH \quad \text{Equation 6}$$

and integrating:

$$\frac{Q}{K_{sat}} \int_0^l dx = - \int_{H1}^{H2} d * dH \quad \text{Equation 7}$$

results in

$$Q = K_{sat} \frac{(H1^2 - H2^2)}{2L} \quad \text{Equation 8}$$

The point of the above mathematical exercise and example was to illustrate the potential complexities of even the simplest groundwater flow problems. The reader may want to find any one of many applied hydraulics books, groundwater hydrology books or fluid mechanics books to explore these theoretical topics further.

Typical values of Ksat are found in many soil and groundwater hydrology textbooks. The following table, Table 1 Typical Ksat for Common Soils (from Bower, Groundwater Hydrology) shows typical values for common soil types:

Table 1 Typical Ksat for Common Soils

Soil Type	Ksat (m/day)
Clay Soils	0.01 to 0.20
Deep Clay Beds	10^{-8} to 10^{-2}
Loam Soils	0.1 to 1
Fine Sand	1.0 to 5
Medium Sand	5.0 to 20
Coarse Sand	20 to 100
Gravel	100 to 1000
Sand and gravel mixes	5 to 100
Clay, sand, and gravel mixes (till)	0.001 to 0.1

What are worth noting in the above table is the large range of Ksat values. The smallest ranges given show factors of 5 between lowest estimates to highest estimates while the largest factors show a range of one million (10^6). The range of typical values of Ksat found in groundwater hydrology texts may in part due to the high variability the soil structure but many Ksat charts reference materials do not consider soil structure at all. The soils structure may have a tremendous impact on the actual Ksat experienced in the field where the onsite dispersal feature is being planned. In particular, for otherwise “tight” soils the structure may be more important than the soils structure-less saturated hydraulic conductivity. Laboratory measurement devices that mimic the defining sketch show earlier for the definition of Ksat invariably destroy the soil structure as the soil is added to the chamber

The implication of these Ksat values and their range on the design of onsite systems can be easily illustrated by the use of typical values published for loam soils. In the two tables that follow values of Ksat for loam of 0.001 cm/sec (approx 1.0 m/day) and 0.0001 (approx 0.1 m/day) are used to illustrate this condition.

The chart shows the potential soil-loading rate for various ground slopes (across the top) and for various depths of soil available for conducting the effluent away from a trench or series of trenches. For example considering a ground slope of 2% and a soil depth of 24 inches a 1.0 m/day Ksat results in an application rate of 0.85 gpd/sq.ft. Reducing the assumed value of Ksat to 0.1 m/day reduces the application rate to 0.085 gpd/sq ft. This would result in a 10-fold increase in the number or length of the required trenches. Text book values of Ksat may have only limited applicability in the design of onsite systems but their range of values illustrates the nature of the problem of accurately determining the soils ability to carry treated effluent away.

D. Linear Loading Rate Analysis

LEARNING OBJECTIVE: understand the concept and application of Linear Loading Rate analysis.

1. Linear Loading Rate Definition

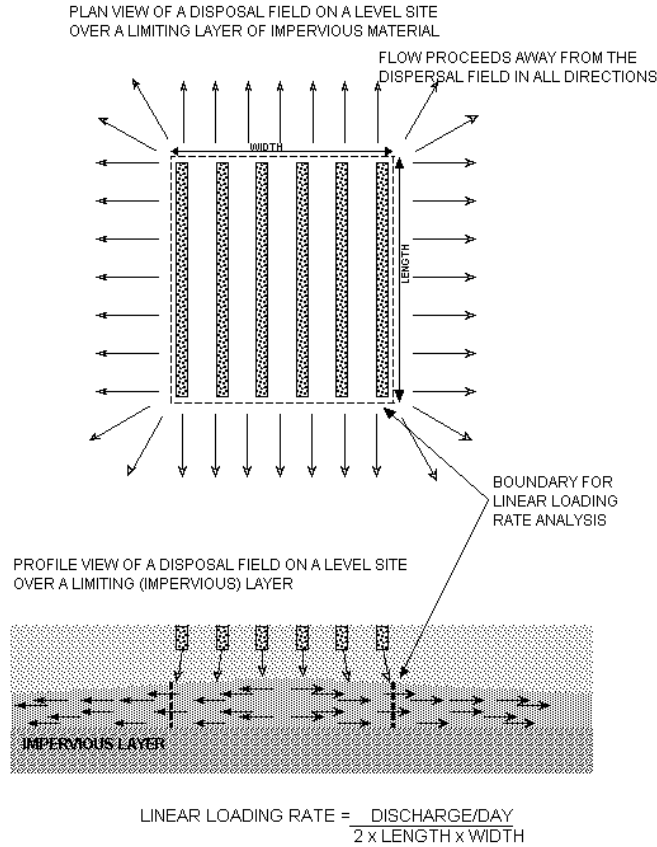
The Linear Loading Rate (LLR) is a tool to evaluate acceptable soil loading rates for a drain field located across a sloping site. A flow line equal in length to the upstream dispersal trench and parallel to it (and the contours) is established to determine the LLR. The LLR can be used to predict unacceptable mounding of effluent over groundwater or over an impervious boundary. Mounding of effluent could be unacceptable if it results in saturated conditions extending to grade at some point downgrade or causes the vertical separation between the point of effluent discharge and the depth to saturated soil to be below the minimum allowable minimum vertical separation (MVS).

