PERMEABILITY DETERMINATION
FOR LANDFILL STUDIES

BY

JOEL GEORGE SMITH
B.S.E., Florida Technological University, 1972

RESEARCH REPORT

Submitted in partial fulfillment of the requirements for the degree of Master of Science in Engineering in the Graduate Studies Program of Florida Technological University

Orlando, Florida
1973
ABSTRACT

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This report reviews the state of the art with respect to permeability determination and sanitary landfills. Characteristics of the soil which determine the permeability are given. Processes which can change the permeability are discussed.

Darcy's Law, the mathematical basis of permeability and its validity are discussed. Laboratory and field methods for determining the permeability are also discussed. Applications of determined permeability for design and management of landfills are also indicated.

Director of Research Report
ACKNOWLEDGEMENTS

I would like to thank my advisor, Mr. J. P. Hartman and my committee members, Dr. W. M. McLellon and Dr. M. P. Wanielista for their technical expertise and wise counsel in helping me prepare this report. My typist, Mrs. Donna Wood, has also greatly contributed with her fine work and patience.
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2. Permeability vs Hydraulic Gradient for Ottawa Sand
3. Sketches of Equipment Used in Laboratory Determination of Permeability
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INTRODUCTION

Permeability is a useful and complex parameter and is a necessary quantitative measure of fluid flow in porous materials. Consequently, permeability has applications in many fields of engineering, particularly in dam seepage, slope stability, settlement analyses, dewatering, ground water flow, etc.

Recent interest in the environmental impact of projects has increased investigation of the impact of sanitary landfills. This solid waste disposal system utilizes the subsurface deposition of refuse and subsequent covering with natural fill. If this matter decomposes in a saturated condition a leachate is formed. The leachate, which is a mineralized water, can reduce the quality of ground waters in the area surrounding the landfill site. The movement of the leachate will depend on the contents, spatial distribution and ground water movement at the site (Remson et al, 1968). The California State Department of Water Resources has classified the acceptability of landfill sites with respect to the transmissibility (determined from the permeability) of underlying geological structures (Coe, 1970).

Until recently most studies had been performed in the arid southwest where leachate transmission did not present serious problems. However, geophysical studies performed in Illinois (Cartwright et al, 1968) utilizing electrical earth resistivity and soil temperature surveys showed a direct relationship between leachate movement and permeability of subsurface strata.
Permeability plays an important role in the site selection and management of sanitary landfills. Thus, expenditure of resources is justified in determination of permeability with accuracy within acceptable limits. Leachate affects on the time rate of change of permeability also deserve investigation.

This report reviews the permeability determination techniques for use in sanitary landfills and indicates areas for future study. Factors affecting permeability, the theoretical basis of permeability, its validity, methods for determining permeability and methods of applications to landfill sites will be described.
CHAPTER I
FACTORS AFFECTING PERMEABILITY

The permeability of a soil is the ease with which a fluid (water) will be transmitted through the soil under the influence of a hydraulic gradient. An increase in permeability corresponds to an increase in the quantity of flow under a constant hydraulic gradient. Physical characteristics of the soil will determine the permeability. Table 1 shows approximate permeability, drainage and soil classification comparisons. For clean sands the permeability can vary from 1.0 to 0.001 cm/sec.

An idealized representation of a sandy soil would be a bed of spheres. Geological processes which form a soil, however, produce a mixture of particles of many different sizes and shapes. Permeability is affected by five major physical characteristics of the soil; particle size and shape, void ratio, composition, soil fabric, and degree of saturation (Lambe, 1969). Particle size influences the sizes of the pores through which the fluid must pass. The pore area is represented by the void ratio or porosity which is indicative of the density of the soil.

The particles in a soil are classified as sand, silt, or clay. Classification is based on grain size and cohesive properties of the soil grains. Various agencies have different criteria for soil classification. The composition of the soil is determined by the relative amount of each type of soil particle. Sand percentages are
TABLE 1
Coefficient of Permeability

<table>
<thead>
<tr>
<th>Permeability (k) (cm/sec)</th>
<th>10^2</th>
<th>10</th>
<th>1.0</th>
<th>10^-1</th>
<th>10^-2</th>
<th>10^-3</th>
<th>10^-4</th>
<th>10^-5</th>
<th>10^-6</th>
<th>10^-7</th>
<th>10^-8</th>
<th>10^-9</th>
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<tbody>
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<td>Good Drainage</td>
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<tr>
<td>Clean Sands</td>
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<td>Mixtures of Sands, Silts and Clays</td>
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</tbody>
</table>

Soils modified by the effects of Weathering and Vegetation

(After, Casagrande 1939)
measured by a mechanical sieve analysis and silt and clay percentages are determined by hydrometric analysis or centrifuge tests (Lambe, 1951).

The fabric or structure of a soil is determined by the arrangement of the soil particles. Arrangement of the particles affects the pore geometry and determines the tortuosity of the path that the water must follow. Degree of saturation is indicative of the amount of air in the soil void spaces. This mixture of two fluids (water and air) increases the resistance to flow through the soil. Determination of permeability is usually performed under completely saturated conditions (volume of air = zero) although the effects of saturation can be accounted for in calculations of seepage rate (Fok, 1970).

