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A Modified Field Infiltrometer Test For Clay Liners

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SYNOPSIS: Regulatory agencies are looking more frequently to in situ field hydraulic conductivity tests for the assessment of a liner's compliance to a specified hydraulic conductivity. Most field tests have evaluated hydraulic conductivity by measuring the infiltration rate of the liner. The infiltration rate can be used to arrive at a hydraulic conductivity value if the hydraulic boundary conditions of the test can be identified or if the head loss at different depths can be measured.

A test fill of a clay liner was evaluated for its saturated vertical hydraulic conductivity. This paper discusses the use of eight tensiometers to measure soil suction at four depths beneath a double ring infiltrometer. The hydraulic conductivity results using the tensiometer data displayed good consistency and agreed well with laboratory test results.

INTRODUCTION

Hydraulic barriers of compacted soil are widely used for covering waste disposal facilities and for lining solid waste landfills, liquid storage ponds, and other impoundments. These soil barriers are generally made of naturally clayey soil or a soil/bentonite mixture. The materials used for these clay covers/liners may have to conform to a design specification such as a certain plasticity index or a minimum bentonite content, however, most often the design specification will require that these barriers must have a hydraulic conductivity not exceeding a specified value.

The determination of the hydraulic conductivity of a clay liner is most often made from laboratory hydraulic conductivity tests, but, recently, regulatory agencies are looking more frequently to field hydraulic conductivity tests for the assessment of a barrier's compliance to a specified impermeability. Field tests may be receiving more attention because it has been suggested that laboratory hydraulic conductivity tests underestimate in situ hydraulic conductivity (Daniel, 1984), because field tests may be better at accounting for any hydraulic defects in the in situ barrier (Stewart and Nolan, 1987), and/or because field tests evaluate hydraulic conductivity on the scale more representative of the hydraulic barrier (Day and Daniel, 1985b).

Most field tests have evaluated hydraulic conductivity by measuring the infiltration rate for the hydraulic barrier. The infiltration rate can be used to arrive at a hydraulic conductivity value if the hydraulic boundary conditions of the test can be identified or if the head loss at different depths can be measured (Daniel and Trautwein, 1986).

The purpose of this paper is 1) to describe the use of tensiometers with an infiltrometer test for evaluating the hydraulic condition of soil suction (soil tension), 2) to discuss options utilizing the tensiometers, and 3) to describe a recent field test where an infiltrometer test using tensiometers was conducted with success. Before proceeding, a clarification of terms is necessary. The terms permeability and hydraulic conductivity are often used interchangeably. Strictly speaking, permeability is a property of the soil independent of the fluid. However, the data collected from field tests is often a measure of the hydraulic conductivity, which is a property of the soil and the fluid passing through it. Therefore, the term hydraulic conductivity is used in this report.

Infiltration Test

There are three broad categories of infiltration tests: the borehole or percolation test, the single ring infiltrometer, and the double ring infiltrometer. Advantages and disadvantages of each type of infiltration test are well documented (Day and Daniel, 1985b; Daniel and Trautwein, 1986). All of these tests measure the loss of water to the soil as infiltration. The borehole test uses the change in the water level in an uncased or cased hole. A single ring infiltrometer pools the water abov the barrier to be tested and reduces the effects of lateral infiltration by being at l
as wide as the barrier is thick. The doub
ring infiltrometer minimizes the effects
lateral infiltration by having two pools
ter. A large pool of water, surrounding
ner pool of water, is the source of all
affected by lateral infiltration. The l
water through the inner ring is used to
mine the infiltration.

Description of Test Apparatus

As an extension of the double ring infil ter described above, a sealed double ring infiltrometer featuring a covered inner ring that eliminates evaporation as a flux term, has been developed by Trautwein Soil Testing Equipment of Houston, Texas. The apparatus consists of two rings: a fiberglass rectangular ring approximately five feet to the side (inner ring) which is positioned in the center of a second rectangular aluminum ring that is approximately twelve feet long on the side (outer ring). Two sets of the Trautwein apparatus were used. A schematic of the test layout and a cross-section are shown in Figure 1. Both rings are filled with water, and the loss of water from the inner ring is measured periodically by weighing a flexible bag that is the reservoir of water for the inner ring (Figure 1). This water loss is the amount of water that has infiltrated the test fill beneath the inner ring. The water level in the outer ring is maintained at a level slightly above the top of the inner ring. The head of water in the inner ring is equal to the outer ring by placing the flexible bag in the water of the outer ring. Submerging the inner ring reduces the effects of temperature changes on measurements of water volume lost through the inner ring.