The term “linear loading rate” seems to have come to the attention of the onsite community in the 1990 Wisconsin Mound Manual by Dr. John Converse (see Converse – Linear Loading Rate paper excerpt with this material). The concept as used in the Wisconsin Mound Manual was simply the division of the total daily discharge of wastewater by either the downside length of the basal area of the mound (on sloping sites) or the downside and upside length of the basal area of the mound on level sites. The end areas are generally not considered in the analysis of liner loading rate for Wisconsin Mounds because the recommended design of Mounds is significantly longer along the slope than it is wide and the end areas are not considered as significant dissipaters of effluent. The concept of liner loading rate as used in the Wisconsin Mound Manual does not take the depth of soil transmitting the effluent away from the mound into consideration.

An analysis of the linear loading rate may mean different things to different people. A *determination* of the linear loading rate could result in a relatively simple set of computations based upon the situation although if groundwater is the limiting layer the concept is subject to interpretation.

a) A determination on level sites over an impervious limiting layer where effluent is expected to dissipate in all lateral directions can arguably make use of the entire perimeter of the disposal field. Figure 8 Linear Loading Rate on Level Site shows this lateral distribution around the entire field.

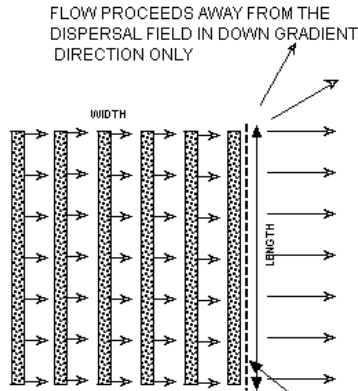
Figure 8 Linear Loading Rate on Level Site



b) A determination on sloped sites over an impervious limiting layer where effluent is expected to dissipate predominately in one lateral direction can arguably make use of only the downgrade boundary of the disposal field. Figure 9 Linear Loading Rate on Sloping Site shows this situation.

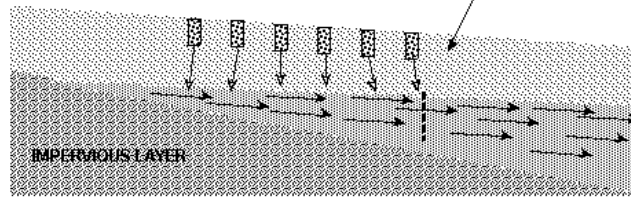
Figure 9 Linear Loading Rate on Sloping Site

PLAN VIEW OF A DISPOSAL FIELD ON A SLOPING SITE OVER A LIMITING LAYER OF IMPERVIOUS MATERIAL



BOUNDARY FOR LINEAR LOADING RATE ANALYSIS

PROFILE VIEW OF A DISPOSAL FIELD ON A SLOPING SITE OVER A LIMITING (IMPERVIOUS) LAYER



$$\text{LINEAR LOADING RATE} = \frac{\text{DISCHARGE/DAY}}{\text{LENGTH}}$$

c) A determination on level or sloping sites over a seasonal groundwater limiting layer

LINEAR LOADING RATE CONCEPTS AND CALCULATIONS

PLAN VIEW OF A DISPOSAL FIELD ON A LEVEL SITE OVER A LIMITING LAYER OF IMPERVIOUS MATERIAL

FLOW PROCEEDS AWAY FROM THE DISPERSAL FIELD IN ALL DIRECTIONS

BOUNDARY FOR LINEAR LOADING RATE ANALYSIS

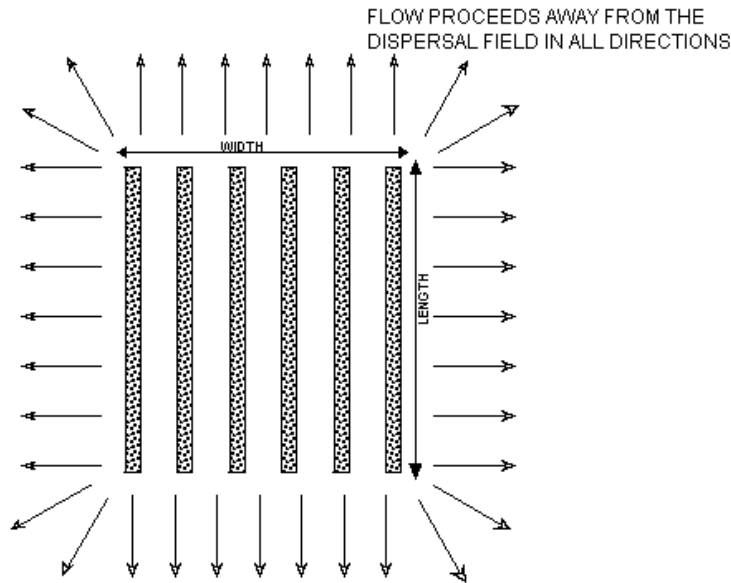
PROFILE VIEW OF A DISPOSAL FIELD ON A LEVEL SITE OVER A LIMITING (IMPERVIOUS) LAYER