These characteristics are qualitative in nature and the assignment of quantitative figures to their relative effect on the permeability has not been particularly successful for natural soil conditions. Table 2 shows the comparison between laboratory permeability on a landfill site (McLellon, 1973) obtained in a constant head permeameter and those recommended for use in the Navfac Design Manual (Navfac, 1971). This manual related the $d_{10}$ grain size and void ratio to the expected permeability. The correlation was not accurate and variation was not predictable. However, an understanding of soil characteristics is required for determining the effects that change in soil condition will have on the permeability. The leachate from a sanitary landfill can be expected to change the permeability of the soil. Predicting such changes can increase the success of the management program at the landfill site.
### TABLE 2

**COMPARISON OF PERMEABILITY OBTAINED BY GRAPHICAL METHODS AND LABORATORY TESTS**

<table>
<thead>
<tr>
<th>Sample Number (McLellon, 1973)</th>
<th>Grain Size $d_{10}$ (mm)</th>
<th>Void Ratio $e$</th>
<th>$k_{DM-7}^*$ $(10^{-4}$ cm/sec)</th>
<th>Permeability $k_{Lab}^{**}$ $(10^{-4}$ cm/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>0.2</td>
<td>0.66</td>
<td>507.0</td>
<td>7.4</td>
</tr>
<tr>
<td>0.80</td>
<td>0.15</td>
<td>0.72</td>
<td>268.0</td>
<td>82.5</td>
</tr>
<tr>
<td>0.160</td>
<td>0.16</td>
<td>0.60</td>
<td>162.0</td>
<td>2.3</td>
</tr>
<tr>
<td>150.70</td>
<td>0.19</td>
<td>0.66</td>
<td>491.0</td>
<td>13.2</td>
</tr>
<tr>
<td>150.150</td>
<td>0.13</td>
<td>0.70</td>
<td>482.0</td>
<td>17.1</td>
</tr>
<tr>
<td>150.230</td>
<td>0.12</td>
<td>0.61</td>
<td>65.8</td>
<td>41.9</td>
</tr>
<tr>
<td>300.0</td>
<td>0.19</td>
<td>0.70</td>
<td>482.0</td>
<td>13.8</td>
</tr>
<tr>
<td>300.80</td>
<td>0.15</td>
<td>0.74</td>
<td>268.0</td>
<td>20.4</td>
</tr>
<tr>
<td>300.160</td>
<td>0.34</td>
<td>0.66</td>
<td>1730.0</td>
<td>8.2</td>
</tr>
<tr>
<td>450.0</td>
<td>0.39</td>
<td>0.66</td>
<td>1420.0</td>
<td>58.2</td>
</tr>
<tr>
<td>450.80</td>
<td>0.22</td>
<td>0.66</td>
<td>698.0</td>
<td>36.4</td>
</tr>
<tr>
<td>450.160</td>
<td>0.12</td>
<td>0.66</td>
<td>127.0</td>
<td>43.4</td>
</tr>
</tbody>
</table>

* $k_{DM-7}$ Permeability from Navfac DM-7 (1971)

** $k_{Lab}$ Permeability obtained from constant head permeability test.
In landfill applications, the soil will act as a straining mechanism on the leachate. Four processes have been defined which operate in removing suspended particles from the leachate (Krone, 1967). These processes are surface straining, bridging, sedimentation, and adhesion and cohesion. Surface straining will increase the resistance to flow as the particles accumulate at the soil surface. Bridging occurs when the particles are larger than the soil pores. Sedimentation can occur due to the fluctuating flow velocity in the soil pores. Adhesion and cohesion occur in an interaction between the surfaces of the suspended particles and the soil particles. Accumulation of suspended solids in the soil pores will promote growth of bacterial slimes which when combined with the above processes can be expected to decrease the permeability of the soil.

The chemical character of the leachate can also affect the permeability. This effect is generally through changes induced in the soil fabric, although plugging of the soil pores by large molecular complexes also must be considered.

Iron oxides and sodium in the leachate will disperse soil particles, increasing the permeability. This occurs when there are clay particles present. However, in all soils iron and calcium can precipitate to form impervious layers which will decrease the permeability. Organic matter in clayey soils will increase the permeability by aggregation of the particles (Horn, 1971). The importance of these chemical effects warrants further investigation with respect to leachate applications.

Physical processes which can decrease the soil permeability are compaction and wet cultivation (Horn, 1971). These factors could
be used to decrease the amount of water entering the cells at a sanitary landfill. Avoiding them would have the reverse effect.

It is seen then that soil permeability can be a parameter of use in managing sanitary landfills. Obviously it is important in the site selection of a landfill. The rate at which water will move from the site is determined from the permeability of the soil. An understanding of the theoretical derivation, and determination of the permeability will be useful.