Initially, the water from the inner ring enters a clay cover/liner which is unsaturated. The water is forced through the barrier by the head of water in the rings and by the soil suction caused by capillary tension. Eventually, the soil beneath the ring will become saturated, and the infiltration rate will approach steady state. If a drainage layer (vented to the atmosphere) is provided beneath the barrier, it would be possible to measure the outflow through the barrier. When the outflow is equal to the inflow, steady state conditions are said to exist. Under these saturated steady state conditions, the boundary condition below the barrier can be easily identified. To arrive at the hydraulic conductivity of the barrier from a measured infiltration rate, either the boundary conditions beneath the barrier must be known, or the head loss between different soil depths must be measured.

Previous application of the Trautwein apparatus (Daniel and Trautwein, 1986) and of single ring infiltrometers (Day and Daniel, 1985b; Stewart and Nolan, 1987) have not measured pore water pressure (soil suction). When the soil barrier is saturated and steady state conditions exist, as verified with a free draining layer underlying the barrier, pore water pressure can be assumed to be zero. However, clay liners with low hydraulic conductivity may take several months to saturate if thicker than a couple of feet.

Theory

The seepage of water into the test fill is driven by the hydraulic gradient caused by the

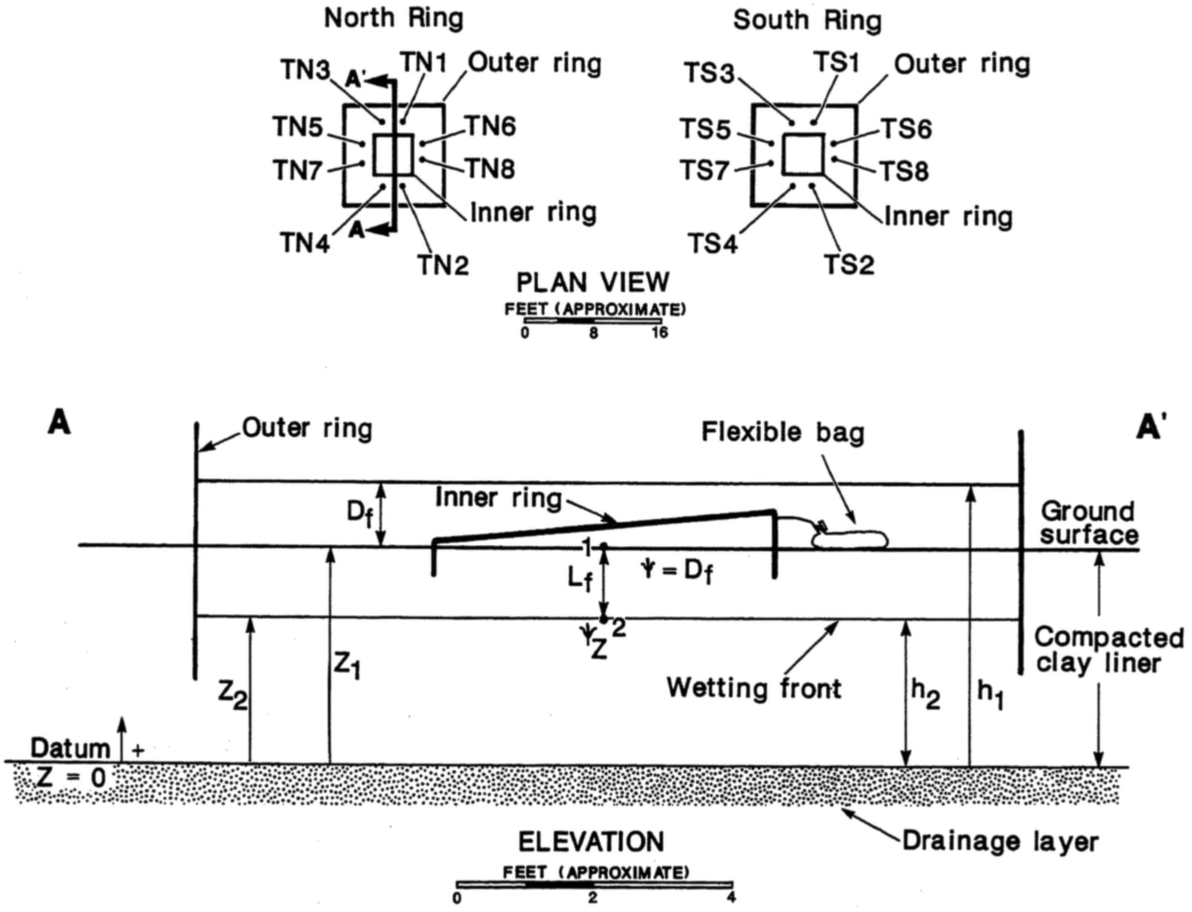

Figure 1. Infiltrometer Test Schematic

ponded water depth and soil suction (tension). The saturated hydraulic conductivity can be computed using a form of Darcy's Law which includes terms for the total hydraulic gradient. The governing equation that describes the flow of water into the compacted clay is developed below. Illustration of the terms and sign convention is shown in Figure 1.

According to Darcy's Law, and by observation of Figure 1 (the flow direction is in the negative Z-direction, downward):

$$q = -K \frac{\Delta h}{\Delta L} \qquad (1)$$

where,

q = flow rate per unit area (L/T)
K = saturated hydraulic conductivity (L/T)
$\frac{\Delta h}{\Delta L}$ = total hydraulic gradient (L/L)

Recognizing that, $\Delta h = h_1 - h_2$ and $\Delta L = Z_1 - Z_2$ between points 1 and 2 (see Figure 1) taken at ground surface and immediately below the wetting front, respectively, Equation 1 becomes:

$$q = -K \left[\frac{(Z_1 + \psi_1) - (Z_2 + \psi_2)}{(Z_1 - Z_2)} \right] \qquad (2)$$

where,

ψ = pressure head
Z = elevation head, and
$h = \psi + Z$.

Rewriting:

$$q = -K \left(\frac{\psi_1}{L_f} - \frac{\psi_2}{L_f} + 1 \right) \qquad (3)$$

where,

$L_f = Z_1 - Z_2$.

By reviewing Figure 1, it can be seen that at point 1 the pressure head is equal to the depth of flooding, or $\psi_1 = D_f$. Also, since the clay fill is unsaturated, the in situ pressure head at point 2 will be negative and can be designated simply as $\overline{\Psi}$, i.e., $\overline{\Psi} = -\psi_2$.

Substituting into Equation 3:

$$q = -K \left(1 + \frac{D_f}{L} + \frac{\overline{\Psi}}{L_f}\right) \qquad (4)$$

The minus sign indicates that flow is in the negative Z direction (downward).

Equation 4 is time dependent. That is, the flow rate per unit area (infiltration rate, q) and the depth of the wetting front (L_f) are interrelated and vary with time. As the wetting front advances, Equation 4 can be used to calculate hydraulic conductivity at various wetting front depths, provided values of soil tension ($\overline{\Psi}$) are measured at these depths using tensiometers.

The position of the wetting front at a point in time is evidenced by the soil tensiometer readings falling to zero. In Equation 4, the length of the wetting front (L_f) is known by noting the depth at which the tensiometer was installed. The depth of flooding (D_f) is taken as an average value. The soil suction ($\overline{\Psi}$) is set equal to the average soil tension preceding the passage of the wetting front. The infiltration rate (q) is determined by weighing the flexible bag (Figure 1) periodically to determine the volume of water lost. The change in volume is then divided by the area of the inner ring and the elapsed time over which the volume change occurred. The values of q, D_f, L_f, and $\overline{\Psi}$, are then used to back calculate K from Equation 4. In addition, it is possible to evaluate K for individual layers of the clay fill, i.e., zero to 6 inches, 6 inches to 12 inches, zero to 12 inches, etc.

The test procedure and analysis methodology described above, is based on the assumptions; Darcy's law applies, the test fill is homogeneous and isotropic, the flow from the infiltration ring is vertically downward, and a discrete and well defined wetting front exists between the saturated soil and the partially saturated soil. The assumption of the well-defined wetting front is valid at early stages of the test, but may not be totally valid at subsequent times since a transition zone between the saturated and partially saturated soil is likely to exist. The errors which could be introduced by the possible limited validity of the assumptions, all result in hydraulic conductivity values which will be too high. Consequently, the test results are conservative.