$$\text{LINEAR LOADING RATE} = \frac{\text{DISCHARGE/DAY}}{2 \times \text{LENGTH} \times \text{WIDTH}}$$

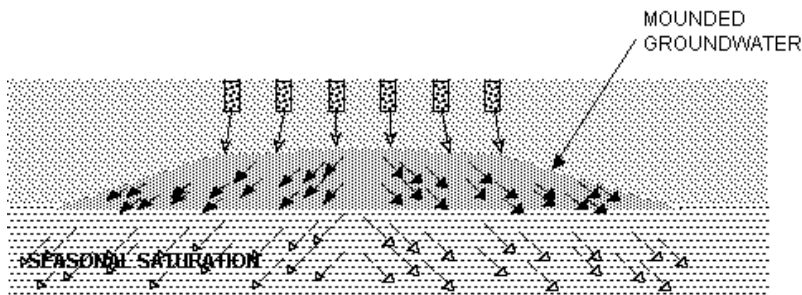
where effluent is expected to dissipate laterally and downward may require more definition of the concept of a linear loading rate and may require more sophisticated groundwater modeling techniques once the term is clarified. Figure 10 Linear Loading Rate on a Level Site Over Groundwater illustrates this situation.

Figure 10 Linear Loading Rate on a Level Site Over Groundwater

PLAN VIEW OF A DISPOSAL FIELD ON A LEVEL SITE
OVER A LIMITING LAYER OF SEASONAL SATURATION (LEVEL SITE)



PROFILE VIEW OF A DISPOSAL FIELD ON A LEVEL SITE
OVER A LIMITING SEASONAL SATURATION LAYER

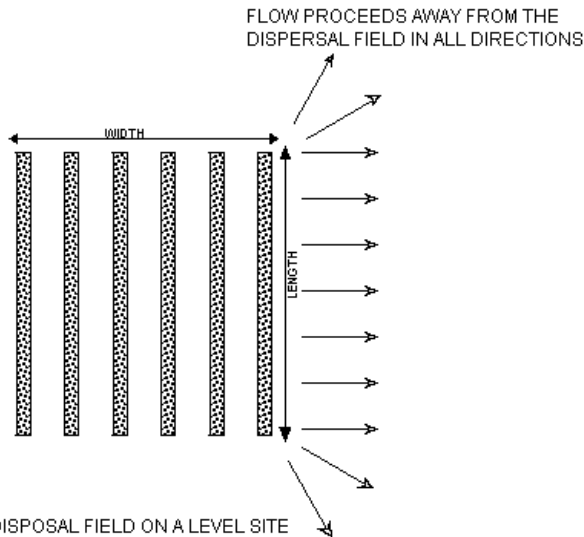


$$\text{LINEAR LOADING RATE} = \frac{\text{DISCHARGE/DAY}}{2 \times \text{LENGTH} \times \text{WIDTH}}$$

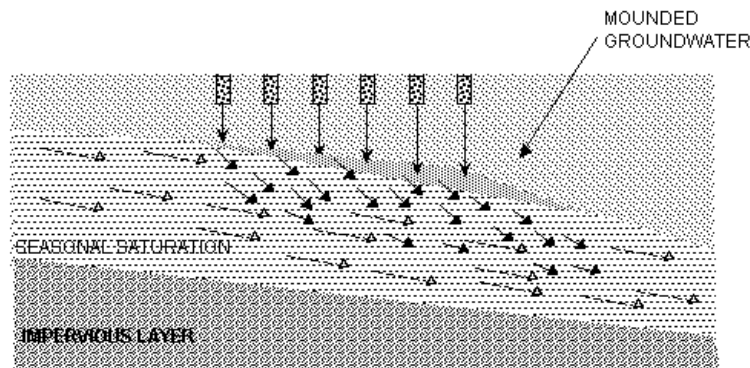
d) A determination on a sloping sites with a seasonal groundwater limiting layer over an impervious boundary where effluent is expected to dissipate laterally and downward may require more definition of the concept of a linear loading rate and may require more sophisticated groundwater modeling techniques. Figure 11 Sloped Site with Impervious Boundary and Seasonal Groundwater illustrates this situation.