DERIVATION OF DARCY'S LAW

Permeability determinations are based on Darcy's Law (Darcy, 1856). In his studies, he related the bulk velocity of flow in porous media to the first power of the hydraulic gradient by the coefficient of permeability. This law is generally assumed valid for laminar flow.

\[ v = ki \]  

(1)

Where:  
\( v \) = Discharge velocity  
\( k \) = Permeability  
\( i \) = Hydraulic gradient

In deriving the differential form of Darcy's Law, the saturated condition is assumed. Consider an elemental soil volume \( dV = dA/d\ell \) (Hautush, 1964), where \( dA \) is the elemental area and \( d\ell \) is the elemental length of the volume. A force equilibrium would exist for the case of steady flow. The individual forces to be considered are the pressure force, weight force and viscous force (See Figure 1). The pressure force, \( f_p \), is the difference of the pressure at the entrance of the volume and the pressure at the end of the volume, \( p - (p + \delta p/\delta \ell) \).
FIGURE 1

Force Diagram Used in Deriving Darcy's Law
The net force $f_p$ is

$$f_p = -n \, dA \, (\delta p/\delta \ell) \, d\ell \quad (2)$$

Where: $n = \text{Porosity (the ratio of void volume to total soil volume)}$

It is assumed that the net forces will be representative of the forces in the individual pores. The weight force is

$$f_g = -\gamma n \, dA \, d\ell \, (\delta z/\delta \ell) \quad (3)$$

Where $\gamma$ is the unit weight of the fluid, $z$ is the coordinate above an $XY$ plane and $(\delta z/\delta \ell)$ is the sine of the angle $d\ell$ makes with the $XY$ plane. The viscous force opposing flow is

$$f_\mu = -\mu c \, dA \, d\ell \, v \quad (4)$$

Where $\mu$ is the dynamic viscosity, $c$ is a constant characteristic of the pore geometry and the soil grain surface contacting the fluid (specific surface) and $v$ is the discharge velocity. Summation of the forces in equilibrium yields:

$$- \delta p/\delta \ell - \gamma (\delta z/\delta \ell) = + \frac{\mu c}{n} \, v \quad (5)$$

The velocity is given by:

$$v = - \frac{\delta (p/\gamma + z)}{\delta \ell} \frac{\gamma n}{\mu c} \quad (6)$$

The intrinsic permeability, $P$, is substituted for $n/c$ and has dimensions of $L^2$ (Walton, 1970). The total head acting on the soil is

$$h = (p/\gamma + z + f) \quad (7)$$

Where $f$ is the elevation of the $XY$ plane above an arbitrary datum.

In differential form with respect to the elemental length, the head is

$$\frac{\delta h}{\delta \ell} = \frac{\delta (p/\gamma + z)}{\delta \ell} \quad (8)$$
The velocity can now be expressed as

\[ v = -\frac{\gamma P}{\mu} \frac{\delta h}{\delta x} \]

(9)

Where the ratio \( \frac{\delta h}{\delta x} \) is the hydraulic gradient. The negative sign indicates the direction of flow and is not used in Darcy's Law. The permeability, \( k \), is equal to \( \frac{\gamma P}{\mu} \). Note that the permeability is proportional to the intrinsic permeability and inversely proportional to the viscosity.

Assumptions in the derivation of Darcy's Law have raised questions with respect to the validity of its application to permeability studies. These are:

1. The flow is laminar
2. The net force and hence the net velocity is representative of the forces and velocities occurring in the soil pores
3. Inertial forces are insignificant.

Further discussion is presented to summarize the work of investigators related to the validity of Darcy's Law. However, the test procedures for determining permeability and seepage rate assume that Darcy's Law is valid.

VALIDITY OF DARCY'S LAW

Much discussion has occurred on the applicability of Darcy's Law to soil permeability. The assumption of laminar flow appears to be the most controversial point in applying the work of Darcy. If the velocity determined by the expression

\[ v = ki \]

(1)

is divided by the porosity of the soil, a seepage velocity is determined.
Although this average seepage velocity is more indicative of the velocity in the soil pores, no method is available for determining the true pore velocity.

Hantush (1964) relates the flow through soils to an analogous flow through capillary tubes and uses the Reynolds number as an index to the limit of laminar flow. The expression for the Reynolds number used is:

\[ R = \frac{\rho v d_{10}}{\mu} \]  \hspace{1cm} (11)

Where:
- \( v \) = Bulk velocity
- \( \rho \) = Fluid density
- \( \mu \) = Dynamic viscosity
- \( d_{10} \) = Mean grain diameter, or diameter such that 10% by weight is of smaller size and 90% is of larger size

He gives the transition of laminar to turbulent flow as occurring at Reynolds numbers of 1 to 10. Darcy's Law would then be applicable since typical Reynolds numbers for soils were shown to be less than unity. No experimental data was shown to support these findings.

Rumer (1964) stated the transition of laminar to turbulent flow was not the cause of non-Darcy behavior. He proposed that resistance to flow should be the summation of the drag forces of each individual particle in the soil, given by Oseen's expression

\[ D = 3\pi \mu d U (1 + 3/16 U \frac{d \phi}{\mu}) \]  \hspace{1cm} (12)

Where:
- \( D \) = Drag force
- \( d \) = Particle diameter
\[ U = \text{Magnitude of parallel stream of uniform velocity} \]
\[ \rho = \text{Fluid density} \]

This expression would be valid up to a Reynolds number of 5. For flow conditions it would be incorporated into an expression such as equation 5. His explanation for deviation from Darcy's Law was an increasing influence of inertial forces in the higher laminar flow velocities. Since the pore velocity cannot be accurately measured, his presentation does not seem of much use in practical application of soil permeability.