The other method used to analyze the infiltrometer test is suggested by the manufacturer of the test apparatus in their technical literature. This method assumes that steady state conditions exist and soil tension does not contribute to the hydraulic gradient. In this method, Equation 4 is evaluated using the same values as used above, but setting soil tension equal to zero. The depth of the saturated zone will be identified by the depth of the tensiometer which reads zero soil suction (i.e., complete saturation).

CASE HISTORY

A test fill was constructed between August 19 and September 4, 1986 on a site just outside of Chemical Waste Management, Inc., Kettleman Hills, California facility. The test fill is approximately 140 feet long by 50 feet wide at the surface, with a depth between three and three and one half feet.

The test fill was underlain with a drainage layer of geonet and geotextile. Prior to hauling soil to the test fill each day, the admix stockpile was moisture conditioned to maintain a moisture content of approximately 30 percent. The admix was placed in approximately eight inch loose lifts. Compaction consisted of wheel rolling by the routing of the scrapers used to haul the soil, and two complete passes with a sheepsfoot compactor.

After the compactor made its last pass, the lift was tested to assure the lift had the proper water content and density, and samples were obtained for laboratory hydraulic conductivity tests. At the end of each day the test fill was wheel rolled to seal the surface and minimize the moisture lost from the soil. Before a new lift was applied, the underlying lift was scarified and moisture conditioned as a method to improve the bonding between lifts.

Field Measurements

After placement and compaction of each lift, a series of field measurements were conducted. The field measurements consisted of nuclear density tests and sand cone density tests for each lift. In addition, samples of admix at each nuclear density test location were obtained for laboratory water content tests and undisturbed shelby tube samples were also obtained from select lifts for laboratory hydraulic conductivity tests. Occasionally, pocket penetrometer tests were also conducted to determine the undrained shear strength of the compacted admix.

Laboratory Tests

Laboratory tests were conducted on bulk samples of the admix stockpile to determine the relationship between the degree of compaction, water content and hydraulic conductivity. Tests included index tests, compaction tests and hydraulic conductivity tests.

Shelby tube samples were taken from select lifts of the test fill. Portions of the shelby tubes were then tested to determine the hydraulic conductivity of each lift. The results of these tests are tabulated in Table 1.

TABLE 1. Test Fill Permeability Results

Tested Depth	Dry Density* (lb/ft^3)	Water Content* (percent)	K (cm/sec)
lift 1	89.0	30.2	4×10^{-9}
lift 2	90.0	30.7	1×10^{-9}
lift 3	92.5	29.5	4×10^{-9}
lift 4	87.8	31.2	7×10^{-9}
lift 5	94.2	26.5	1×10^{-8}
lift 6	97.6	24.3	2×10^{-8}
lift 7	92.7	30.7	2×10^{-9}

* Density and water contents from lift near permeability sample.

$1\ lb/ft^3 = 16.02\ kg/m^3$

INFILTROMETER TEST

Site Preparation and Installation

The test fill was prepared for the infiltrometer by using a motor grader and a vibratory drum roller to level and smooth the test fill's surface. The surface was lightly sprayed with water and then covered with a black plastic tarp.

The outer rings for both sets of infiltrometers were positioned on the tarp and their outlines were marked on the tarp to locate the trenches for the rings. The outer rings trenches were cut with a Ditch Witch, series 1420, to a depth of 18 inches. The inner rings were positioned in the center of the outer rings and their outlines marked. The five inch deep trenches for the inner rings were cut by hand using mason's hammers.

The outer ring trenches were sealed with bentonite pellets while the outer ring was in the trench. The trenches for the inner rings were sealed with a thin layer of bentonite pellets in the bottom of the trenches and the remainder of the depth filled with a viscous Voclay grout. The inner rings were placed in their trenches during the grouting process.

The inner ring was checked for leaks by adding a little water to it. The outer ring was flooded until the inner ring was slightly submerged, then the inner ring was partially filled. The outer ring was filled to its final depth and the inner ring was topped off. The bags and hoses for the measurement of water infiltrating through the inner rings were attached, the inner ring was purged of air, and the tests began.