Figure 11 Sloped Site with Impervious Boundary and Seasonal Groundwater

PLAN VIEW OF A DISPOSAL FIELD ON A SLOPED SITE
OVER A LIMITING LAYER OF SEASONAL SATURATION (LEVEL SITE)



PROFILE VIEW OF A DISPOSAL FIELD ON A LEVEL SITE
OVER A LIMITING SEASONAL SATURATION LAYER



$$\text{LINEAR LOADING RATE} = \frac{\text{DISCHARGE/DAY}}{\text{LENGTH}}$$

From a groundwater movement perspective there is a difference between the movement of water horizontally over an impervious boundary and vertically into a pre-existing saturated zone of seasonal groundwater. The concept of a linear loading rate over a seasonal high ground water table (or evidence of one) becomes a little sketchy. If groundwater exists a short distance under the dispersal field the soils between the dispersal field and the seasonal ground water are likely to be previous to water and would support a downward movement under the influence of gravity. The water would then both commingle with the existing seasonal ground water (when it is there) and move in whatever direction it is moving, or, the water would continue downward moving into the same soils that the seasonal ground water would move through if it were there. Hypothetically, if the seasonal groundwater is perched over a shallow impervious boundary then the linear loading rate analysis proceeds as it would for the simple case where the impervious boundary is the limiting layer and the effluent being dispersed moves laterally. If the depth of natural seasonal groundwater is relatively thin over the impervious boundary than the additional effluent can have a significant impact. If the

depth is greater over a deeper limiting layer a more sophisticated groundwater-modeling methods may again need to be pursued.

2. Application of Tyler's Method For Linear Loading Rate Analysis

LEARNING OBJECTIVE: Apply Tyler's method for Linear Loading Rate Analysis

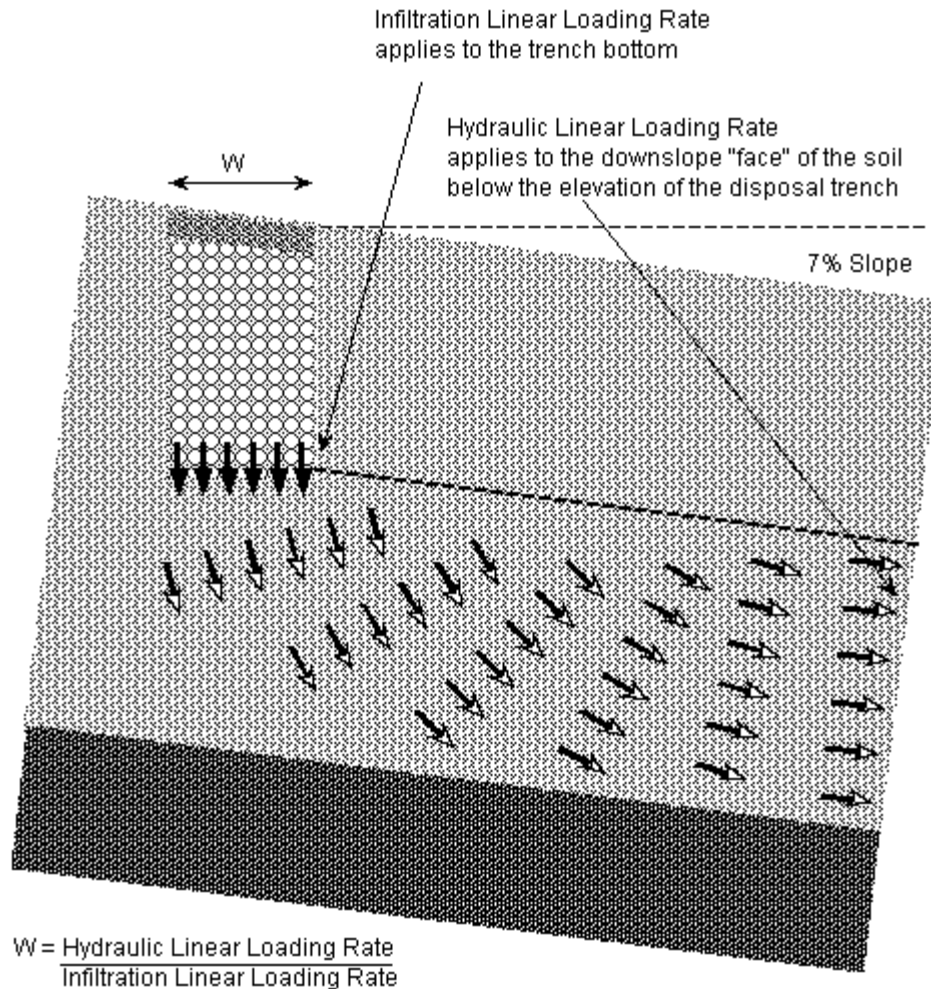
Accompanying this module is an analysis (Tyler) of and recommendations for Linear Loading Rates that address both the movement of groundwater through the soils and also the movement of effluent across the clogging mat and into the soils immediately adjacent to the trench.