York (1970) found that although Darcy's Law was not valid for flow in coarse grained materials it was valid for hydraulic gradients of 4 to 1. At other gradients, the permeability was found to vary with the hydraulic gradient. This finding is supported by others (Anandakrisnan et al, 1964); (Burmister, 1954). Lane (1964) stated this variation at higher hydraulic gradients was due to specimen changes, such as densification, grain rearrangement, and removal of air bubbles from the fluid. The other authors did not reach these conclusions however.

The variation in permeability with hydraulic gradient, has led to the introduction of new mathematical equations for flow. Some of these are (Harr, 1962)

\[ i = a v + b v^n \]  \hspace{1cm} (13)

and (Anandakrishnan et al, 1964)

\[ v^n = k'i \]  \hspace{1cm} (14)

Where: \( a, b \) are constants

\( k' = \) Coefficient of turbulent flow
\[ \eta = \text{Turbulence exponent} \]

The major problem in applying equations of this type is that two or three parameters must be determined rather than one. They add complexity to the problem rather than solving it.

To give a quantitative idea of the magnitude of the inaccuracy, Table 3 and Figure 2 give the permeability of Ottawa sand at different hydraulic gradients and porosities. The data are calculated from constant head permeability tests performed by the author.

The major error in applying the permeability determined in tests to actual conditions seems to lie in the difference in the hydraulic gradient which is used in the test and that which exists in the field condition. If the field gradient can be estimated the test condition can duplicate it with sufficient accuracy (Anandakrishnan et al, 1964). Ward (1964) states that:

1. Any equation for flow should reduce to Darcy's Law as the velocity decreases.

2. Constants should be characteristic of the fluid and the media and should not vary with the velocity.

The foregoing discussion summarizes the debate on the validity of Darcy's Law. Various authors suggest that the assumptions inherent in Darcy's Law limit its applicability to natural soil conditions. Limitations of their discussions must also be considered. Hantush related the flow in soils to flow in capillary tubes. Based on this relation the flow should change from laminar to turbulent at a critical Reynolds number. However, this phenomenon has not been shown for soils (Lee, 1968). Rumer's conclusions are intuitively correct, but no method of applying them to soil application is available. The same applies to the formulas of Harr and Anandakrishnan and
<table>
<thead>
<tr>
<th>Porosity (n)</th>
<th>Hydraulic Gradient (i)</th>
<th>Permeability ($k_{20}$)*</th>
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<td>0.42</td>
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<td>3.80</td>
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</table>

* $k_{20}$ Expressed as $10^{-2}$ cm/sec at $20^\circ$C
Permeability vs. Hydraulic Gradient for Ottawa Sand (Note TABLE 3)
Varadarajuler. The latter presented methods for determining the required constants based on a permeability test in which Darcy's Law was assumed correct. However, this assumption is made in all the permeability tests examined. Experience has shown that these tests can be used in a practical application, within the limits of accuracy of each method.
CHAPTER II
LABORATORY DETERMINATION OF PERMEABILITY

Determination of permeability in the laboratory is done by three methods: constant head, falling head and oedometer. The oedometer method is primarily used for consolidation studies and does not seem to have specific applications in sanitary landfill analyses.

The soils on which the tests are performed are either undisturbed or remolded samples. Remolded samples are not necessarily representative of field samples, in that soil structure is altered during the remolding process. Sample disturbance must be minimized to reproduce natural conditions as accurately as possible.

A schematic diagram of the apparatus used in the constant head test is shown in Figure 3a. The permeability is determined by collecting a volume of water, Q, in time, t. The relationship is then:

\[ k = \frac{QL}{hAt} \]  \hspace{1cm} (14)

Where:
- \( L \) = Sample length
- \( h \) = Constant head
- \( A \) = Area of the sample

Rearranging terms yields:

\[ \frac{QL}{At} = kLh \]  \hspace{1cm} (15)

or Darcy's Law:

\[ v = ki \]  \hspace{1cm} (1)
FIGURE 3 Sketches of Equipment Used in Laboratory Determination of Permeability.
Note that the above test directly relates a superficial velocity of flow (discharge velocity) to the hydraulic gradient and is also useful for turbulent conditions (Anadakrishnan et al, 1964).

Evaporation of water from the supply reservoir can effect results in tests run over long periods of time. York (1970) shows that with proper precaution, errors from this source can be avoided.

The falling head device is shown in Figure 3b. The equation for permeability is derived by noting the quantity of flow per time, \( q \), is:

\[
q = \frac{dh}{dt} \cdot A_t
\]  

(16)

Where:

- \( dh \) = Change in head
- \( dt \) = Change in time
- \( A_t \) = Area of the tube

Also from Darcy's Law:

\[
q = \frac{k \cdot h \cdot A}{L}
\]  

(17)

Where:

- \( q \) = Quantity of flow per time
- \( k \) = Permeability
- \( h \) = Head
- \( L \) = Sample length
- \( A \) = Sample area

Equating (16) and (17) yields:

\[
k \cdot h \cdot A = -\frac{dh}{dt} \cdot A_t \quad \text{or} \quad \frac{-dh}{h} = \frac{KA_t}{LA} \cdot dt
\]  

(18)

(19)

Applying the boundary condition, as time changes from 0 to \( t \), \( h \) changes from \( h_1 \) to \( h_2 \) and integrating yields:
Rearranging:

\[
\frac{kA_t}{t} = -\ln \frac{h_2}{h_1} = \ln \frac{h_1}{h_2}
\]  

(20)

For this test, Darcy's Law has also been assumed valid (equation 17). Also the laboratory procedure can not be used to duplicate a field condition where a constant hydraulic gradient exists.