Data Collection

Each inner ring has a heavy duty flexible bag of water connected to it in order to provide a volume of water to replace that water which infiltrated the test fill beneath the inner ring. The bags were fitted with no volume change valves to allow new (refilled) bags to be exchanged after the original bags were depleted without any water loss. The difference in the initial and final weight of the bag is the amount of infiltration that occurred over that particular time period. During the first days of the test, several bags of water were needed each day. In less than a week, each bag was changed once a day.

In addition to the amount of infiltration, data collected daily for each ring included: depth of water in the outer ring, temperature of the water, and tensiometer values. Notes about the condition of the water (i.e., algae growth) and the test fill, and the weather were recorded when deemed relevant. Periodically, the inner rings were purged of any air which may have collected in the inner rings. The air volume was measured and the the accumulative infiltration adjusted to include this volume. In general, the volume of air purged from the system was very small compared to the volume of water infiltrating on a daily basis.

Results Based on Using Tensiometers

Both the North and South Rings have two tensiometers, each at the depths of 6, 12, 18, and 24 inches (a total for both rings of 16 tensiometers). The tensiometers register the passage of the wetting front by displaying a soil tension of zero. All of the tensiometers went to zero at different times; therefore, the values used to solve for the hydraulic conductivity are different for each of the tensiometers. Figures 2 and 3 present the accumulated infiltration as a function of time for the North and South Rings. Figure 4 presents two examples of the soil tension versus time. The values used in Equation 4 are listed in Table 2.

Figure 2. Accumulative Infiltration Versus Time (North Ring)

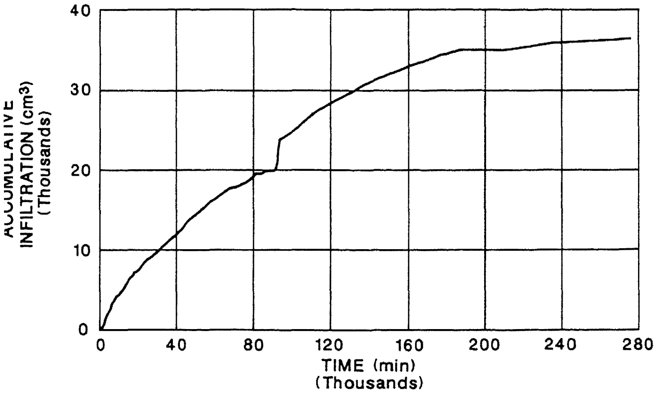

Figure 3. Accumulative Infiltration Versus Time (South Ring)

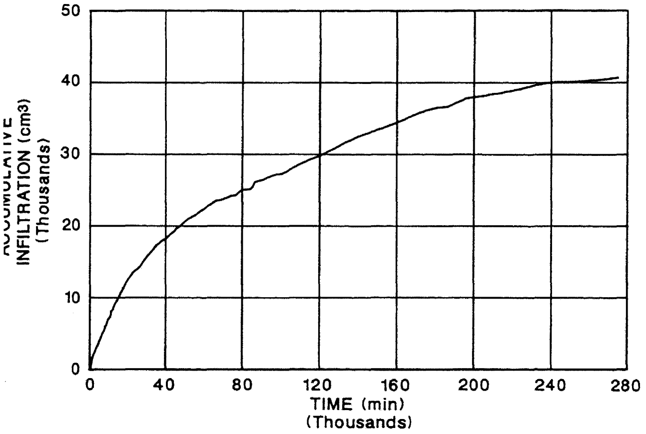

Figure 4. Soil Tension Versus Time at 24 Inch Depth (North Ring)