Tyler's model is presented below and has the following assumptions:

- A limiting layer exists beneath the trenches.
- Effluent escapes the trenches only at the bottom of the trench
- The movement of effluent across the clogging mat and into the local soils is referred to as "Infiltration Linear Loading Rate".
- The movement of effluent away from the trench is referred to as the "Hydraulic Linear Loading Rate"
- The movement of effluent away from the trench is related to the ground slope which is assumed to equal the hydraulic gradient used in Darcy's equation.

Figure 12 Tyler's Linear Loading Rate Analysis Model is shown below and is the basis for the example problem that follows.

Figure 12 Tyler's Linear Loading Rate Analysis Model



The reader is referred to the accompanying paper for further background. The example problem in the paper is expanded below with commentary.

Tyler's Example from Accompanying Paper:

Assumptions:

1. Soil Type = Silt Loam (SIL) with weak (1) sub angular blocky structure (BK).
2. Infiltration loading rate = 0.4 gpd/ft² (if BOD > 30 mg/l) or 0.6 gpd/ft² (if BOD < 30 mg/l)

Note that the application rate is applied to the bottom of the trench only.

3. Tyler proposed hydraulic linear loading rate = 3.0 gpd/ft for this soil at 7% slope and 14 inches deep.

Note:

The linear loading rate does not double with a doubling of the soil depth.

4. By this method the single and only trench width needs to be:

$$3.0 \text{ gpd/ft} / 0.4 \text{ gpd/ft}^2 = 7.5 \text{ ft}$$

With such a trench width the absorption into the bottom of the trench is equalized with the dispersal down slope for every foot of trench length.

In situations in which credit is also given for sidewall and bottom the equivalent approach would dictate that the sidewall perimeter (or area per foot of trench) multiplied by the allowed infiltration-loading rate should be equal to or less than the associated linear hydraulic loading rate for that soil.

Consider the same soil, soil depth and slope with septic effluent (>30mg/l). The hydraulic linear loading rate would be 3.0 gpd/ft {37.3 L/day/m} and the infiltration-loading rate would be 0.4 gpd/ft² {16.3 L/day/m²}. Therefore the maximum allowable sidewall upgrade from the dispersal soil (or absorption surface) would have to be:

$$3.0 \text{ gpd/ft} / 0.4 \text{ gpd/ft}^2 = 7.5 \text{ ft} \{2.29 \text{ m}\}$$

By Tyler's method this would be the maximum allowable total of sidewall depth (2x) and bottom (1x) counted for dispersal, any more would violate the hydraulic loading rate recommendations unless lower application rates are used.

A potential problem with this method is Tyler's definition of infiltration distance which he defines basically the same as many communities define the vertical separation. Some communities have a minimum vertical separation of 5 ft {1.52 m} or greater for septic tank effluent. This is significantly greater than the maximum value in his table (4 ft {1.22 m} for minimal sloped sites and less for steeper sites.) The increases he shows in horizontal linear loading rates with increased depth are very modest and not simply proportional to depth. Notice how a doubling of depth has only a slight increase on the horizontal linear loading rates recommended in Tyler's chart. This puts a significant limitation upon how short a trench can be. For the soil, soil depth and slope considered above we would see the same 150 ft {45.7 m} minimum trench length that Tyler recommends.

Also note that this assumes only one trench across a given portion of a slope. If more than one trench is stacked up parallel to each other the situation is more critical because all the effluent from all the trenches stacked up the slope needs to be accounted for in the analysis.

Tyler's method is quite rational but will be very difficult for many soil types and lot sizes.

E. Insitu Measurement of Saturated Hydraulic Conductivity.

Various attempts have been made to enable a determination of the K_{sat} for a given soil in its natural setting with its structure intact (**insitu**). Such a determination of K_{sat} will overcome many of the problems with K_{sat} values extracted from soils or groundwater hydrology text that make a laboratory determination of K_{sat} .

The ideal **insitu** determination of K_{sat} makes use of a constant head test as does the laboratory test discussed above.

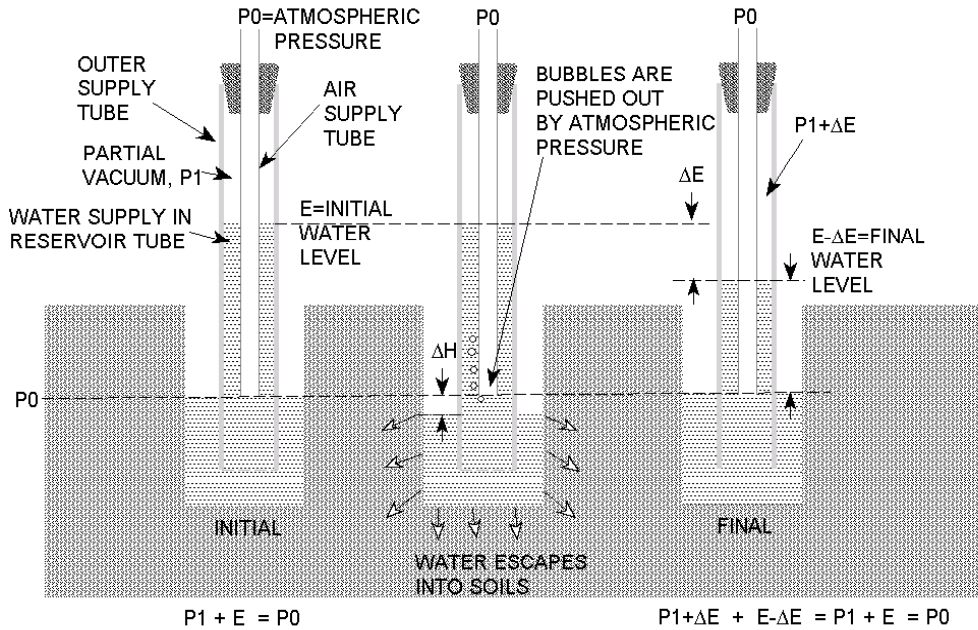
Various devices have been proposed to allow a constant head test to proceed in the soils without the need for a technician to be in constant attendance adding water to maintain the starting elevation.