In engineering work, permeability is generally reported at a standard temperature of 20°C, and the temperature correction is given by (Lambe, 1969):

\[
k_{20} = \frac{\mu_T}{\mu_{20}} k_T
\]  

(22)

Where:

\[k_{20} = \text{Permeability at } 20°C\]

\[k_T = \text{Permeability at temperature } T\]

\[\mu_{20} = \text{Viscosity of water at } 20°C\]

\[\mu_T = \text{Viscosity of water at temperature } T\]

This correction can cause variation by a factor of 1.8 at 0°C to 0.7 at 40°C.

Several problems exist in laboratory permeability measurements. Complete sample saturation must be accomplished before the permeability is representative of the sample. Saturation can be accomplished by subjecting the sample to a partial vacuum before water is passed through the sample. Disturbance of the sample must be avoided in this procedure. Leakage between the sample and the permeameter wall has been shown in some cases to affect the determined permeability by as much as a factor of 35 (York, 1970). This problem can be avoided.
by use of a membrane between the sample and the wall. Sample disturbance can occur in transporting the soil from the field to the laboratory and should be minimized.

The samples used in laboratory tests are usually less than 10 cm in diameter and yet are often assumed to be representative of the field condition. This can be an obvious problem. Since there are also several sources of operator error in the laboratory tests, the reliability of the results may be questioned. An alternative which can increase the reliability is determination of permeability by field tests.
CHAPTER III
FIELD DETERMINATION OF PERMEABILITY

The decision to conduct field permeability tests is influenced by several characteristics of the test. First, the soil is tested in its natural conditions. Sample disturbance is minimized in carefully controlled tests. Secondly, sources of error due to the laboratory equipment evaporation and wall leakage, are avoided. Lastly, the area which influences the test results in larger and hence, a more representative figure for the permeability should result. Three methods of field testing, the auger hole, single tube and double tube, will be discussed. A diagram of the equipment is shown in Figure 4.

THE AUGER HOLE METHOD

The auger hole method was developed by van Bavel and Kirkham (1948), and is the most practical for permeability measurements below a ground water table (G.W.T.). The procedure is to auger a hole to a depth below the ground water. The hole is allowed to fill and the highest level attained is assumed to be the G.W.T. height. Water is then pumped from the hole and the hole allowed to refill several times to flush out soil pores in the sides of the hole. A measurement is then taken of the change in water level with respect to time, \( \frac{dh}{dt} \), during subsequent pumping and refilling operations.
(a) Auger Hole Method

(b) Single Tube Method

(c) Double Tube Method

Figure 4 Sketches of Equipment Used for Field Determination of Permeability
This method is subject to an exact mathematical analysis based on the depths to a more pervious or impervious layer (van Bavel and Kirkham, 1948). Reasonable results have been obtained by using:

\[
\frac{dh}{dt} = \frac{k r}{S d^3} \frac{dh}{dt}
\]

Where:
- \( k \) = Permeability
- \( h \) = Depth of water in the hole
- \( r \) = Radius of hole when \( \frac{dh}{dt} \) is determined
- \( \frac{dh}{dt} \) = Rate of rise of water in hole at depth \( h \)
- \( S \) = Coefficient dependent on ratios of \( h/d \) and \( r/d \)

The coefficient, \( S \), decreases with increasing values of \( h/d \) and \( r/d \). A graphical method for determining this coefficient is presented by Spangler and Handy (1973).

The horizontal permeability is determined by this method because the flow into the hole is mainly horizontal. It is particularly useful for determining the horizontal permeability of nonisotropic soils. For auger holes in layered soils, the permeability is a composite of the permeabilities of the individual layers (Spangler and Handy, 1973). This method is very simple and has practical applications to the sanitary landfill, where the water table is near the surface.

THE SINGLE TUBE METHOD

The single tube method is very similar to the auger hole method, and was developed by Frevert and Kirkham (1948). For this method, after the hole is dug, a tight fitting tube is driven into the hole.
Repeated emptying of the hole is not necessary because flow does not pass through the sides of the hole. The hole is allowed to fill to determine the G.W.T. level and then emptied to begin the test.

The rate of rise of the water level is measured with respect to time. Permeability is determined by:

\[ k = \frac{A \ln \left( \frac{h_o}{h_f} \right)}{E \cdot t} \]  

(24)

Where:

- \( A \) = Area of the tube
- \( h_o \) = Initial depth of water level below the G.W.T.
- \( h_f \) = Final depth of water level below the G.W.T.
- \( t \) = Time required for the water level to rise from \( h_o \) to \( h_f \)
- \( E \) = Coefficient

The coefficient, E-factor, is determined from the diameter of the tube, the depth of the tube below the G.W.T. and the shape of the bottom of the hole. For a horizontal hole bottom, the simplest case, \( E \) increases with increasing diameter and decreases with increasing depth to diameter ratio. The dimensions of \( E \) are length, and it is determined by electrical analog studies. Values are tabulated in Spangler and Handy (1973).