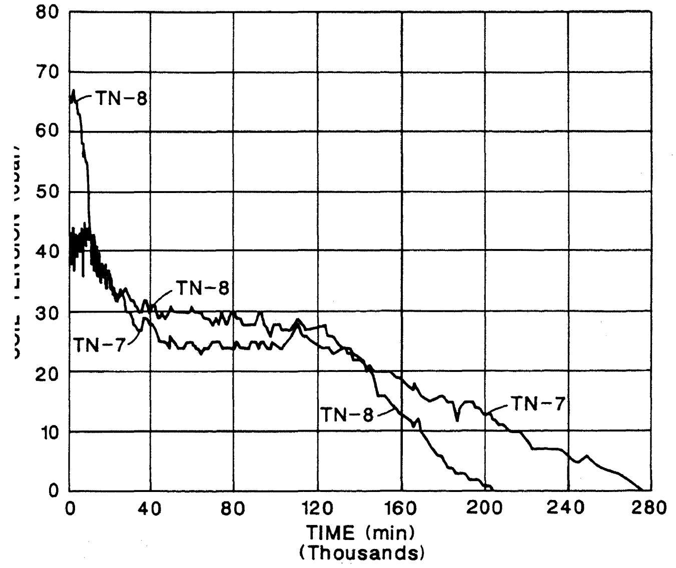

TABLE 2. Infiltrometer Test Values

Tensiometer	q (cm/sec)	D_f (in)	L_f (in)	Ψ (in)
TN-1	3.1×10^{-7}	11.1	6	60.3
TN-2	2.3×10^{-7}	11.1	6	100.5
TN-3	1.1×10^{-7}	11.4	12	76.4
TN-4	1.6×10^{-7}	11.4	12	76.4
TN-5	9.9×10^{-8}	11.5	18	92.5
TN-6	1.3×10^{-7}	11.4	18	100.5
TN-7	4.1×10^{-8}	11.5	24	56.3
TN-8	5.5×10^{-8}	11.4	24	56.3
TS-1	4.3×10^{-7}	10.8	6	112.6
TS-2	4.2×10^{-7}	10.8	6	74.4
TS-3	1.5×10^{-7}	10.8	12	60.3
TS-4	1.9×10^{-7}	10.8	12	48.2
TS-5	8.7×10^{-8}	10.9	18	56.3
TS-6	8.6×10^{-8}	10.9	18	30.2
TS-7	6.4×10^{-8}	10.9	24	68.3
TS-8	8.0×10^{-8}	10.9	24	64.3

NOTE: Ψ (in) = Ψ (cbars) x 4.02

1 in = 2.54 cm

The hydraulic conductivity for the soil between two particular depths is the geometric mean of the separate values for that range of depth. Applying the values given in Table 2 to Equation 4, the range of hydraulic conductivity for the soil of depths for each six inch thickness of soil is presented in Table 3.

TABLE 3. Summary of Infiltrometer Test Accounting for Soil Tension

Soil Depths (in)	Range in Permeability (cm/sec)	Geometric Mean (cm/sec)
0 - 6	$1 \times 10^{-8} - 3 \times 10^{-8}$	2×10^{-8}
6 - 12	$1 \times 10^{-8} - 3 \times 10^{-8}$	2×10^{-8}
12 - 18	$1 \times 10^{-8} - 3 \times 10^{-8}$	2×10^{-8}
18 - 24	$1 \times 10^{-8} - 2 \times 10^{-8}$	1×10^{-8}

1 in = 2.54 cm

The composite hydraulic conductivity for the soil from zero to twenty four inches deep is calculated by applying to Equation 5, the mean hydraulic conductivity for the different ranges of depth.

Applying the geometric means given in a preceding paragraph, the equivalent hydraulic conductivity for the depths from zero to twenty four inches is 2×10^{-8} cm/sec.

Results Without Considering Soil Suction

An alternative methodology suggested for an sis of the test provided by the manufactur based upon the assumption that soil sucti not a factor in the hydraulic gradient. assumption is valid only when steady st reached. Assuming steady state is att when and where the soil is saturated, 4 may be applied if the soil suction term is set to zero. The resulting hydraulic conductivity ranges are shown in Table 4.

Applying these values to Equation 5, the equivalent hydraulic conductivity for the depths from zero to twenty four inches is $6x10^{-8}$ cm/sec.

$$K = \frac{D}{\frac{d_{0-6}}{K_{0-6}} + \frac{d_{6-12}}{K_{6-12}} + \frac{d_{12-18}}{K_{12-18}} + \frac{d_{18-24}}{K_{18-24}}} \qquad (5)$$

where K = equivalent hydraulic conductivity (L/T)

D = total depth (L) (= 24 in)

d_{a-b} = depth of the zone a to b (L) (= 6 in)

K_{a-b} = hydraulic conductivity of the zone a to b (L/T)