The Mariotte Principle can be used to develop a device that will maintain a constant head in a small test hole. At least two such devices are available commercially, which exploit this principle.

- Guelph Permeameter by Soilmoisture Equipment Corp
- Amoozegar meter

The Mariotte principle allows for incremental additions of water to a test hole while also allowing precise measurements of the volume of water added. Figure 13 Mariotte Principle used for Insitu Determination of K_{sat} illustrates the basic operation of the device.

Figure 13 Mariotte Principle used for Insitu Determination of K_{sat}



WATER IN WELL REMAINS AT THE LEVEL OF THE AIR INLET TIP
 EXPLANATION:
 AS WATER ESCAPES INTO THE SOILS, THE WATER IN THE WELL DROPS SLIGHTLY BELOW THE LEVEL OF THE AIR INLET TIP. THE PRESSURE AT THE END OF THE AIR SUPPLY TUBE BECOMES SLIGHTLY LESS THAN ATMOSPHERIC PRESSURE. ADDITIONAL AIR BUBBLES INTO THE WATER SUPPLY TUBE INCREASING THE PRESSURE IN THE WATER SUPPLY TUBE. THIS CAUSES THE WATER IN THE WATER SUPPLY TUBE TO DROP UNTIL AIR PRESSURE AND THE HEAD OF WATER IN THE WATER SUPPLY TUBE AGAIN EQUAL ATMOSPHERIC PRESSURE.

F. Constant-head Well Permeameter Method: In-Depth Background (Aziz Amoozegar)

The constant-head well permeameter method is perhaps the most versatile procedure for measuring K_{sat} of the vadose zone from near the soil surface to a depth exceeding a few meters. This technique, also known as the shallow well pump-in method, borehole permeameter or borehole infiltration test (Amoozegar and Warrick, 1986; Philip, 1985; Stephens, 1992), was originally developed in the early 1950's, but because of its versatility and ease of measurement has received the most attention by soil scientists and hydrologists during the last decade. In this technique, the steady-state flow rate of water (Q) under a constant head (H) at the bottom of a cylindrical auger hole of radius r is measured, and K_{sat} is calculated by an appropriate equation using Q , H , and r . Because the flow from the auger hole is three dimensional, the K_{sat} value depends on both horizontal and vertical flow.

Originally, it was believed that the procedure for measuring K_{sat} by this technique might take a few days and requires a considerable amount of equipment and a large quantity of water (see Luthin, 1978). Developments in the theoretical evaluation of water flow in three dimensions and modifications of the field procedure for measuring K_{sat} by this technique (Amoozegar, 1992; Philip, 1969; Reynolds et al., 1983; Stephens and Neuman, 1982a and 1982b; Talsma, 1970; Talsma and Hallam, 1980) have indicated that steady-

state flow rate from a small diameter (4 to 10 cm) cylindrical auger hole under a constant depth of water may be reached within a short time (a few hours). Because the amount of water needed to fill a small diameter hole to create the desired head of water is not substantial, measurement of K_{sat} for most practical cases may be accomplished with a few liters of water in 2 to 3 hours (see Reynolds et al., 1983; Talsma and Hallam, 1980).

To measure K_{sat} , a hole of radius r is dug to the desired depth using a hand auger. For most practical applications, a 4 to 10 cm diameter hole is suitable for this purpose, but an approximately 6 cm diameter hole is recommended. The bottom of the hole is cleaned with a planer auger to form a cylindrical cavity. To minimize the effects of smearing the sidewall during hole construction, a round brush can be used to scrape off the wall of the hole. [For commercially available augers, planer augers, and brushes see the section on the Auger-hole Method.] If smearing occurs as a result of soil being too wet or clayey, brushing may not remove the smearing. After cleaning the bottom of the hole and measuring the depth of the hole, a constant depth of water H is maintained at the bottom of the hole (see Fig. 6). To maintain a constant depth of water at the bottom of the hole a mariotte syphon system (Amoozegar and Warrick, 1986) or a float system (Luthin, 1978; Stephens et al., 1987) can be used. The mariotte syphon system shown schematically in the previous section(Figure 13 Mariotte Principle used for Insitu Determination of Ksat) and the ones available commercially are suitable for relatively small diameter (4 to 10 cm) holes, whereas the float system can be used only in a hole where a small floating device can be installed in the hole.