For accuracy in the single tube method, several precautions are necessary. It is necessary that no leakage occurs between the tube and the hole. Also, the bottom of the hole should not be disturbed. Emptying the hole and allowing it to refill may flush out the pores at the surface.
The flow in the single tube method is vertical and this increases the effect of the vertical component of permeability. This method and the auger hole method are designed for use below the ground water table. The double tube method is designed for permeability determinations above the ground water table.

THE DOUBLE TUBE METHOD

The double tube method is apparently the most accurate field determination technique of permeability. The procedure and calculations are summarized here. For a theoretical derivation of the method, the reader is referred to the literature Bouwer (1961). It is primarily used for above the ground water table permeability determination.

The apparatus consists of an inner and outer tube each of which is connected to a stand pipe. The minimum ratio of inner and outer tube radii is 1.7, however accurate results have been achieved with a ratio of 1.6. An undisturbed surface at the bottom of the auger hole is necessary for use of the test.

The test procedure begins by driving the outer tube into the auger hole to penetrate to a depth of about 5 cm below the bottom of the hole. The soil is saturated by filling the outer tube and the inner tube is then emplaced in the outer tube. Guides in the outer tube are recommended to maintain concentricity. A penetration of 2 cm for clayey soils and 2 to 3 cm for sandy soils is recommended. The depth of penetration, \(d\), must be known for calculations. The inner tube is connected to the inner tube standpipe (ITS) by a plastic hose. The water level is maintained in the outer tube and the plastic hose
is coiled with no downward bends to prevent air locks after the cover is bolted to the outer tube. With the cover in place, water reservoir is connected to the system at C, and the outer tube standpipe (OTS) is filled. When the standpipes are full, the reservoir connection is changed to point D. Pressure differences between the inner and outer tubes must be minimized to prevent "blow out" of the soil between the inner and outer tube. Valve A is then closed and the rate of water fall is recorded for the ITS while the level in the OTS is maintained at a constant level. The ITS is refilled and valve A is opened. The rate of water fall for both tubes recorded for the same distance as before. The procedure is repeated until correlating data is attained. It is also recommended that ten times the amount of time required for the first level change is allowed before the second rate is determined.

The level changes are then plotted with respect to height and time axes. Figure 5 illustrates the procedure. Permeability is determined from the equation

\[ k = \frac{R_v^2}{F_f R_c} \left( \frac{\Delta H}{H dt} \right) \]  (25)

Where: \( R_v \) = Effective radius of the ITS
\( R_c \) = Radius of the inner tube
\( \Delta H, \int H dt \) = (Note Figure 5)
\( F_f \) = The flow factor, dimensionless

The flow factor, \( F_f \), is determined by:

\[ F_f = \frac{Q_H}{\pi H R_c k} \]  (26)
FIGURE 5
Graphical Determination of $\Delta H_t$ and $\int H dt$ for the Double Tube Method
Where: \( Q_H \) = The rate of flow at H
\( H \) = Distance of the water level in the inner tube above (positive) or below (negative) of the level of water in outer tube

For graphical solutions of \( F_f \), the flow factor is a function of the depth to a more permeable or impermeable strata underlying the tested strata \((D)\), and the depth of the penetration of the inner tube \((d)\). The first factor is expressed as \( D/R_c \) and the latter as \( d/R_c \) for solutions in Bouwer (1961). For the permeable layer case, the flow factor decreases for increasing values of \( D/R_c \) and \( d/R_c \). For the impermeable case, the flow factor increases with decreasing values of \( d/R_c \) and \( D/R_c \).

Data presented by Bouwer based on sand model experiment indicates that this method is very accurate. It can also be modified to determine a relationship between the vertical and horizontal permeability components of the soil (Bouwer, 1963). It does require knowledge of the geological characteristics of the area in which the test is performed. Experience in running the test is also necessary.

Field determinations of permeability by these methods have several advantages over the laboratory tests. The permeability is determined for a larger volume of the soil strata than for the laboratory tests. Sources of error from equipment are reduced, however, complexity is introduced in the double tube method. The accuracy to which the coefficients of each method, \((S, E, F_f)\) are determined will affect the accuracy of the tests. It may also be desired to determine the permeability for an entire strata of soil. For this, field pumping tests are recommended.
FIELD PUMPING TESTS

The field pumping test data is used to relate the shape of the cone of depression of a pumped well to the aquifer's ability to transmit water. The shape of the cone of depression is determined by measurements taken in wells near the pumped well. Most work performed to date in this field has been by the U.S. Geological Survey (Land, 1967). In their work, permeability is expressed in Meinzer units (rate of flow of water in gallons per day through one- \( \text{ft}^2 \) or a section under the influence of a hydraulic gradient of 1 ft/ft). The standard temperature is 60°F. This requires consideration in interpreting the Survey's data.