TABLE 4. Summary of Infiltrometer Test Without Accounting for Soil Tension

Soil Depths (in)	Range in Permeability (cm/sec)	Geometric Mean (cm/sec)
0 - 6	$8 \times 10^{-8} - 2 \times 10^{-7}$	1×10^{-7}
6 - 12	$6 \times 10^{-8} - 1 \times 10^{-7}$	8×10^{-8}
12 - 18	$5 \times 10^{-8} - 8 \times 10^{-8}$	6×10^{-8}
18 - 24	$3 \times 10^{-8} - 6 \times 10^{-8}$	4×10^{-8}

1 in = 2.54 cm

Discussion of Results

The tensiometers had different values and registered zero soil suction at different times for the same depth. This is expected for a material that had different moisture contents when placed. In the North Ring, one of the tensiometers at the 18 inch depth measured zero soil suction after one of the 24 inch tensiometers on the opposite side of the North Ring had measured zero. This could be the result of different moisture contents and compactive efforts of the placed material causing a greater hydraulic conductivity on one side of the infiltrometer. Tensiometers TN-3, TN-5, and TN-7 all recorded zero soil suction after their counterparts. A review of the field data for the density and moisture content of the soil from 12 to 18 inches revealed one test on the south side of the North Ring which had a low density and a high moisture content. This moisture/density relationship may result in a higher hydraulic conductivity, but the true effect can not be quantified.

The scale of the infiltrometer tests appears to be evaluating the overall hydraulic conductivity of the test fill rather than the hydraulic conductivity of discrete samples, which is accomplished in laboratory tests. The hydraulic conductivity results of the infiltrometer tests utilizing the tensiometers are consistent between the North and South Rings and between the soil at different depths.

The infiltrometer test results, which accounted for soil tension, compared well with the laboratory results for hydraulic conductivity. The geometric mean of the laboratory results is $5x10^{-9}$ cm/sec, and the geometric mean of the infiltrometer tests is $2x10^{-8}$ cm/sec. The difference between the means of less than an order of magnitude is considerably better than previous investigations (Day and Danniel, 1985a). The good results in the field tests underscore the importance of quality control during construction of the test fill and installation of the infiltrometer.

The analysis of the infiltrometer data without accounting for soil tension reveal that the differences between the infiltrometer and the laboratory results may be as much as two orders of magnitude. If the full thickness of the test fill were considered saturated when only the first six inches were saturated, the gradient would be 1.31 ($1 + Df/Lf = 1 + 11/36$). The resulting hydraulic conductivity would range from $2x10^{-7}$ cm/sec to $3x10^{-7}$ cm/sec. Clearly this method of analysis gives an overly conservative upper bound of the hydraulic conductivity of the material.

At the end of the test, the system can be assumed to be saturated through its entire depth. No water was noted to drain out the underlying drainage layers. Since the system drains to the edge of the test fill, any water draining from the system would be hard to notice because of the high evaporation of the site and low flow rates. Using the last infiltration rates for the two infiltrometer sets ($1.5x10^{-8}$ cm/sec and $1.6x10^{-8}$ cm/sec), and assuming steady state, the hydraulic conductivity would be $1x10^{-8}$ cm/sec.

The test fill took over 200 days to be saturated through the first two feet. To shorten the time to conduct the test, the tensiometers could have been used to identify the saturated depth. If the analysis included the use of the tensiometers to identify the depth of saturation without accounting for the soil tension, the hydraulic conductivity results after the first six inches of the test fill were saturated would only be $8x10^{-8}$ cm/sec to $2x10^{-7}$ cm/sec. The first six inches took just over 21 days to be completely saturated. Therefore, use of the tensiometers just to note the saturated depth does not improve the results significantly, while using the tensiometers to quantify soil suction does improve the analysis significantly.

CONCLUSIONS

An in situ hydraulic conductivity test using two sets of sealed double ring infiltrometers was conducted with the soil suction measured by tensiometers. The infiltrometer tests have worked to adequately characterize the in situ hydraulic conductivity of the clay liner. The results display good consistency between the North and South Rings and between the soil at different depths. In addition, the laboratory hydraulic conductivity tests conducted on samples of the test fill correlate well with the results from the infiltrometer. Using the tensiometers and accounting for soil tension could reduce the testing time by approximately a factor of ten. Using the tensiometers merely to identify the saturated thickness may improve the correlation between the laboratory and field hydraulic conductivity tests, but not improve it significantly.

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