In the constant head permeameter the constant head (depth) of water at the bottom of the hole (i.e., H) is set at the desired level by moving the adjustable air tube inside the constant-head tube up or down such that the distance d between the water level in the hole and the tip of the air tube inside the reservoir, i.e., the reference level on the permeameter, corresponds to the distance h_1 from the tip of the air tube to the water level in the constant head tube. The rate of flow of water into the soil is determined by measuring the change in the height of water in the reservoir (i.e., h_2 in Fig. 6) with time multiplied by the cross sectional area of the reservoir.

After establishing a constant head of water, water is allowed to infiltrate the soil until steady state is achieved (i.e., the rate of water flow into the soil Q under the constant depth of water becomes constant). For practical applications, it can be assumed that steady state is achieved when three consecutively measured Q 's are equal. The depth of water in the hole should be measured accurately a few times to assure that a constant head is maintained throughout measurement. The depth of water in the hole can be determined accurately by subtracting the distance between the water level in the hole and a reference level on the surface (e.g., d in Fig. 6) from the depth of the hole measured from the same reference level (D in Fig. 6).

Talsma and Hallam (1980) developed a simple permeameter for maintaining a constant depth of water at the bottom of a small diameter (in the order of 6 cm) auger hole. Reynolds et al. (1983) modified Talsma and Hallam's permeameter and introduced the Guelph permeameter with approximately 3 L of useful water capacity. [NOTE: The

Guelph permeameter is available from Soilmoisture Corp., Santa Barbara, CA.] Amoozegar (1989a, 1992) presented yet another small portable permeameter, the Compact Constant Head Permeameter (CCHP), with 5 L of useful water capacity for measuring K_{sat} to 2 m depth using a 4 to 10 cm diameter hole. With accessories, the CCHP can be used to measure K_{sat} at deeper depths. [NOTE: The Compact Constant Head Permeameter is available from Ksat, Inc., Raleigh, NC.] Jenssen (1989) introduced a marioette type infiltrometer with a lightweight plastic liner that fits snugly inside the auger hole for preserving the cylindrical shape of the hole.

Various equations and approaches (models) are available for determining K_{sat} based on the steady state flow rate(s) of water at the bottom of the hole and other field data (Q , H , and r). The early equations, including the Glover solution (Zangar, 1953), only consider the saturated flow of water around the auger hole, whereas several recent studies consider both saturated and unsaturated flow of water (Elrick et al., 1989; Philip, 1985; Reynolds et al., 1985; Stephens et al., 1987; Stephens and Neuman 1982a). The saturated flow component of these analyses is parametrized by K_{sat} , and the unsaturated flow component is represented by an unknown parameter that must be determined independently, estimated from selected soil properties, or calculated simultaneously (Elrick et al., 1989; Philip, 1985; Reynolds and Elrick, 1986; Stephens et al., 1987). To develop their models, Reynolds et al. (1985) and Philip (1985) used the matric flux potential

$$\phi = \int_{h_i}^h K(h) dh \quad [11]$$

[NOTE (Not in the published chapter): Matric flux potential is given by Phi on the left side and the right side is the integral of K(h) dh from h sub i to h for all h less than or equal to zero. Here, h sub i is the initial soil water pressure head.]

and the hydraulic conductivity function

$$K(h) = K_{sat} \exp(\alpha h) \quad [12]$$

[NOTE (Not in the published chapter): The right hand side is Ksat times exp (Alpha times h), where Alpha (units of 1/length, e.g., 1/m) is an empirical constant.]

developed by Gardner (1958). In the above equations, h_i (L) is the initial soil water pressure head, α (L^{-1}) is an empirical constant, and the other parameters are as defined previously. Stephens and Neuman (1982a) and Stephens et al. (1987) used capillary properties obtained from $K-h$ curves or water retention models (e.g., van Genuchten, 1980) to represent the unknown parameter related to the unsaturated flow in their regression equations.