In aquifer analysis, aquifers are categorized in three general types: 1. completely confined aquifers (artesian); 2. aquifers confined by leaky strata; 3. water table aquifers. Methods for analysis are categorized as equilibrium and non-equilibrium. Analyses by the equilibrium method requires a stabilized cone of depression, a constant flow rate from the pumped well and known drawdown distances in wells at two different radii from the pumped well. The formula for these conditions is given as (Walton, 1970):

\[
k = \frac{Q \ln \left( \frac{r_2}{r_1} \right)}{2 \pi m \left( s_1 - s_2 \right)}
\]

Where:
- \( k \) = Permeability
- \( Q \) = Pumping rate
- \( r_1, s_1 \) = Distance and drawdown in observation well 1 respectively
- \( r_2, s_2 \) = Distance and drawdown in observation well 2 respectively
m = Thickness of the aquifer

Use of the equilibrium formula is based on the following assumptions (Lang, 1967):

1. The aquifer is homogenous, isotropic and infinite in a real extent
2. The well penetrates and receives water from the entire aquifer
3. The coefficient of permeability is constant in all places and all times
4. The flow is laminar.

Non-equilibrium conditions in aquifers are analyzed by applying the appropriate boundary conditions to the differential equation governing flow for the particular case (Walton, 1970). For artesian aquifers the differential equation is:

\[
\frac{\partial^2 h}{\partial r^2} + \frac{1}{r} \frac{\partial h}{\partial r} = \frac{S}{T} \frac{\partial h}{\partial t}
\]  \hspace{1cm} (28)

Where:

- \( T \) = Transmissibility
- \( S \) = Specific storage
- \( h \) = Height of cone of depression above underlying aquiclude (datum)
- \( r \) = Distance from well to observation point
- \( t \) = Time

Applying the boundary conditions as \( h \) approaches \( h_o \), \( r \) approaches infinity for \( t \) greater than 0, where \( h_o \) is the height of the piezometric surface above the datum, and initial conditions \( h(r,0) = h_o \) yields (Walton, 1970):

\[
S = \frac{Q}{4 \pi T} \int_0^{\infty} \frac{e^{-u}}{u} \frac{u}{r^2 s/4T} \, du
\]  \hspace{1cm} (29)
Where: \( S = \text{Drawdown} \ (h_0 - h) \)
\( Q = \text{Pumping rate} \)

The integral form in the above equation \( \int_{u}^{\infty} e^{-u} \frac{du}{u} \), is read as the well function of \( u \). This analysis assumes in addition to the above (Lang, 1967):

1. The pumped well has an infinitesimal diameter.
2. Water released from storage is discharged instantaneously.

Analysis by the above formula requires a logarithmic plot of \( u \) versus \( W(u) \), called a type curve. The observed data are plotted on a logarithmic plot of the same scale and superimposed on the type curve with parallel axes to obtain a best fit of the field data and type curve. The displacement of the major axes is then used to determine the transmissibility and hence permeability. For a more detailed review of non-equilibrium formula analysis, the reader is referred to Walton (1970) or Parcher and Means (1968).

Water table aquifer analysis is more complex. However, such a condition seems to apply to sanitary landfill applications more than the other two types. Because of the slow drainage in this type of aquifer the cone of depression occurs in three stages (Lang, 1967). In the first stages, water is released from storage as in an artesian aquifer. However, this represents only a small portion of the water available. Secondly, slow drainage of material above the cone of depression acts as a source of recharge causing drawdown to decrease. Thirdly, the cone spreads more rapidly until the cone of depression reaches a source of recharge or a boundary. In the third stage, incremental release of water from storage is small due to the large area involved. This progression requires superposition on the type
curve for data taken early in the test and data taken later in the test. The reader is referred to Walton (1970) for more detailed explanation.

Several conditions must be considered before pumping is applied to sanitary landfill site exploration. Pump test data would provide an accurate picture of the actual field conditions. The permeability can be determined for both the horizontal and vertical components. However this method is recommended for aquifers approaching homogenous conditions (Mansur and Dietrich, 1965). This may not be the field situation, particularly for water table aquifers, where impervious strata may invalidate the test results.

Another consideration is the accuracy inherent in the data analysis. A best fit between field data and type curves is matter of judgement. Experience would be required for accuracy. The cost of the pumping test method is high. Several wells are required, and equipment for pumping and level measurements are required. Therefore, the use of this method may be limited, where less expensive techniques for permeability determination are judged to be sufficient.

In all the permeability determination methods discussed there are sources of inaccuracy. Operator error, coefficient determinations, non-ideal conditions and the degree that the sample represents the field condition are cited as sources. Quantization of the accuracy for a general case or even a particular method is extremely difficult. The use of several different test procedures (or repeating of procedures) should result in a range of permeabilities representative of a given site or strata. The range of permeabilities and their useful design
would depend on the desired accuracy of the application to the field condition and the economic cost which could be justified.
CHAPTER IV
APPLICATION OF PERMEABILITY

Initial site explorations are extremely important before the location of the landfill is chosen. Access, topography and soil conditions should be considered. Increasing awareness of the need for environmental preservation dictates protection for the surrounding ground waters. Because the leachate is carried through the soil, the importance of soil permeability in site selection consideration is emphasized. A representative permeability determination is then necessary.