The Glover solution, which ignores the unsaturated flow in its analysis (Zangar, 1953), has been recommended for calculating K_{sat} when the distance between the bottom of the

hole and any impermeable layer below the hole (s) is, $2H$ (Amoozegar and Warrick, 1986; Bouwer and Jackson, 1974). The Glover solution is

$$K_{sat} = CQ/(2\pi H^2) \quad [13]$$

[NOTE (Not in the published chapter): Inside the parentheses on the right is 2 times Pi times H to the power 2.]

where

$$C = \sinh^{-1}(H/r) - [(r/H)^2 + 1]^{1/2} + r/H. \quad [14]$$

For determining K_{sat} , Zangar (1953) recommended that the height of water in the hole be at least 10 times the radius of the hole (i.e., $H, 10r$) and reduced Eq. [14] for large H/r values to

$$C = \sinh^{-1}(H/r) - 1. \quad [15]$$

Amoozegar (1989b), however, recommended that Eq. [14] be used in the Glover solution (Eq. [13]) for all H/r values. 5. When $s < 2H$, K_{sat} can be calculated by

$$K_{sat} = 3Q \ln(H/r) / [\pi H(3H + 2s)]. \quad [16]$$

[NOTE (Not in the published chapter): In bracket on the right is Pi times H.]

Other equations and approaches available for measuring K_{sat} are the Laplace analysis, simultaneous equations approach of the Guelph permeameter method (Reynolds and Elrick, 1985), the fixed approach of Elrick et al. (1989), the Philip (1985) analytical model, and Stephen and Neuman (1982a) and Stephens et al. (1987) regression equations. The Laplace analysis is fairly similar to the Glover solution and only considers the saturated flow of water around the auger hole. The simultaneous equations approach is for calculating K_{sat} and α (Eq. [11]). For nonhomogeneous soils this approach may result in negative values for either K_{sat} or α (Amoozegar, 1989b; Reynolds and Elrick, 1985; Wilson et al., 1989). The fixed approach of Elrick et al. (1989) does not result in negative K_{sat} , but the parameter must be estimated based on the soil texture and structure. Elrick et al. (1989) presented the values for four different types of soils, and Amoozegar-Fard et al. (1983) reported the values compiled by a number of investigators for different soils. The regression equations developed by Stephens and Neuman (1982a) and Stephens et al. (1987) are based on numerical evaluation of water flow from an auger hole under a constant depth of water. These equations can be used for calculating K_{sat} provided that the parameter related to the unsaturated flow is independently determined.

There is no universal agreement among investigators on the equation or approach most suitable for calculating K_{sat} by the constant-head well permeameter technique. Comparisons of the available equations and approaches have indicated that the K_{sat} values obtained by various models and approaches are similar for most practical cases. Because of the simplicity of the Glover solution, we recommend that K_{sat} be calculated by Eq. [13]

using the C factor from Eq. [14] if $s < 2H$. The one restriction for using the Glover solution for the constant-head well permeameter method is that H/r must be > 5 . When $s < 2H$, K_{sat} can be calculated by Eq. [16]. The equations and approaches that include the parameter for the unsaturated flow can also be used if the unsaturated parameter for the soil under consideration is determined independently. For details and the requirements of the models please see the respective references.

As mentioned earlier, smearing of the auger hole sidewall during its construction may reduce the infiltration rate and yield a lower than actual K_{sat} value. Wilson et al. (1989) have discussed the possibility of removing the smearing by allowing the soil inside the hole to dry before measuring K_{sat} . To eliminate the smearing effect, Campbell and Fritton (1994) used an ice pick to remove a thin layer of the soil from the bottom section of shallow holes. This methodology, however, may increase the size of the hole and is not practical. Air entrapment in the wetted zone around the auger hole (see Stephens, 1992) might also affect the measured K_{sat} value. Stephens et al. (1987) have shown that air entrapment might result in significantly lower K_{sat} values. According to them, forcing CO_2 into the pores around the auger hole before water application to the hole can reduce the impact of entrapped air on the measured K_{sat} . Air entrapment is a problem for all the field methods for measuring K_{sat} of the vadose zone. To prevent the collapse of the auger hole sidewall and to preserve the cylindrical shape of the auger hole in sandy soils a section of perforated pipe can be inserted in the hole. A 2-in PVC well casing is well suited for 6 cm diameter holes.

G. Percolation Test

Another way to determine a sites application rate is the percolation test. The percolation test is generally used in combination with a soil application relationship. The percolation test itself is an empirical relationship of infiltration capabilities of the soils based upon an approximation to a falling head permeameter test. K_{sat} values are not determined from the percolation test and attempts to develop consistent relationships between the percolation rate and other methods of measuring K_{sat} have generally been unsuccessful.

Various methods have been used to relate the percolation test results (generally given as minutes/inch) to recommended long term application rates for septic tank effluent or other sewage effluents. The most common relationships are of the form:

$$Q = \frac{C}{\sqrt{T}}$$

where:

Q = application rate in gallons/sqft/day

C = empirical coefficient generally ranging from 2 through 5

T = percolation rate in minutes/inch

These formulas relate the percolation rate to application rates derived from field data for in use systems considered to be working adequately in a variety of soils and settings.

Although this method has been reasonably successful, today, it is no longer considered a sufficient way to predict long term soil absorption rates. Among the cited defects of the test and procedure for estimating the application rate are:

- a) High statistical variability of the percolation test itself
- b) High variability of applied effluent
- c) High variability of soil loading patterns through time
- d) Uncertain mix of saturated flow phenomena and unsaturated flow phenomena

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