The permeability determined in the field pumping tests is representative as an average of the entire strata tested. The horizontal and vertical components of the permeability can be determined by calculation (Mansur and Dietrich, 1965). For flow conditions in strata, a representative permeability is determined by mathematical analysis.

For flow parallel to the orientation of the strata the arithmetic mean of the permeability is used.

$$k_p = \frac{k_1 + k_2 + k_3 + \ldots + k_n}{n}$$  (30)

Where:

- $k_p$ = representative permeability for parallel flow
- $k_1, k_2, k_n$ = permeabilities of the strata
- $n$ = number of strata

For flow perpendicular to the strata orientation a harmonic mean is used:

$$k_s = \frac{n}{1/k_1 + 1/k_2 + \ldots + 1/k_n}$$  (31)
Where: $k_s =$ representative permeability for series flow

It has been shown, by electrical analog studies, that the geometric mean is most representative in anisotropic strata of soils (Bouwer, 1969). It is determined by

$$k_g = \sqrt[n]{k_1 \times k_2 \times \cdots \times k_n}$$

(32)

Where: $k_g =$ geometric mean

The analyses used to arrive at a representative permeability will depend on the flow condition and the soil conditions.

The rate of leachate migration at a sanitary landfill will be no greater than the rate of ground water movement. For water table conditions, the hydraulic gradient is determined from piezometric surface measurements. The velocity can then be determined using the permeability by Darcy's Law. For migration downward into a ground water table, infiltration rate analyses are required. Methods of determining this rate are given by Fok (1970). The permeability of the soil is required for this determination.

Dispersion perpendicular to the direction of flow can be expected. Methods to determine this dispersion are presented in Li and Lai (1966). Varying concentrations of pollutants carried by the leachate can be expected. This variance will also be proportional to the permeability of the soil. (Cartwright and McComas, 1968). A method of pattern analysis is given in Legrand (1965). Predicting this dispersion is the major task in choosing sites for landfills; controlling it is the management responsibility.

Three alternatives exist in choosing and managing a landfill site. These are (Remson, et al, 1968):

1. Consolidate and stabilize the site as soon as possible
2. Delay degradation as long as possible

3. Control degradation so that leachate production will be within acceptable limits.

The first alternative would require an area of high permeability with compaction of cover materials minimized. Localization of leachate to prevent migration to ground waters must be accomplished. At the Orange County site, geology prevents migration (McLellan, 1973). The second would require a nearly impermeable site with highly compacted cover material. For the third alternative compaction requirements for the cover material could be specified with respect to the permeability. Soil additives may also be used to further control the permeability.

If permeability is used as a control parameter, it should be determined to a high degree of accuracy. At present, the laboratory and field methods may not achieve desired accuracy. The use of many tests and the determination of some statistical variation could be a possible solution to the accuracy problem, although it would be very, very expensive. This suggests that further research needs to be accomplished to determine the reliability of permeability tests. Experience gained in this area could be valuable to the environmental preservation task, if the costs are reasonable.
CHAPTER V
SUMMARY AND CONCLUSIONS

A review of soil permeability theory, determination methods and applications to sanitary landfills is presented. The characteristics of soils and their relationships to permeabilities is also discussed. Several conclusions have been reached in this report.

The complete validity of Darcy's Law when applied to soil seems doubtful. The law fails to account for several factors which are significant for certain conditions, such as turbulent flow and inertial forces. However, no alternative theory has been proposed which has practical applications to the natural soil case. Darcy's Law has been successfully applied in past work and can be used if limitations are respected.

Permeability may be measured in the laboratory or the field by various methods. The size of the sample or area of influence must be considered in interpreting test data. Factors which must be considered in choosing tests methods are sample disturbance, equipment and operator sources of error, accuracy required, and cost. The faith placed in results is a matter of experience and judgement.

Several considerations in applying permeability to landfill applications are apparent. The test results may yield only a range of values. The geometric mean of these values is suggested as the most representative of the field conditions particularly for anisotropic layered soils. Field pumping tests would give a representative perme-
ability for an entire stratum; however, it is not applicable to the above case. The leachate can be expected to change the permeability of the soil. Prediction of these changes has not yet been determined and requires further research.

The California State Department of Water Resources has classified the acceptability of the landfill sites with respect to transmissibility. Their classification is very general, and indicates the accuracy of permeability tests currently available is sufficient. Further study particularly in retrospective analysis should improve on this classification method.

In managing sanitary landfills, permeability can be used to determine cover characteristics. Compaction and use of soil additives are recommended for limiting permeability. Management techniques should be directed toward limiting or eliminating leachate migration from the landfill site. This migration could be quite costly particularly where municipal water resources are derived from ground water sources. The cost must be measured in terms of removing pollutants from surrounding ground waters. The accuracy of available determination methods of permeability may limit its application in managing landfills.

The present state of the art indicates a methodology in locating and managing sanitary landfills. Based on geologic/hydrologic evaluations, permeability including determinations, a reasonable judgement of site acceptability can be made. Management practice can use permeability to help specify operational procedures if sufficient accuracy is obtainable. Most important a monitoring program must be established to determine the rate of leachate migration within or from the site. The leachate migration might limit further use of the landfill site or re-
quire construction of control structures. Research at the landfill site will provide information for future improvement of the sanitary landfill technique for solid waste disposal.
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